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Chapter 1 - Introduction

The treatment and disposal of wastewater in developing countries is of prime importance for environmental and public health reasons. The simplest method of municipal wastewater treatment is through the use of waste stabilization ponds or lagoons. Lagoons are simple earthen basins in which wastewater is treated by the removal of particulate matter and the biological degradation of settled solids. Waste stabilization ponds rely on lengthy detention times and environmental factors (wind, solar radiation) for treatment efficiency.

This report centers on the design of lagoons following a chemically enhanced primary treatment (CEPT) stage in Tatui, Brazil. The present treatment facilities in Tatui, which consist of an anaerobic lagoon followed by a facultative lagoon, are over-loaded, and hence insufficient. There exists a proposed design to replace the existing facilities by aerated lagoons followed by settling tanks. This design is proposed by the environmental agency for the state of São Paulo.

The design of the lagoons that will follow the CEPT stage will be done using empirically derived guidelines found in literature, and subsequently tested using a model previously fitted on other lagoons in Brazil. The model is an adapted version of a dynamic "bio-geo-chemical" lagoon model developed by Raymond Ferrara in 1978. The model will also be modified to model aerated lagoons, in order to predict the efficiency of the SABESP design.

Chapter 2 will introduce lagoons, and review the processes involved in the three main lagoon types. Chapter 2 will also review literature in terms of empirical design guidelines. The chapter will conclude by proposing a design for the lagoons to follow the CEPT stage at Tatui.

Chapter 3 will introduce the Ferrara model, and exhibit the adapted Ferrara model. The data used to fit the models will be explained, and a model to predict lagoon temperature will be developed. The models on lagoons and aerated lagoons will be developed and studied. Finally, chapter 3 will show the results of the use of the models on the proposed designs.

Chapter 4 will conclude the report and propose recommendations for both the models and the lagoon design.

Chapter 2 - Wastewater Stabilization Ponds

Introduction

The primary purpose of wastewater treatment is the reduction of pathogenic contamination, suspended solids, oxygen demand and nutrient enrichment. Waste stabilization ponds are a cheap and effective way to treat wastewater in situations where the cost of land is not a factor. The goal of this chapter is to review the different types of waste stabilization ponds. This chapter will also introduce the design of the lagoons for the CEAGESP treatment plant in Tatui.

Wastewater Stabilization Lagoons: A Review

The Advantages of Wastewater Stabilization Ponds

Conventional treatment of liquid wastes involve the use of energy intensive mechanical treatment systems, and are the norm in developed countries (Arthur, 1983.) However, they are not the best option for less developed countries. Indeed, conventional treatment schemes were developed due to climatic and area constraints. These constraints are often not the case in developing countries. Moreover, the use of energy intensive mechanisms is not desirable in less developed countries, where energy supply is not reliable. Further, conventional treatment facilities require regular high-skilled maintenance, a thing that is either too expensive or impossible to find in developing countries.

Stabilization ponds offer many advantages over conventional treatment schemes. One of their most important advantages is their ability to remove pathogens (WHO EMRO

Technical Publication No. 10, 1987.) For conventional systems, pathogen removal is only attained with tertiary treatment, such as the use of maturation ponds or chlorination. In addition, stabilization pond systems are much less costly, for both capital costs and maintenance costs. Pond systems are a viable option for both large and small populations. Moreover, wastewater stabilization ponds exhibit what is known as the "reservoir effect", which enables the pond to absorb both organic and hydraulic shock loadings. The following section will introduce and describe the different types of wastewater stabilization ponds.

Types of Stabilization Ponds

There are three main types of stabilization ponds: anaerobic, facultative and maturation. This section will outline the mechanisms involved in the three main types of ponds, and will describe their loading capacities and efficiencies.

Anaerobic Ponds

Anaerobic ponds, which are lacking oxygen except at a thin layer at the surface, rely totally on anaerobic digestion to achieve organic removal. Anaerobic digestion is a two-stage process. The first stage is putrefaction, and the second stage is methanogenesis. Putrefaction is the bacterial degradation of organic matter into organic acids and new bacterial cells. In methanogenesis, methanogenic bacteria break down the products of putrefaction into methane, carbon dioxide, water, ammonia and new bacterial cells.

Anaerobic ponds operate under heavy organic loading rates (usually greater than 100g BOD/m^3.d). Anaerobic ponds thus contain no dissolved oxygen, and algae are only present on a thin film at the surface). The main mechanism of BOD removal in anaerobic ponds is by sedimentation of settleable solids, and subsequent anaerobic digestion in the resulting sludge layer. The typical design and efficiency values for anaerobic ponds can be seen in Table 2-1.

Table 2-1: Anaerobic Pond Design Criteria

Source	Optimal Depth [m]	Surface Loading [kg/ha.d]	Detention Time [d]	BOD Removal [%]	TSS Removal [%]	Optimal Temperature [C]
Metcalfe & Eddy (1993)	2.5 – 5	225 – 560	20 – 50	50 – 85	20 – 60	30
WHO EMRO Technical Report No. 10 (1987)	2.5 – 5	> 1,000	5	50 – 70	NA	25 – 30
Lagoon Technology International (1992)	2 – 5	> 3,000	1 – 2	75	NA	25
World Bank Technical Paper No. 7 (1983)	4	4,000 – 16,000	2	NA	NA	27 – 30

It is obvious that there is a great range of values for surface loading rates for anaerobic ponds. It has been widely recognized that this type of design criterion is insufficient for anaerobic ponds. Indeed, the preferred loading rate design value should be expressed with respect to volume, and not surface area (Metclafe & Eddy, 1993). The typical value for volumetric loading rate for an anaerobic pond is 100 – 400 g BOD/m^3/day.

Anaerobic ponds are used as the primary stage in the pond treatment process. A primary facultative pond can, however, replace them. Facultative ponds are discussed in the following section.

Facultative Ponds

Facultative ponds take their name from the facultative bacteria that populate them. Facultative bacteria are capable of adaptive response to aerobic and/or anaerobic conditions. Facultative ponds degrade organic matter through different processes depending on the depth layer considered. Figure 2-1 presents a schematic of the processes involved in facultative ponds.

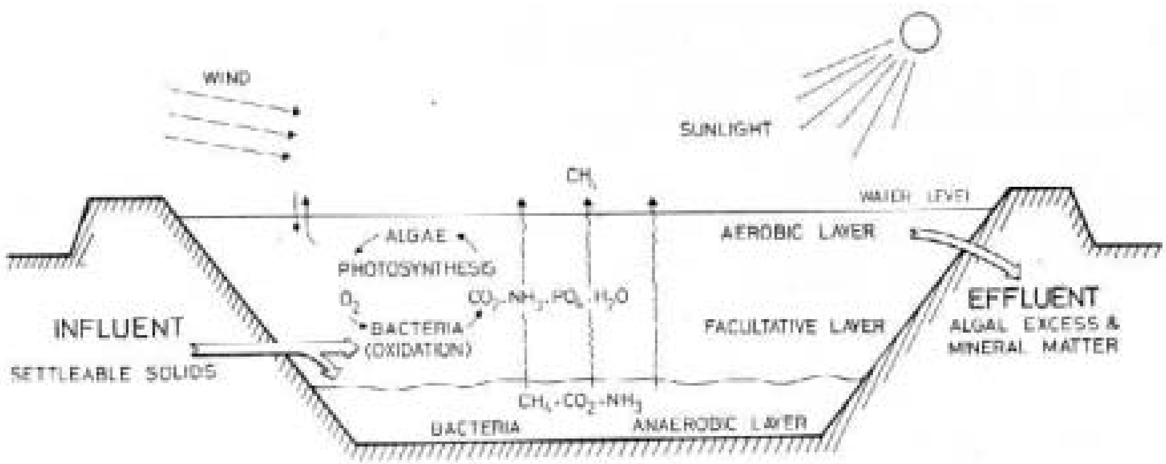

Figure 2-1: Processes involved in Facultative Ponds

As can be seen in Figure 2-1, facultative ponds have three biologically-active layers. In the bottom, where sludge accumulates, organic matter is degraded anaerobically. In the top layer, the organic matter is degraded aerobically due to the presence of dissolved oxygen produced by photosynthesis occurrence in algae. Finally, in the middle layer, the facultative layer, dissolved oxygen is present some of the time, fed from the upper layer.

The transformations occurring in a facultative pond are generally from biodegradable organic matter to living organic matter (i.e. algae, bacteria, protozoa, etc.). In their

Technical Paper No. 10, the WHO state that the biochemical oxygen demand generated from living organisms such as algae is not necessarily detrimental to the environment.

Table 2-2 presents the design criteria for facultative ponds. Again, there are some discrepancies in the literature, but these discrepancies are mostly due to their reference to different geographic locations, and hence different climatic conditions.

Table 2-2: Facultative Pond Design Criteria

Source	Optimal Depth [m]	Surface Loading [kg/ha.d]	Detention Time [d]	BOD Removal [%]	TSS Removal [%]	Optimal Temperature [C]
Metcalfe & Eddy (1993)	1.2 – 2.5	60 – 200	5 – 30	80 – 95	70 – 80	20
WHO EMRO Technical Report No. 10 (1987)	1.5 – 2	200 – 400	NA	80	NA	20 – 30
Lagoon Technology International (1992)	1 – 2	100 – 400	NA	70 – 80	NA	NA
World Bank Technical Paper No. 7 (1983)	1 – 1.8	200 – 600	NA	NA	NA	15 – 30

Maturation Ponds

Maturation ponds are placed last in the pond treatment system, if they are used at all. They are very shallow, and generally occupy very large surface areas. Their main function is the reduction of pathogenic organisms. Maturation ponds are also known to remove some algae and some nutrients, but this is not their principal function. The processes by which the pathogens are removed are multiple, and include sedimentation, lack of food and nutrients, solar ultra-violet radiation, high temperatures and pH, natural predators, toxins and natural die-off.

The general design values and efficiencies of maturation ponds are presented in Table 2-3.

Table 2-3: Maturation Pond Design Criteria

Source	Optimal Depth [m]	Surface Loading [kg/ha.d]	Detention Time [d]	BOD Removal [%]	TSS Removal [%]	Optimal Temperature [C]
Metcalfe & Eddy (1993)	1 – 1.5	≤ 17	5 – 20	60 – 80	NA	20
WHO EMRO Technical Report No. 10 (1987)	1 – 1.5	NA	5 – 10	50 – 60	NA	NA
Lagoon Technology International (1992)	1 – 1.5	NA	NA	NA	NA	NA
World Bank Technical Paper No. 7 (1983)	1.2 – 1.5	NA	5	NA	NA	NA

Design of the Lagoon System to follow the CEPT System

Having briefly reviewed the various types of wastewater stabilization ponds, the present task is to select the appropriate lagoon type and size to treat the CEPT settling tank or lagoon effluent in our proposed design for Tatui, Brazil. This section will present the approximate CEPT tank effluent characteristics under three different flow regimes associated with estimated population growth. Subsequently, an appropriate lagoon design will be proposed, and the estimated effluent quality presented.

CEPT Settling System Effluent Characteristics

The raw influent characteristics to the Tatui-CEAGESP treatment plant, the CEPT expected removal efficiencies, and the CEPT effluent characteristics are presented in Table 2-4. The values for predicted flows and influent quality are taken from the 1992

SABESP report on the sanitation situation in Tatui, and the values for removal efficiencies are taken from the jar test data.

Table 2-4: Estimated CEPT System Influent[1] and Effluent under Three Flow Regimes

Year	Flow [L/s]	Influent BOD [kg/d]	Influent TSS [kg/d]	Effluent BOD [kg/d]	Effluent TSS [kg/d]
1995	135	2945	1491.9	1472.5	298.4
2005	161	3843	1779.2	1921.5	355.9
2015	244	5823	2696.5	2911.5	539.3

Given the data of Table 2-4 and a present available lagoon surface area of 5.5 ha, the following three options are available:

1. Anaerobic pond(s) (in parallel) followed by facultative pond(s) (in parallel).

2. Facultative pond(s) (in series).

It is also necessary to design lagoons that will fit in the area available after using the in-pond CEPT treatment option (i.e. 4 ha).

3. Anaerobic pond followed by a facultative pond.

These three options are examined in detail in the next two sub-sections.

[1] Raw influent is flow and BOD loading are taken from SABESP Edital 1992. Influent TSS is approximated to 130 mg/L. Removal efficiencies are estimated at 50% for BOD and 80% for TSS.

Option 1: Parallel Anaerobic Ponds in Series with Parallel Facultative Ponds

Option 1 involves a number of small anaerobic ponds operating in parallel. The necessity for multiple anaerobic ponds is dictated by the principle that anaerobic ponds only function properly under minimal loading. Since the effluent loading is predicted to increase by one third from 2005 to 2015, it is necessary to provide multiple ponds to accommodate the growing load, while maintaining a minimal load in each pond.

Table 2-5 presents the anaerobic pond volumes required under the three different loading scenarios.

Table 2-5: Anaerobic Pond Design under Three Loading Regimes

Year	BOD Loading [kg/d]	Volumetric Loading [g/m3/d]	Anaerobic Pond Volume Required [m^3]	Number of Ponds (3.5 m depth)	Pond Area [ha]	Detention Time [d]
1995	1,472.5	105	14,000	1	0.4	1.2
2005	1,921.5	137	14,000	1	0.4	1
2015	2,911.5	104	28,000	2	0.4	1.33

The design values for volumetric loading and detention time in Table 2-5 respect the minimal quantities to achieve anaerobic conditions. The detention time is on the short side, but given that most of the suspended solids are removed in the CEPT tanks, a detention time of one day is presumed to be sufficient. Under these conditions, it is estimated that the anaerobic lagoons will achieve a further 50 % reduction of the BOD load.

Under option 1, the effluent from the anaerobic ponds will be directed to a facultative pond. The design for this facultative pond is seen in Table 2-6.

Table 2-6: Facultative Pond Design under Three Loading Regimes

Year	BOD Loading [kg/d]	Surface Loading [kg/ha/d]	Facultative Pond Area Required [ha]	Number of Ponds (2.5 m depth)	Pond Area [ha]	Detention Time [d]
1995	736.25	245	3	2	1.5	6.4
2005	960.75	320	3	2	1.5	5.4
2015	1,455.75	323	4.5	3	1.5	5.3

Again, the detention times for the facultative pond system are quite short, but they are still within limits. The predicted effluent characteristics from this system are shown in Table 2-7.

Table 2-7: Predicted Effluent Quality for Option 1

Year	BOD Influent to Anaerobic Ponds [mg/L]	Anaerobic Pond BOD Removal Efficiency [%]	BOD of Influent to Facultative Ponds [mg/L]	Facultative Pond BOD Removal Efficiency [%]	Final Effluent BOD [mg/L]
1995	126.2	50	63.1	60	25.2
2005	138.1	50	69	50	34.5
2015	138.1	50	69	50	34.5

Option 2: Facultative Ponds in Series

The second option makes no provision for anaerobic ponds. Rather, the CEPT tank effluent is directed immediately into a facultative pond. Table 2-8 shows the design values for this option.

Table 2-8: Facultative Pond Design under Three Loading Regimes

Year	BOD Loading [kg/d]	Surface Loading [kg/ha/d]	Facultative Pond Area Required [ha]	Number of Ponds (2.5 m depth)	Pond Area [ha]	Detention Time [d]
1995	1,472.5	268	5.5	1	5.5	11.8
2005	1,921.5	350	5.5	1	5.5	9.9

2015	2,911.5	530	5.5	1	5.5	6.5

The effluent quality out of the facultative pond is shown in Table 2-9.

Table 2-9: Predicted Effluent Quality for Option 2

Year	BOD Influent to Facultative Pond [mg/L]	Facultative Pond BOD Removal Efficiency [%]	Final Effluent BOD [mg/L]
1995	126.2	70	37.9
2005	138.1	65	48.3
2015	138.1	60	55.2

Option 3: Anaerobic pond followed by facultative pond (Restricted area)

The third option uses less area than the two previous options due to the presence of a CEPT lagoon, where the chemical coagulation and settling takes place. It is assumed that the CEPT lagoon will have the same removal efficiencies as the CEPT tanks. This assumption represents a gross underestimation, since the CEPT lagoons have a hydraulic retention time (HRT) of 1 day and the CEPT tanks have a HRT of 1 hour.

Table 2-10 presents the design of the anaerobic lagoon under option 3.

Table 2-20: Anaerobic Pond Design under Three Loading Regimes

Year	BOD Loading [kg/d]	Volumetric Loading [g/m3/d]	Anaerobic Pond Volume Required [m^3]	Number of Ponds (4 m depth)	Pond Area [ha]	Detention Time [d]
1995	1,472.5	57	26,000	1	0.65	2.2
2005	1,921.5	74	26,000	1	0.65	1.9
2015	2,911.5	112	26,000	1	0.65	1.2

The design values for volumetric loading and detention time in Table 2-10 are below the minimal quantities to achieve anaerobic conditions for years 2005 and 2005. However, the anaerobic lagoon in Riviera de São Lorenço (refer to chapter on lagoon modeling for details about Riviera de São Lorenço) exhibited low volumetric loadings, and still achieved an average 50% removal efficiency. Under these conditions, it is therefore estimated that the anaerobic lagoons at Tatui will achieve a further 50 % reduction of the BOD load.

Under option 3, the effluent from the anaerobic ponds will be directed to a facultative pond. The design for this facultative pond is seen in Table 2-11.

Table 2-11: Facultative Pond Design under Three Loading Regimes

Year	BOD Loading [kg/d]	Surface Loading [kg/ha/d]	Facultative Pond Area Required [ha]	Number of Ponds (3 m depth)	Pond Area [ha]	Detention Time [d]
1995	736.25	223	3.3	1	3.3	8.5
2005	960.75	291	3.3	1	3.3	7.1
2015	1,455.75	441	3.3	1	3.3	4.7

Again, the detention times for the facultative pond system are quite short, but they are still within limits (c.f. Table 2-2). The predicted effluent characteristics out of this system are shown in Table 2-12.

Table 2-12: Predicted Effluent Quality for Option 3

Year	BOD Influent to Anaerobic Ponds [mg/L]	Anaerobic Pond BOD Removal Efficiency [%]	BOD of Influent to Facultative Ponds [mg/L]	Facultative Pond BOD Removal Efficiency [%]	Final Effluent BOD [mg/L]
1995	126.2	50	63.1	50	31.5
2005	138.1	50	69	50	34.5
2015	138.1	50	69	50	34.5

Conclusions

In this chapter, waste stabilization ponds (lagoons) were reviewed. These lagoons were shown to have many advantages over more conventional wastewater treatment methods. The second part of the chapter considered the design of lagoons to follow the chemically enhanced primary treatment stage for Tatui, Brazil. Three lagoon configuration options were presented to follow the CEPT stage in Tatui. None of these options was retained as the "best" option. Indeed, all options produce comparable predicted effluent qualities. However, the third option is proposed as the one that should kept as the final conceptual design, because of the reduced area that it occupies. Area constraints are very limiting in Tatui. Moreover, the third option is suitable for the two possible CEPT applications: in-tank and in-pond.

Chapter 3 - Lagoon Modeling

Introduction

Mathematical modeling not only summarizes accumulated data, but it also provides an essential analytic tool. Models can act as compact data generators, as well as form the basic framework for hypothesis testing. Furthermore, models can generate data where it was absent. Interpolation between data points can be achieved with a model, and so can extrapolation. In any science, modeling the data is an efficient way to keep a record while notably increasing its potential usefulness.

Modeling the processes that occur in a waste stabilization pond is an essential part of this project. Indeed, the model will compare the proposed design with that of a CEPT system and smaller lagoons. The model will also be useful for lagoon sizing and configuration.

The Ferrara Model

Introduction

The waste-stabilization pond model proposed by Raymond Ferrara describes both hydraulic transport and biological and chemical transformation of material. The model was developed in 1978, and was extensively tested on waste stabilization ponds in the United States. The Ferrara model is a dynamic mathematical model for predicting the effluent quality of stabilization ponds. Ferrara and Harleman (1981) show that the fully mixed hydraulic assumption was valid for most waste stabilization ponds. This means that the underlying hydraulic assumption in the model is that the concentration of all model variables is uniform in the entire pond. The implications of assuming the ponds to be fully mixed are that the predicted efficiency will be worse than a plug-flow model. However, the fully mixed assumption ignores dead-zones and short-circuiting.

Governing Principles of the Model

Waste stabilization ponds are an extension of natural systems, and it is therefore appropriate to use similar modeling approaches. The bio-geo-chemical part of the Ferrara model is based on five general principles:

1. Mineralization of organic compounds: assumed to be first-order with respect to organic matter concentration.

2. Organism growth: proportional to organic matter concentration.

3. Net loss of material by settling of non-biodegradable organic matter, precipitation and adsorption of inorganic phosphorous, and denitrification: assumed to be first-order.

4. Atmospheric re-aeration of CO_2: first-order reaction with respect to difference between saturation and actual concentration of CO_2.

5. Removal of fecal coliform by death and predation: assumed to be first-order.

The Ferrara model was developed and tested in 1978 with pond treatment systems in Corinne, Utah and in Kilmichael, Mississippi.

Adapted Version of the Ferrara Model

The complexity of a model is directly related to its accuracy of simulation. However, complex models need more parameters, and require more sophisticated solution techniques. The usefulness of a model is dictated by the data available to the modeler. In our case, the data available and output desired were much related. Indeed, in Brazil, the main effluent constraints pertaining to environmental legislation revolve around oxygen demand. There are no legal constraints as to the nutrient or pathogenic contents of wastewater. The model was therefore restricted to three governing equations. These equations are Equations 3-1 through 3-3.

$$\frac{d(OC)}{dt} = \frac{Q_i}{V}(OC)_i - \frac{Q_e}{V}(OC) - R_{12}(OC) - R_{1S}(OC) + R_{21}\left[\frac{IC}{K_{SC}+IC}\right](OC) \quad \textbf{(3-1)}$$

$$\frac{d(IC)}{dt} = \frac{Q_i}{V}(IC)_i - \frac{Q_e}{V}(IC) + R_{12}(OC) + R_{20}\left(CO_{2_S} - CO_2\right) - R_{21}\left[\frac{IC}{K_{SC}+IC}\right](OC) \quad \textbf{(3-2)}$$

$$\frac{d(FC)}{dt} = \frac{Q_i}{V}(FC)_i - \frac{Q_e}{V}(FC) - R_{8S}(FC) \quad \textbf{(3-3)}$$

The legend to these equations is presented in Table 3-1.

Table 3-1: Legend for Equations 3-1 to 3-3

SYMBOL	DEFINITION
OC	Concentration of organic carbon
IC	Concentration of inorganic carbon
FC	Number of fecal coliforms per unit volume
Q	Flow rate
$_i$	Subscript for influent
$_e$	Subscript for effluent
V	Volume of pond
R_{12}	Transformation rate from organic carbon to inorganic carbon

R_{21}	Transformation rate from inorganic carbon to organic carbon
R_{20}	Atmospheric re-aeration rate
R_{1S}	Organic carbon net loss rate
K_{SC}	Half-saturation constant for carbon
R_{8S}	Overall fecal coliform decay rate

Reaction rates R_{12}, R_{21}, R_{1S} and R_{8S} are temperature dependent. The value for these reaction rates is known for a temperature of 20^{o} Celcius. They are corrected to take into account the lagoon temperature with Equation 3-4.

$$R_{XY}^{T} = R_{XY}^{20} \cdot \theta^{(T-20)} \quad \textbf{(3-4)}$$

The three governing equations of the MIT-Ferrara Model were programmed using the Runge-Kutta 4th Order algorithm for numerical approximation.

Modeling the Riviera de São Lorenço Data

Background

Riviera de São Lorenço is a summer resort located about 140-km northeast of São Paulo. A private company, Sobloco, manages the water supply and sanitation for Riviera. The resort-city is fully sewered. The wastewater treatment plant for Riviera is a system of lagoons. The raw influent is directed through an anaerobic pond, and it is subsequently directed to one of three facultative ponds (see Figure 3-1 for Riviera de São Lorenço WWTP schematic.)

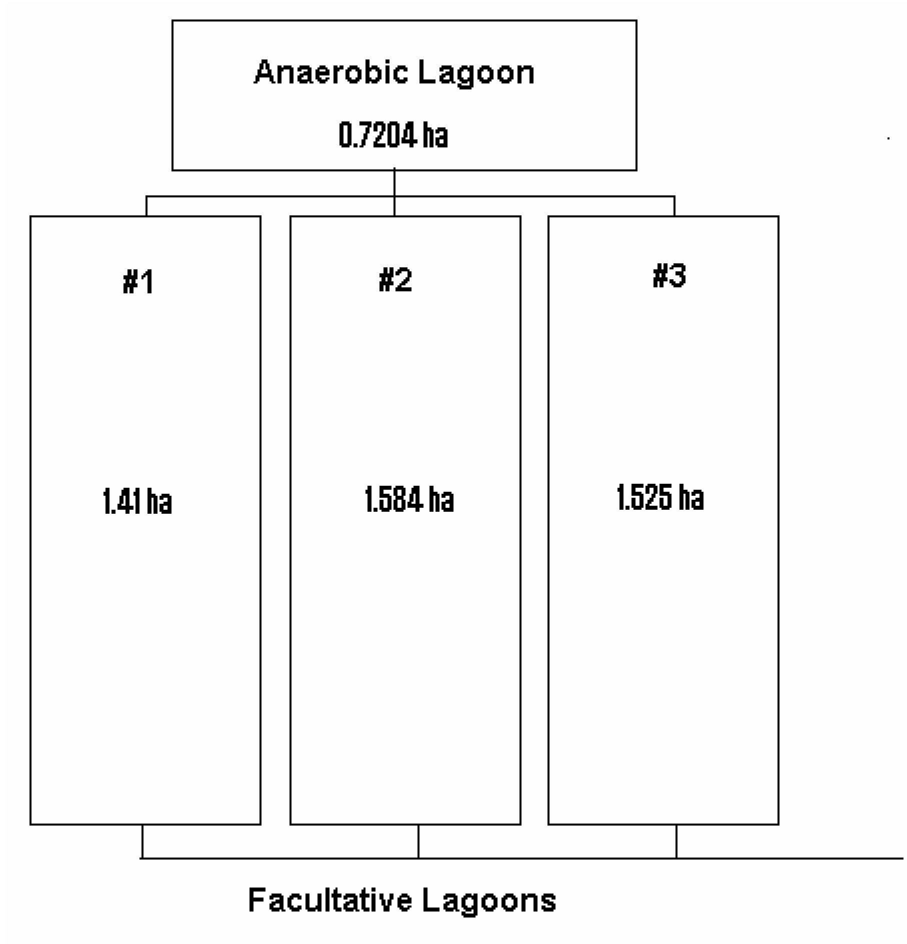

Figure 3-1: Riviera de São Lorenço Treatment System Schematic

It is widely accepted that the WWTP at Riviera is the best operated in the state of São Paulo (Personal communication with Dr. Ricardo Tsukamoto, 1999). Moreover, the lagoons are monitored regularly in terms of water quality and organic-load removal efficiency. Data from the Riviera de São Lorenço wastewater treatment plant was obtained through Dr. Ricardo Tsukamoto, who keeps a close contact with the Riviera staff. The quality and quantity of data available from Riviera are ideal for model-fitting purposes. Indeed, the Ferrara model had previously only been applied to waste stabilization ponds in the United-States. It was therefore necessary to fit the model to Brazilian data, before using it in a predictive mode.

Although the characteristics of Riviera and Tatui are entirely different, both treatment systems under consideration treat domestic waste.

The Riviera Data

The data available from Riviera is of high quality. However, there are some missing values in the data set. The Ferrara model requires a steady stream of daily values for organic loading (in the form of concentration of organic carbon), inorganic loading, inflow rate, outflow rate and pond temperature. None of the latter was complete in the data set provided. It was therefore necessary to fill the gaps with statistically generated or modeled data.

The COD removal efficiency of the Riviera lagoon system is depicted in Figure 3-2, where monthly COD averages are shown for the raw influent, the anaerobic pond effluent and the combined facultative pond effluent. It should be noted that since the three facultative ponds are configured in parallel, the monthly COD values were averaged over the three ponds. These values were computed for a period lasting from the 24th of December 1997 until the 25th of February 1999.

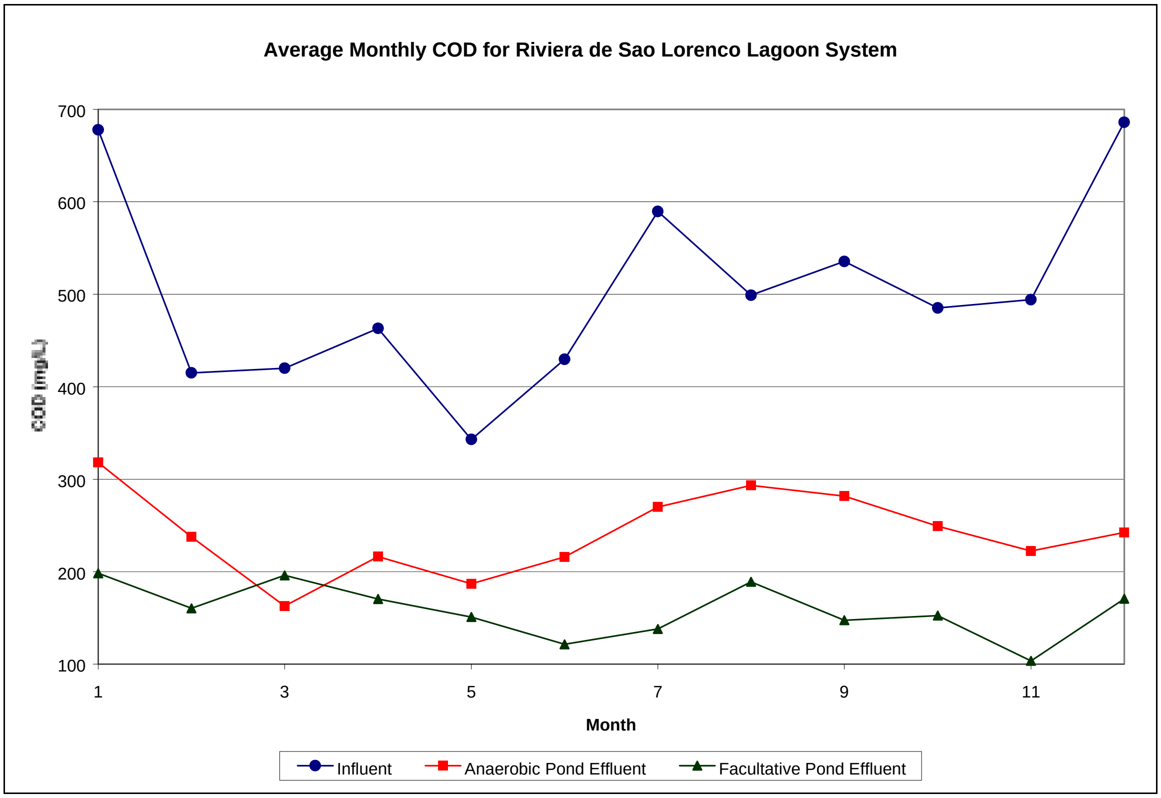

Figure 3-2: Monthly Averaged COD values for the Riviera de São Lorenço Lagoon System

The yearly average COD removal efficiency in the anaerobic pond is of 51.4%, whereas the average facultative pond removal efficiency is of 37.1%. The data that was made available for the Riviera system represents the period running from the 24th of December 1997 until the 25th of February 1999. The monthly COD averages are therefore only representative of 1998, except for the months of January and February, which represents an average of 1998 and 1999. It is important to note that the second facultative lagoon was undergoing maintenance from the 19th of June to the 17th of December 1998, period during which it was unused. Also, the third facultative lagoon was only put into service on the 10th of June 1998, and the first facultative lagoon was not loaded for the months of June through August, in order to load up the third facultative lagoon. Consequently, the facultative removal efficiency depicted in Figure 3-2 is representative of facultative lagoons 1 & 2 for the first half of the year, and lagoons 1 & 3 for the second half of the year. This might explain the low removal efficiencies witnessed in the first half of the year. The second facultative lagoon, due for maintenance, probably skewed the efficiencies on the downside. If the first half of the year is omitted in the calculation of average facultative pond COD removal efficiency, the averaged COD removal is 42.5% in the facultative lagoons.

Riviera Lagoons Loading, Detention Time and removal Efficiencies

The lagoons at Riviera were examined in terms of organic loading, detention time and removal efficiency. The objective of this study was to compare the performance of the lagoons at Riviera with the generic performances cited in the literature.

Figure 3-3 represents the removal efficiencies for all ponds as compared to the surface loading of the ponds. The three low removal efficiencies that can be seen for the facultative ponds at low surface loadings are for the months of March, April and May 1998. These are the three months that precede the second facultative pond maintenance schedule. On the other side, the two highest removal efficiencies for the facultative ponds, which occur at the same surface loading range, are for the months of September and November 1998. It is thought that the data available for the Riviera ponds, although of high quality, is not sufficient to propose firm conclusions. Indeed, the processes that govern the inner-workings of waste-stabilization ponds are quite complex, being influenced by climactic, environmental and anthropogenic factors. Thus, a lengthy dataset is required in order to smooth out the external factors, especially the anthropogenic disturbances (as is the present case). Moreover, the year 1998 is characterized by many changes in the management of the ponds at Riviera. A new facultative pond was added, and an existing facultative pond was put in maintenance. It is therefore suggested that the only valid dataset available from Riviera de São Lorenço is that of the anaerobic pond, because it was the least subject to anthropogenic disturbances.

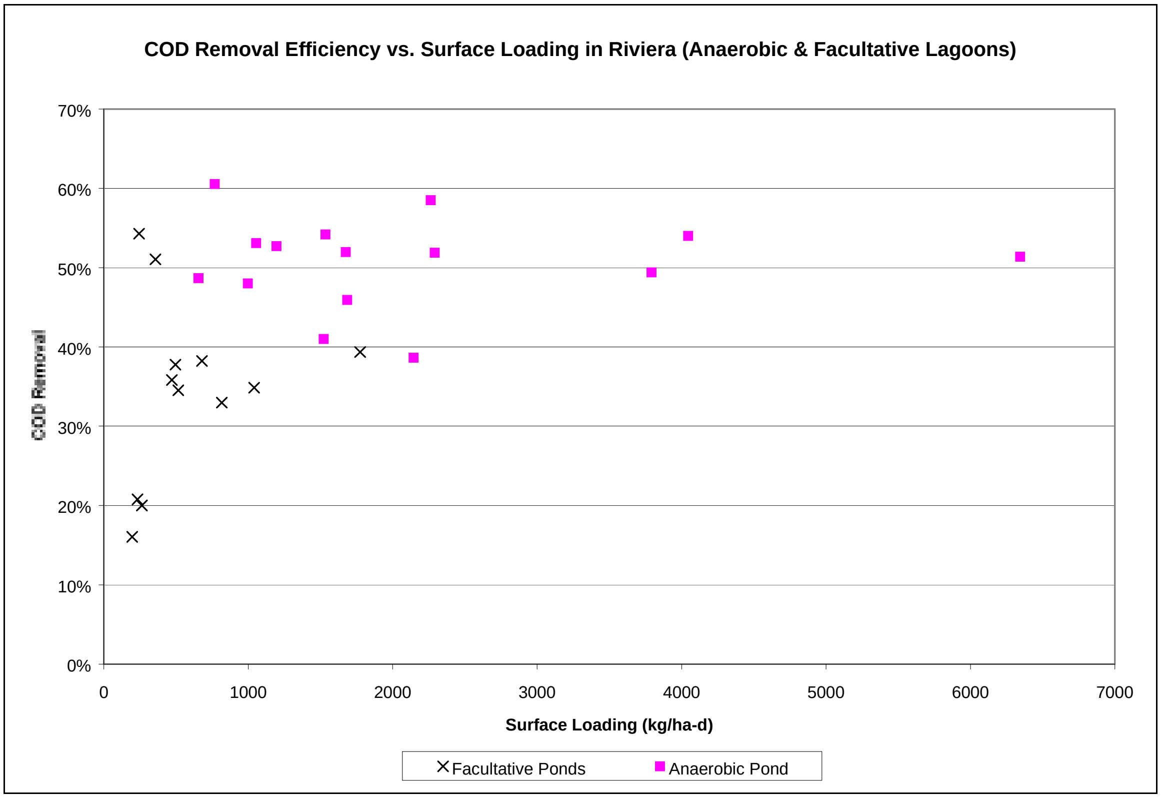

Figure 3-3: Removal Efficiency vs. Surface Loading, Riviera de São Lorenço

Due to the very low loading of the anaerobic pond, Figure 3-3 presented the anaerobic pond loading on a surface area basis. During certain periods of very low loading, the anaerobic pond might act as a facultative pond. It is observed that the anaerobic pond performs much better than the facultative pond under the same surface loading. However, the anaerobic pond is twice as deep than the facultative ponds (3 m vs 1.5 m). This enables the anaerobic pond to have a much deeper anaerobic layer when it acts as a facultative pond, thereby increasing efficiency.

Figure 3-4 presents the anaerobic pond removal efficiency as compared to volumetric loading. It has been shown in the previous chapter that anaerobic pond loading is best measured on a volumetric basis and not a surface basis.

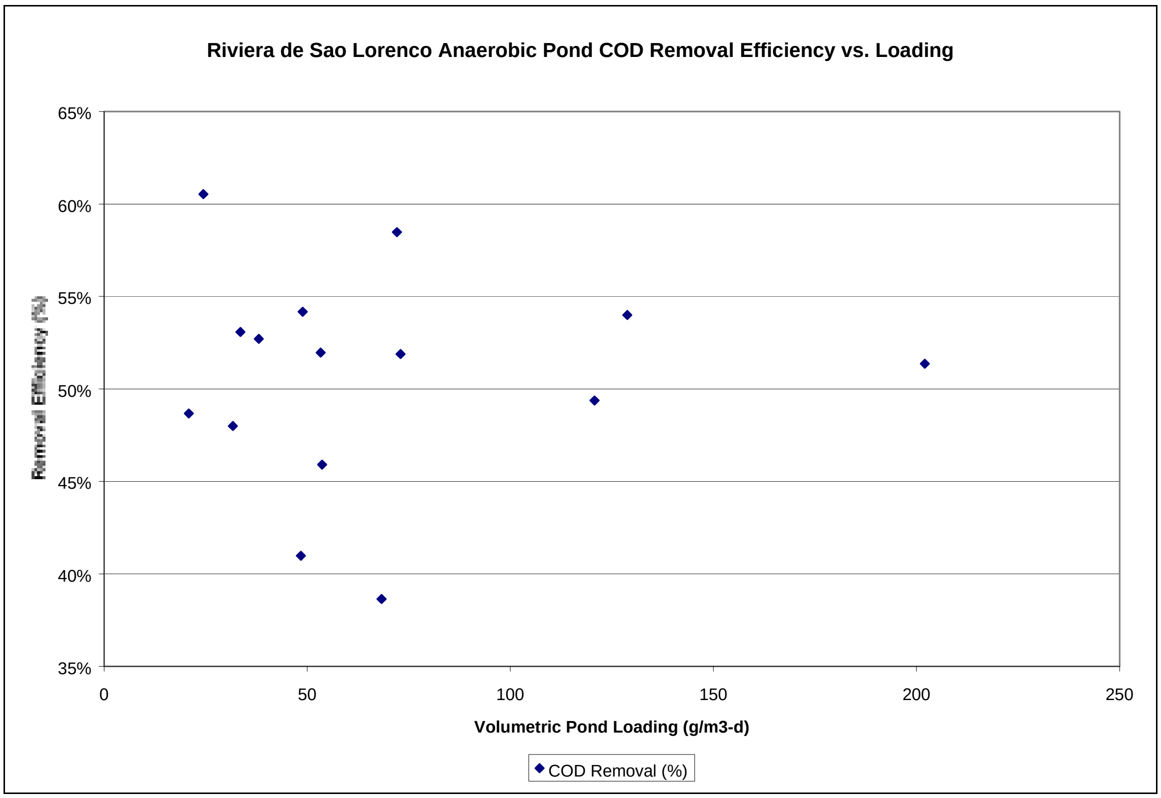

Figure 3-4: COD Removal Efficiency vs. Volumetric Loading, Riviera Anaerobic Pond

Figure 3-4 exhibits quite a scatter of removal efficiencies. No clear rule can be drawn as to the relation between loading and removal efficiency. The mean COD removal efficiency is 50.7%, and the standard deviation about that mean is of 5.8 percentage points. Although the literature cites 100 [g/m^3-d] as the minimal loading for an anaerobic pond to achieve a fully anaerobic state, the data indicates that the present anaerobic pond achieves quite a regular removal over a range of 25 – 200 [g/m^3-d]. The implications of this are quite interesting. Indeed, if all anaerobic ponds behave similarly, this would imply that an anaerobic pond could be designed to have a long lifetime, being able to cope with increased loading. It also implies that the minimum of 100 [g/m^3-d] rule can be foregone.

Figure 3-4 presents the anaerobic pond COD removal that is not lagged by the appropriate hydraulic retention time. The removal efficiencies lagged by the retention time are presented in Figure 3-5. The average of the removal efficiencies is 45% and the standard deviation is 16%. These statistics are biased, however, by some negative removal efficiencies, which are remnants of the technique used to lag the effluent COD data by the lag time. Indeed, lag times were calculated on a weekly basis (i.e. related to weekly average flow), and this might have responsible for negative removals.

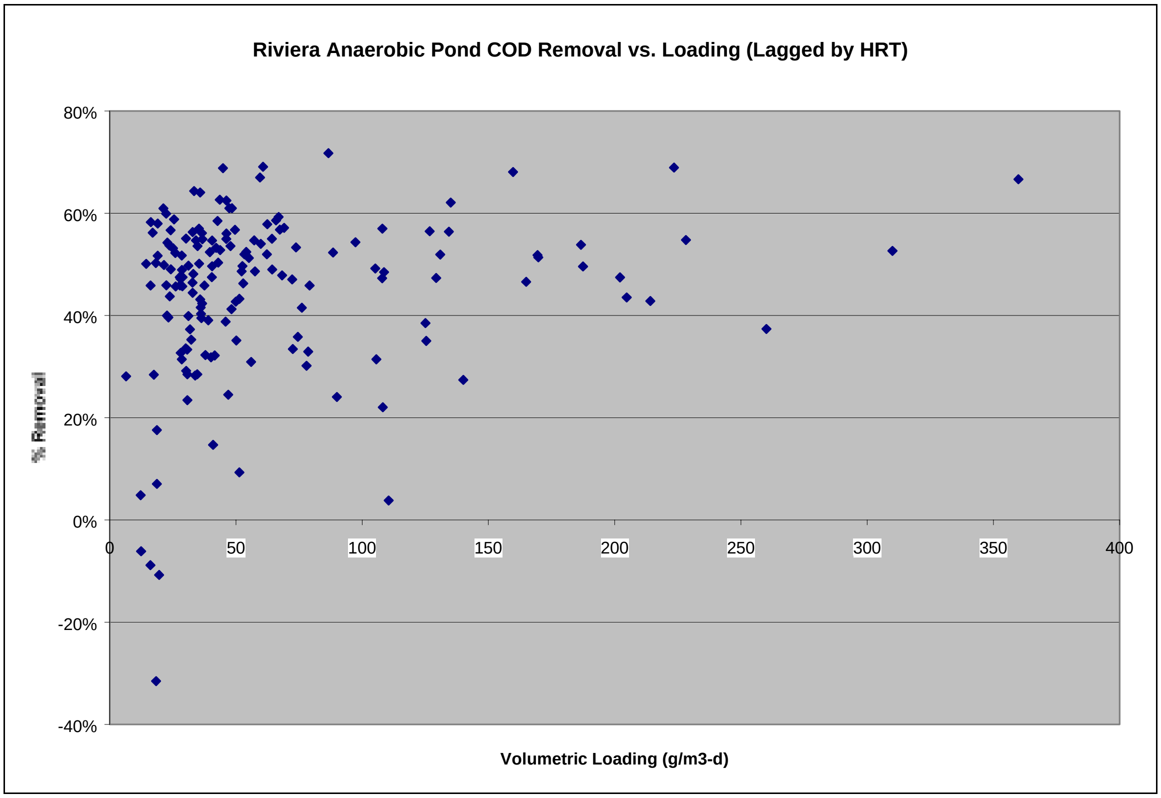

Figure 3-5: Anaerobic Pond COD Removal (Effluent Lagged by HRT)

Organic Loading Data

Organic loading is measured in terms of concentration of COD and BOD_5. On the days where data was missing, artificial data was generated by linearly interpolating between two known points. In most cases data was missing for one to three consecutive days. It is thought that interpolation is acceptable to fill in data for such a small duration.

Inorganic Loading Data

Inorganic loading is necessary for the Ferrara model in terms of inorganic carbon concentration and carbon dioxide concentration. None of these data were available from

Riviera. Indeed, these types of readings are very rarely done in simple WWTPs such as Riviera. Data points were therefore artificially created to satisfy the model's needs.

Inflow and Outflow Rates

The flow data available from Riviera presented two problems. First, there were some days during which no data was available. Second, flow rates were only available into and out of the whole treatment system. There were no flow rates available for the respective lagoons.

On the days where flow data was unavailable, points were created by linearly interpolating between two know points. For the flow rates to and from respective ponds, the following scheme was developed. Since all inflow enters the anaerobic lagoon, and the outflow from the anaerobic lagoon is directed to three facultative ponds set in parallel, the only data point missing is the flow from the anaerobic pond to the facultative system. Infiltration and evaporation influence the change in flow between lagoons. Because both infiltration (seepage) and evaporation can be related to the surface area of the lagoons, and due to the fact that the anaerobic lagoon occupies approximately one third of the surface area that the facultative ponds occupy (on a use-weighted basis for the time period), flow rates between the anaerobic pond and the facultative pond were interpolated one fourth of the way between the inflow and outflow of the whole system.

Lagoon Temperature Modeling

Lagoon temperature is not monitored at all at the Riviera WWTP. It was therefore necessary to generate temperature data for the lagoons using meteorological data from

Santos, a city that lies 50 kilometers south of Riviera. These meteorological data were obtained from a database maintained by Columbia University, and accessible through the web at http://ingrid.ldgo.columbia.edu/SOURCES. The following paragraphs will describe the temperature modeling procedure.

The temperature model is based upon a very simple heat balance for the water body. This heat balance for a completely-mixed system is expressed in Equation 3-5.

Accumulation = inflow – outflow ± surface heat exchange **(3-5)**

The term labeled "inflow" represents the heat entering through the inlet stream. Accordingly, the term labeled "outflow" represents the heat lost through the pond outlet. The last term, "surface heat exchange" represents the heat gained, or lost, through the air-water interface of the pond. It should be noted that this model does not take the energy exchange with sediments into account. The latter can be quite significant in shallow systems such as lagoons.

The "inflow" and "outflow" terms are described by Equations 3-6 and 3-7.

$$\text{Inflow} = Q*\rho*C_p*T_{in}(t) \qquad \textbf{(3-6)}$$

$$\text{Outflow} = Q*\rho*C_p*T \qquad \textbf{(3-7)}$$

Where: Q = Flow rate of water coming in the pond or leaving it

ρ = Density of the water

C_p = Heat capacity of water

T = Temperature of water (as function of time for influent temperature)

It should be noted that T_{in}, the temperature of the pond influent, was unavailable. For modeling purposes, this temperature was assumed to be constant at a value of 25°C (refer to the sensitivity analysis of the pond influent temperature, for a more detailed discussion of the ramifications of this assumption).

The surface heat exchange term is a combination of five processes. Figure 3-1 presents a schema of all processes involved in surface heat exchange. These processes, as seen in Figure 3-6, can be grouped in two different ways. First, we can distinguish the radiation versus non-radiation terms, and the second way to group them is to distinguish between terms that are dependent of the water body temperature or not.

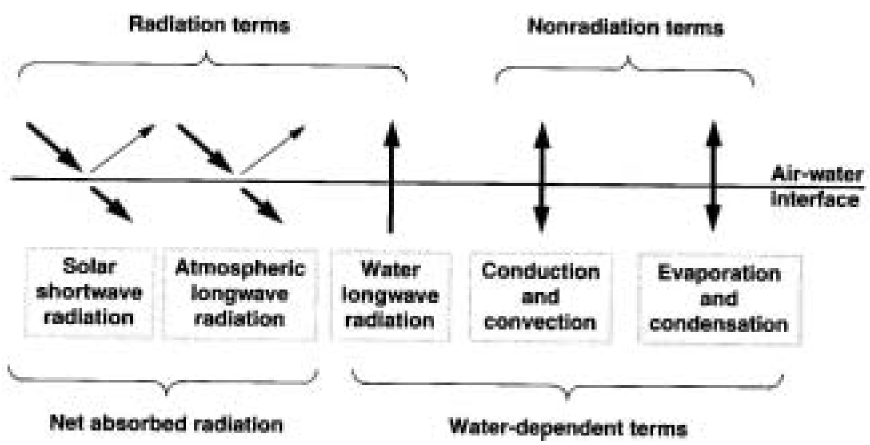

Figure 3-6: Schema of Surface Heat Exchange Processes (Chapra, 1997)

The net surface heat exchange can be represented as

$$J = J_{sn} + J_{an} - (J_{br} + J_c + J_e) \quad \textbf{(3-8)}$$

where: J_{sn} = net solar shortwave radiation

J_{an} = net atmospheric longwave radiation

J_{br} = longwave back radiation from the water

J_c = conduction

J_e = evaporation

The net shortwave solar radiation is taken from the meteorological data. In the present case, the closest available weather station that had a good historical record of meteorological data was Santos, which is located approximately 50 kilometers south of Riviera de São Lorenço. The rest of the terms from Equation 3-8 can be calculated from other data, such as wind speed, dry bulb and wet bulb temperatures. It should be noted that the three latter terms are a function of the pond surface temperature, which is in our case the unknown. Equations 3-9 through 3-12 represent the terms involved in the surface heat exchange. The atmospheric longwave radiation is expressed as

$$J_{an} = \sigma*(T_{air}+273)^4*(A+0.031\sqrt{e_{air}})*(1-R_L) \quad \textbf{(3-9)}$$

(Stefan-Bolzmann Law) (Atmospheric attenuation) (Reflection)

where: σ = the Stefan-Bolzmann constant ($11.7*10^{-8}$ cal $(cm^2\ d\ K^4)^{-1}$)

T_{air} = Air temperature (^oC)

A = a coefficient (0.5 to 0.7)

e_{air} = air vapor pressure (mmHg)

R_L = reflection coefficient (0.03)

The water longwave radiation term is expressed as

$$J_{br} = \varepsilon\,\sigma * (T_s + 273)^4 \qquad \textbf{(3-10)}$$

where: ε = emissivity of water (0.97)

T_s = water surface temperature

The conductive heat transfer is expressed as

$$Jc = c_1 * f(U_w) * (T_s - T_{air}) \qquad \textbf{(3-11)}$$

where: c_1 = Bowen's coefficient ($\approx$ 0.47 mmHg $^oC^{-1}$)

$f(U_w)$ = dependence of heat transfer on wind velocity = $19 + 0.95 * U_w^2$

U_w = wind speed as measured at a height of 7m above water surface (ms^{-1})

The evaporative heat loss can be expressed as

$$J_e = f(U_w) * (e_s - e_{air}) \qquad \textbf{(3-12)}$$

where: e_s = saturation vapor pressure at water surface

e_{air} = vapor pressure of overlying air

The saturation and air vapor pressures can be calculated from the surface water temperature and dry bulb temperature respectively as

$$e = 4.596 * e^{\,(17.27*T / 237.3 + T)} \qquad \textbf{(3-13)}$$

The Lagoon Temperature Model

The Ferrara Model used to dynamically predict lagoon effluent quality assumes that the lagoon is hydraulically fully mixed, and therefore, this assumption will remain for the temperature modeling. Consequently, the water surface temperature term that was included in the equations in the previous section is analogous to the lagoon temperature. Also, the data acquired from the web-based database was daily averaged data (a part from the net solar radiation, which was averaged monthly). Thus, the temperature was modeled as a daily steady-state phenomenon. This assumption of steady-state implies that the "J" term on the left-hand-side of Equation 3-8 is set to zero. The remaining equation was numerically solved to find the lagoon temperature. It should be noted that, as mentioned before, there was no available data for influent temperature (needed for equation 3-6), and it was therefore assumed to remain constant at a value of 25^{o}C. A sensitivity analysis of the resulting lagoon temperature with respect to influent temperature will be provided later on.

The model results are seen in Figure 3-7.

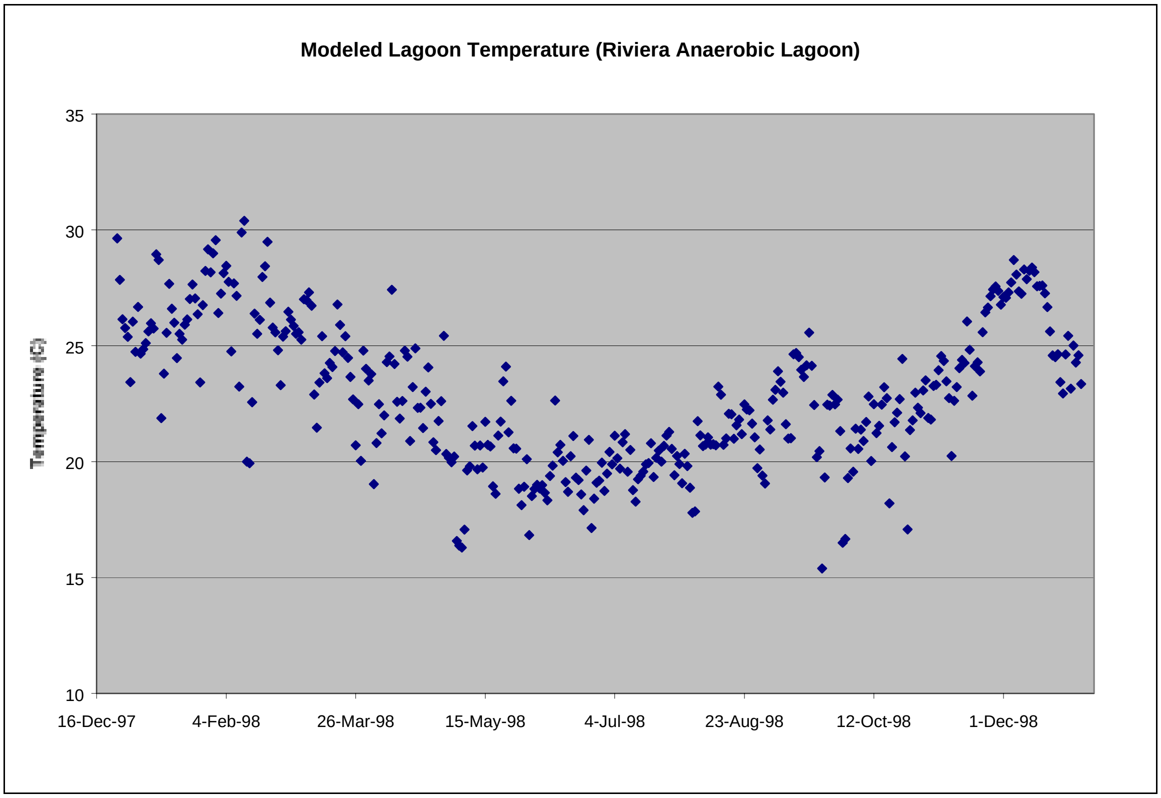

Figure 3-7: Modeled Temperature of Riviera Anaerobic Lagoon

The modeled temperature series has a mean of 22.9°C and a standard deviation of 3°C about the mean. The maximum-modeled temperature lies at 30.4°C (10th of February 1998), whereas the minimum-modeled temperature is 15.4°C (22nd September 1998).

No validation analysis can be done for the modeled temperature, as there is a complete absence of data about temperature of lagoons at Riviera de São Lorenço. This is a major short coming of the model as applied to Riviera. Indeed, it could be argued, and rightly so, that it is unscrupulous to model the lagoons at Riviera without any possible subsequent validation of the model. In defense of the approach taken, it could be said that the time constraints of the M.Eng. thesis are limiting, and therefore the scientific rigors of mathematical modeling should be relaxed for the purpose of this exercise.

The influent and resulting lagoon temperatures seem to concur with the little information available through personal communication with Dr. Ricardo Tsukamoto (via email, Monday March 8th 1999), an engineer who has had extensive experience at the Riviera site. Indeed, Dr. Tsukamoto quoted four influent and four lagoon temperature values for some days of 1994. The values are shown in Table 3-2.

Table 3-2: Riviera Temperature Data (Dr. Ricardo Tsukamoto, Monday March 8th 1999)

DATE	INFLUENT TEMPERATURE (^{O}C)	EFFLUENT TEMPERATURE (^{O}C)
01/24/94	26	28
02/28/94	27	28
03/07/94	26	26
04/04/94	26	26
05/02/94	26	26

Sensitivity Analysis for Influent Pond Temperature

In this section, the sensitivity of the temperature model will be examined as a function on the assumed influent temperature. The influent temperature will be varied between the values of 15 and 30 degrees Celcius, and be kept constant during the modeled period.

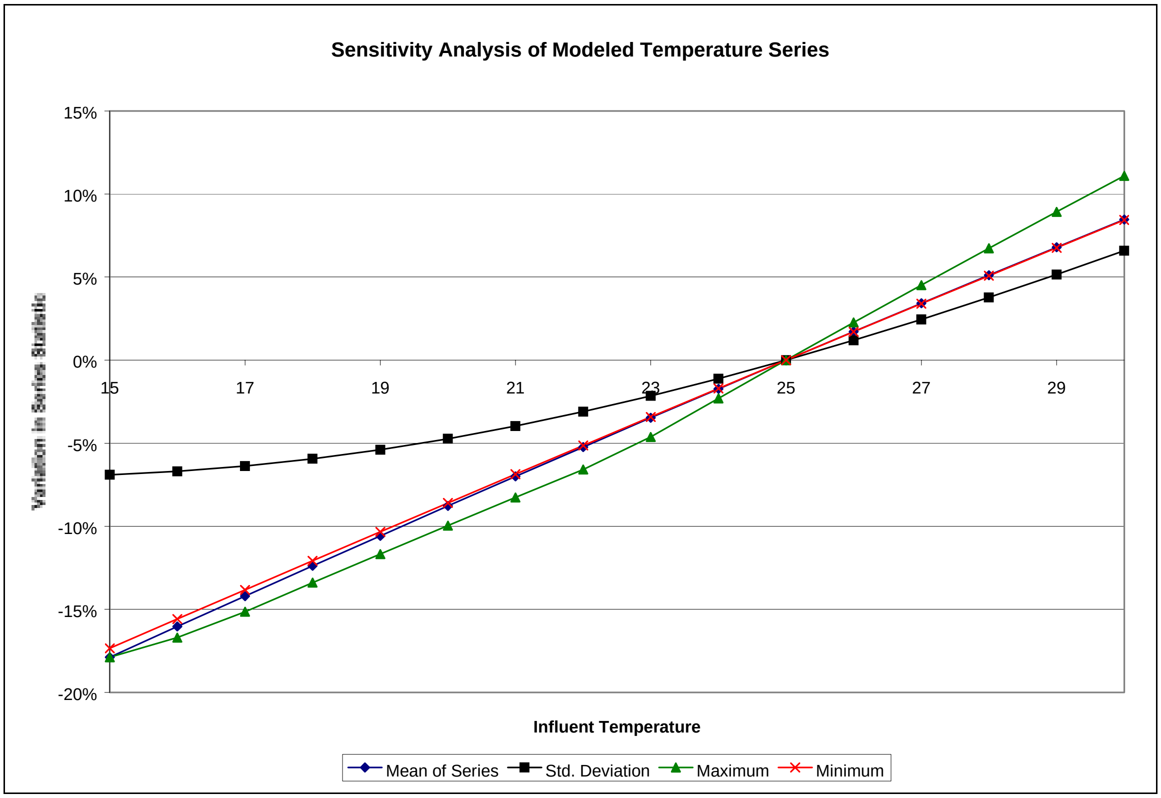

Figure 3-8: Plot of Sensitivity Analysis Results

The plot of the sensitivity analysis (Figure 3-8) shows that the mean, maximum value and minimum value of the series vary by about 10% when the influent temperature is increased or decreased by 25% ($5^{o}C$). The standard deviation of the series varies very little, by a maximum of 5%. This sensitivity analysis, from which we can conclude that the model is moderately sensitive to influent conditions, must be complemented by a sensitivity analysis of the lagoon model with respect to pond temperature. Should the pond model output vary a lot with temperature, then the choice of influent temperature in the pond temperature model is significant. The model fitting, discussed in the following section, will be undertaken using the temperature modeled with a constant $25^{o}C$ influent.

Model Fitting

The model fitting, or calibration process, was done by manual iteration. One of the four kinetic parameters is changed, and the resulting sum-of-squared errors is evaluated. That same parameter is changed until the sum-of-squared errors (SSQ) has reached a minimum. The next parameter is then varied, and the same SSQ minimization is achieved. The model fitting process is best described by Figure 3-9.

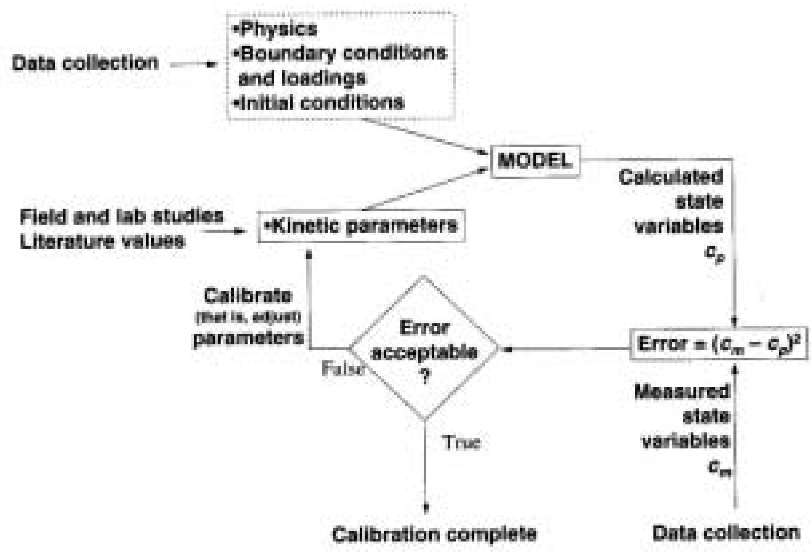

Figure 3-9: Schema of Model Calibration Procedure (Chapra, 1998)

In the iteration process, a second "goodness-of-fit" measure was used: R-square, comparing modeled and observed series. However, the sum-of-squared error was given the priority.

Given that the model fitting procedure was manual (i.e. change the parameter, and run the model), the risks that the fitted model parameters represent a local optimum and not a global one are great. The models presented in the following sections are the best models achieved given the time constraints.

The Riviera Anaerobic Pond Model

The modified Ferrara model applied to the Riviera de São Lorenço anaerobic lagoon data is presented in Figure 3-10. Visual inspection of the model reveals that the fit is rather good. The fitting involved 44 individual iterations of the model, and two iteration-sets for each parameter. This means that the parameters were optimized one-by-one, and when all four parameters had been optimized, the process was started again with the newly optimized values as starting points. Although the total number of iterations pales in comparison with the number of iterations that would have been accomplished had the fitting process been computerized, it is thought that the model achieved approaches the best-possible fit. Indeed, the fitted parameters are extremely close to the parameters for the Kilmicheal and Corinne ponds that were found by Ferrara in 1978. The fitting process tried to keep the parameters as close to the Ferrara parameters. This tends to prove that the modeling framework used is robust. Indeed, if the parameters for ponds in the United States are similar to those for ponds in Brazil, it tends to prove the validity of the model.

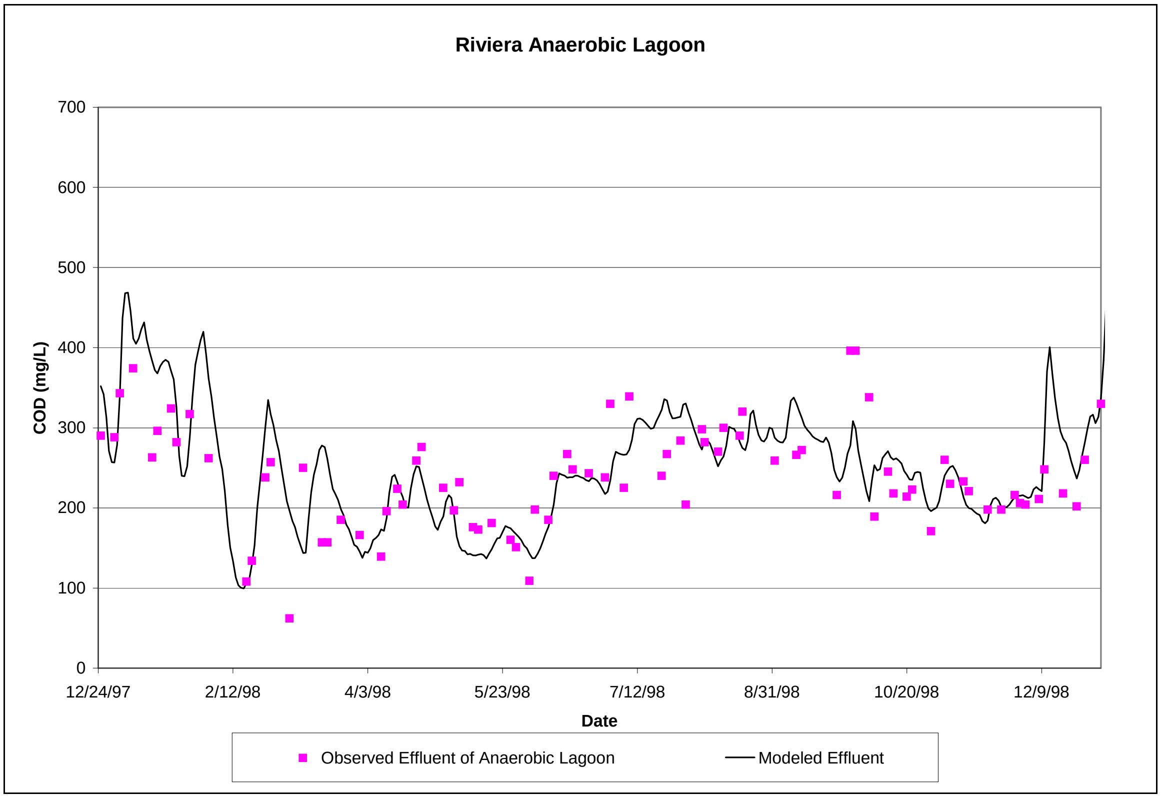

Figure 3-20: Riviera Anaerobic Lagoon Model

The final estimated parameters for the Riviera Anaerobic Lagoon Model are shown in Table 3-3. The Corinne Pond model parameter values were included for comparison purposes. These are the values that Raymond Ferrara had fit to the first facultative pond in Corinne (Utah) in 1978.

Table 3-3: Parameters for Riviera, Corinne & Kilmicheal Models (20^{o}C)

PARAMETER	Estimated Value for Riviera Anaerobic Lagoon	Values for First Facultative Pond in Kilmicheal, MI (Ferrara, 1978)	Values for First Facultative Pond in Corinne, Utah (Ferrara, 1978)
R12 *[day^{-1}]*	0.05	0.05	0.05

R21 *[day^{-1}]*	0.02	0.04	0.085
R1S *[day^{-1}]*	0.04	0.02	0.02
R20 *[day^{-1}/m depth]*	8.64	8.64	8.64

Model Sensitivity to Lagoon Temperature

The underlying assumption in the lagoon temperature model that the influent temperature is constant needs to be assessed, as to its consequence on the lagoon model. Figure 3-11 presents the modeled effluent curves for three different lagoon temperature time-series. The three lagoon temperature time-series are based on the assumptions of constant 20^{o}C, 25^{o}C and 30^{o}C influent.

The models based on the three different influent temperatures are very close to each other in Figure 3-6. It is concluded that the Riviera de São Lorenço Anaerobic Pond model is practically not influenced by variations in pond influent temperature, and therefore the assumption of constant pond influent temperature for the lagoon temperature model is validated. Indeed, the variations produced by an influent temperature change are not great, and since it is safe to assume that the temperature of the influent varies between 20 and 30^{o}C, an assumption of a constant 25^{o}C influent is acceptable.

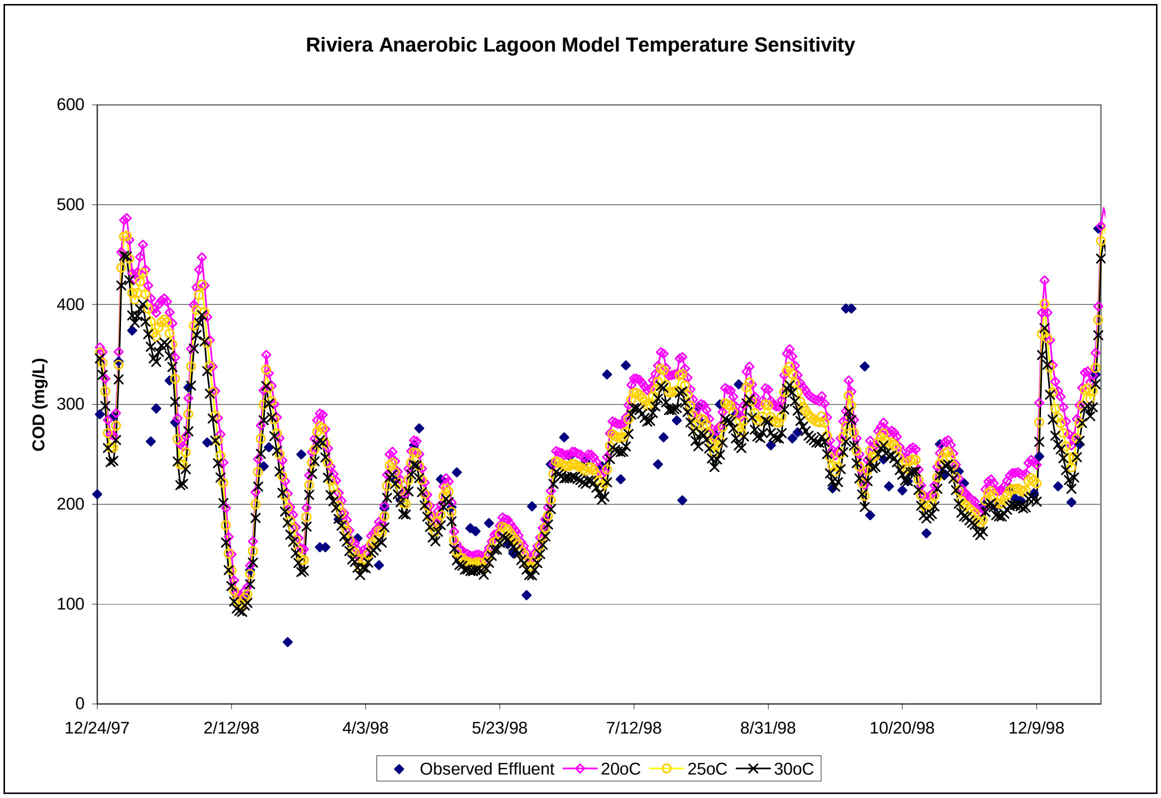

Figure 3-31: Riviera Anaerobic Lagoon Model Sensitivity to Lagoon Temperature

As previously stated, the anaerobic lagoon at Riviera de São Lorenço is very lightly loaded in terms of organics. It has been stipulated that this might lead the anaerobic lagoon to act as a facultative lagoon, with a aerobic layer on the top of the pond profile. The fitted model will therefore be tested on the facultative lagoons of Riviera by keeping all the parameters. The only parameter change will occur for R_{20}, which will be scaled for the different pond depth.

Modeling the Riviera Facultative Lagoons

Riviera de São Lorenço operates three facultative ponds arranged in parallel. For modeling purposes, it is proposed to model the three facultative lagoons as one lagoon.

Indeed, this will greatly simplify the task by ignoring the separation of flow between the three lagoons, for which no data is available. The temperature model is applied to the facultative lagoons and other missing data is interpolated as was done for the anaerobic lagoon. The model developed for the anaerobic pond in Riviera de São Lorenço will be used in a predictive mode on the “consolidated” facultative pond at Riviera.

Temperature Modeling for the Facultative Pond

To model the facultative pond temperature, the results from the anaerobic pond temperature model were used as influent temperatures. The facultative pond temperature model output is show in Figure 3-12.

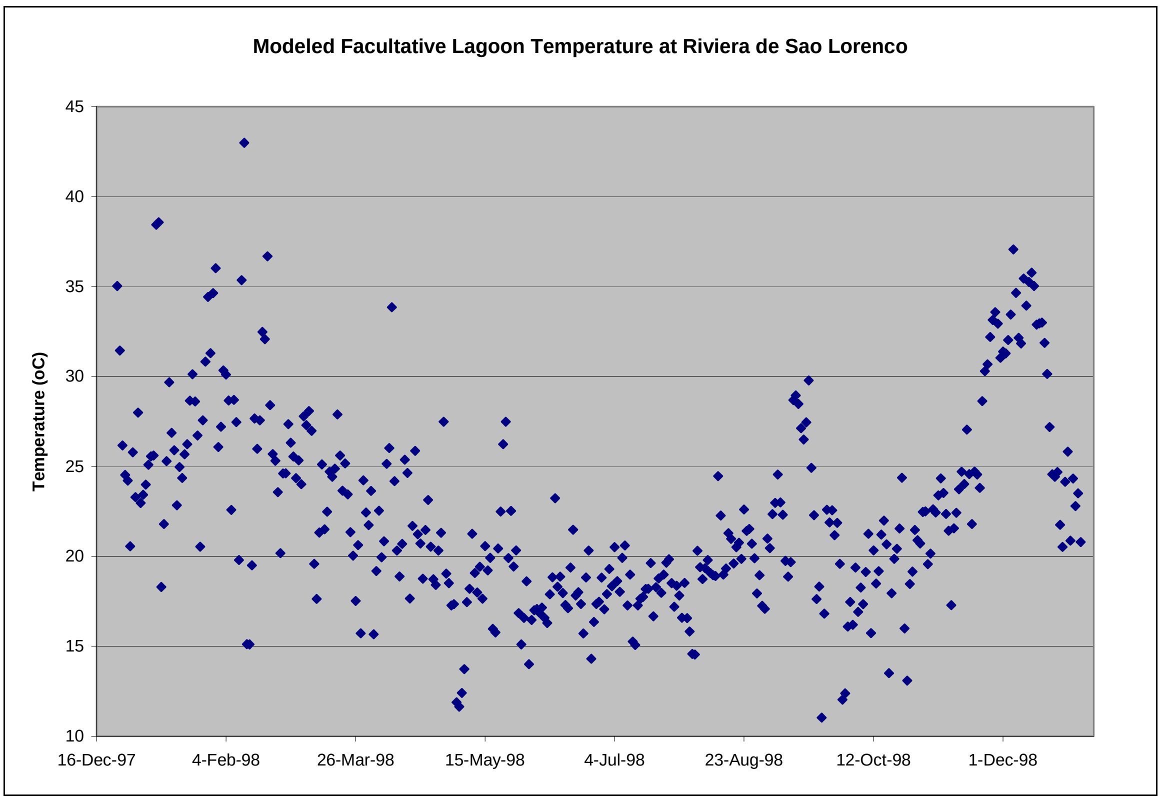

Figure 3-42: Modeled Temperatures for Facultative Lagoon at Riviera de São Lorenço

The modeled series of lagoon temperatures presented in Figure 3-12 has a mean of 22.5°C, a standard deviation about the mean of 5.45°C, a maximum value of 43°C, and a minimum value of 11°C. The range of the modeled facultative temperature series, which is of 32°C, is much greater that that of the modeled anaerobic temperature series, which is of 15°C. This increase in range, and in standard deviation is due to the fact that the influent temperature is not constant, as it was for the anaerobic pond temperature model. The plausible errors associated with the constant influent temperature have propagated onto the temperature model of the facultative pond. However, in the absence of any validating data, the modeled temperatures for the facultative pond shall be accepted, and used in the facultative pond model.

Modeling Riviera Facultative Ponds

The temperatures that were modeled for the facultative pond at Riviera de São Lorenço were input into the adapted version of the Ferrara model. The value for R_{20} was changed from 25.92 [day^{-1}] to 12.96 [day^{-1}], to account for the depth of the facultative lagoon, which is half of the depth of the anaerobic lagoon. Indeed, the rate of loss of inorganic carbon to the sediment layer is directly related to lagoon depth (Ferrara, 1978).

The modeled facultative ponds of Riviera de São Lorenço are presented in Figure 3-13, along with the observed effluent quality. The modeled series' basic statistics are close to those of the observed series. The mean of the modeled series is 139 [mg/L], and that of the observed series is 162 [mg/L], which represents a difference of 19%. The standard deviations of the modeled and observed series are 43.8 [mg/L] and 44 [mg/L] respectively. The model performs poorly at reproducing the shape of the observed series.

Indeed, the correlation coefficient between both series is a modest 0.34. Visual inspection of the modeled and observed series (see Figure 3-13) reveals that the modeled series is an "exaggerated" version of the observed series. However, the model does seem to capture the general essence of the observed series. Indeed, the averages of both series are somewhat close. The model therefore seems to perform well on a general scale without capturing the details involved.

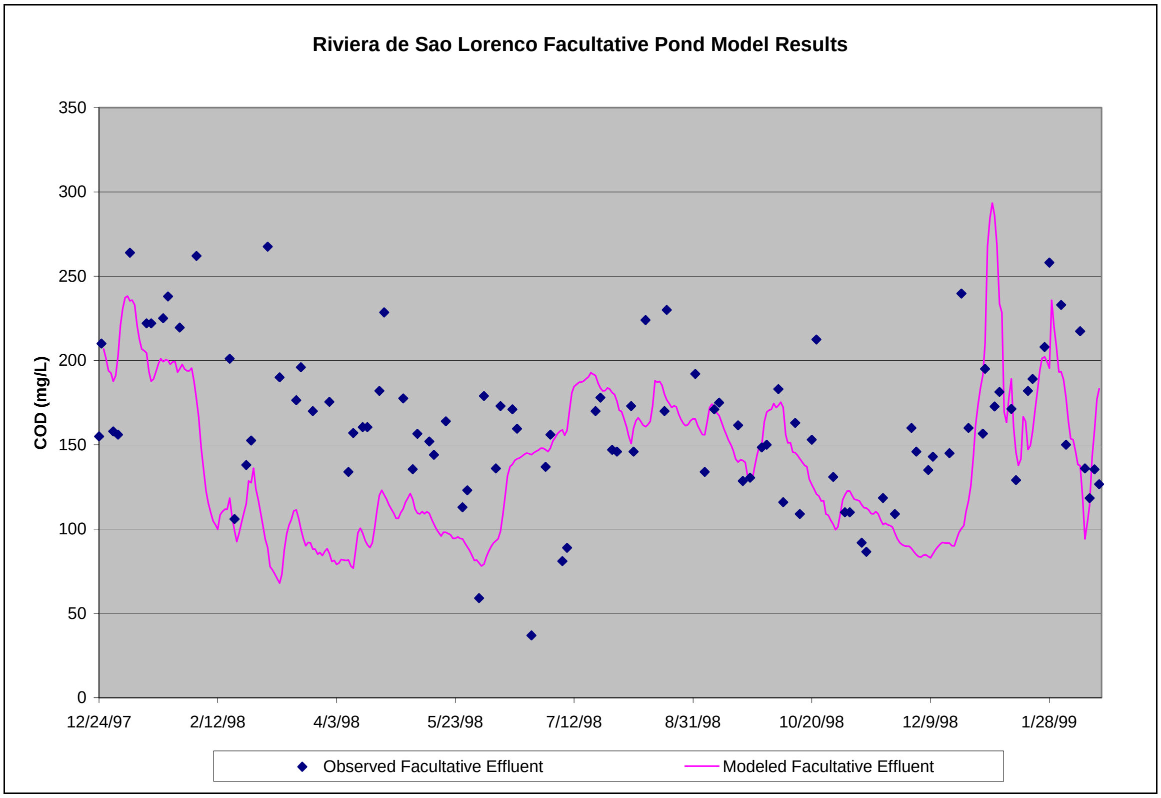

Figure 3-53: Modeled vs. Observed Facultative Pond Effluent at Riviera de São Lorenço

Both the anaerobic pond model and the facultative pond model are deemed suitable for use as design aids for the lagoons at Tatui. For comparison purposes, a model of an aerated lagoon followed by a sedimentation tank was developed upon data acquired from

Dr. Albert Pincince on the Amman (Jordan) wastewater treatment station. This model will serve to estimate the performance of the SABESP proposed design.

The Jordan Aerated Lagoon Model

This section presents a model for the aerated lagoons at the As-Samra wastewater treatment station in Amman, Jordan. As-Samra is the biggest waste-stabilization pond treatment station in the world, with 187 hectares of waste stabilization ponds designed to accommodate an average daily flow of 68,000 m^3/day. The facilities at As-Samra consist of three parallel tracks of anaerobic ponds, facultative ponds and maturation ponds in series. Figure 3-14 presents the schematic of the As-Samra treatment facilities.

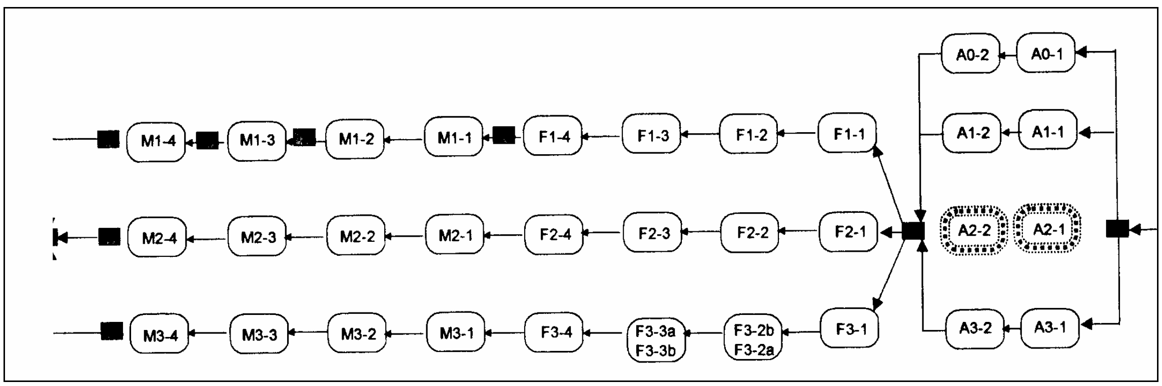

Figure 3-64: Treatment Scheme at As-Samra (Eller, 1998)

Figure 3-14 shows that there are four parallel tracks of two anaerobic ponds in series, followed by three tracks of four facultative ponds and four maturation ponds in series. The aerated lagoons are ponds M1-3 and M1-4, which are the two last maturation ponds on the first treatment track. The M1-3 maturation pond is fully-mixed and the M1-4 maturation pond is only partially mixed, to allow for settling (Eller, 1998).

The data on the As-Samra treatment system was given by Dr. Albert Pincince, a vice-president at Camp Dresser Mckee, a consulting firm that had been involved in the upgrading of the As-Samra treatment facility. It is said that it has been 10 years that the As-Samra facility was overloaded (Eller, 1998), with an average daily flow in excess of 150,000 m^3/day, as compared to the design 68,000 m^3/day.

The Data at As-Samra is presented in Figure 3-15, where daily COD values are plotted for the two-pond system (M1-3 and M1-4).

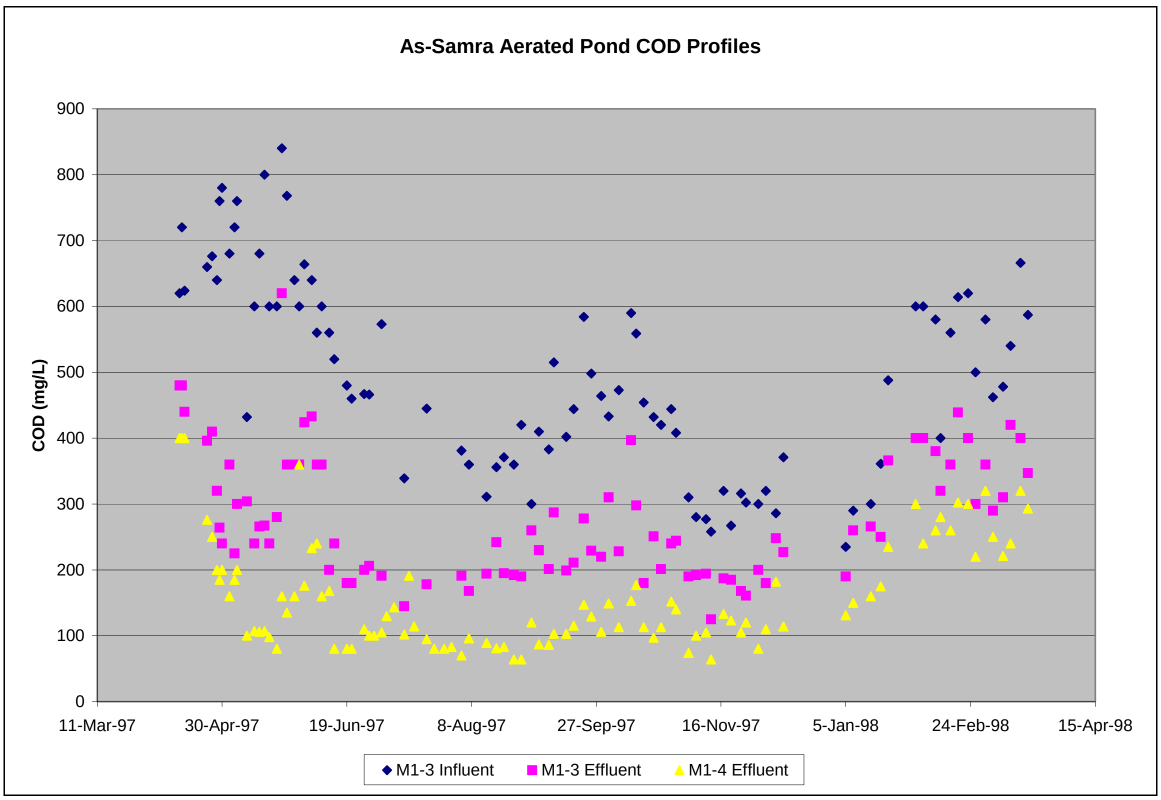

Figure 3-15: Daily COD Values for As-Samra Ponds M1-3 and M1-4

The influent to M1-3 has a mean of 498 [mg/L] of COD, and its standard deviation is quite high at 150 [mg/L]. The averaged COD removal efficiency of pond M1-3 is 44%;

that of pond M1-4 is 42.7%, and the overall COD removal efficiency of both ponds is 68%.

Figure 3-16 presents the COD loadings averaged on a monthly basis for both ponds.

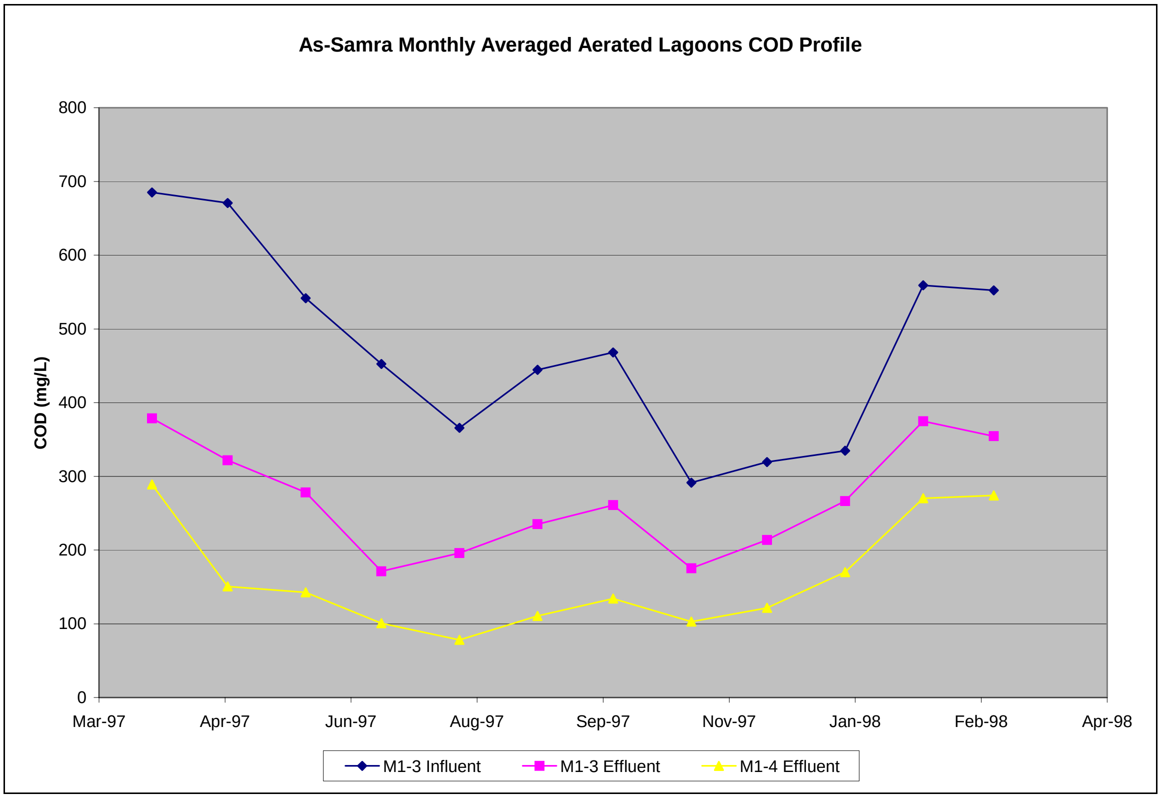

Figure 3-16: Monthly Averaged COD Profile for Aerated Lagoons at As-Samra

The effluent requirements are met 50% of the time in terms of BOD5, which is an effluent concentration of 50 [mg/L]. The colder winter and spring months did not achieve the effluent requirements.

The aerated maturation ponds have 46 aerators of 37.5 kW each. When the aerators were installed, the ponds had to be dug deeper to allow for an appropriate detention time; the resulting depth was of 2.85 meters, for a total pond volume of 153,350 m^3.

The modified Ferrara Model was applied to the combination of both M1-3 and M1-4 ponds. Indeed, both these ponds form a system. It could be argued that the assumption of fully-mixed flow is wrong for the combination of these two ponds. However, the model was fit using that assumption. It was thought that the implications of this error were not great enough to substantially affect the model. Moreover, by using the fully mixed assumption, it is implied that both ponds form one inextricable system; and they do: in the first pond, aeration occurs, and in the second pond the particulate organics settle. This can be viewed as one system with two steps.

The model was fit using the same procedure as outlined for the anaerobic pond model in Riviera de São Lorenço. The results of this model-fitting are presented in Figure 3-17.

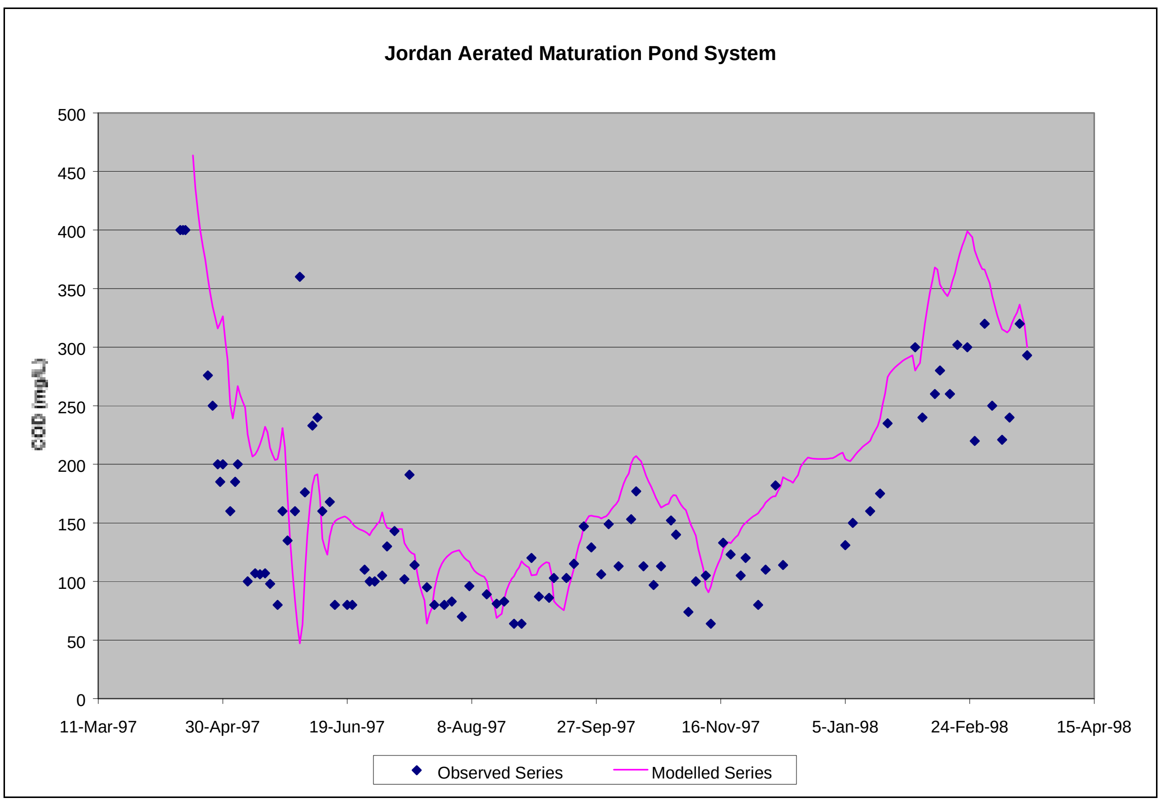

Figure 3-17: As-Samra Aerated Pond Model

The model performed quite well on the Jordan data. Indeed, the modeled series correlates quite well with the observed series, as can be seen in Figure 3-18.

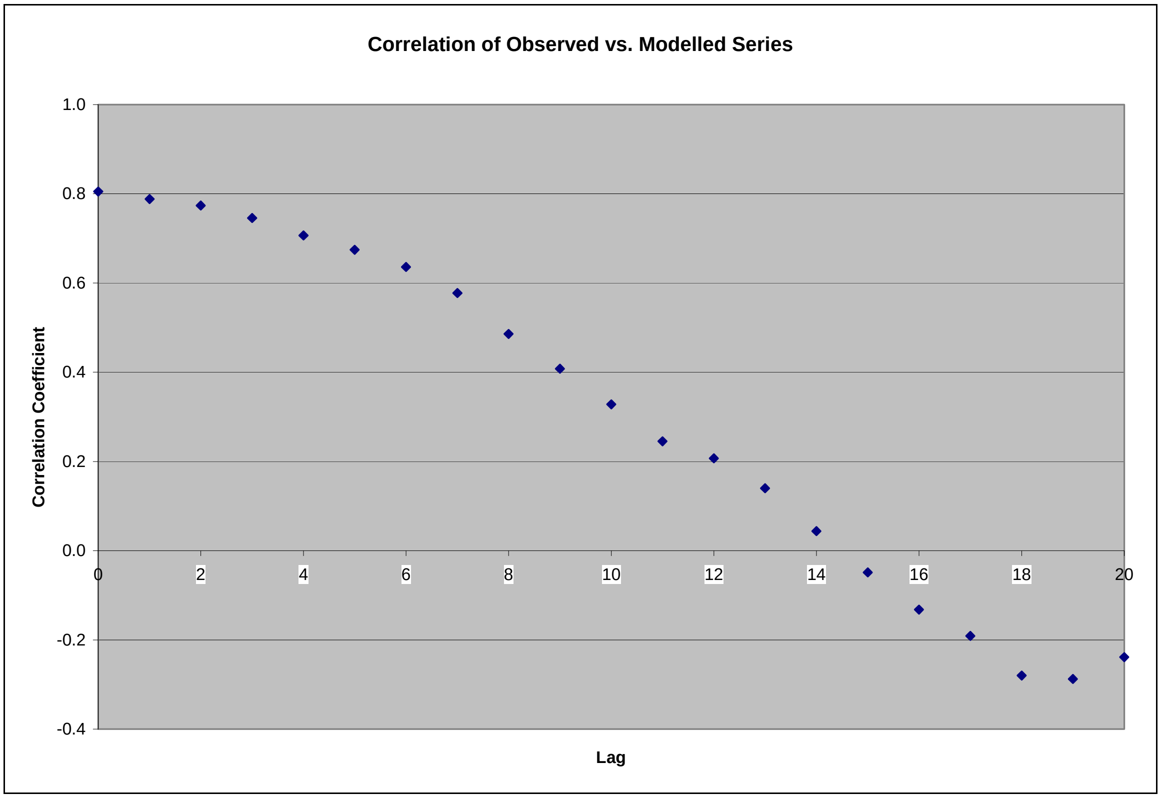

Figure 3-18: Correlation between Modeled and Observed Series of COD Effluent at As-Samra

The highest correlation is observed at lag-zero, which means that the modeled effluent is "synchronized" with the observed effluent quality. The lag-zero correlation coefficient is quite high at 0.8. Moreover, the "r-squared" value comparing modeled and observed series is 0.65, which is much higher that the 0.15 value exhibited for the Riviera de São Lorenço anaerobic pond model. The "r-squared" value is a measure of the linearity of the relation between the two compared series. In other words, if the modeled values were plotted against their respected observed values, the resulting graph would be a straight line if the "r-squared" value were one.

The modeled effluent COD converts to a removal efficiency of 59.3% for the two ponds, whereas the observed effluent represents a removal efficiency of 64.2%. These two values are quite close, and indicate that the model performed quite satisfactorily.

The fitted-mode parameter values are exhibited in Table 3-4.

Table 3-4: Model Parameter Values for As-Samra Model

PARAMETER	AS-SAMRA MODEL	RIVIERA MODEL	CORINNE POND MODEL
R12 *[day^{-1}]*	0.01	0.05	0.05
R21 *[day^{-1}]*	0.07	0.02	0.085
R1S *[day^{-1}]*	0.16	0.04	0.02
R20 *[day^{-1}/m depth]*	23.86	8.64	8.64

It is interesting to compare all three treatment stations just by observing the parameters for the modified Ferrara model. Indeed, the Riviera and Corinne models have the greatest transformation rate from organic to inorganic carbon (R12), whereas the As-Samra model has the smallest. This would tend to indicate that there is a greater digestion rate of organic material at Riviera. This makes sense, since the Riviera model is for a lightly loaded anaerobic pond, where the processes in the lower layer of the pond are all about organic material digestion. On the other hand, the As-Samra pond model takes its organic material removal in the R1S parameter, which represents net loss of organic material. The

R1S parameter for the As-Samra model is eight times higher than that of the Corinne pond model, and 4 times higher than that of the Riviera model. The As-Samra model represents a combination of two ponds, the first one of which is fully mixed, which means that there is absolutely no settling. The second pond of the As-Samra model is only partially mixed to allow for settling. The goal of aerated lagoons is to transform the soluble organic material to particulate organic material, and then to allow it to settle out.

In conclusion, the model worked very well on the As-Samra data. Both the Riviera pond model and the As-Samra pond model will be used in a predictive mode to assess the efficiencies of the proposed designed lagoons to follow the chemically enhanced primary treatment stage at Tatui, and the proposed SABESP aerated lagoon design for Tatui.

Assessing Effluent Quality from Design Situations

The two proposed designs (CEPT & lagoons, Aerated lagoons) for the upgrading of the CEAGESP treatment facility in Tatui will be assessed in this section. The method used to assess these designs will be to apply the models that were developed in the previous sections to the average influent conditions.

The average influent conditions are outlined in Table 3-5. The models will be applied to these inflow conditions, and the predicted efficiency of the designs will be deducted from the results of the simulations.

Table 3-5: Average Influent Characteristics

Design	Lagoons Following CEPT	SABESP Aerated Lagoons
Average Influent COD *[mg/L]*	250	500
Average Flow *[L/s]*	161	161

The simulation results for the SABESP aerated lagoons and sedimentation tank system are presented in Figure 3-19.

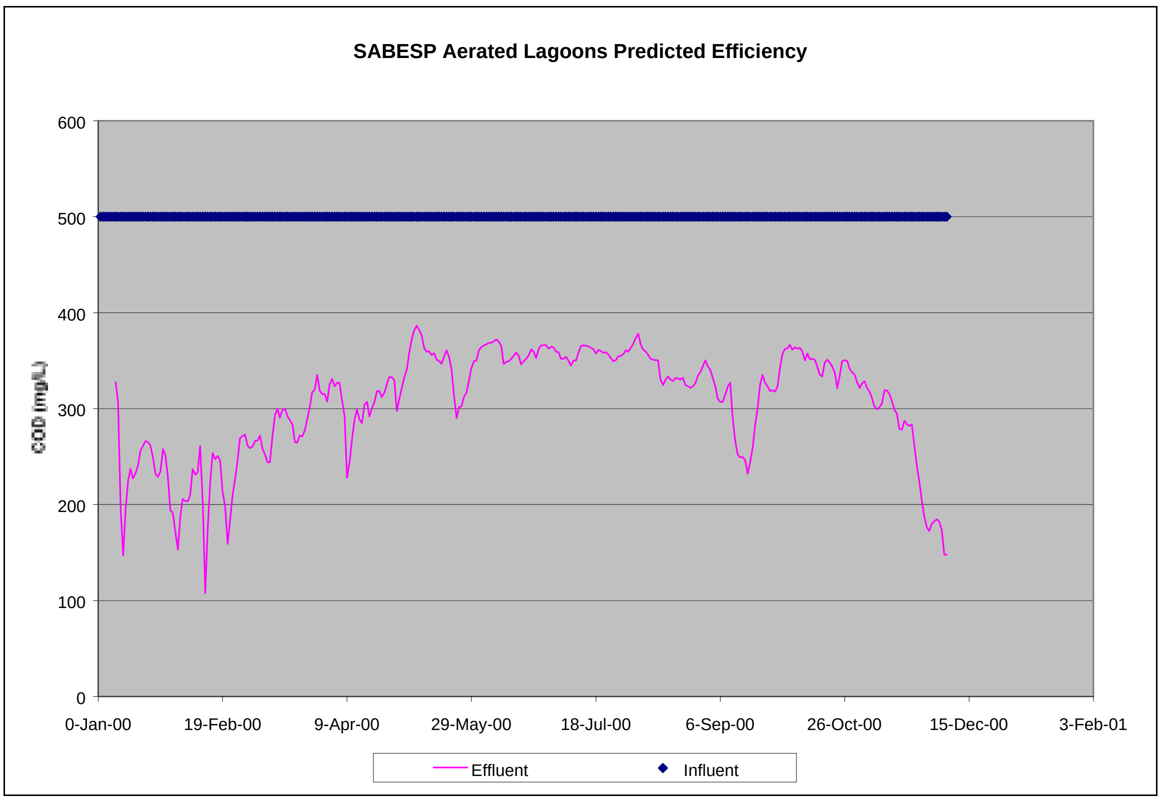

Figure 3-79: Modeled SABESP Aerated Lagoons & Settling Tanks

The average removal efficiency found from the As-Samra model, as applied to the SABESP aerated lagoon design, is 38.2%. This is quite contradictory with the estimated removal efficiency of 95% predicted for the SABESP design. However, the results of this modeling exercise are not to be taken into consideration, since no attention was given to the power of the aerators, nor the relative size of the settling tank to the aeration lagoon. Much more effort has to be given to the aerated lagoon model before it can be used in a predictive mode. The use of it in this section only serves to underline its imperfection.

To calibrate the model in terms of aeration power, it would be necessary to determine which of the parameters is affected by the amount of aeration injected into the lagoon. This could be done by studying one system in depth, observing the variations in effluent quality with changes in aeration intensity. Moreover, separate models should be developed for the aeration lagoon and the settling tank. Although they can be accepted as a system, it is better to separate them, in order to isolate the processes that take place in the lagoon and in the tank.

The next steps to be taken for the Jordan model are to model the lagoon as a anaerobic pond with no aeration (the average yearly volumetric loading to the first maturation pond at As-Samra is 97.5 g/m^3-d), and compare the results of this model with those of the aeration pond model (for the first pond model), and thereby deduce the added efficiency achieved by aeration. This would permit a crude analysis of the added benefit of aeration, and possibly could allow for a crude estimate of the aeration coefficient to be included in the model. Again, it has to be stressed that lack of time and lack of data are the culprits for the presentation of such a summary study.

The next step is the application of the Riviera model to the lagoons that have been designed to follow the CEPT stage in the proposed design for the CEAGESP treatment plant in Tatui. Unlike the previous case, this model was fitted on data from a location 160 km east of where it shall be used in a predictive mode. The model was fitted on a facility of approximately the same size as the one it shall be used on in a predictive mode, and both treat municipal wastewater excusively.

The results of the simulation on the lagoon system proposed to follow the CEPT stage in Tatui is presented in Figure 3-20.

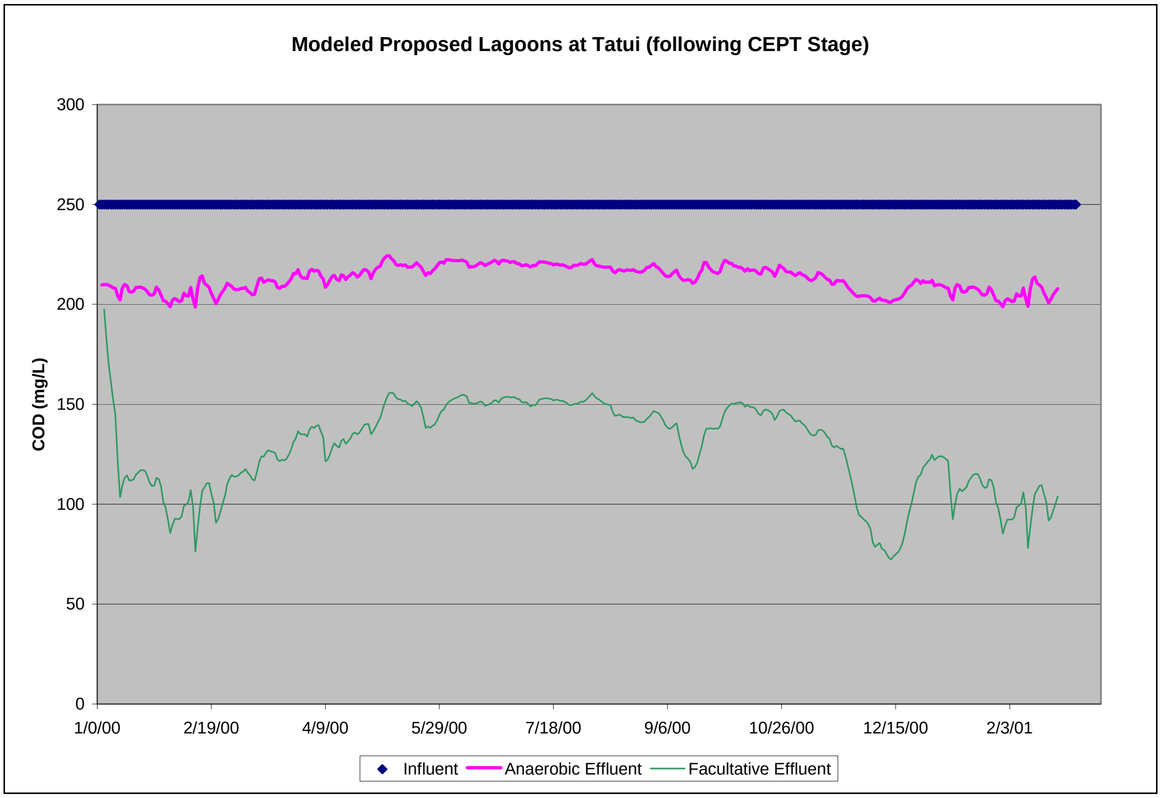

Figure 3-80: Modeled Efficiency of Proposed Lagoons to follow CEPT stage

The model exhibits an average COD removal of 14.85% for the anaerobic lagoon, and an efficiency of 40% for the facultative lagoon. The overall efficiency of the modeled system is 48.75%. These figures are much lower than the predicted values that had been taken from the literature. Indeed, the literature prompted the use of a total efficiency of 75%, which was on the conservative side.

It should be noted that the major difference between the Riviera ponds and those designed to follow the CEPT stage at Tatui is in the hydraulic retention time (HRT). Indeed, the yearly averaged HRT for the anaerobic pond in Riviera is 7.5 days, whereas that of the designed anaerobic pond in Tatui is of 1.9 days. The Facultative ponds at Riviera have a yearly averaged detention time of 22.5 days, whereas the proposed facultative ponds for Tatui have a detention time of 7.1 days. There is a factor of about three between detention times of the Riviera and Tatui lagoons. However, space constraints in Tatui limited the designed lagoons to their present design retention times.

The population served by the wastewater treatment facilities in Tatui and Riviera are both around 50,000. However, this population represents the peak loading for Riviera, a loading that is only experienced in the peak months of January and December. It is during these two months that the treatment plants at Riviera and Tatui are comparable. The estimated organic loading of the Tatui treatment station is of 135 g/m^3-d of COD, whereas the peak loading for Riviera lies around 120~200 g/m^3-d of COD. Table 3-6 presents the monthly loading and COD removal efficiency for the anaerobic pond at Riviera.

Table 3-6: Peak Loading and COD Removal at Riviera Anaerobic Pond

MONTH	Loading [g/m3-d]	Detention Time [d]	COD Removal [%]
December 97	129	4	54
January 98	121	4	49.4
December 98	202	3	51.4

Table 3-6 presents loadings and detention times that are in the range of those expected at Tatui. Indeed, the average expected loading at Tatui is 135 g/m3-d, and the average detention time of the designed anaerobic lagoon at Tatui is 2 days. It is therefore expected that the Tatui anaerobic pond will achieve 50% removal of COD, and not 15% as predicted by the Riviera model. The cause for the shortcoming of the Riviera model lies in the seasonal variations in loading of the Riviera treatment system. The model was fit for the entire year, and not the peak months only. A new model for the peak season is presented next, and the predictions for the Tatui pond associated with this new model are also shown. The model predicts an average COD removal for the peak season (December and January) at Riviera of 44.15%. The observed COD removal during the peak season is 45.25%. the model parameters are seen in table 3-7.

Table 3-7: Riviera Anaerobic Pond Peak-Season Model Parameters

PARAMETER	Riviera Peak-Season Model	Riviera Year-Round Model
R12 [day-1]	0.11	0.05
R21 [day-1]	0.02	0.02
R1S [day-1]	0.04	0.04
R20 [day-1]	8.64	8.64

This model was achieved after 12 iterations, starting with the parameters from the year-round model. The model results are shown in Figure 3-21.

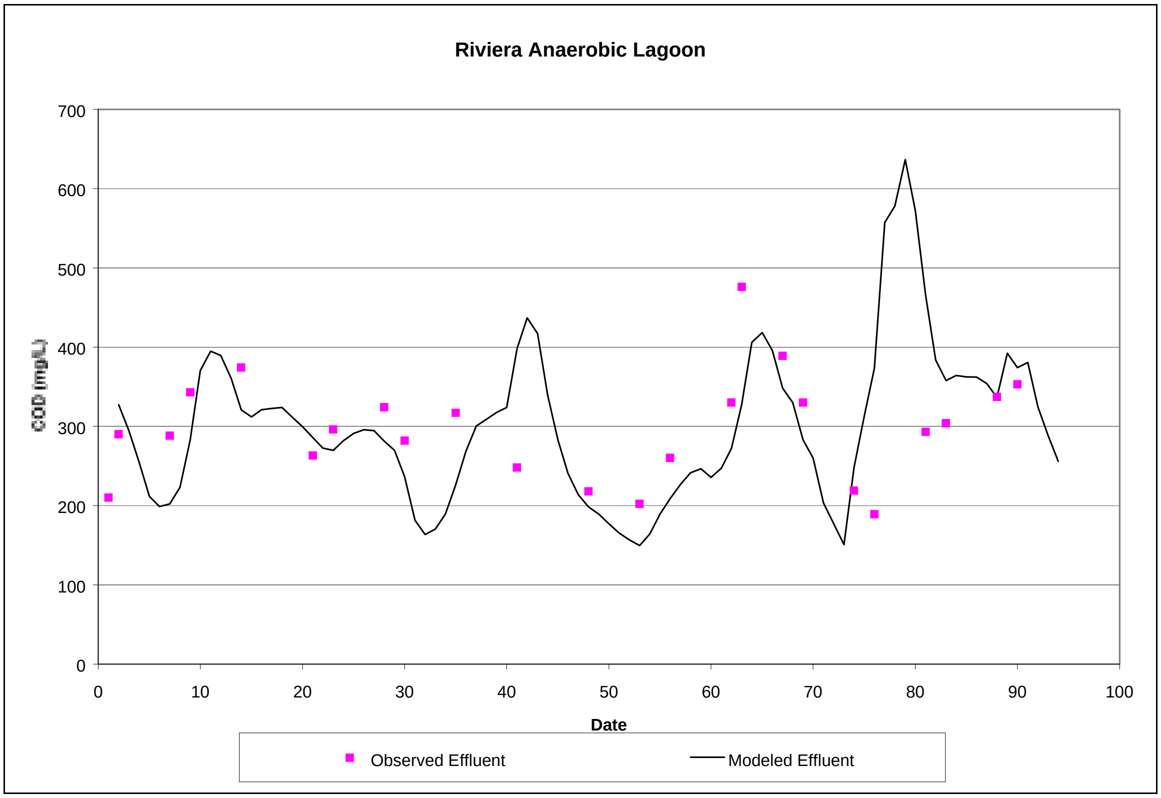

Figure 3-21: Riviera Anaerobic Pond "Peak-Season" model

This "Peak-Season" model was used in a predictive mode for the proposed anaerobic lagoon at Tatui. Indeed, the loadings experienced during the peak-season at Riviera are similar to the year-round loading that will be experienced in Tatui. The resulting predicted efficiency is 25.5%. The modeled results are presented in Figure 3-22.

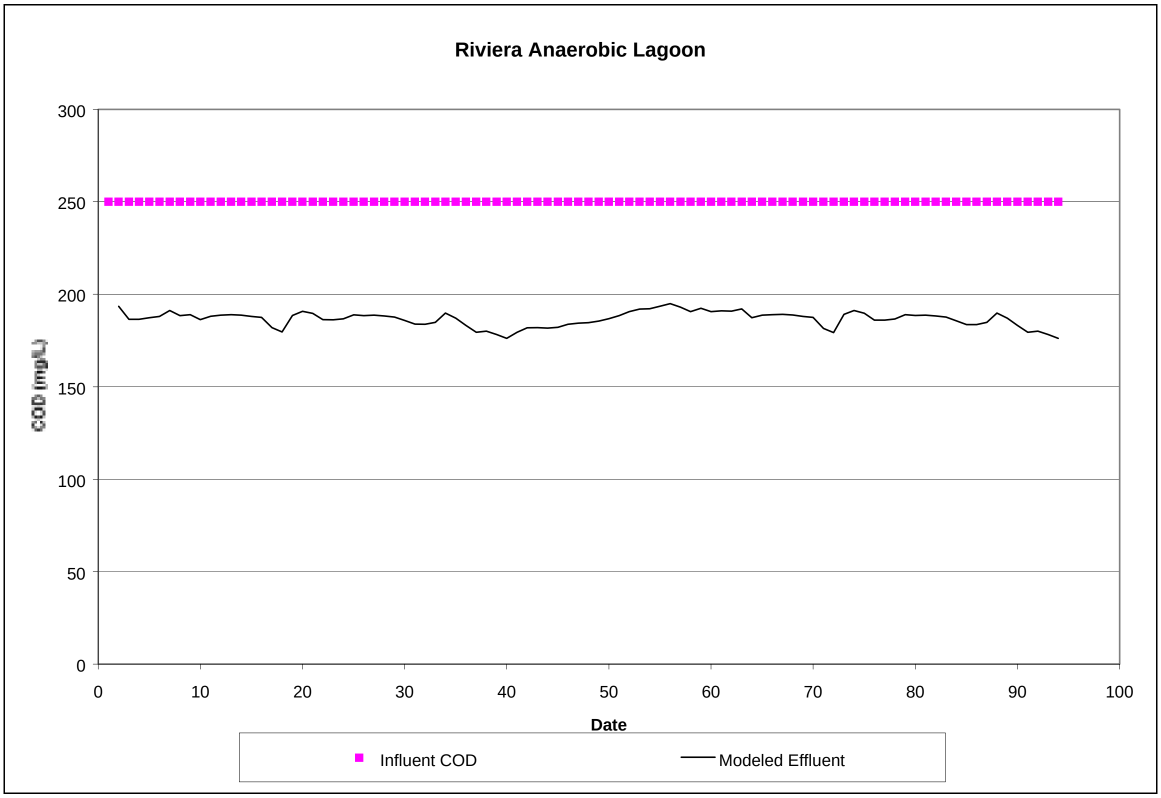

Figure 3-22: Predicted Effluent from Tatui Anaerobic Pond

The models' poor performance is due to the short detention time of the designed anaerobic lagoon at Tatui. This has prompted a redesign of the anaerobic pond for Tatui. The newer design provides an anaerobic pond of 1.8 hectares, 4.5 meters deep. This provides an average detention time of 5.8 days, and the modeled predicted removal efficiency (using the year-round Riviera anaerobic pond model) can be viewed in Figure 3-23. The average volumetric loading to this redesigned pond is 43g/m^3-d. this volumetric loading was the reason for which the year-round Riviera anaerobic pond model was used, and the peak-season model. Indeed, the peak-season model was fit onto data that had an average loading of 150g/m^3-d, while the year-round model was fit with data corresponding to an average loading of 68g/m^3-d.

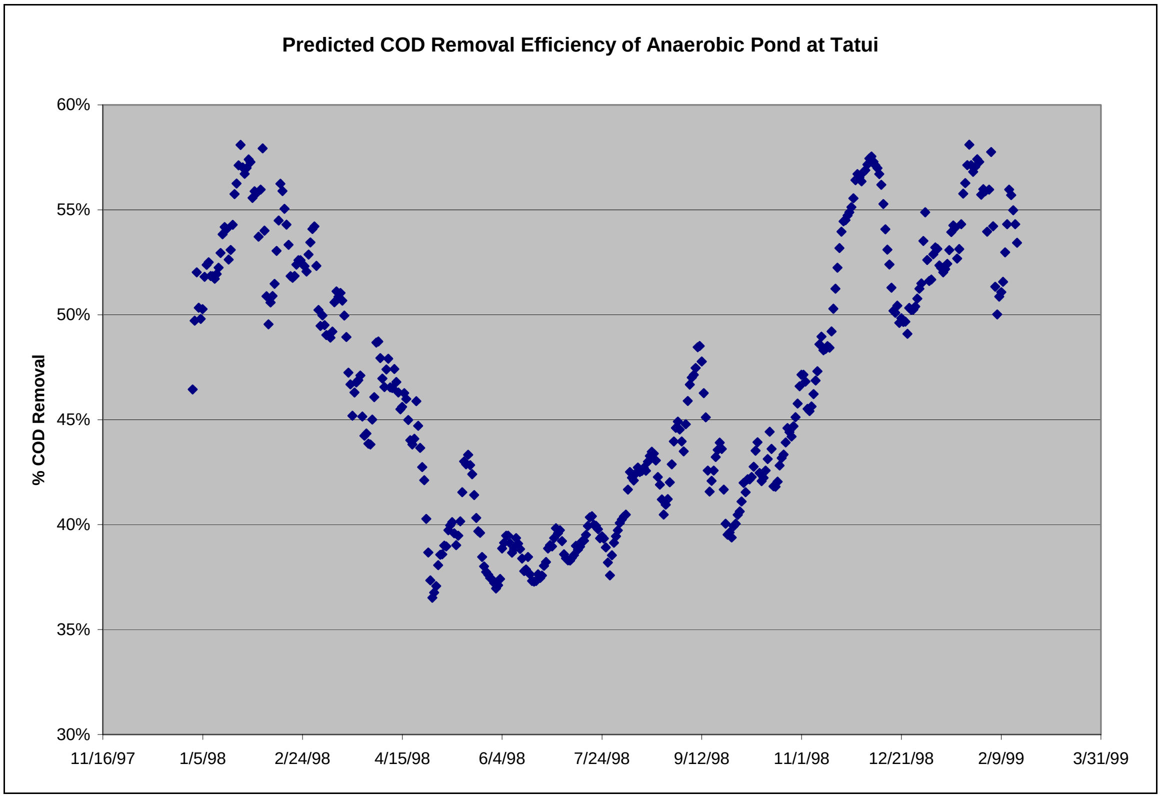

Figure 3-23: Predicted Effluent of Redesigned Anaerobic Pond

The average yearly COD removal efficiency of the redesigned anaerobic pond is 47%. The variations in removal seen in Figure 3-23 are due to temperature only. Indeed, the predicted removal efficiencies were derived from the model with constant COD concentration and flows.

Chapter 4 - Conclusions and Recommendations

This report forwarded a design for lagoons to follow a chemically enhanced primary treatment stage for the city of Tatui in South East Brazil. The preliminary design was done using empirical guidelines taken from the literature. These empirical design values are not site specific, and consequently their applicability to various scenarios is questionable.

It was therefore sought to develop a model that might aid the design process. However, data short-comings and time restrictions have hindered the development of an appropriately validated model, useful for the present purpose. The model framework was taken from Raymond Ferrara's 1978 doctoral thesis. It had been tried and tested for facultative ponds in the United-States (South West), and its performance was acceptable in the predictive mode.

The Ferrara model was simplified to account for the data available in Brazil, and also for the effluent characteristics that were needed. The model was fit to data from a waste stabilization pond system on the coast of South West Brazil. The model was also fit on data acquired from a wastewater treatment station in Amman, Jordan.

The model developed on the Brazilian data was used to predict the effluent quality of the proposed lagoon design to follow the CEPT stage in Tatui. Results were far below expectation, and prompted a redesign of the lagoons to follow the CEPT stage. The redesigned lagoon are shown to achieve the desired removal efficiency.

The data that was available for the treatment plant in Jordan was on a sequence of two aerated lagoons. The first of these lagoons was fully mixed, and the second was only partially mixed to allow for settling. The model was fit on the combination of both ponds. That is to say that the two ponds did not have their respective models. The model fit the data quite beautifully. However, the model did not take the power of aeration into account, and therefore could not be applied to the proposed SABESP design for Tatui. The reasons for this obvious shortcoming of the model were lack of data. No relation could be drawn between aeration intensity and removal efficiency. However, it was proposed to compare the same lagoon modeled as an anaerobic pond, and compare this model with the aerated pond model in order to deduce the parameter for aeration power in the model.