EXECUTIVE SUMMARY

This report deals with the conceptual design of two alternatives to the proposed SABESP design for upgrading the wastewater treatment facility for the city of Tatui, Brazil. The MIT-Group alternatives use chemically enhanced primary treatment (CEPT). CEPT is used to enhance wastewater treatment efficiency, and may be used in conjunction with lagoons. The lagoons discussed in this report are designed with the help of the adapted MIT dynamic nutrient cycling model, which was developed by Raymond Ferrara and Dr. Donald R.F. Harleman in 1978.

Two design alternatives (Alternatives 1 and 2) are presented in this report, and compared with the design proposed by SABESP. The SABESP design is composed of mechanically aerated lagoons followed by settling lagoons. Alternative 1 uses pre-pond CEPT, and Alternative 2 uses in-pond CEPT. The former is the addition of chemical coagulants to concrete sedimentation basins before subsequent lagoon treatment, while the latter is the addition of the chemical coagulants directly to a settling lagoon.

The two alternatives are compared with the SABESP design on two fronts: for removal efficiency and financially. Both design alternatives have satisfactory removal efficiencies in comparison with the proposed SABESP design. The capital costs for Alternative 1 and 2 are 76% of the proposed SABESP design capital cost. The operations and maintenance costs associated with Alternative 1 are 72% of the O&M costs of the proposed SABESP design, and those of the in-pond CEPT (Alternative 2) are 32% of the O&M costs of the SABESP aerated lagoon design.

For Alternative 1, the sludge is composted and landfilled. For Alternative 2, the sludge is digested in the CEPT-pond for two years, and is subsequently dewatered in sludge drying beds, much like the process for the SABESP design.

Pre-pond CEPT is a well-proven technology and is used in large-scale plants in San Diego and Hong-Kong. In-pond CEPT has a more limited experience, being principally used in Norway. It is found that the in-pond CEPT option, referred to as Alternative 2, is the most efficient. Although there is limited experience on In-Pond CEPT, it is recommended that it be used in Brazil, and monitored to gain experience in warmer climates, where it should be more effective.

It is therefore recommended that the upgrading of the CEAGESP treatment plant in Tatui be done using the Alternative 2 in-pond CEPT design outlined in this report. In-Pond CEPT presents an appropriate technology for many situations in Brazil, and is much more cost effective.

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1 INTRODUCTION

This report centers on the design of a wastewater treatment facility for a small city (Tatui) in the state of São Paulo, Brazil. The current facilities at the CEAGESP treatment site in Tatui are overloaded and poorly maintained [Gotovac, 1999]. The CEAGESP site consists of an anaerobic lagoon followed by a facultative lagoon. The lagoons have an overall chemical oxygen demand (COD) removal efficiency of 42% [Gotovac, 1999]. The CEAGESP lagoons currently serve a population of 50,000.

To replace the CEAGESP lagoons, the current proposed design set forth by SABESP, consists of mechanically aerated lagoons followed by settling lagoons, having a total area of 2.02 ha. The sludge is to be digested in the settling lagoons for a period of two years, and then dewatered in sludge drying beds. The assumed efficiency of the design is 95% removal of biochemical oxygen demand (BOD) [SABESP Edital, 1997].

Through contacts with Brazil, the MIT Group was given permission to visit the current facility, to view the design of the proposed SABESP facility, and to create their own design(s). The design(s) by the MIT group would be looked at, upon completion, and would be considered as a possible replacement of the proposed design for the upgrading of the current, severely overloaded facility. In January 1999, the MIT Group went to Tatui, São Paulo, Brazil to conduct a field study of Tatui's wastewater treatment facility (CEAGESP). The group consisted of Dr. Donald R.F. Harleman (Ford Professor Emeritus at MIT), Susan Murcott (Research Affiliate at MIT), Christian Cabral, Frédéric Chagnon and Domagoj J. Gotovac (MIT graduate students).

Jar tests and other field tests were undertaken to quantify the efficiency of the present treatment system in Tatui, and to determine the optimum chemical coagulant dosage, optimum polymer dosage, optimum coagulant/polymer combination, and the necessary settling time for the optimum dosage/combination. The goals of the project were to design a more efficient and less costly treatment facility, and to do so with the limitation that the designed facility must occupy an area no greater than that currently occupied by CEAGESP.

Alternative schemes for the design of a new wastewater treatment facility are exposed in this report. The chosen treatment method for Alternatives 1 and 2 (proposed by the MIT Group) is Chemically Enhanced Primary Treatment (CEPT). This is the enhancement of conventional primary treatment by adding chemical coagulants to promote more efficient settling. It is proposed to study the use of CEPT as pre-treatment, in order to enhance the performance of waste stabilization lagoons. This is called Pre-Pond CEPT, and is referred to as Alternative 1 in this report. For the purposes of this project report, the acronym CEPT is used as the general term for enhancing conventional primary sedimentation basins (whether or not followed by lagoons). Alternative 2 consists of chemically enhanced waste stabilization lagoons (by dosing in the lagoon), and is called In-Pond CEPT.

Alternatives 1 and 2 were designed to achieve an average effluent of 60 mg/L BOD5 (5-day biochemical oxygen demand), which was the only specified effluent parameter for the design of the treatment systems. The average raw sewage influent characteristics for the planned upgrade include the following: inflow rate [Q] = 161 L/s; influent [BOD_5] = 276 mg/L; influent [TSS] = 200 mg/L [SABESP Edital, 1998]. In order to achieve treated effluent BOD5 of 60 mg/L, the average BOD5 overall removal of the treatment system should be about 80%.

Section 2 of this report gives the reader a brief introduction to chemically enhanced primary treatment (CEPT). Section 3 will outline the design of the CEPT stage alternatives for the CEAGESP treatment facility. Section 4 describes the use of a dynamic waste-stabilization-pond model to design the lagoons that will utilize the CEPT technology. Section 4 will compare the financial aspects of the two MIT alternatives and the SABESP designs, and the final section will provide recommendations for the CEAGESP treatment facility upgrade.

2 INTRODUCTION TO CEPT

2.1 Introduction

Chemically Enhanced Primary Treatment (CEPT) is the process by which chemicals (metal salts) and/or organic polyelectrolytes (polymers) are added to primary sedimentation basins to enhance the removal of solids (TSS), organic matter (measured as BOD or COD) and the nutrient phosphorous from wastewater via coagulation and flocculation.

The utilization of CEPT basically involves using a conventional primary treatment facility and adding chemicals to it [the addition of chemical coagulants to increase the efficiency of simple lagoons is called chemical precipitation in ponds, or, in-pond CEPT (Hanaeus, 1991)].

It is important to note that the chemicals added in CEPT are the same ones commonly added in potable water treatment, and that there is largely no residual iron or aluminum in the supernatant from the metal salts (Harleman & Murcott, 1992).

One of the key benefits of CEPT sedimentation basins is that they can be operated at overflow rates much greater than those of conventional primary settlers[1], while still maintaining a high removal rate of TSS and BOD. Operation at a high overflow rate allows for the construction of smaller basins, thus, a lower capital cost. CEPT also provides the opportunity for reductions in size of subsequent treatment units [or it can increase the

[1] The average overflow rate of conventional primary sedimentation basins ranges from 800 – 1200 gal/ft^2 (35 – 50 m/d) at average flow, whereas chemically enhanced primary sedimentation basins are often operated at overflow rate of 1500 – 2000 gal/ft^2 (60 – 80 m/d) at average flow.

capacity of existing conventional primary treatment plants], such as activated sludge basins when used in a combined primary and secondary biological treatment facility[2].

In the case of a coastal city, CEPT is ideal since the removal of TSS is very high and the removal of BOD is sufficient so as to not impact oxygen concentrations in the ocean since the mixing is great. This is precisely the case in two of the largest operating CEPT facilities in the world (Point Loma, California, and Hong Kong). But CEPT is also appropriate for in-land wastewater facilities. It is utilized for phosphorus removal by a number of facilities which discharge their effluent into the Great Lakes (Harleman & Murcott, 1992). A high removal rate of TSS is always desired due the adsorption of toxins to particulates. Thus, CEPT is also a "detoxifying" process. As noted, CEPT can remove a high amount of phosphorus, which can prevent the eutrophication of waters. Biological secondary treatment removes TSS and BOD at a very high efficiency, but does not effectively remove phosphorus, and produces nitrates (Morrissey, 1990). If this effluent does not undergo nutrient removal before it is released into a body of water, eutrophication can occur. The algal blooms often accompanying this kind of nutrient loading will deprive the water body of oxygen, which would, in effect, be the same as releasing a high-BOD effluent into that body of water. It should be noted that CEPT treatment does not preclude subsequent biological treatment. CEPT treatment makes any subsequent treatment smaller and less costly due to the fact that BOD5 removal averages 55% for CEPT versus approximately 30% for conventional primary treatment.

---

[2] CEPT is also an effective means of preparing wastewater for disinfection. With its high removal of TSS, CEPT effluent can easily and effectively be disinfected via chlorination and ultra-violet irradiation. Where odor is regulated, iron salts help control hydrogen sulfide.

CEPT has been around for over one hundred years, yet it is not as commonly used as would be expected upon analysis of its performance. The notion was that CEPT utilized far too great an amount of coagulants and therefore incurred high costs and also dramatically increased sludge production. But, CEPT, most notably low-dose CEPT, is used today with a minimal coagulant dosage (10 - 50 mg/L). The theory of sludge increase is a misconception since the chemicals themselves make only a slight contribution to sludge production[3]. The greatest portion of the increase of sludge production is due to its increased efficiency of TSS removal in the primary clarifiers (or the in-pond CEPT lagoon). This is the goal of CEPT, the increase of TSS removal, TP removal, and BOD removal, in the sedimentation process.

CEPT is a relatively simple technology providing a very low cost, effective (high level of treatment), which is an easily implemented process (Harleman & Murcott, 1992).

2.2 Financial Benefits of CEPT

In addition to what is needed for conventional primary treatment facilities or simple lagoon facilities for in-pond CEPT, the addition of metal salts and/or a polymer will only require tanks for the chemicals and injection equipment[4].

Table 2-1 presents data comparing the costs of primary treatment, secondary biological treatment, and chemically enhanced primary treatment.

---

[3] The amount produced by Alternative 2 is 460 kg/d on a dry weight basis (15% of the total sludge produced), and the amount produced by Alternative 3 is 275 kg/d on a dry weight basis (less than 10% of the total sludge produced).

[4] These expenses are very low, especially when compared to aerators for aerated lagoons. Aerators involve a large capital investment, and a great deal of maintenance, for its parts and for the cost of energy consumption. Therefore, eliminating the use of these aerators will reduce capital and maintenance costs (this is in reference to comparing Alternative 1 to Alternatives 2 and 3).

Table 2-1:Comparison of Costs for Different Treatment Levels (National Research Council, 1993)

	CAPITAL COSTS ($/GPD)	O&M COSTS ($/MG)	TOTAL COSTS
Primary Treatment	0.9 – 1.1	205 – 240	450 – 550
Biological Secondary Treatment	2.4 – 2.6	320 – 410	930 – 1,130
Low Dose CEPT	1.1 – 1.4	230 – 280	550 – 680

This demonstrates how low-dose CEPT costs minimally more than primary treatment, and only about half as much as secondary treatment[5]. Yet, the removal efficiencies show CEPT's superiority, as discussed in the next section.

2.3 Efficiency of CEPT

Table 2-2 displays the efficiencies of the aforementioned treatment schemes[6].

[5] Little data exists as to the cost comparison of in-pond CEPT treatment and is therefore not in Table 2-1.

[6] Again, the three processes represented in Table 2-2 are very common treatment methods which have been thoroughly studied and therefore have ample information on their efficiencies. However, data on the performance of in-pond CEPT lagoons and CEPT as pre-treatment is scant and cannot be put into this table. But, examples of their efficiencies are located in Chapter 3.

Table 2-2: Removal Efficiencies of Different Treatment Methods (National Research Council, 1993)

	TSS (%)	BOD (%)	TP (%)	TN (%)	FOG[7] (%)
Primary Treatment	55	35	20	15	51
Biological Secondary Treatment	91	85	30	31	98
Low-Dose CEPT	85	57	85	37	71

Table 2-2 illustrates how CEPT enhances the removal of TSS and its associated BOD; through chemical coagulation and flocculation, followed, of course, by settling of the floc.

The data, based on a survey of 100 wastewater treatment plants in the United States, show CEPT's superiority over conventional primary treatment. And, by incorporating the cost analysis in Table 2-1, its superiority over secondary biological treatment.

2.4 Ease of Implementation

A conventional primary treatment process is very simple, consisting of bar screens, a grit chamber, and primary clarifiers (see Figure 2-1). The implementation of an in-pond CEPT facility is even simpler since it involves a lagoon in lieu of the primary clarifiers. To upgrade a conventional primary treatment facility to a CEPT facility, basically all that is needed is the addition of a chemical coagulant and potentially a (see Figure 2-2). With CEPT's high surface overflow rate, the sedimentation basins will not need to be large (when compared to conventional primary sedimentation basins). And the use of rectangular sedimentation basins, as Alternative 2 proposes, will allow the use of common walls, which will reduce capital costs.

[7] Fats, Oil, and Grease.

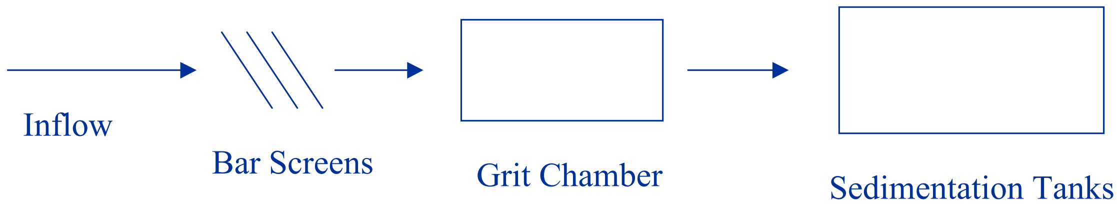

Figure 2-1: Schematic of Conventional Primary Treatment

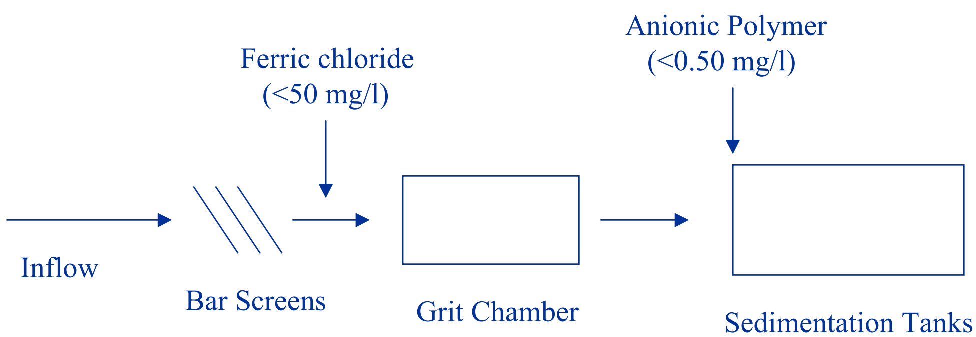

Figure 2-2: Schematic of CEPT Upgraded Primary Treatment[8]

2.5 Coagulation and Flocculation

Coagulation and flocculation are the processes by which CEPT, pre-pond CEPT, and in-pond CEPT demonstrate their great efficiency over conventional primary treatment. It should be noted that the purpose of CEPT is not only to settle non-settleable particles (such as colloids). It is also the intention of CEPT to increase the settleability of all particles, thus decreasing the settling time and size of the sedimentation basin(s), increasing the overflow rate, which in turn will increase treatment capacity. These phenomena were studied in the jar tests undertaken at Tatui. For an explanation of the jar tests, their data and analysis, and a complete chemical analysis (including field tests), see Appendix A-2.

[8] For upgrading, refer to Harleman and Morrissey (19).

2.6 Existing CEPT Facilities

Throughout the United States and the rest of the world, CEPT is implemented at several facilities. See Appendix A-1 for a study on existing CEPT facilities in the United States.

2.7 Summary

CEPT is an efficient, cost-effective and easily implemented wastewater treatment technology, based on data from large-scale municipal wastewater treatment plants in Southern California with multiple years of operating experience under a CEPT regime. The addition of chemical coagulants and/or polyelectrolytes allows for the increased removal of total phosphorus, total suspended solids and its associated biochemical oxygen demand. The increased removal of TP, TSS and BOD is accompanied by increased settling rates of particles, which allows for the design of smaller basins and greater overflow rates.

Based on our jar tests in Tatui using a variety of Brazilian and American metal salts and polymer products, the optimum chemical coagulant for Tatui is a ferric chloride produced by NHEEL. The procedure for determining the optimum chemical coagulant and coagulant dosage is presented in Appendix A-2.

3 DESIGN

3.1 Introduction

This chapter will discuss and explain how the Alternative facilities were designed, focusing mainly on Alternative 1 and Alternative 2 for Tatui with minimal design analysis of the proposed design by SABESP. It should be noted that each alternative will use the combined bar screen-grit chamber unit designed by SABESP for the proposed design.

3.2 Present Pond System at Tatui

The present treatment system of CEAGESP consists of an anaerobic lagoon followed by a facultative lagoon. The system is severely overloaded, which is why it will be upgraded. See Figures 3-1 and 3-2.

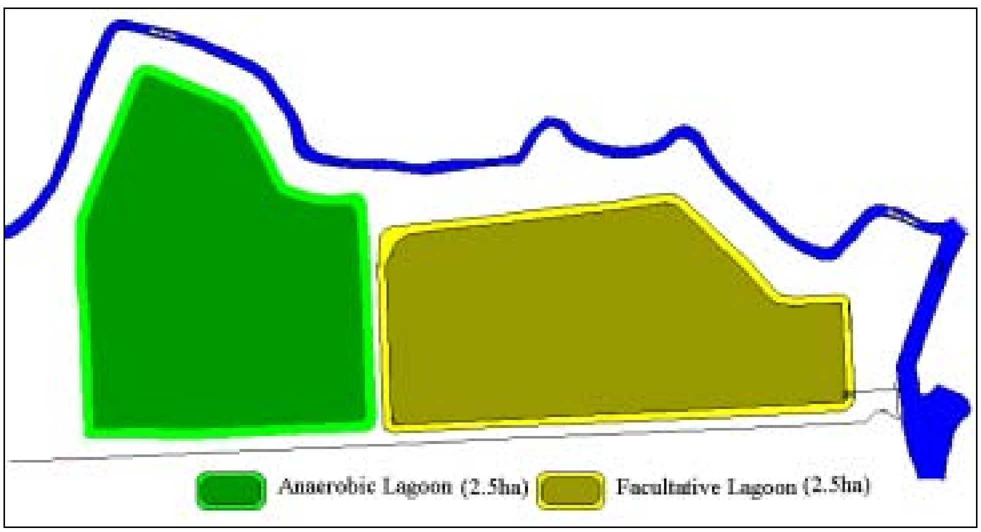

Figure 3-1: Present CEAGESP Pond Layout

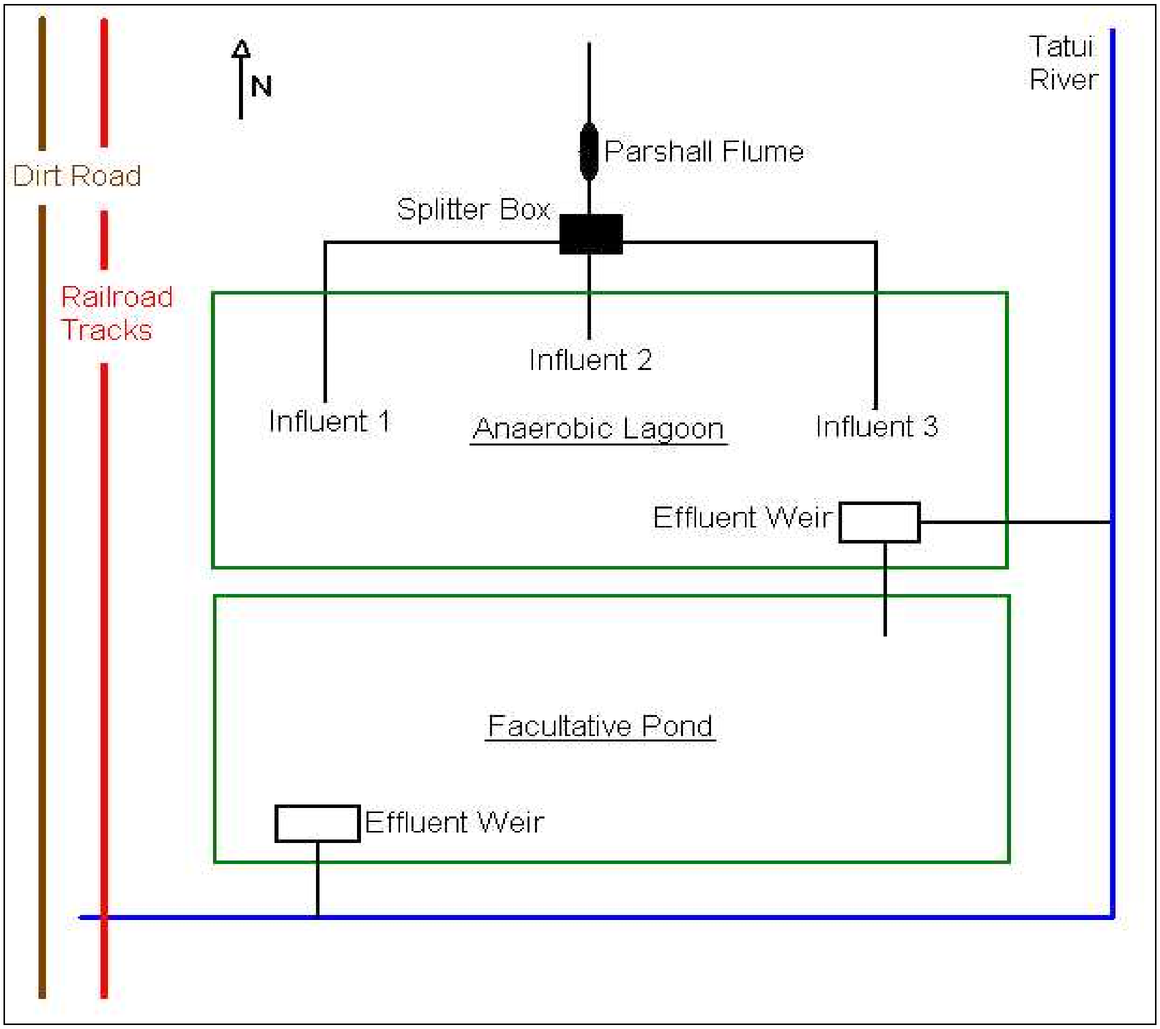

Figure 3-2: Present CEAGESP Pond Schematic

As can be seen in the schematic of the CEAGESP facility in Figure 3-2, part of the anaerobic effluent is directly discharged into the river. After the anaerobic lagoon, the other portion goes into the facultative lagoon. The anaerobic lagoon is almost completely filled with sludge, there is a high degree of short-circuiting, and the lagoon is thus operating well below design expectations. From the results of the field sampling and testing at CEAGESP, it was determined that the anaerobic lagoon had a COD removal efficiency of only 35%, whereas a properly operated anaerobic lagoon should remove 50-85% of the BOD_5 (Metcalf & Eddy, 1991). The facultative lagoon had a COD removal efficiency of 26%, whereas a properly operated facultative lagoon should remove 80-95% of the BOD_5 (Metcalf & Eddy, 1991). It is often found that BOD_5 removal does not equal COD removal, but they are related, and removal efficiencies are close. Thus, although it can not be stated, for example, that the facultative lagoon is only removing 26% of the expected 80-95% of the BOD_5, it is certain that the system is not performing up to par. Nevertheless, the COD measurements are a useful indicator of its current level of efficiency, or lack thereof.

3.3 Proposed SABESP Design

The proposed system consists of four aerated lagoons (the lagoons were often referred to as "tanks" by SABESP officials, thus the labeling in Figure 3-3) equipped with five aerators each rated at 15 hp. Four settling lagoons follow these aerated lagoons. The settled sludge will remain in the lagoon for two years (during which time it will digest and become stabilized) and will subsequently be pumped by a pump barge into the sludge drying beds. The design was undertaken by SABESP, and no analysis can be performed on the methods of design since the calculations are undisclosed. [See Figure 3-3.]

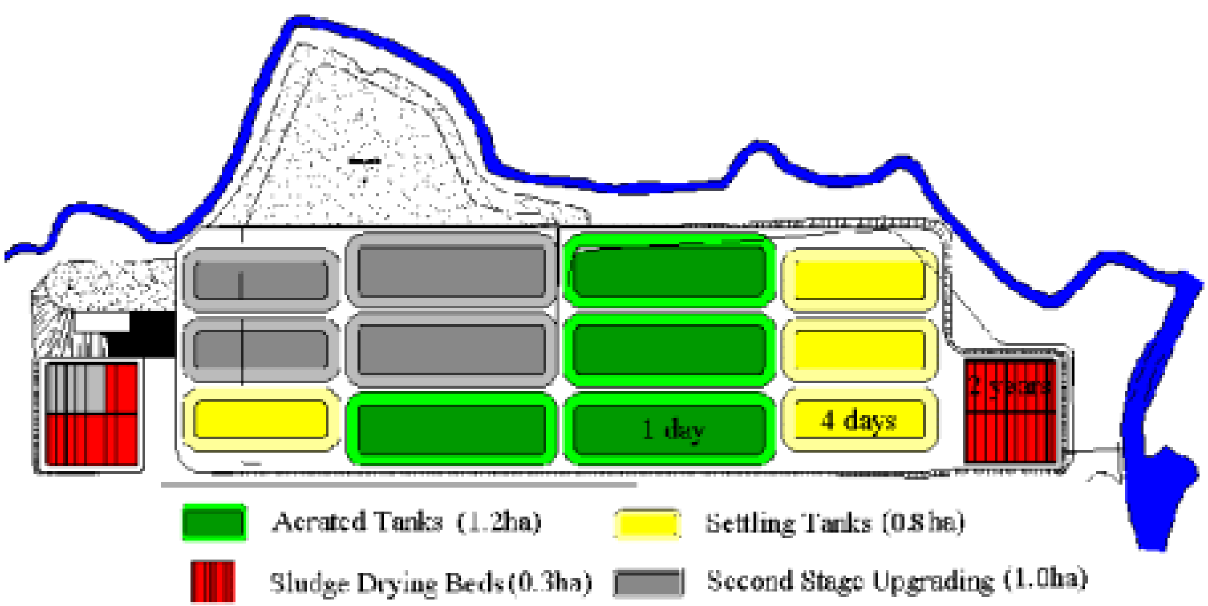

Figure 3-3: Layout of Proposed SABESP Design

Figure 3-3 shows more than four aerated lagoons and four settling lagoons. This is because the SABESP design calls for building four aerated lagoons and four settling lagoons at first, then expanding the facility by adding two more settling lagoons in the future. This expansion also entails building more sludge drying beds and purchasing more surface aerators.

The first stage of the proposed SABESP upgrade consists of four aerated lagoons whose total surface area is approximately 12,000 m^2, with a depth of 3.5 m. Thus, the total volume is 42,000 m^3, which yields a hydraulic retention time of 3 days. The four settling lagoons have a total surface of 8000 m^2 and a depth of 3 m. Thus, the total volume is 24,000 m^3, yielding a hydraulic retention time on the order of 2 days.

The design proposed by SABESP was verified, in order to evaluate the expected efficiency of the design. The calculations were done along the Metcalfe & Eddy (1991) guidelines. The mixing requirements for the aerated lagoons as designed demand 900 hp, assuming that a the energy requirements for mixing alone are 0.6 hp/1000 ft^3 (21 hp/1000m^3) [Metclafe & Eddy, p.611]. However, the design includes only 300 hp for the aerated lagoons. The design is therefore insufficient in terms of mixing energy following the Metcalfe and Eddy (1991) guidelines.

3.4 Alternative 1: Pre-Pond CEPT

Alternative 1 is the first of two alternative design proposals by the MIT group. The treatment system consists of three chemically enhanced sedimentation basins followed by an anaerobic lagoon, followed by the existing facultative lagoon. The sludge from the chemically enhanced sedimentation basins will be pumped to a filter press and subsequently composted (windrow composting). The anaerobic lagoon will occupy part of the space of the present anaerobic lagoon at CEAGESP, and will have a surface area of 1.8 ha. This design alternative will use the CEAGESP facultative lagoon as it presently is (2.5 ha), with no size modifications. See Figure 3-4 for its layout.

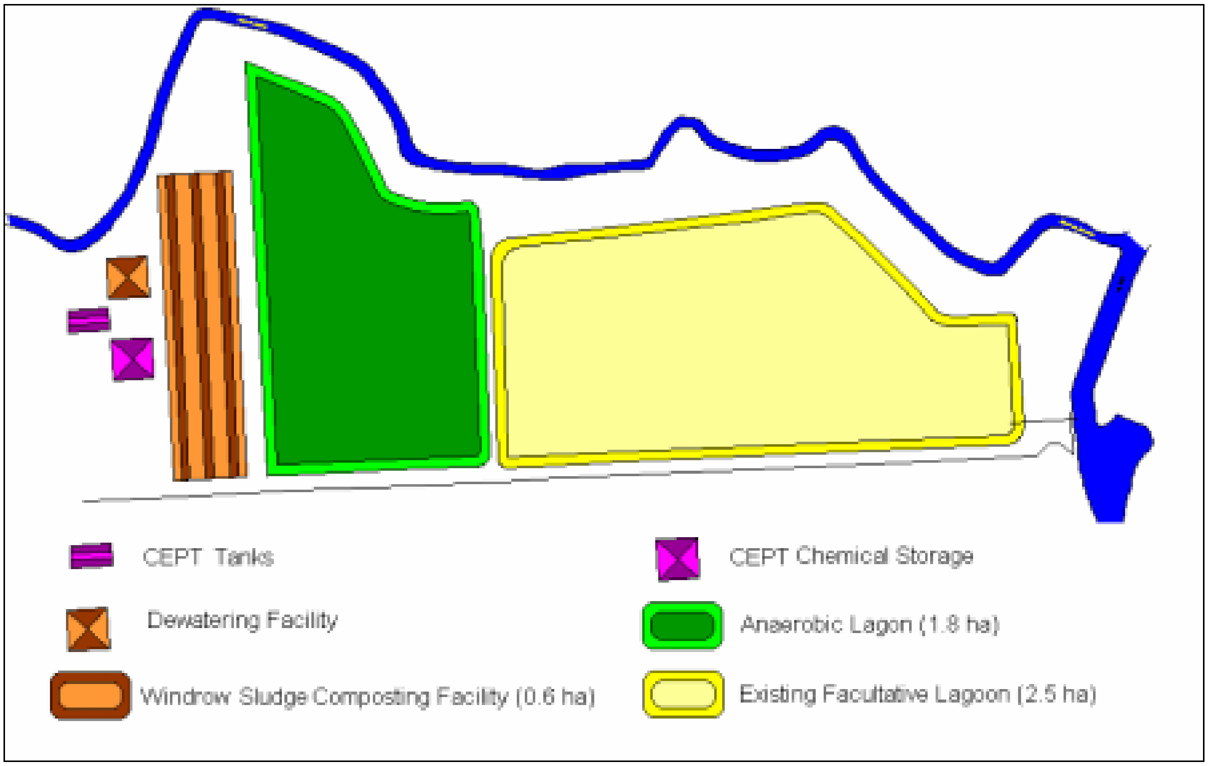

Figure 3-4: Alternative 1 Layout

3.5 Alternative 2: In-Pond CEPT

The second alternative is also a design of the MIT project. It is an in-pond CEPT facility. The wastewater first enters a CEPT lagoon (called a "CEPT settling lagoon" in Figure 3-5). Then the wastewater proceeds into an anaerobic lagoon (of 1.8 ha), and then into the existing facultative lagoon (again, the current one at CEAGESP of 2.5 ha). The sludge in the in-pond CEPT lagoon will be pumped out by a pumping barge after a two-year residence time and will be dried in sludge drying beds. This is the same sludge handling process in the proposed SABESP design for upgrading CEAGESP. See Figure 3-5 for its layout.

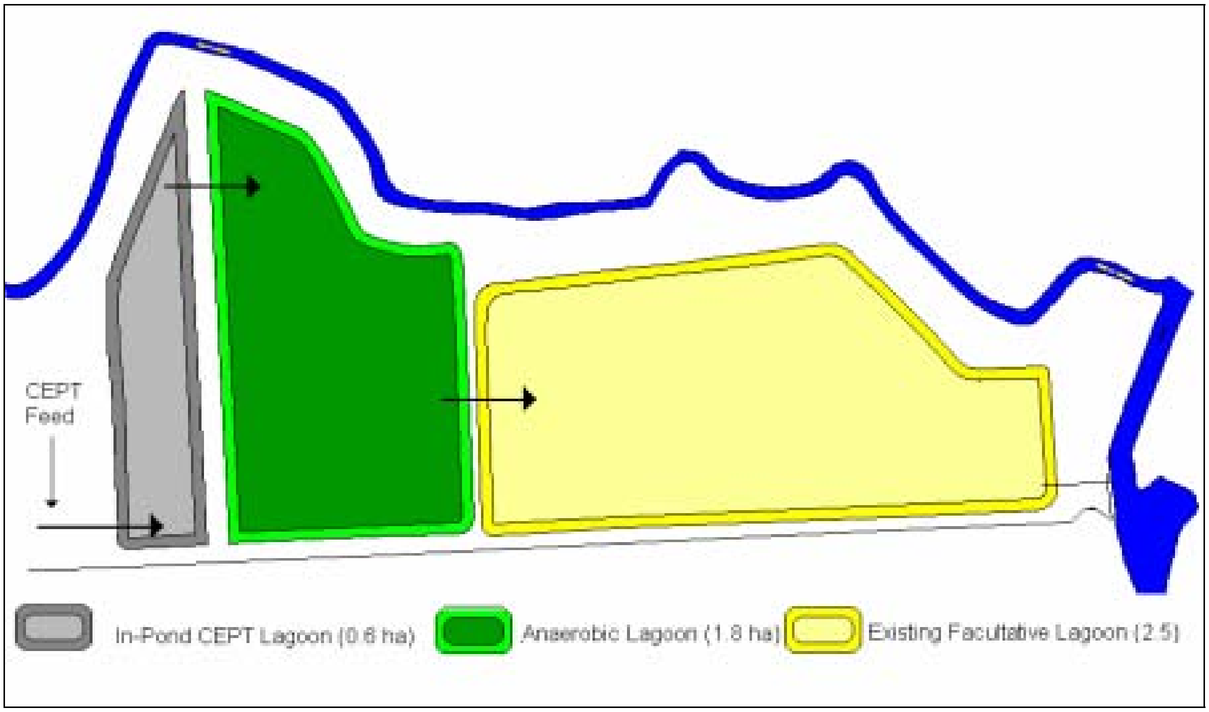

Figure 3-5: Alternative 2 Layout

3.6 Bar Screens & Grit Chambers

The bar screens for Tatui were designed by SABESP. The specifications for design state that the width of each bar is 3/8 inches, with a thickness of 1½ inches and a bar spacing of ¾ inches. This (manual) bar screening facility is a combined bar screen/grit chamber. That is, the head of the grit chamber is a bar screen. This unit will be the same one used in Alternatives 1 and 2. The unit has been analyzed with scrutiny and semblance to common design practice and is deemed appropriate for Alternatives 1 and 2.

3.7 CEPT Sedimentation Basins

Sedimentation basins (often referred to as sedimentation tanks or primary clarifiers) are, most often, the unit process after a grit chamber. The purpose of sedimentation basins is the removal of suspended solids and their associated BOD. The main parameters for the design of a sedimentation basin include the surface overflow rate and the detention time.

The surface overflow rate (OFR) is defined as the volume of wastewater divided by the surface area of the basin (in units of length per time). That is, $OFR = \frac{Q}{SA}$. The OFR value is also the basis for design of a chemically enhanced primary sedimentation tank (as shown below in the calculation for sizing the sedimentation basins).

Detention time is the amount of time that wastewater spends in the basin. Conventional primary sedimentation basins are designed to have detention times varying between 1.5 hours and 2.5 hours (typically, 2 hours). Chemically enhanced primary sedimentation basins can be designed at much shorter detention times due to increased settling velocities from the addition of chemical coagulants. This allows for the option of treating more wastewater in the same amount of space as compared to a conventional tank. Or, the tank can be sized smaller, but still treat the same amount of wastewater that a conventional basin can.

Alternative 1 CEPT Basin Design

The basis for the design of the sedimentation basins for Alternative 1 is the settling test. As can be seen by the NHEEL (a $FeCl_3$ producer in Brazil) jar settling tests (Figures 3-6, 3-7). This settling test will determine the OFR for the actual basins. To correlate this with an actual sedimentation basin, the following equation must be used:

$$\mathrm{OFR} = \frac{H}{t_d}$$

Where,

OFR = Overflow rate (m/d)

H = Distance from water surface to sampling port in Jar Test (m)

t_d = necessary settling time (d).

In Figure 3-6, it is shown that 50% COD removal is reached in 3 minutes, and in Figure 3-7, 80% TSS removal os also reached in 3 minutes. It should also be noted that the TSS removal after 3 minutes without chemical addition was only 53%.

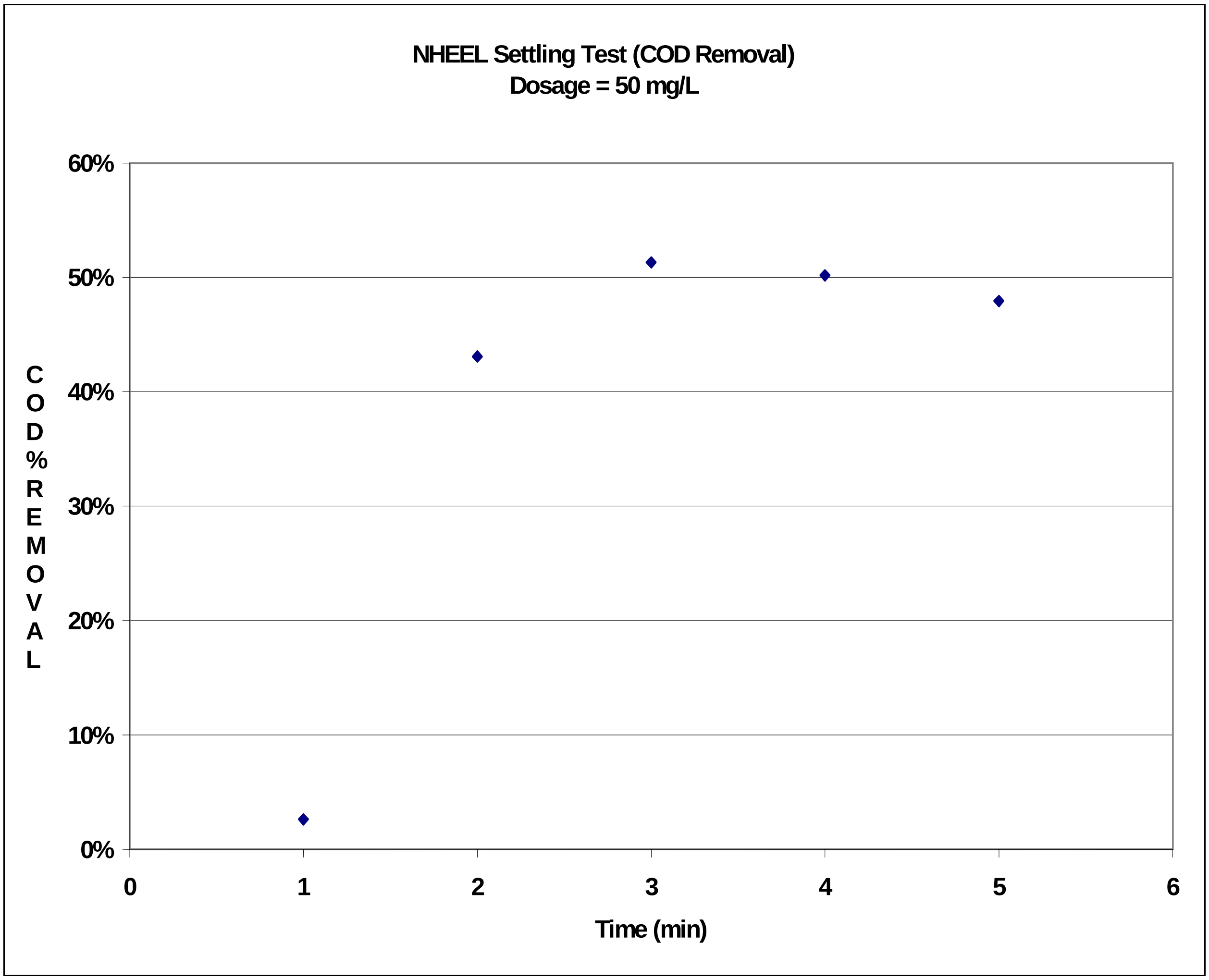

Figure 3-6: NHEEL Settling Test Results (COD)

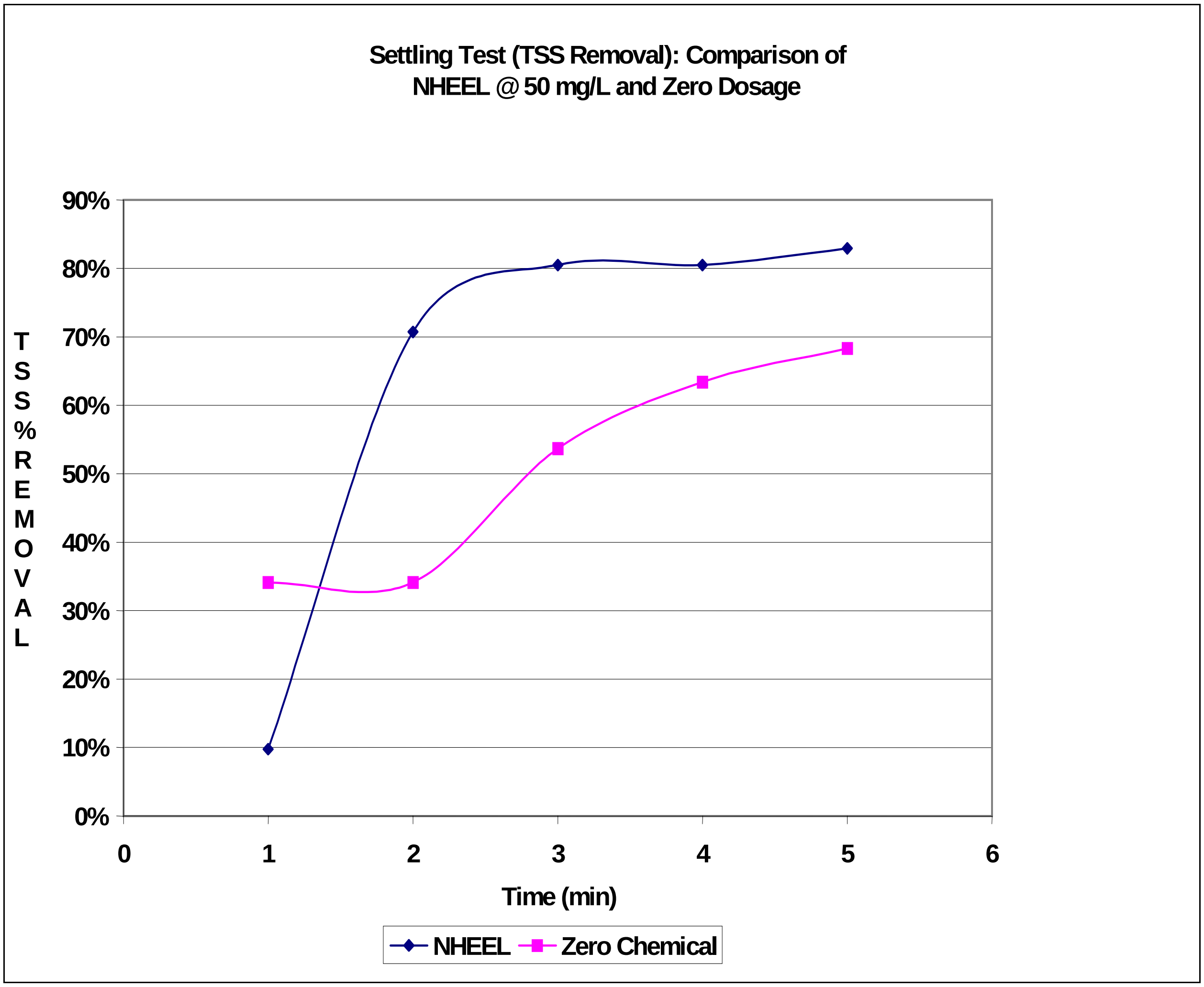

Figure 3-7: NHEEL v Zero Chemical Settling Test Results (TSS)

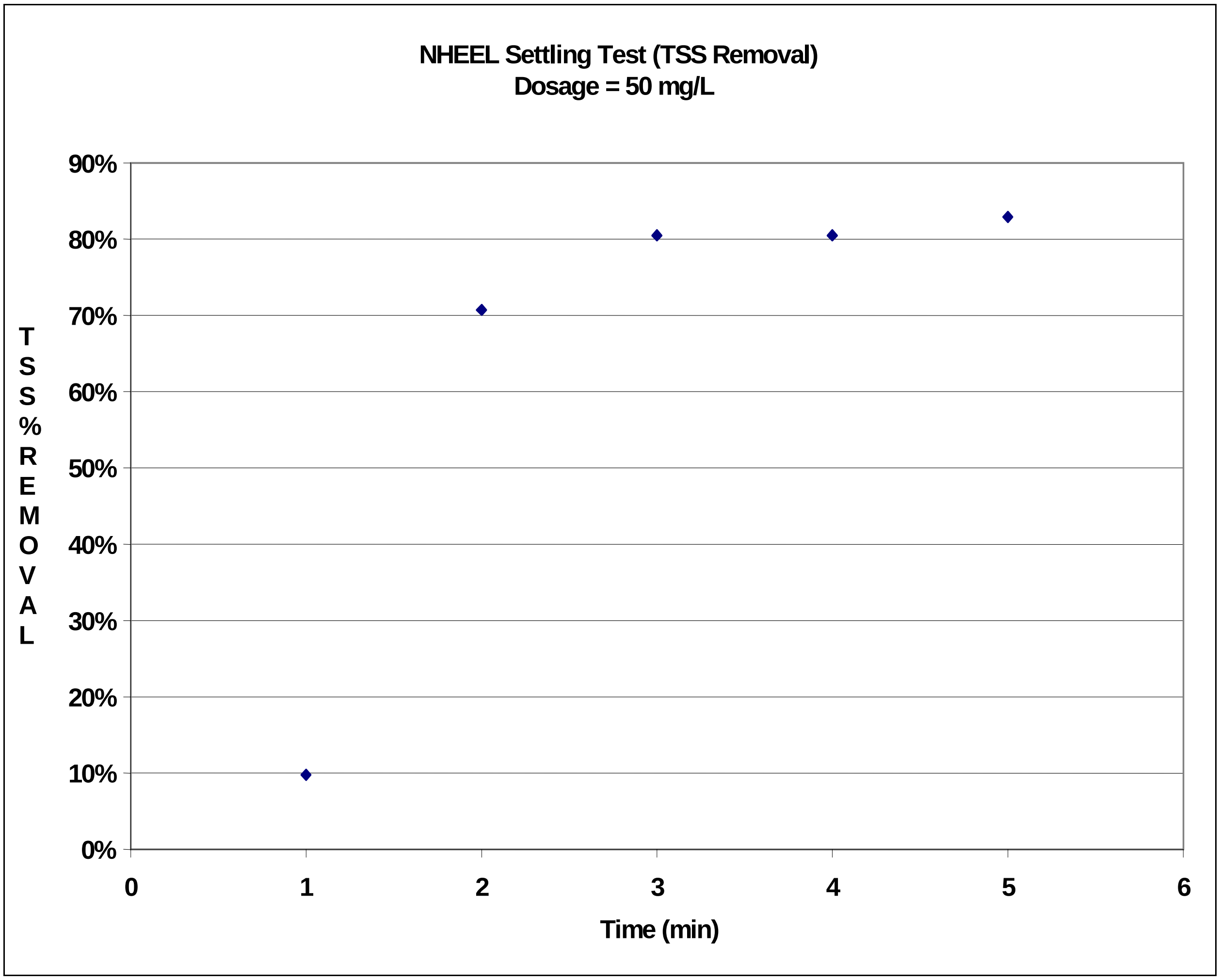

Figure 3-8: NHEEL Settling Test Results (TSS)

The settling height in the jar test beaker was 6 inches (0.1524m) with a 2-liter sample volume. Thus,

$$\text{OFR} = \frac{.1524m}{3\min \cdot \frac{1hr}{60\min} \cdot \frac{1d}{24hr}} = 73 \text{ m/d.}$$

This OFR for CEPT test is about twice as high as that for conventional primary treatment. With this value (73 m/d), the CEPT basins were designed as follows:

> Sedimentation basins are designed for average flow, thus the design value for flow is 161 L/s (with a BOD_5 of 276 mg/L), as specified by SABESP. Another design parameter is that the tank length to width ratio should be at least 5:1 to ensure horizontal flow [the same ratio applied to L:H]. To be able to handle the flow without having over-sized basins, and to allow sufficient capacity to handle flow if one basin has to go down for repair, more than one basin is necessary. For Alternative 1, three basins were chosen as sufficient. For Alternative 2, a lagoon is used instead of a basin. These kind of simple lagoons do not have down time for mechanical failure due to the fact that there are no mechanical parts in the lagoon.

Chemical Dosage (C_c) = 50 mg/L ($FeCl_3$)

Chemical Dosing Period = 12 hr/d (8am – 8pm)

Q = 161 L/s = 13911 m^3/d

Basin Height (H) = 3.5 m

Basin Width (W) = 3.5 m

Basin Length (L) = 19 m

Surface Area (SA) = W·L = 66.5 m

Volume (per basin) (V) = 232.75 m^3

Total Volume (total of all three basins) (V_T) = 698.25 m^3

Overflow Rate (OFR) = $\frac{Q}{3 \cdot SA}$ (a factor of 3 to account for each basin)

= 70 m/d (less than design value, $\therefore$ OK)

Detention time (t_d) = $\frac{V_T}{Q}$

= 1.20 hrs.

The peak flow is 224.33 L/s. With this flow, the OFR is 97 m/d with a detention time of 0.86 hours.

The sludge produced in each sedimentation basin will be manually raked into the basin's sludge hopper for pumping to the filter press.

3.8 In-Pond & Pre-Pond CEPT Treatment Systems

The objective of lagoons, and all wastewater treatment systems, are the removal of TSS, BOD, and a host of contaminants which can pose a threat to the environment. The removal of BOD is accomplished partially through the removal of organic carbon. The major pathways for its removal includes the separation of particulate organic matter into the bottom of the lagoon's sludge bed or by the aerobic degradation of organic matter into CO_2, or during anaerobic conditions, turning the organic carbon into CO_2 and CH_4 (Hanaeus, 1991). Algae, which play a major role in simple lagoons (through the supply of oxygen), can utilize this CO_2 produced in the degradation of organic carbon. But in chemically enhanced lagoons it is chemical precipitation which is the governing treatment process, not algae (Hanaeus, 1991). From a study conducted by Hanaeus (1991) of Finnish CEPT lagoons, it was discovered that the lagoons achieved a BOD_7 removal efficiency of 77%, with a range of 43-88%. These same plants achieved an average phosphorus removal efficiency of 60%, with a range of 20-100%. As in simple stabilization lagoons, short-circuiting of lagoons is detrimental. Thus, it is this hydraulic factor which is the greatest deterrent to the successful operation of chemically enhanced lagoons. To combat this, baffles will be placed in the in-pond CEPT lagoon of Alternative 1 in an attempt to prevent short-circuiting.

In-pond CEPT lagoons are most prevalent and have been most-extensively studied in Scandinavia (where they are called fellingsdams). There are three types of fellingsdams (Balmer *et al*., 1987). The first is pre-pond precipitation. This is what Alternative 1 is, the addition of chemicals in a tank separate from, and before, the lagoon system. The second type is in-pond precipitation. This is what Alternative 2 is, the addition of chemicals at the head of the lagoon system. The third type of fellingsdam is post-pond precipitation. As is indicated by its name, it is the addition of chemicals to lagoon effluent in a separate tank. A study from Balmer (1987) of 56 Norwegian treatment plants (fellingsdams) found that the average removal of TSS was 87.6%. The average removal of BOD was 83.0%, a removal efficiency of COD of 76.9%, and an average removal of TP was 91.6%. It should be noted that not all of these were in-pond precipitation facilities. Of these, that is, the in-pond precipitation facilities, the removal of COD ranged from 68-83%, the removal of TP ranged from 70-94%, and the removal of TSS ranged from 80-93%. Balmer *et al*. (1987) conclude their paper by stating that the simplest and cheapest fellingsdam is the in-pond precipitation mode, which gives the satisfactory results of 70-90% BOD-reduction and 85-95% TP-reduction.

In the planning and operation of fellingsdams, Balmer *et al*. (1987) suggest addressing three matters. The first matter is the need for operator attendance. Alternative 1 is designed to have one engineer, one operations worker, and two maintenance workers (all on-site). Alternative 2 (and Proposed SABESP Design) is designed to have an on-site engineer and an on-site operations worker. The second matter is sludge production. For Alternative 1, the sludge production is calculated in Section 3.6, and the sludge will be removed from the sedimentation basins throughout the work-day of the facility's staff. For Alternative 2, sludge production has also been calculated in Section 3.6 and the in-pond CEPT lagoon is sized to have a sufficient detention time even when the sludge has accumulated for 2 years. For the proposed SABESP design, SABESP also has made undisclosed calculations and maintains that its settling lagoons are sized to accumulate sludge for 2 years and still operate efficiently. The third matter is the possibility of odors. Although a normal lagoon will produce odors, CEAGESP is not close to residences, so this is not a significant concern in the design. In the current 2-lagoon system of CEAGESP, the odors were not strong at all, and the visit was in the summer time, when temperatures are high and microbial activity is also high, and still no odor problems.

The in-pond CEPT lagoon will function as an extremely large (earthen) sedimentation basin which is sized properly to store the accumulated settled solids (sludge) for a period of two years. The facility staff will maintain the lagoon; most importantly, as algae form in the lagoon, they will be removed and disposed of. The sludge will accumulate at the bottom of the lagoon and will anaerobically digest. Lagoons become anaerobic due to microbiological activity and the BOD entering the lagoon. The influent BOD is 276 mg/L, and since approximately 50% (based on jar tests) will be removed through chemical coagulation and sedimentation, the concentration of the sludge on the bottom will be about 140 mg/L. This concentration is sufficient over the specified surface area (6,000 m^2) to create an anaerobic condition. Also, microbial activity doubles with a rise of (approximately) 10ºC, and Tatui is in a warm climate (with an average temperature of approximately 22ºC). Thus, with the warm temperature and high rate of microbial activity, dissolved oxygen will be depleted and an anaerobic state in the in-pond CEPT lagoon will be achieved rather quickly. In fact, Narasiah *et al*. (1990) indicate that in high water temperatures in ponds (greater than 18˚C [64˚F]), organic sludge may be completely decomposed. The biodegradation of the accumulated bottom sludge is of great importance in a high-water temperature setting as Tatui, when it may be more rapid than solid decomposition (Hanaeus, 1987). The sludge will digest similarly to sludge in an anaerobic digester, and also in the same way sludge digests in a lagoon without chemical addition. And since the temperature is fairly high, the sludge will "self-digest" quicker than in the studied facilities in Scandinavia.

The in-pond CEPT lagoon will be in the same spot the current anaerobic lagoon is located. But, the current design calls for a surface area of 0.6 hectares (6,000 m^2), whereas the current anaerobic lagoon is 2.5 hectares. Thus, an earthen dike will be placed in the current anaerobic lagoon at a specified distance to achieve a 0.6-hectare surface area for the in-pond CEPT lagoon. The following is the design specifications for the in-pond CEPT lagoon:

Chemical Dosage (C_c) = 30 mg/L of NHEEL ($FeCl_3$).

Depth (H) = 4.5 m

Volume (V) = 27000 m^3 (without sludge deposition)

Volume (V) = 9200 m^3 (with sludge deposition of 2 years)

Surface Area (SA) = 6,000 m^2

Overflow Rate (OFR) = 2.31 m/d

Residence Time (t_d) = 16 hr (with sludge deposition of 2 years)

Residence Time (t_d) = 47 hr (without sludge deposition)

Chemical Dosing Period = 12 hr/d

An advantage of upgrading the current facility with the in-pond and pre-pond chemical precipitation options (Alternatives 2 and 1, respectively) is that the current facultative lagoon can still be used as a form of treatment for the produced wastewater while the facility is built.

3.9 Sludge Quantity

The quantity of sludge produced is what will determine the size of sludge handling facilities and often the choice of processes for treating the sludge. The quantity of sludge produced by conventional primary treatment is equal to the quantity of TSS removed by the treatment process. Thus,

$$S_p = Q \cdot TSS_{rem} \cdot 10^{-3} \tag{3-1}$$

where:

S_p = Dry weight of raw sludge produced (kg/d)

Q = Influent flow rate (m^3/d)

TSS_{rem} = Concentration of suspended solids settled/removed (mg/L)

10^{-3} = Conversion factor for liters to m^3 $(\frac{1000L}{1m^3})$ and mg to kg $(\frac{1kg}{10^6 mg})$

This, of course, is in terms of dry solids. Therefore, this value must be turned into a volume by using the sludge %solids content and sludge density.

CEPT sludge calculations must include the chemicals which precipitate out ($Fe(OH)_3$) and the precipitates formed in the removal of phosphorus. The equation, from Murcott and Harleman (1992), to include these variables, is:

$$S_p = Q \cdot [TSS_{rem} + F \cdot P_{rem} + K \cdot C_c] \cdot 10^{-3} \tag{3-2}$$

where:

F = Stoicheometric factor for $FePO_4$ removal (1.42 for $FeCl_3$, a trivalent metallic salt)

P_{rem} = Quantity of phosphorus removed (mg/L)

K = Constant (0.66 for $FeCl_3$; 66% by weight of the $FeCl_3$ precipitates out as $Fe(OH)_3$),

C_c = Concentration of chemical coagulant added (mg/L)

For Alternative 1, the sludge produced is determined by:

Q = 13911 m^3/d

TSS_{rem} = 180 mg/L *(90% TSS removal in CEPT Tanks is assumed for sludge production calculations, although the settling test shows 80% removal. This enables a more conservative design for sludge handling)*

F = 1.42

P_{rem} = 3 mg/L

K = 0.66

C_c = 50 mg/L of $FeCl_3$ (produced by NHEEL)

%Solids = 4%

Sludge Density = 1025 kg/m^3.

Thus,

S_p = 3022 kg/d

$$\text{Wet Sludge} = \frac{S_p}{\%solids} = 75{,}559 \text{ kg/d}$$

$$\text{Sludge Volume} = \frac{S_p}{(\%solids)(\rho)} = \frac{WetSludge}{\rho}\ 74.1 \text{ m}^3\text{/d.}$$

This is the amount of sludge that will necessitate handling on a daily basis[9]. Thus, the sludge handling facilities will be designed to handle this volume.

For the sludge production of Alternative 2, the difference from Alternative 1 is that C_c = 30 mg/L. Following the same calculations as done above for Alternative 1, the above results are:

S_p = 2838 kg/d

Wet Sludge = 70,698 kg/d

Sludge Volume = 69.58 m^3/d

This sludge volume, as stated, is the sludge produced daily. But, the sludge will be allowed to accumulate in the in-pond CEPT lagoon for two years. Thus, due to compaction and

[9] It should be noted that the calculations for sludge accumulation in the in-pond CEPT lagoon and the sedimentation basins, C_c is not adjusted for the fact that chemical dosing is not a 24-hr process. Thus these sludge calculations represent a conservative figure.

anaerobic digestion, the volume and amount of solids after two years cannot simply be determined by multiplying the above results by 2 years. The amount of sludge accumulation is calculated as follows:

The sludge in the lagoon is assumed to degrade at a rate of 50% of volatile solids (VS), which represent 75% of the total solids (TS), per year. The remaining solids will be called non-volatile solids (NVS), which are also often called Fixed Solids (FS). The following calculations will determine the TS, VS and NVS accumulation per year and then determine the amount of dry solids after 2 years:

$TS = S_p \cdot 365$ d/yr = 1,036,137 kg/yr

$VS = TS \cdot 75\%$ = 777,103 kg/yr

$NVS = TS \cdot 25\%$ = 259,034 kg/yr

$$\text{Dry solids (kg)} = 2 \cdot (NVS) + (0.5)^2 \cdot (VS) + 0.5 \cdot (VS) \qquad \textbf{(3-3)}$$

$$= 1100896 \text{ kg/2yr}$$

Thus, 53.2% ($\frac{1,100,896}{(2)(1,036,137)}$) remains from what was produced over a two year period. The result is that after two years of using the in-pond CEPT lagoon, there will be 1,100,896 kg of dry solids. Assume, for a moment, it occupies its original volume (that is, no compaction), this would yield a %solids of 2.125%. But, the sludge is compressed via sludge compaction (sludge settling on top of sludge). Assuming that compaction results in a decrease in sludge height of 65% (thus, a height of 35% compared to no compaction), the resulting density of sludge is 60 kg of dry solids per m^3 of wet sludge (6% solids). The %solids is calculated as follows:

$$= \frac{Solids\,Pr\,oducedPerDay}{VolumeOfSludgePerDay \cdot (1 - \%compaction)}$$

$$= \frac{151.9\frac{kg}{d}DrySolids}{(69.58\frac{m^3}{d}WetSludge) \cdot (1 - 0.65)}$$

≈ 6 % solids

Thus, the new density (of the wet sludge) is

(6%)(1.5 g/cm^3) + (94%)(1.0 g/cm^3) = 1030 kg/m^3.

Above, 1.5 g/cm^3 is the density of dry solids, and 1 g/cm^3 is the density of water.

With this calculated density (of the wet sludge), we can calculate the volume of wet sludge produced after two years, and the weight of wet sludge.

Wet sludge weight after two years $= \frac{DrySolids(2yrs)}{\%solids}$

$$= \frac{1,100,896kg}{6\%}$$

=18,348,266 kg (after 2 yrs)

The wet sludge volume is the wet weight divided by the wet density (1030 kg/m^3). Thus, the volume is

Volume $= \frac{18,348,266kg}{1030kg/m^3}$

$$= \frac{17814 m^3}{2 yr} \text{ [that is, after 2 years residence time]}$$

$$= 24.4 \text{ m}^3/\text{d}.$$

Thus, dry solids production per day, including decomposition and compaction is

$$= (24.4 m^3 / d)(1030 kg / m^3)(6\% solids)$$

$$= 1510 \text{ kg dry solids per day}.$$

The calculations above will determine the number of drying beds necessary. The calculations for the number of sludge drying beds are in Section 3.11.

3.10 Chosen Sludge Handling Methods

Many different methods can be utilized to process sludge. The main difference in most approaches is the cost.

The options to be discussed below were chosen based on technical feasibility (often simplicity) and a cost-benefit analysis. Many processes were eliminated due to high cost, such as anaerobic and aerobic sludge digestion. Also, the options below require very little technical expertise and maintenance. Land is readily available at the Tatui site, thus the facility was designed with this important characteristic (*i.e.*, available land).

Proposed SABESP Design

SABESP's design calls for the sludge to accumulate in the settling lagoons and have a residence time of two years. After this two-year residence time, when the sludge will anaerobically digested, the sludge will then be pumped (by a barge) into sludge drying beds (conventional sand drying beds).

Alternative 1: Pre-Pond CEPT

For Alternative 1, sludge will be pumped from the CEPT sedimentation basins to a filter press. The sludge will be properly mixed with bulking agents and/or an amendment. It will be composted in a windrow composting facility. The compost will be disposed of by landfilling, or given away (free) as fertilizer (for example, to be applied to eucalyptus trees, which are widely used in Brazil as a raw material for "pressed" boards).

Alternative 2: In-Pond CEPT

The second alternative/design for Tatui does not call for the use of chemically enhanced sedimentation basins. The design involves a "chemically enhanced lagoon" which is located where the current anaerobic lagoon is (but does not take up the whole lagoon space). Thus, the sludge will settle in the lagoon and accumulate at the bottom of the lagoon. After a period of two years (a time during which the sludge volume will become stable and volatile solids will be reduced at a rate of 50% per year due to anaerobic microbial activity in the sludge), the sludge will be pumped from the bottom of the lagoon, by a barge, into sludge drying beds at the rate at which the sludge is being produced. The residence time in the sludge drying beds is 24 months. The sludge will be mechanically removed after its 24-month residence time and will be subsequently landfilled, or given away as free fertilizer, if possible.

3.11 Storage and Pump Calculations

The chemical dosing for Alternatives 1 and 2 will be handled by a pump and storage facility. The following are the pump and storage calculations for each design for the use of NHEEL, the chosen chemical coagulant.

Pump Calculations

Alternative 1: Pre-Pond CEPT

Amount of chemical required:

$= Q_{max} \cdot C_C$

$= (224L/s)(50mg/L)(86{,}400s/d)(\frac{1kg}{10^6 mg}) = 970kg/d$ (dry basis)

(dry weight = 970 kg/d) = (wet weight = ?) x (% in solution = 38%)

wet weight = $\frac{970kg/d}{0.38} = 2550kg/d$ (liquid solution).

Pump Calculations:

Q_{max} = 224 L/s = .224 m^3/s

%Solids = 38%

Density (ρ) = 1.4 kg/L

C_c dry = 50 mg/L

C_c liquid = $\frac{C_C dry}{\%Solids} = \frac{50mg/L}{.38} = 132mg/L$

Mass Flux = Q ·C = (806.4 m^3/hr)(.132 kg/m^3) = 107 kg/hr

Pump Capacity = $\frac{MassFlux}{\rho} = \frac{107kg/hr}{1.4kg/L} = 76.5L/hr$ = 500 gpd

Thus, 500 gpd (gallons per day) is the necessary pump capacity.

Alternative 2: in-Pond CEPT

For Alternative 2, the same calculations pertain, with the substitution of Cc = 30 mg/L, instead of 50 mg/L. This yields a requirement of 1530 kg/d of liquid solution, and a necessary pump capacity of 300 gpd.

Chemical Storage Tanks

The storage facility for the chemicals will have a volume designed to store ten days of peak flow demand.

Alternative 1

Volume = (Pump Capacity)(10 days)

= (500gpd)(10d) = 5000 gallons = 19,000 L

Thus, a storage tank of 20,000 L will be used, which will provide necessary freeboard.

Alternative 2

Volume = (300 gpd)(10d) = 3000 gallons = 11,625 L

Thus, a storage tank of 12,000 L will be used, which will also provide necessary freeboard.

The chemical storage tanks will be filled as deemed necessary by the plant engineer, about every ten days.

3.12 Existing Design

The existing facility (CEAGESP) at Tatui is a very simple lagoon treatment system. To make it even simpler to operate, the facility had no provision for sludge processing. Thus as the lagoons are dredged for the new construction to begin, the sludge handling it will receive is its removal and subsequent landfill disposal.

3.13 Windrow Composting (Alternative 1)

The chosen sludge-handling mode for Alternative 1 is Windrow Composting. The windrow method will be done by mixing and turning the piles to supply the microorganisms with oxygen, to control the temperature of the piles, and to remove excess moisture. Before the piles can be formed, the sludge will be thickened in a filter press. Then amendment and/or bulking agents will be added to the dewatered sludge. The amount of amendment and/or bulking agents can not be presented here since there has not been an analysis of the (dewatered) sludge to determine the optimum mixture. Thus, this is a conceptual design of what Alternative 1 uses to treat its produced sludge. The windrows should be mixed at a ratio of approximately 3:1 with wood chips[10], and formed into windrows of about 8 ft high and 12 feet wide, with a spacing on the order of 8 feet between the piles.

3.14 Sludge Drying Beds

Proposed SABESP Design

Since SABESP made this design and the calculations are undisclosed, the following is only the sizing and number of drying beds:

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[10] Source: Conversation with Michael Bryan-Brown of Green Mountain Technologies, Whitingham, VT (May 3rd & 6th, 1999).

Bed Length = 25 m

Bed Width = 5 m

Number of Beds = 32

Land Requirement = 4000 m^2.

Alternative 2

Alternative 2 will dry the in-pond anaerobically-digested sludge in sludge drying beds. The calculations are:

From the sludge accumulation calculations and digestion calculations for the in-pond CEPT lagoon (in Section 3.6), the loading of dry solids is 1508 kg/d with a density (ρ) of 1030 kg/m^3 at a 6% solids content. This daily dry solids load amounts to 1,100,913 kg after two years (the designed residence time). The sludge will be pumped out of the in-pond CEPT lagoon at the rate at which it accumulates (1508 kg/d dry solids), which is 24.4 m^3/d (from the sludge quantity calculations). From Metcalf & Eddy (1991) design recommendations, the chosen loading rate is 125 kg/m^2·yr. The drying beds are designed to have a residence time of 2 years. Thus, the loading rate is 250 kg/m^2·2yrs. The required surface area (SA) is calculated by dividing the dry load by the loading rate (for the two-year period), as so:

$$SA = \frac{TotalDrySolids}{SolidsLoadingRate} = \frac{1{,}100{,}913kg}{250kg/m^2} = 4400m^2$$

SABESP's design calls for the use of 25 x 5 m-drying beds. Alternative 3 will use the same (SABESP-approved) dimensions. This surface area will necessitate the use of 36 drying beds of the designed size, which require a total land area of 4500 m^2

(0.45 hectares). The sludge drying beds will be constructed with roads for access for trucks and other large vehicles involved in their cleaning and/or maintenance.

4 LAGOON MODELING

The design of waste stabilization ponds is usually done along empirical guidelines that are not site, or quality, specific. This section outlines the modeling procedure for the design of waste stabilization ponds to follow the CEPT stage in Tatui, Brazil. The modeling framework is an adapted version of the dynamic nutrient cycling model developed by Raymond Ferrara and Dr. Donald R.F. Harleman in 1978 (c.f. Appendix-B).

4.1 Model Fitting

The adapted model parameters were fit onto data from the waste stabilization facilities at Riviera de São Lorenço, a small resort city located 50km north of Santos, which is 80 km East of São Paulo. The lagoons Riviera de São Lorenço serve a peak population of 50,000, which is similar to the population served by the CEAGESP facilities in Tatui. The data available from Riviera is discussed in Appendix-B. The model was fit on the first pond (designed as an anaerobic pond), and the resulting model and data comparison are presented in Figure 4-1.

The São Lorenço data is used to fit the model parameters due to the fact that the influent characteristics are comparable to those at Tatui. The average raw sewage influent COD at Riviera is of 500 mg/L, and the averaged removal efficiency of the whole treatment system is of 67.5% (c.f. Appendix-B for more details on the Riviera WWTP).

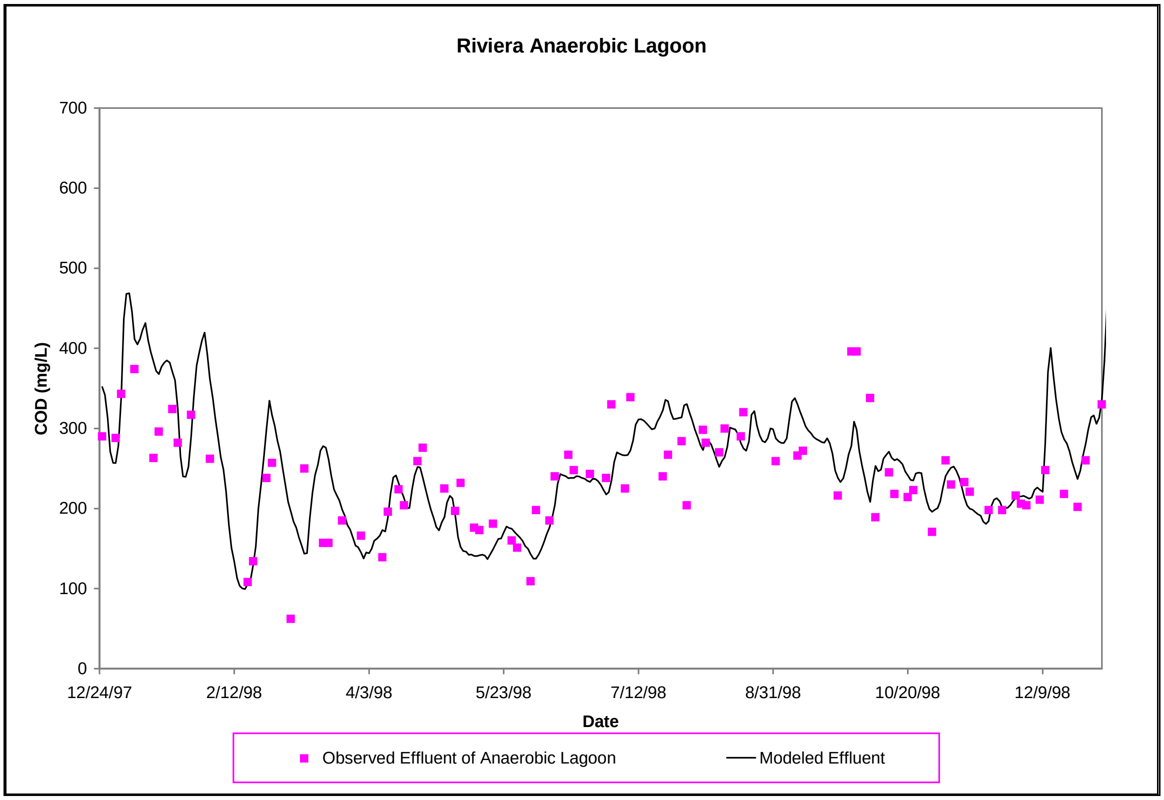

Figure 4-1: Riviera Anaerobic Lagoon Model

Visual inspection of the model reveals that the fit is rather good. The four model parameters were optimized one-by-one, and when all four parameters had been optimized, the process was re-iterated, much like a Newton-Raphson optimum search. The reader is referred to Appendix-B for more details on the model parameters and model fitting procedure. It is thought that the model achieved approaches the best-possible fit. The fitted parameters are extremely close to the parameters for the Kilmicheal and Corinne ponds that were found by Ferrara & Harleman in 1978 (c.f. Table 4-1).

The final estimated parameters for the Riviera Anaerobic Lagoon Model are shown in Table 4-1. The Corinne and Kilmicheal 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) and in Kilmicheal (MI) in 1978.

Table 4-1: 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 & Harleman, 1978)	Values for First Facultative Pond in Corinne, Utah (Ferrara & Harleman, 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

4.2 Use of the Model in Predictive Mode for Pond Design at Tatui

The Riviera model, presented in the last section, is used in this section to size waste stabilization ponds that will follow the CEPT stage at the CEAGESP facility in Tatui. The CEPT stage effluent characteristics that are presented in Table 4-2 are a result of the jar testing done on-site in Tatui during the month of January 1999.

Table 4-2: Predicted CEPT Stage Effluent Characteristics

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
Design	161	3843	1779.2	1921.5	355.9
2015	244	5823	2696.5	2911.5	539.3

Using the Riviera model with the design values as inputs, it is found that a 1.78 ha pond of 4.5 meters depth will achieve an average yearly COD removal of 46.5%. The predictive model output is shown in Figure 4-2.

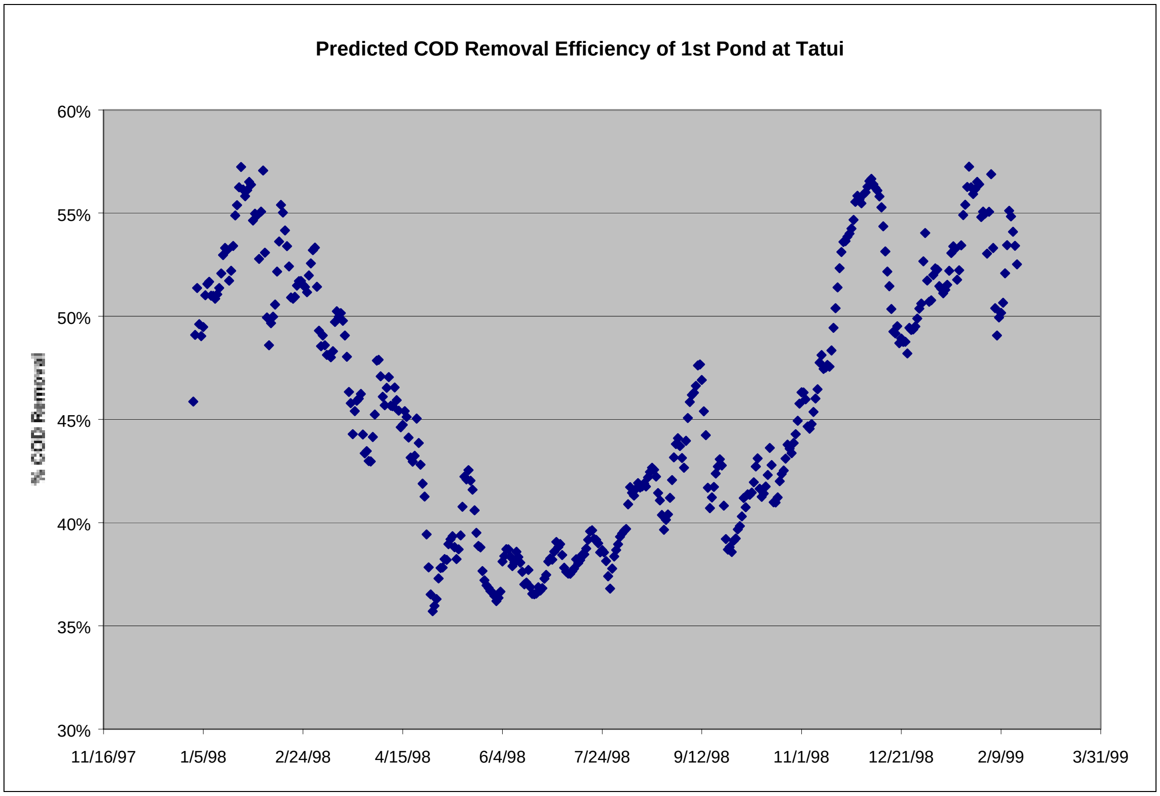

Figure 4-2: Predicted COD Removal Efficiencies for 1st Pond at Tatui

The design of this anaerobic pond would permit the use of the existing second pond at Tatui (facultative pond) as final polishing for the effluent. The existing facultative pond was modeled, and its removal efficiencies are exhibited in Figure 4-3. The average BOD removal for the existing facultative pond is 20.4%.

As discussed in Section 3 of this report, the predicted average COD removal of the CEPT stage is 50% for the pre-pond CEPT option (Alternative 1) and 60% for the in-pond CEPT option (Alternative 2) (c.f. Section 3 of the report). The design conditions for the CEAGESP treatment facility, outlined in a report by SABESP in 1992, are for an average influent BOD5 concentration of 276 mg/L. Table 4-3 presents the average predicted effluent BOD5 concentration from the designed first pond for Alternatives 1 and 2.

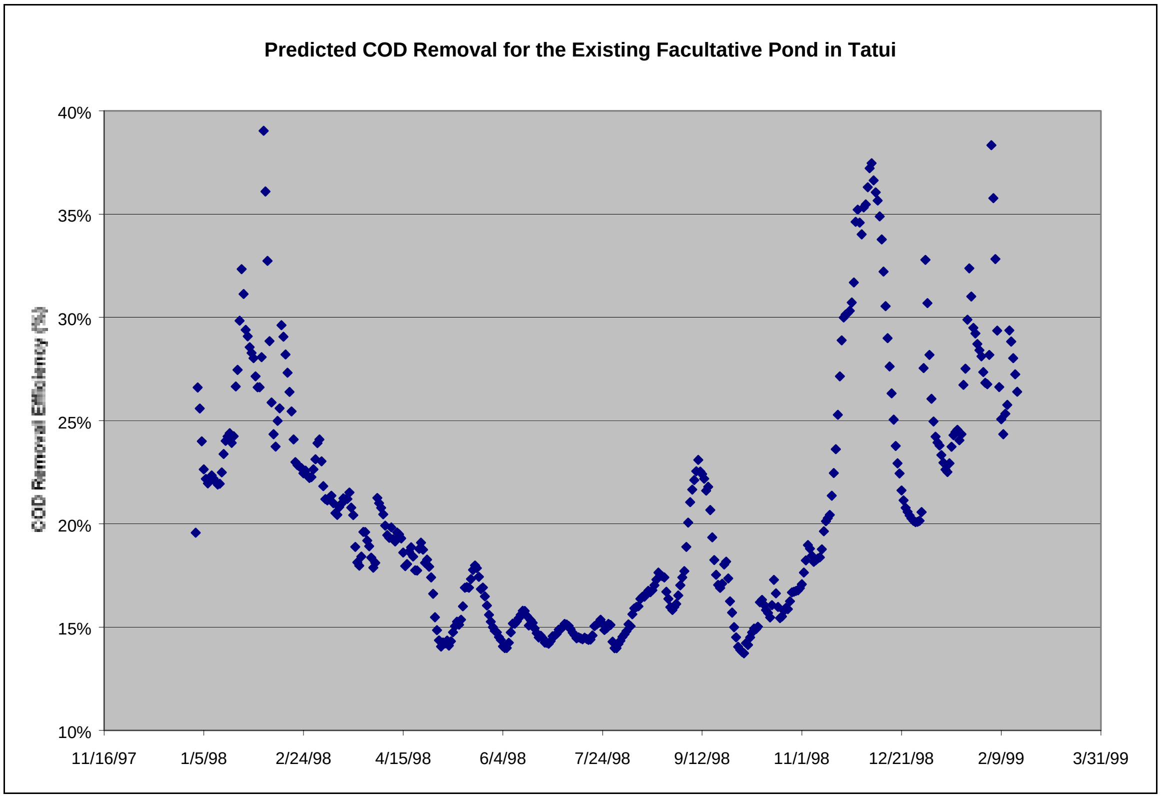

Figure 4-3: Predicted COD Removal for Existing Facultative Pond in Tatui

The yearly averaged final effluent predictions satisfy the required effluent quality limit of 60 mg/L of BOD for both alternatives 1 & 2.

Table 4-3: Predicted Effluent Qualities from Ponds

Treatment Alternative	Average Influent BOD5 [mg/L]	CEPT Effluent BOD5 [mg/L]	First Pond Effluent BOD5 [mg/L]	Existing Facultative Pond Effluent BOD5 [mg/L]
1	276	138	74	59
2	276	110	59	47

5 FINANCIAL ANALYSIS

5.1 Introduction

The purpose of this chapter is to evaluate the financial costs of the MIT-CEPT designs (Alternatives 1 and 2), and to compare them to the costs of the SABESP-Aerated lagoon design. To accomplish a fair comparison, costs will be tallied using the same assumptions that the SABESP design does.

In Brazil, the design consultant firm is responsible for quantifying the design in terms of specific tasks (i.e. units of labor, equipment and material usage). Each of the tasks is included in a database managed by SABESP. This database provides cost per unit of these services.

5.2 Methods for Cost Comparisons

The SABESP cost database presents the aggregated cost to accomplish a specific task. For example, the cost of moving one cubic meter of soil includes the cost of labor and transportation of the soil, and its unit is currency per cubic meter of soil, per kilometer of distance to transport.

The method to compare the costs between the two MIT-CEPT designs and the SABESP design therefore relies on a comparison between the various SABESP units. Consequently, the MIT-CEPT design costs will be estimated using the SABESP standard, in order to provide an accurate cost comparison.

The cost comparison will rely on a quantitative comparison using the various SABESP task units (i.e. volume of earth moved, foundation reinforcements, etc.). The CEPT budget will neglect the differences in the predicted pipe installation as well as all items related to the power station construction since it would not be representative.

SABESP's quantifying system assumes that there are no unknown variables for the construction. This means, for instance, that all the information regarding quantities of rock demolition, although estimated, will represent the actual amount of worker and machinery rent hours, as well as the volume of rock demolished and transported.

Regarding special units for these quantities, global items (represented as GB, which is the SABESP unit for “global”) include all services and/or amount of supplies necessary to accomplish the entire specified task. The lists presented in the next pages use the same nomenclature as SABESP’s lists.

5.3 SABESP Pricing Structure

The SABESP pricing system represents the estimated price to accomplish a unit service including all the necessary related items. For example, the price of soil removal deeper that 4 meters includes: worker-hours, machinery-rent-hours, and material used. However, the unit is m^3 and the price corresponds to soil digging beneath four meters.

With this pricing structure, it is difficult to estimate price reduction factors, such as economies of scale, or the construction company profit. The service taxes also vary geographically, but the SABESP prices remain the same.

In this study two out of four SABESP’s lists of services and equipment are important: list-3, hydraulic equipment, and list-4, electric equipment. The price unit for these lists is GB (global). The services that are related to the installation of all the equipment included in those lists are evaluated in two different SABESP budget items. These items bear titles such as “installation of hydraulic equipment of list-3” or “installation of electrical equipment of list-4”.

List-3 includes several hydraulic items and the aerators required equipment. There are three specifications related to aerator items: floating aerators (15hp), iron cables (diameter 3/16"), and aerator fixing structures. These items will be excluded from the Alternative A2 and A3 CEPT design budget, and the installation price will be reduced accordingly.

List-4, with all electrical equipment necessary for the whole wastewater treatment plant, will remain the same. However, it is important to notice that since no aerators will be used in the CEPT treatment system alternatives, there would be a slight decrease in this price. By using the same price, the MIT-CEPT budget will therefore be conservative. This can be seen as a buffer for any unexpected costs.

Moreover a fifth price list will be included for the MIT-CEPT design alternatives. This list will consist of all the equipment required for the coagulant addition: pumps, flow meter and storage tanks.

5.4 The Design Alternatives for Upgrading Tatui's Wastewater Treatment Plant (WWTP)

The Tatui treatment system presently is composed by two lagoons, one anaerobic and the other facultative, in a 5 ha area in the suburbs of the city. The efficiency and the condition of the CEAGESP-WWTP was evaluated by Milton Tomoyuki Tsutiya and Orlando Zuliani Cassettari in 1992 (c.f. Appendix-D for the translation of their work). In 1998, a bid was placed in order to upgrade one of the City's WWTP (called CEAGESP).

The MIT CEPT Project consists of the study of the present design of the WWTP in Tatui and three plans for upgrading the system. The design presented by SABESP in the bid for the system's upgrading, was designed by Ampi and approved by Eduardo Pericle Colzi in 1996. Alternatives 1 and 2 rely on Chemically Enhanced Primary Treatment (CEPT) for the removal of total suspended solids (TSS) and its related biological oxygen demand (BOD) as explained in Section 2 of this report.

For the schematic of the three treatment plans and the area distribution see Section 2. Table 5-1 presents the areas and depths required for each treatment for the three alternatives.

Table 5-1: Design lagoon areas and depths of treatment plans

Treatment	SABESP Area (ha)	Design Depth(m)	Pre-Pond Area (ha)	CEPT Depth(m)	In-Pond Area (ha)	CEPT Depth(m)
Aerated	1.2	3.5	NA	NA	NA	NA
Settling	0.8	4.0	NA	NA	0.6	4.5
Anaerobic	NA	NA	1.8	4.5	1.8	4.5
Facultative	NA	NA	2.5(*)	1.5	2.5(*)	1.5

(*) EXISTING LAGOON

It is important to notice that the facultative lagoon of 2.5 ha of alternatives 1 and 2 is only used for polishing and clarifying the effluent from the anaerobic lagoon and therefore will remain the same as it is presently. Thus, no capital investment will be done in order to improve the facultative lagoon condition.

In the next section we estimate the capital cost (CC) and the operation and maintenance costs per month (O&M) for SABESP Design and Alternatives 1 and 2. The first section presents the final table of total CC of the three alternatives. The services and supplies are grouped in 15 group items. In terms of CC, the differences in quantities between the three alternatives are seen in 5 items: *Soil Movement, Foundations and Structures, Supplies, Sludge Treatment, and Other* (installation of hydraulic and electric equipment).

Total Capital Cost for Treatment Plans

The budget for the construction of lagoons treatment systems, neglecting the land price, consists essentially of land movement, foundations and structures, and wastewater treatment. The following sections divide the construction budget of the three alternative WWTP in 15 main items. Figure 5-1 shows the general distribution of costs for the three treatment plans.

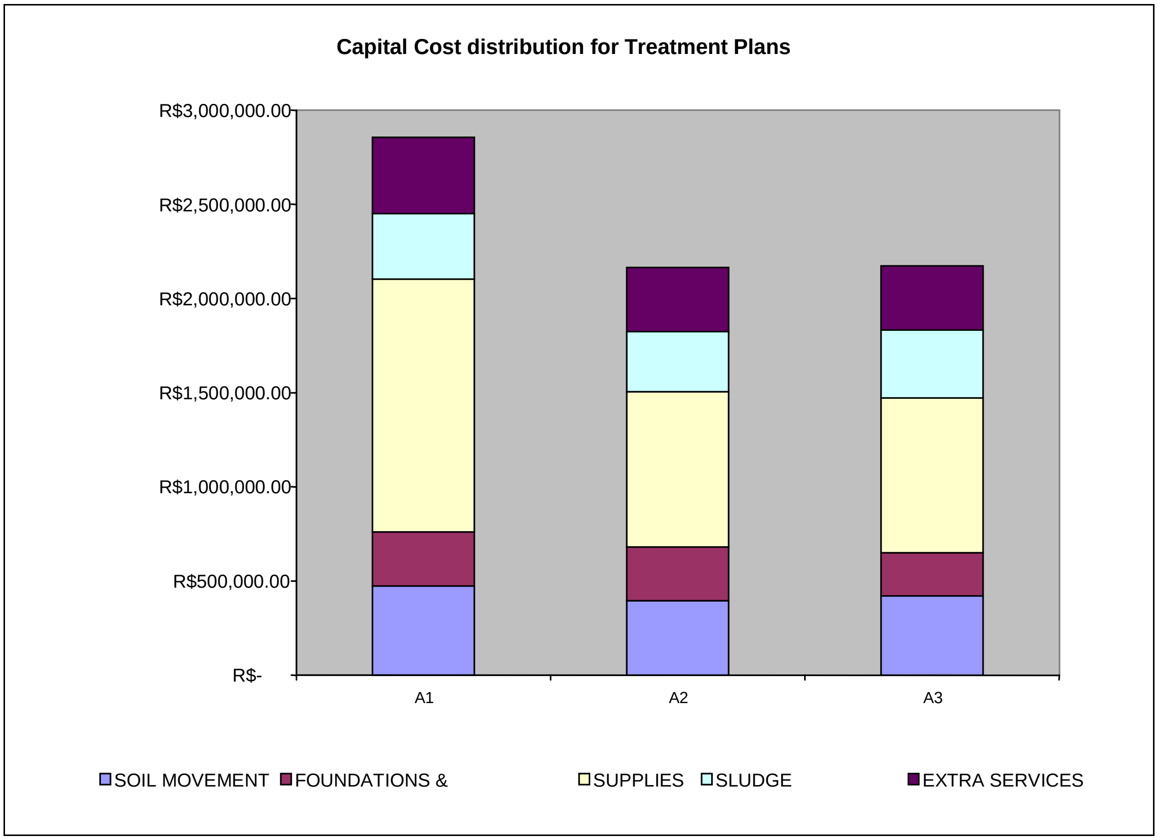

Figure 5-1: Capital Costs of Alternatives 1, 2 and 3

As mentioned aboveysis, the cost of these WWTP alternatives are ultimately a comparison between SABESP's required services and materials, including equipment, and those of the CEPT designs. Table 5-2 presents the construction budget for the three alternatives. All prices are given in Brazilian Reais. The exchange rate used is R$1.20 per US$ 1.00 referring to the year 1998.

Table 5-2: Construction Costs for A1, A2 and A3

		SABESP	Alternative 1	Alternative 2
Front	Specification	Total Price [R$]	Total Price [R$]	Total Price [R$]
1 Total	Adm/General facilities	16,926.22	16,926.22	16,926.22
2 Total	Technical Services	8,045.70	8,045.70	8,045.70
3 Total	Preliminary Services	74,520.00	74,520.00	74,520.00
4 Total	Soil Movement	473,825.53	396,137.02	421,700.52
5 Total	Drainge & Pumping	3,581.50	3,581.50	3,581.50
6 Total	Foundations & Structures	287,347.45	286,062.43	228,576.83
7 Total	Pipe Installation	14,059.23	14,059.23	14,059.23
8 Total	Pavement	37,995.00	37,995.00	37,995.00
9 Total	Alvenaria	35,458.81	35,458.81	35,458.81
10 Total	Painting	85,715.79	85,715.79	85,715.79
11 Total	Urbanization	27,422.46	27,422.46	27,422.46
12 Total	General services	1,500.00	1,500.00	1,500.00
13 Total	Supplies	1,342,490.61	823,349.26	823,349.26
14 Total	Special services	1,270.00	1,270.00	1,270.00
15 Total	Sludge treament	347,103.93	320,000.00	360,765.97
16 Total	Other	98,620.17	31,526.40	31,526.40
Total Global	ETE – CEAGESP (1st stage)	2,855,882.39	2,163,569.82	2,172,413.69

The CC of Alternatives 1 and 2 of CEPT WWTP upgrading are about 30% cheaper than the SABESP design. For the detailed costs for these alternatives, refer to Appendix-C. Below we describe the items where CETP technology represents a capital cost change. For the detailed budget for each item of all three alternatives, refer to Appendix -C.

Construction Budget

This section describes the 3 budgets of the three alternatives. Most of the item's prices were based on using Brazilian suppliers. All the prices refer to 1998, when SABESP evaluated the SABESP budget. All the pipe installations for the filter press, CEPT tank and CEPT storage facility are included.

Soil Movement

Generally, the most expensive part of a lagoon system WWTP construction budget, 30% to 35%, is the soil movement. Since, lagoons are essentially topographic arrangements, its price is related to the volume of soil displaced (excavated, borrowed and filled), the amount of sludge dredged (and disposed), trench excavation (for the pipe system), and dikes (compacted and protected with pre-cast concrete slabs). Since lagoon treatment relies on natural stabilization, the design criterion for anaerobic lagoons is volumetric BOD load. Aerated lagoons, on the other hand, base their design criterion on power for the mechanical aeration to stabilize (oxidize) the organic matter in the wastewater. The settling lagoons have to maintain a minimum hydraulic detention time for the maximum flow. The result is that Alternatives 1 and 2 require 90% and 120%, respectively, the area of SABESP Design. Refer to Section 3 of this report for the specifications and details of the design of the three alternatives. Appendix-C presents the required areas, depths as well as an estimate of the amount of required services and supplies, such as, excavation and dredging volumes, concrete slab areas, dike lengths, etc.

Foundations and Structures

Foundations and structures represent 20% to 30% of a WWTP budget. This item contains the concrete related sub-items including services and materials for the construction of the facility's devices and sub-items related to foundations. There are only two differences between the Alternatives in respect to structures and foundations. In Alternative 1, a R$ 80,000.00 CEPT tank is included and the length (and total price) of dikes is about 88% less than SABESP Design. Since the chemical addition for Alternative 2 will occur in the inlet of the anaerobic lagoon, no CEPT tank is necessary, however, the dikes' cost is about one fourth of the SABESP cost for the CEPT alternatives.

Supplies

In the item called Supplies, all the piping system is included. We estimate the same budget for the pipe network for the three alternatives, however the lists of hydraulic equipment do change (List 3, Appendix-C). In SABESP Design, List 3 includes general hydraulic parts and all the aerators related equipment, for Alternatives 1 and 2 the price of this equipment is deducted from the final price of the list (List 3 without aerators, Appendix-C). List 4, with the electrical equipment, assumes the same price for the three plans. List 5 consists of the pumps, chemical tanks and other equipment related to the CEPT technology so these costs are included in Alternatives 1 and 2. The overall cost of supplies is approximately the same for Alternatives 1 and for the SABESP Design. However, the cost of supplies for Alternative 2 is 40% less than SABESP's. Appendix-C shows the description and price of all the lists of prices (Lists 3, 5 and 6, Appendix-C).

Sludge Treatment

The item called Sludge Treatment includes all the structure, foundations and equipment for the sludge handling. The sludge treatment system used for SABESP Design is the same as the one used for Alternative 2, i.e. a pump boat to pump the sludge from the bottom of the lagoon and SDBs to dewater the sludge. Alternative 1, however, uses composting instead of pump boat. For Alternative 2 the sludge is pumped from the CEPT tank to the filter press.

After dewatering it is mixed with wood chips or ashes and set on windrows for the final stabilization. The price of composting, is a rough estimate since there is no previous Brazilian experience using it in municipal wastewater treatment.

Installation of Hydraulic and Electric Equipment

The installation of hydraulic and electric equipment item is considered as 10% and 20%, respectively, of the price of the equipment.

Operational and Maintenance Costs

This section will price the monthly operation and maintenance expenditures of operation of the three plans. SABESP's design requires a complex operation because of the aeration system. The calibration of aerators requires a permanent efficiency control executed by the WWTP staff. The calibration of this equipment is based on the efficiency of the volume of air mixed, however, their efficiency changes during their lifetime use. For the O&M evaluation of the SABESP Design the lifetime of the aerators is considered as 10 years. In SABESP Design the aerators energy consumption and maintenance are estimated to be around R$25,000.00. On the other hand, operation of a lagoon wastewater treatment system is simple. In Brazil, it generally requires one operation assistant to control the vegetation growth at the borders of the lagoons and the alga growth in the lagoons and to maintain the site (fences, cleaning of facilities, etc.). The operation of a CEPT tank was designed to require one sludge removal per day. The experience of the operation staff also could reduce the chemical consumption by learning about low loading hours. For alternatives 1 and 2, the chemical concentration would initially be 50 ml/L and 30 ml/L, respectively, 12 hours per day. The pump boat used in Alternatives 2 and SABESP Design to remove the sludge from the settling lagoons and its O&M would cost R$ 3,000.00/month.

The total O&M cost varies among the three plans. The price of the CEPT's O&M alternatives is lower because is consists mainly of labor, which in Brazil is low. However, for Alternative 1 the price of O&M is 2.3 times the O&M cost for Alternative 2 due to the inclusion of composting which is estimated as the salary of two extra assistants, the maintenance of the filter press and the bio-solids handling (tractor and conductor). Nevertheless, both final CEPTs alternatives' O&M costs are lower than SABESP's. Alternative 1 is 28% lower and Alternative 2 is 69% lower.

SABESP Design

In the SABESP design the considerations for the monthly cost of the O&M of the facility essentially include staff salary, aerators energy consumption and maintenance. Table 5-3 shows the O&M for SABESP Design.

Table 5-3: O&M for SABESP Design

	Unit	Quantity [month^{-1}]	Price/unit [R$]	Total price/month [R$]
Pump Boat (consumption & maintenance)	R$/month	1	3000	3,000
Energy consumption of aerators	Hp	300	76.67	23,000
Assistants	R$/month	2	1,200	2,400
Engineer	R$/month	1	3,000	3,000
TOTAL				**31,400**

Alternative #1

The operation of a CEPT facility is simple and relatively cheap. The typical CEPT plant would have a monthly cost as a function of the price and quantity of chemicals used as well as the operational staff salary. The pumps used to ensure the proper chemical dosage and mixing require very low energy consumption and the price of chemicals represent around 10% of the final cost of O&M.

Table 5-4 has the cost of the optimum dosage of iron-salts chosen for the treatment. It was determined through jar tests (c.f. Appendix-A2). The price of the ferric chloride was considered US$ 180/ton.

Table 5-4: Optimum dosage of iron-salts chosen for the Alternative 1 treatment (Assuming a 50 mg/L dosing of $FeCl_3$)

Mass of chemical		Volume of chemical		Price
Kg/day (dry)	Kg/month (dry)	L/day	L/month	R$ / month
348	10,433	248	7452	1,565

The price of composting was estimated based in two main aspects: the resulting bio-solids have no market and the composting process is basically hand labor. Sludge from WWTP in Brazil is not commonly commercialized for agriculture and, in this budget analysis, it is not considered. The composting requires two assistants for the filter press and a tractor with operator. The time constrains for the preparation of this project and the MIT CEPT Project resulted in a rough estimation of composting price in Brazil. Table 5-5 presents an estimation of monthly cost of Alternative 1.

Table 5-5: O&M for Alternative 1

	Unit	Quantity	Price/unit [R$]	Total price/month [R$]
Energy consumption of pumps	R$/month	1	500	500
Chemical consumption	Kg	10,433	0.15	1,564.92
Energy consumptio of dewatering system	R$/month	1	2,000	2,000
Composting (tractor maintenance& fuel, operator,and related items)	R$/month	1	10,000	10,000
Pump Boat energy&maitenance	R$/month	1	3,000	3,000
Assistants	R$/month	2	1,200	2,400
Engineer	R$/month	1	3,000	3,000
TOTAL				**22,464.92**

Alternative #2

For Alternative 2 the optimal chemical concentration required for the settling of the particles in the lagoon is much lower than in Alternative 1 since the detention time is much higher than in the settling tank. The detention time in the Alternative 1 is around one hour while in the Alternative 2 it is around 1,5 days. Refer to the Design Chapter for details about optimum chemical dosage. Table 5-6 is the cost of the optimum dosage of iron-salts and polymer chosen for the treatment.

Table 5-6: Optimum dosage of iron-salts chosen for the Alternative 2 treatment (Assuming a 30 mg/L dosing of $FeCl_3$)

Mass of chemical		Volume of chemical		Price
Kg/day (dry)	Kg/month (dry)	L/day	L/month	R$ / month
209	6260	149	4471	1,127

Alternative 2 requires mainly an assistant to maintain the facility, the chemicals, and an engineer to supervise. Table 5-7 shows the monthly expenditure for Alternative 2.

Table 5-7: O&M for Alternative 2

	Unit	Quantity [$month^{-1}$]	Price/unit [R$]	Total price/month [R$]
Energy consumption of pumps	Hp	1	500	500
Pump Boat	R$/month	1	3,000	3,000
Chemical consumption	Kg	6,259.7	0.18	1,126.74
Assistants	R$/month	2	1,200	2,400
Engineer	R$/month	1	3,000	3,000
TOTAL				**10,026.74**

5.5 Concession Analysis for Tatui

There are several possible criteria for the choice of the best treatment plan, the one with the best overall efficiency, the one with the minimal environmental impact, the one with the minimal required area, etc. Since the overall efficiencies of the three plans are similar

(Gotovac, 1999), and they occupy the same area, the goal in this project is to select the best alternative using cost as the screening mechanism. To accomplish it, investments parameters such as capital cost, operation and maintenance cost, present value, payback period, internal rate of return and benefit cost ratio are compared in a 10-year concession scenario.

A concession is an effective way to analyze the investment for the construction and operation of a WWTP in Brazil. In this section, I present a comparison of the three hypothetical concession alternatives as a means to evaluate the CEPT designs. To accomplish this financial comparison, I chose three investment parameters: present value (PV), internal rate of return (IRR) and payback period (PP). The revenues of these hypothetical concessions would come from 25% of a wastewater treatment tariff of R$ 0,50/m3, i.e. R$ 0,125/m3 of wastewater treated. I am assuming an average population growth for the concession period of 1,5% per year. The PV of a project is the most important parameter for an investment analysis, it considers the CC plus the entire O&M annual costs. Both CEPT designs, alternatives 1 and 2, have significant reduction in PV (23% and 46%, respectively), relative to SABESP Design largely because of the difference in the O&M costs. The IRR is the standard comparison index for long term projects. In general, a concession IRR has to be greater than 15% to be considered profitable. The PP is the number of years required for an investment to be "bankable." This means the number of years to pay the initial investment and return some profit to the investor after the concession is over. In Alternative 1, the revenues will never pay the initial investment from the NPV point of view. Table 5-8 presents the PV, IRR and PP for the three plans considering a 10-year project life and a financing of 12% percent per year. This relatively short project design period of 10 years was in order to return the operation of the facility shortly to the Municipality. There is no insurance or inflation considered, since both these items can vary considerably.

Table 5-8: PV, IRR and PP for thr three treatment plans

	SABESP [R$]	Alternative 1 [R$]	Alternative 2 [R$]
O&M [/year]	376,800.00	269,579.04	120,320.91

Capital Cost (CC)	2,855,882.39	2,163,569.82	2,172,413.69
Present Value	4,984,886.43	3,686,751.52	2,852,253.66
CC amortization (/year)	505,445.96	382,917.60	384,482.82
Payback Period (years)	Non payable	8	3
Internal Rate of return (IRR)	3%	15%	23%
Present Value of Revenues	6,180,154.76	6,180,154.76	6,180,154.76
Benefit Cost Ratio	1.2	1.7	2.2

5.6 Summary

This chapter describes the budget comparison of two CEPT designs and the proposed SABESP design. The most important expenditures of a concession are the O&M costs, which, as estimated in this chapter, are much lower for the CEPT alternatives. O&M is an important parameter because when calculating the present value of a project, which is composed of capital cost and the O&M expenditures during the life of the project, it represents from 74% (for SABESP Design) to 57% (for Alternative 2) of the present value of the investment. Section 5 of this report shows the possible savings using CEPT in lagoon treatment systems when compared to aerated lagoons followed by sedimentation basins. Finally the chapter describes a hypothetical concession of Tatui WWTP as a means to compare the three alternative investments.

5.7 Conclusion

Financial parameters (PV, IRR, PP and BC) were used as the most important criteria to screen treatment alternatives. Applying these criteria leads us to the conclusion that Alternative 2 is the most suitable one, since the O&M cost for this alternative is around 1/3 of SABESP's expected monthly expenditure. As for Tatui, many other small cities in developing countries are waiting for appropriate technologies, such as CEPT, to allow their system to operate efficiently without major investments.

6 DISCUSSION AND RECOMMENDATIONS

Three designs were reviewed in this report for the upgrading of a wastewater treatment facility in Tatui, Brazil. The design proposed by SABESP, the São Paulo State environmental agency, is composed of mechanically aerated lagoons followed by settling lagoons. Alternatives 1 and 2, proposed by the MIT-Group, center on Chemically Enhanced Primary Treatment (CEPT). For both Alternatives 1 and 2, a series of lagoons is provided to further the treatment. Alternative 1 deals with the addition of chemical coagulants before the lagoon system, while Alternative 2 deals with the addition of chemical directly in the first lagoon of the lagoon system.

The three plans were compared in three ways: for removal efficiency of organic matter measured as BOD, for sludge handling options and most importantly, for costs. Regarding treatment efficiencies, the three plans are in the same range of BOD removal. As for O&M, Alternative 2 is by far the least expensive, and the easiest to operate and maintain, mostly due to its lack of mechanical devices. Furthermore, Alternative 2 is the least dependant on electricity. The chemical pumps can be powered by a small generator in case of a power shortage, whereas aerators, the most important source of treatment for the SABESP plan, necessitate 300 hp to keep them running at design specification. And if the chemical pumps for Alternatives 1 & 2 break, it is a very small capital investment to purchase an extra chemical pump as a back-up. It was found that Alternative 2 is the most cost effective. Indeed, while achieving the required level of removal, its capital cost is only 57% of the cost of the first plan proposed by SABESP, while the O&M costs only represent 32% of the SABESP design.

Moreover, Alternatives 1 and 2 use the existing facultative pond, thereby decreasing the construction costs, and using the area more efficiently. Alternatives 1 and 2 are also highly adaptable to increases in inflow rate and organic loading. Indeed, future upgrading of the CEPT facility only requires increasing the chemical dosage and dredging the existing facultative pond to increase its removal efficiency (c.f. Section 4 of the report).

This CEPT facility can also act as a pilot plant for other small cities in Brazil that have overloaded pond systems, since CEPT has never been applied to lagoon systems in Brazil. The CEAGESP facility will therefore potentially serve as an example of CEPT application to lagoon systems. The additional cost and effort of monitoring the overall CEPT lagoon system performance, including the rate of sludge accumulation will be more than repaid by its usefulness in future wastewater treatment system upgrades.

Many countries in this day and age are facing similar domestic infrastructure problems. One main problem is increased wastewater flow due to increased population. The average city in Brazil has a yearly population increase of about 2%. This population increase entails an increase in water consumption, and thus also an increase in wastewater production. Wastewater treatment facilities are largely absent in Brazil (10~20% of coverage), and since many of those that exist are old and overloaded, they will necessitate upgrading or replacement. A common practice has been to add aerators to the first lagoon, and then allow the wastewater to settle in subsequent lagoons in quiescent waters. The problem with this is the capital and O&M costs associated with the aerators. A better upgrading option is in-pond CEPT. As is shown by the cost analysis of Chapter 5, in-pond CEPT, in the Tatui case, is the most cost-effective.

In conclusion, CEPT is an appropriate and effective technology for a situation such as this, and should therefore be used before mechanical aeration and activated sludge treatment options.

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APPENDIX-A-1: CHEMICALLY ENHANCED PRIMARY TREATMENT

Introduction

The purpose of this appendix is to describe several CEPT facilities in the United States.

Existing CEPT Facilities

See Table A1-1 and the following discussion for data and descriptions of CEPT facilities currently being operated in the USA.

Table A1-1: Characteristics of Existing CEPT Facilities

PLANT	% REMOVAL (YEARLY AVERAGES)					COAGULANT DOSAGE [mg/L]	POLYMER DOSAGE [mg/L]
	TSS	BOD_5	FOG	TKN	PHOSPHORUS		
Point Loma	86	59	70	-	92	25	0.1
Orange County	75	50	53	15.4	NA	20	0.2
JWPCP	78	42	-	-	-	0	0.15
Hyperion	88	54	-	-	-	6	0.08

Point Loma

The Point Loma Plant is located in (Point Loma) San Diego, CA and is one of the largest operating CEPT plants in the world. Point Loma serves 1.8 million citizens in Southern California and treats approximately 187 million gallons per day (MGD), with a peak handling capacity of 240 MGD (910,000 m^3/d). The facility is currently undergoing reconstruction/expansion, which will add a couple of sedimentation basins, and anaerobic sludge digesters.

The Point Loma facility was given a waiver from secondary biological treatment since it is a coastal city and its utilization of CEPT gave it a high quality effluent suitable for discharge to the Pacific Ocean. The California Ocean Plan, which Point Loma abides by, calls for a process in which the effluent would not affect the ocean's dissolved oxygen concentration by more than 10% below ambient levels (Harleman & Murcott, 1992). Table A1-2 presents the characteristics of Point Loma's CEPT facility.

Table A1-2: Point Loma Influent and Effluent Characteristics in 1998

Parameter	Influent Concentration (mg/L)	Effluent Concentration (mg/L)	%Removal
TSS	284	39	86
BOD_5	259	106	59
TP	6.2	0.5	92
FOG	32.3	9.9	70

As Table A1-2 indicates, Point Loma has a high removal efficiency of TSS, TP, and FOG. The removal efficiency of BOD_5 is not as high, compared to full conventional primary plus secondary biological treatment facilities, but is sufficient to be in compliance of the California Ocean Plan.

Point Loma's facility includes bar screens, followed by an aerated grit chamber, followed by rectangular chemically enhanced primary sedimentation basins (12 basins). The influent receives sodium hydroxide (NaOH) and hydrogen peroxide (H_2O_2), to control odor problems. In the grit chamber, the collected grit is dewatered by a cyclone separator. The separated grit is hauled off to a hazardous waste landfill in Arizona, and the supernatant is returned to the head of the plant. The iron salt ($FeCl_3$ at a dosage of 25 mg/L) is added before the aerated grit chamber, which maintains a velocity of 2 ft/s. The $FeCl_3$ is stored in two 10,000-gallon tanks, which are refilled about two times daily, and fed to the influent by 2-horsepower centrifugal pumps. The polymer (anionic) is added at the head of the rectangular sedimentation basins at a dosage of 0.10 mg/L. The polymer is stored in a tank of a capacity of 6500 gallons which is refilled once every three to four days. The polymer is pumped to a smaller tank where it is made up and then pumped to the flumes of the primary clarifiers for injection. The 12 rectangular sedimentation basins have an average detention time of 1.5 hours at average flow, and contain three cells per basin. The basins are equipped with baffles to ensure horizontal flow, although short-circuiting still occurs. A revolving rake collects the sludge and floating scum. Their average overflow rates are about 2000 gpd/ft^2 (81.4 m/d), with a range of 670 – 2411 gpd/ft^2, depending on the flow.

Six anaerobic digesters, following a two-stage process, are currently treating the sludge. Four of them are first-stage digesters, and the subsequent two are the second-stage digesters. The sludge residence time for the first stage is approximately 15 days, and 3-5 days in the second stage. Each digester has a volume of approximately 4 million gallons, which process a total of 145 dry tons of sludge per day. The methane (CH_4) produced from the anaerobic digesters is currently being burned off due to the reconstruction of the plant which recently tore down the generation facility in order to a build newer and more efficient generator facility. Upon completion of reconstruction, the CH_4 will be utilized to generate power for the whole plant, with the excess being sold to the local power utility (San Diego Gas & Electric). This power generated is also used to heat the sludge prior to entering the anaerobic digesters in order to make the influent sludge the same temperature as the sludge present in the digesters.

Operational and maintenance problems, as with all wastewater treatment facilities, do exist. The sedimentation basins are cleaned and scheduled for other maintenance periodically, usually every three months, with a major overhaul once a year. The basins are only shut down if a major problem comes up. Many little problems exist, such as failure of chemical pumps and chains for the sedimentation basins. The main maintenance involved with the anaerobic digesters is associated with the influent sludge heaters.

The plant's effluent is screened to remove any grease that may pass through the system, and is not chlorinated. The ocean outfall at Point Loma, as in many other plants, is discharged at a great distance from the coast (here, 4.5 miles), and at that point is dispersed through diffusers.

OCSD

The Orange County Sanitation District (OCSD)[11] in Orange County, California utilizes CEPT on 100% of its influent, in which 50% of the advanced primary effluent undergoes secondary biological treatment (which takes place in two plants). The remaining 50% are discharged directly into the ocean through its ocean outfall with effluent from secondary treatment. The data provided in Table A1-3 is the OCSD's CEPT data only, and therefore does not include the effects of the secondary biological treatment. The primary basin design overflow rates are 700 gpd/ft^2 (30 m/d). The amount of wastewater treated is an average of 240 MGD. The outfall is five miles long and has a one-mile long diffuser on the end of it with 500 small (2-inch) ports.

Table A1-3: OCSD Influent and Effluent Characteristics in 1998

Parameter	Influent Concentration (mg/L)	Effluent Concentration (mg/L)	%Removal
TSS	240	60	75
BOD_5	230	115	50
TKN	39	33	15.4
FOG	51.2	24.3	53

The coagulant used is ferric chloride ($FeCl_3$), dosed at 20 mg/L which is fed upstream of the grit chamber. The polymer used is an anionic polymer, dosed at 0.2 mg/L, which is fed at the inlet to the primary clarifiers. Note, again, that all of the data presented is for the efficiency

[11] The data on OCSD was provided through contact with Mr. Robert Ooten from the facility.

of the chemically enhanced primary clarifiers only, that is, not including the performance of OCSD's secondary biological treatment.

JWPCP[12]

The Joint Water Pollution Control Plant (JWPCP) in Los Angeles County, CA treats an average of 380 MGD (1.13 Mm^3/d). The plant uses CEPT in conjunction with secondary biological treatment. Here, approximately 60% of the CEPT effluent undergo secondary biological treatment; the combined effluent is discharged into the Pacific Ocean. JWPCP has two outfalls. One outfall is approximately 2250-m long, 60-m deep and equipped with a 1340-m diffuser. The second outfall is approximately 2500-m long, 60-m deep and equipped with a y-shaped diffuser with each arm extending approximately 670m. The plant operates at an overflow rate of greater than 1300 gpd/ft^2 (53 m/d) (Morrissey, 1990).

HTP[13]

Hyperion Treatment Plant (HTP) is located in Playa Del Rey, CA. HTP serves over 4 million customers from the Los Angeles area and treats an average of 360 MGD (1.36 Mm^3/d). The plant is a CEPT and (soon to be) full secondary facility.

The average influent BOD is 290 mg/L, with an average effluent BOD of 135 mg/L. The influent TSS is 320 mg/L, with an effluent TSS of 40 mg/L. The chemical coagulant at HTP is dosed upstream of the plant.

[12] Data on JWPCP obtained from Morrissey, 1990.
[13] Data provided via conversation with Mr. Mike Noguchi and Y.J. Shao of HTP (March, 1999).

APPENDIX A-2: JAR TESTS & CHEMICAL ANALYSIS

Introduction

The purpose of this chapter is to demonstrate the analysis and results of the jar tests and its accompanying data.

Since CEPT is the addition of chemical coagulants and polymers to enhance the removal of TSS and its associated BOD_5 and TP, it is important to evaluate and analyze the chemicals utilized, as well as the analysis of the data obtained.

Methods and Procedures

The six parameters analyzed in the CEAGESP facility were pH, TSS, chemical oxygen demand (COD), total phosphates as PO_4—(TP), total sulfates as SO_4^{2-}, and temperature. The parameters were chosen based upon the following factors: due to time constraints and technological deficiencies of the lab, BOD_5 was not able to be determined; the parameters are representative of the behavior of the lagoon and the efficiency of the chemicals/polymers utilized in the jar tests; and because many of these parameters (pH and temperature) indicate the condition of the present system.

Laboratory Study and Setup

The purpose of the field trip was to asses the efficiency of the CEAGESP treatment lagoon system in Tatui, and to conduct jar tests on the raw influent to predict the efficiency of a proposed CEPT treatment plant option, as well as selecting the optimum chemicals and dosages.

The tests conducted in Tatui concentrated on assessing the COD, and TSS of samples from both the pond system and the jar tests.

Jar Tests

The jar test is a common laboratory procedure, which will be used to determine, empirically, the optimum operating condition for Alternatives 2 and 3. The jar test procedure is presented in Table A2-1. The table shows the chief mixing regime used. It should be noted that different mixing regimes were used, such as when the recycling of chemical sludge was tested, or when polymer addition was omitted.

Table A2-1: Jar Test Mixing Regime

Steps	Mixing Intensity (rpm)	Mixing time
Raw water	100	15 sec
Primary Coagulant Added (metal salt)	100	30 sec
Polymer added	100	30 sec
Medium mixing	70	2.5 min
Slow mixing	30	2.5 min
Settling	0	5 min

COD Analysis

The COD test measures the oxygen equivalent of the organic matter in a wastewater sample that can be oxidized chemically using dichromate in an acid solution ((Franzini *et al*., 1992). COD was measured using the Hach's adaptation of Standard Methods.

TSS Analysis

TSS were measured using Standard Methods. The procedure involves filtering the samples and drying the filters, with subsequent weighing of them to determine the TSS.

Lagoon Sampling

The stay in Tatui involved seven days of lagoon sampling. The following section will explain the sampling procedure.

A typical sampling exercise would start by measuring the height of water flow through the Parshall flume. The group would then proceed by taking a forty-liter sample out of the splitter box. The sampling of these 40L would be carried out by using a bucket to collect raw sewage, and dumping its contents into one of four 10L jugs. Smaller 1L samples would also be taken at the outlet weir of the anaerobic pond and at the outlet weir of the facultative pond. Temperature and pH were measured at each sampling point.

24-hr Lagoon Test

Part of the lagoon sampling entailed a 24-hr sampling test to assess the diurnal variations of the influent characteristics and the performance of the lagoon systems. Table A2-2 displays the results from the 24-hr sampling test.

Table A2-2: 24-Hour Lagoon Sampling Data[14]

	Anaerobic Lagoon	Facultative Lagoon	Total System
COD %Removal	35.5	26.2	52

[14] These results do not reflect the fact that part of the anaerobic effluent did not undergo further treatment.

TSS %Removal	25.3	43.3	54.5

Chemical Coagulants

The chemical coagulants utilized in the jar tests were ferric sulfates and ferric chlorides. These were the main coagulants tested, although, a few aluminum salts were also tested in compliance with SABESP's requests. The chemical coagulants will be referred to by their commercial names: Sanechlor, Kemwater, NHEEL, Liex, and Eaglebrook. For data on all of the jar tests, the chemical coagulants and polymers used, see the jar test database at the end of this appendix (Appendix A-2).

Sanechlor

Sanechlor jar tests were run on the 16th and 17th of January. This chemical is produced in Sao Paulo, Brazil by Sanechlor Produtos Quimicos Ltda. This chemical coagulant is 40% ferric sulfate (solids) by weight and is 11% iron by weight.

As the results show in the jar test database, a satisfactory removal of COD and TSS are achieved by a dosage of Sanechlor higher than that of the three ferric chlorides tested. Although the dosage is higher, the price of the chemical is approximately 60% of the ferric chlorides (a major factor in selecting the best chemical coagulant). The concentration range of 40 mg/L to 60 mg/L represents a removal efficiency of 38%-67% of COD and 64%-94% removal of TSS. To determine the ideal combination and dosage of chemical coagulant with polymer, a jar test was run with 46 mg/L of Sanechlor and a polymer dosage of 0.25 mg/L. This jar test removed 92% of TSS, but due to lack of COD reagent, a COD analysis was not performed. A problem with Sanechlor, as with Kemwater and all other sulfate-containing chemical coagulants, is the sulfate (SO_4^{2-}) in the compound. The sulfate in Sanechlor disassociates from the $FeClSO_4$ and upon entering anaerobic conditions is oxidized and forms hydrogen sulfide (H_2S), thus increasing odor problems, which would most definitely occur in Alternative 2.

Due to the fact that a larger dosage of Sanechlor is necessary to achieve removal efficiencies equivalent to that of the ferric chlorides tested, the use of Sanechlor would necessitate a larger storage tank, and a pump with a larger capacity. This, combined with its containing sulfate and its jar test performance, was influential in Sanechlor's elimination from being the optimum coagulant.

NHEEL

NHEEL jar tests were run on the 16th and 17th of January. This chemical is produced in Brazil. This chemical coagulant is 38% ferric chloride (solids) by weight and is 34% iron by weight.

As is evident upon inspection of the jar test database, NHEEL performed very well. Its ideal dosage was between 40 mg/L and 50 mg/L, which achieved COD removal efficiencies of 57%-68% and TSS removal efficiencies of 84%-96%.

NHEEL was chosen as the optimum coagulant, at a dosage of 50 mg/L for Alternative 1, and at a dosage of 30 mg/L for Alternative 2. Alternative 1 necessitates a larger dosage because the influent has a detention time of 1.2 hours, whereas Alternative 2 has a residence time on the order of one to two days. This dosage range produced a clear supernatant and large flocs.

Eaglebrook

The Eaglebrook jar tests were run on January 15th and 17th. This chemical is produced in Schereville, IN by Eaglebrook, Inc. This ferric chloride is 40% solids and 34% iron.

From analysis observations, in conjunction with visual observations, Eaglebrook formed the best floc (largest). It achieved COD and TSS removal efficiencies of 60%-72% and 92%-96%, respectively, at dosages of 40 mg/L to 50 mg/L. In the cost analysis, however, Eaglebrook was not a viable choice as the optimum chemical coagulant since it is not currently produced (or even available) in Brazil, and there is no information as to how much it would cost if provided in Brazil. The chemical was tested in Tatui, Brazil because it is a well-known chemical coagulant in the U.S. with a reputation of achieving high removal efficiencies. It served as a measuring stick for the performance of the other ferric chlorides.

Liex

As with NHEEL, so too is Liex a ferric chloride produced in Brazil. It contains 40% solids, and is 34% iron. The Liex jar tests were run on January 15th and 17th.

Liex achieved COD removal efficiencies of 62%-71% and TSS removal efficiencies of 76%-82%, with dosages of 40 mg/L to 50 mg/L. Liex performed very similar to NHEEL, and is the same price. But, Liex formed smaller flocs and had inferior COD and TSS removal efficiencies when compared to NHEEL. Furthermore, NHEEL outperformed Liex in the side-by-side coagulant comparison of January 16th. For these reasons ijt was not chosen as the primary coagulant.

Kemwater

Kemwater is a ferric sulfate produced in Brazil by Kemwater Brazil S.A. It contains 43% solids, and has a density of 1.58 g/cm^3. Kemwater was tested in jar test on January 15th and 17th.

Kemwater achieved the lowest removal efficiencies of all of the chemical coagulants. It achieved removal efficiencies of 40% for COD and 48% for TSS at a dosage of 60 mg/L. It also was inferior to all other chemicals in visual observations of flocs formed and supernatant clarity. A direct comparison of Kemwater to Sanechlor is possible since they are a similar type of coagulant and cost the same amount in Brazil. Since Kemwater was evidently inferior to Sanechlor, it too was not chosen as the optimum coagulant. As stated in the section describing Sanechlor, the Kemwater product we tested contains sulfates. It is

$Fe_2(SO_4)_3$, thus it will release three sulfates for each $Fe_2(SO_4)_3$ added to the wastewater. Again, this will increase odor problems through H_2S production.

Chemical Sludge Recycling as a Coagulant

Facilities do exist in which chemical sludge from CEPT is used as a coagulant, most notably in France. The jar tests conducted in Tatui tested the use of chemical sludge, collected on different occasions with different circumstances and different mixing regimes. The first time chemical sludge was collected was on January 16th where NHEEL was the chemical used, which yielded poor results, largely due to high mixing speeds which broke up the floc. The next jar test run with chemical sludge was also on January 16th, immediately after the one mentioned above. This jar test involved the addition of NHEEL and NHEEL-generated sludge. This jar test yielded better results than the previous one with chemical sludge, as would be expected due to the addition of NHEEL and a lower mixing speed for the chemical sludge to prevent flocs from breaking. Though the results were better than those of chemical sludge addition alone, they did not perform well compared to equivalent dosages of NHEEL without the addition of chemical sludge. The last jar test run on chemical sludge was performed on January 21st, in which Sanechlor-generated sludge was produced and used with Sanechlor and polymer #17. Due to lack of supplies, only COD tests were conducted, with no raw sample to compare removal efficiencies to. Yet, the direct readings of COD are sufficient for a comparison. This jar test run involved two jars with the above procedure, the third jar did not receive chemical sludge. As is evident in the jar test database, jar #3 (ID# 187), which received no chemical sludge, performed the best. Thus, it was decided that the recycling of chemical sludge will not enhance the treatment of Tatui's wastewater.

Other Chemical Coagulants

Various other chemical coagulants were used in the jar tests. Coagulants such as Alum were used to make observations as to their applicability to Alternatives 1 and 2 for CEAGESP. Since none performed well, no further analysis was undertaken.

Lime was not chosen as a candidate for the optimum chemical coagulant. One of the reasons is because lime coagulants generate huge amounts of sludge. Another reason is that lime forms particles with settling velocities that are too fast, and thus can clog the inlet pipes to the CEPT lagoon or the CEPT basins (Hanaeus, 1991). Also, Alternatives 1 and 2 will not dose the coagulant for 24 hours which can pose a problem for lime dosing facilities. When lime dosing terminates, the pH will drop substantially and this change in pH will bring parts of the lime sludge into solution. Indeed, lime could be analyzed for other non-lagoon facilities, but for lagoon facilities with non-24-hour dosing, it is not an optimum choice.

Polyelectrolytes

Polymers are added to wastewater to improve the settleability of solids through particle bridging. Four main (anionic) polymers were used for the Tatui Jar Tests. Other polymers (non-ionic and cationic) were provided by SABESP, but did not perform well and thus data was not collected on them. The main polymers tested, on January 19th, were polymers #13, #15, #17, and #19 produced by the General Alum & Chemical Corporation of Maine. The chemical coagulant used in each jar test is present in the jar test database.

It should be noted that no tests were undertaken to determine the effect of only polymers on the removal of TSS and its associated BOD. Thus no comparison (qualitative or quantitative) can be made between dosing only polymer versus polymer with coagulant or dosing just coagulant. But, it was visually observed that polymers dramatically inrease floc size and dramatically decrease settling times.

Polymer #13

Polymer #13 is an anionic polymer with a low (10%) charge density and a high molecular weight (6 million Daltons). This polymer performed the worst, as can be seen in the jar test database, and was thus not used as the ideal polymer.

Polymer #15

Polymer #15 is an anionic polymer with a high (40%) charge density and a high molecular weight (6 million Daltons). Polymer #15 performed very well, approximately as well as polymer #17, but in the comparison tests of polymer #15 versus polymer #17, polymer #17performed better (according to visual observation).

Polymer #17

Polymer #17 is an anionic polymer with a medium charge density (20%) and a medium molecular weight (4 million Daltons). As stated above, polymer #17 outperformed polymer #15 in "head-to-head" jar tests, and was subsequently chosen as the best polymer based on limited testing. Since polymer #17 was chosen as the best polymer, it was used on January 20th in a comparison test between Sanechlor and NHEEL to determine the optimum coagulant-polymer combination. Then, on January 21st, it was used to determine the ideal polymer and coagulation dosage. As stated above (and below), this was not determined due to time and equipment restraints.

It can not be said for the obtained results that a metal salt/polymer combination is recommended; only that certain polymers out-performed others.

Polymer #19

Polymer #19 is an anionic polymer with a high charge density (40%) and a very high molecular weight (8 million Daltons). As can be seen in the jar test data database, polymer #19 performed rather poorly and was subsequently not chosen as the optimum polymer.

Other Polymers

As stated, SABESP provided the MIT group with different polymers (cationic and non-ionic) which they were interested in testing. The performance of these polymers was poor and was thus not analyzed further in jar tests.

Optimum Polymer Dosage

One of the objectives of a jar test is to determine the optimum coagulant-polymer combination and the optimum dosages of both.

Due to time and supplies constraints, it was not possible to determine the optimum dosage and combination of chemical coagulant with polymer. Moreover, a metal salt coagulant/polymer system is somewhat more complex and expensive than a singe coagulant system. Therefore, there will be no polymer dosing for the design of the CEAGESP facility in Tatui. The only enhancement to the wastewater process will be metal salt addition (specifically, NHEEL).

Jar Test Plots and Jar Test Database

This portion of the appendix is a collection of the data obtained from the jar tests and other lab work done in Tatui, Brazil in January, and its subsequent analysis. The analysis involved directly in the design is presented throughout the thesis, but, the analysis of other data is not. That is, this appendix contains plots and data which were analyzed, but an explanation of the analysis is not presented. Other plots included chemical coagulant comparisons and the data from the 24-hr lagoon sampling test.

Terms

This section will explain the terms used in the data in the subsequent pages.

ID#: This is to follow data from page to page.

Date: The date the test/experiment was done.

Time: The time at which the test/experiment was done. NA will refer to all data that is Not Available.

Sampling Location: Where the sample came from. “Splitter box” is where the flow is split to the three pipes into the anaerobic lagoon, and the overflow into the river. “B1” is the head of the anaerobic lagoon. “G1” is the effluent of the anaerobic lagoon. “J1” is the effluent of the facultative lagoon.

Sampling Time: The time of sampling.

Sample (Jar#): When this is a number, it is the jar number in the jar test (labeled 1 through 4, form the left-most jar, to the right-most jar). “Raw” means a raw sample, one which did not undergo any jar test procedure. B1, J1, and G1 refer to the same as in sampling location. For the ones labeled 1min, 2min…5min, this means the exact minute of sampling during the settling tests.

Coagulant: The specific coagulant used. Zero means the “zero jar” in which no coagulant was added although the sample was part of a jar test run. “Sl” indicates sludge (such as “NHEEL + Sl.”

Chemical Dosage: The amount of chemical dosed to the jar.

Polymer: The type of polymer dosed. None means no polymer was injected. “S-non” is a non-ionic polymer provided by SABESP, as “S-cat” is a cationic and “S-an” is an anionic polymer, both provided by SABESP.

Polymer Dosage: The amount of polymer dosed to each jar.

COD: The measured COD in the sample.

COD %Removal: The amount of COD removal achieved by the jar test, as compared to a raw sample(s).

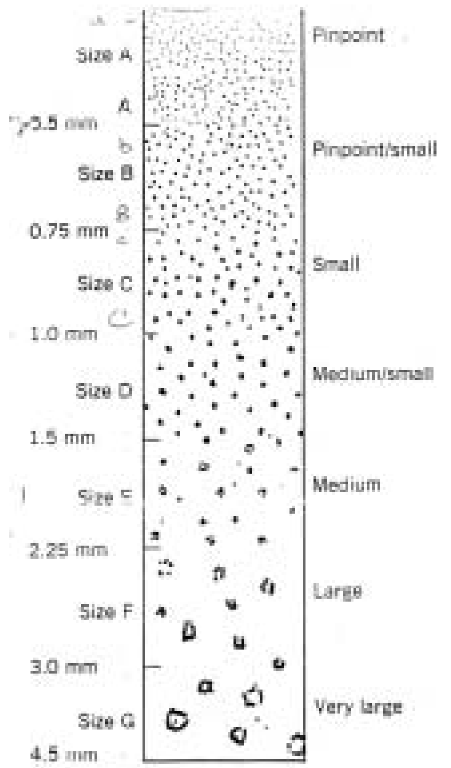
TSS: The measure TSS of the sample.

TSS %Removal: The amount of TSS removal achieved by the jar test, as compared to a raw sample(s).

SO_4: The detected amount of $S0_4$-sulfates in the sample.

PO_4: The detected amount of PO_4-phosphates in the sample.

Floc Size: A visual observation of floc as compared to the following figure:

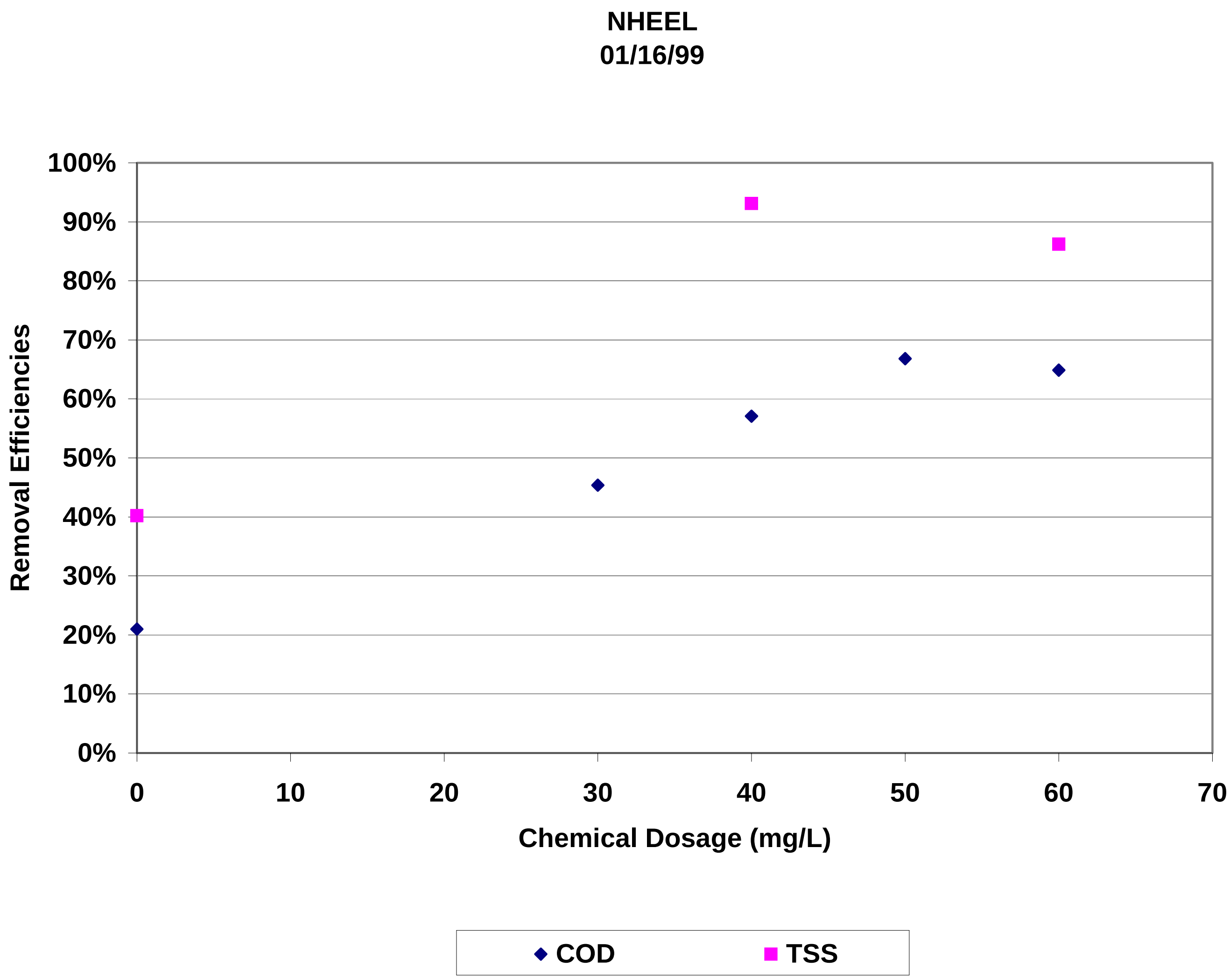


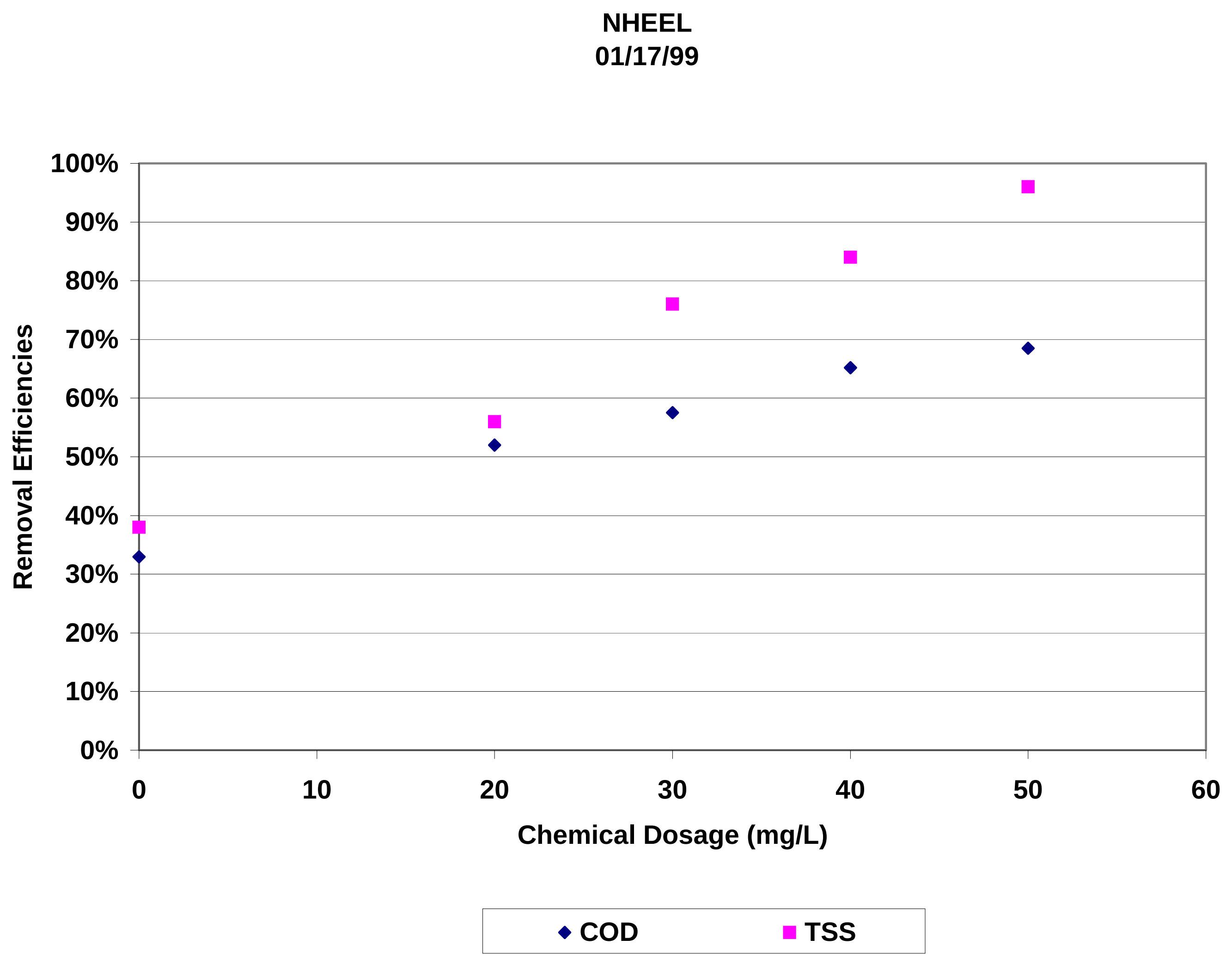
Visual Observation: That which was observed during the jar test or in a sample specimen.

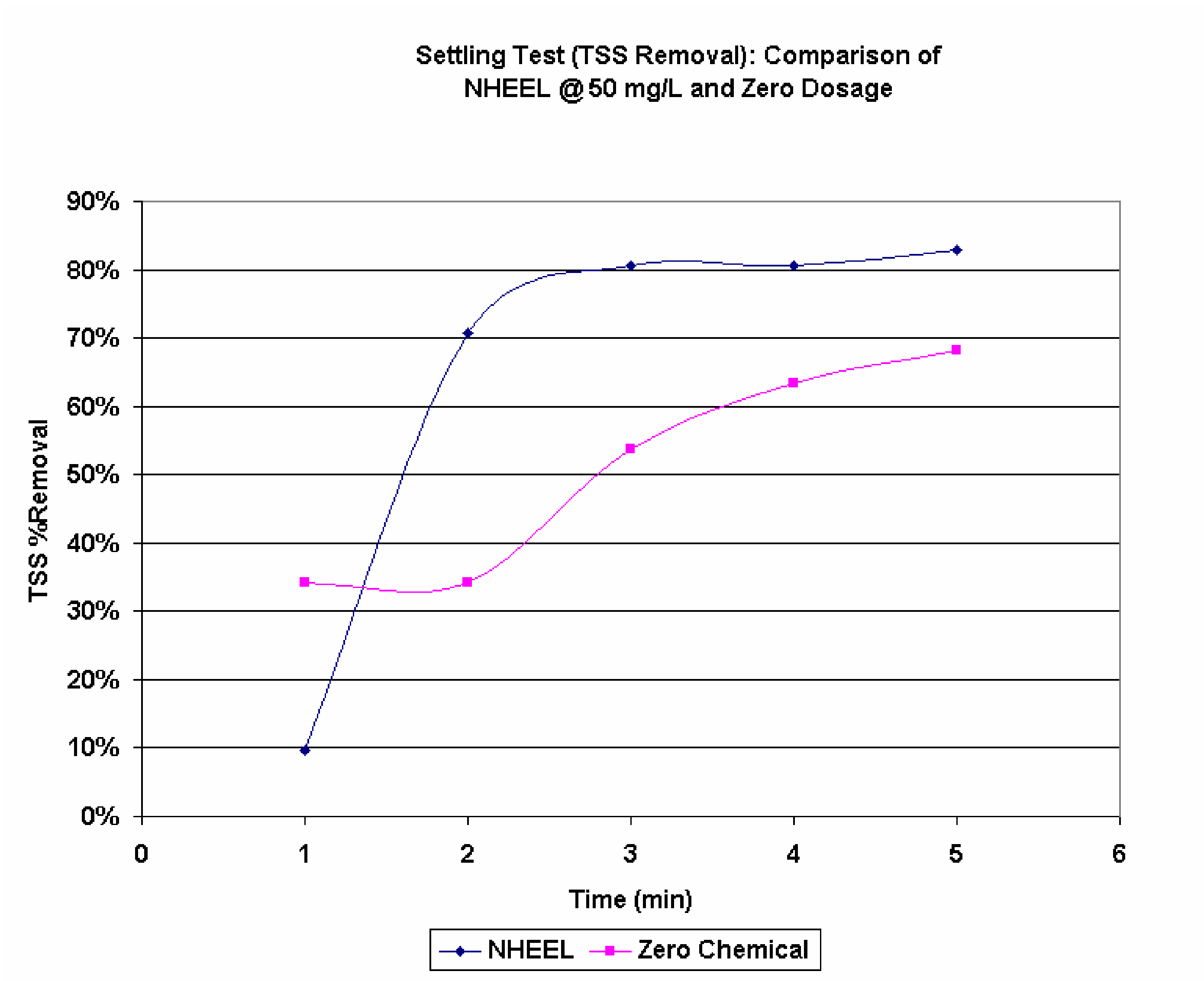
Sample Volume: The volume of wastewater in each jar.

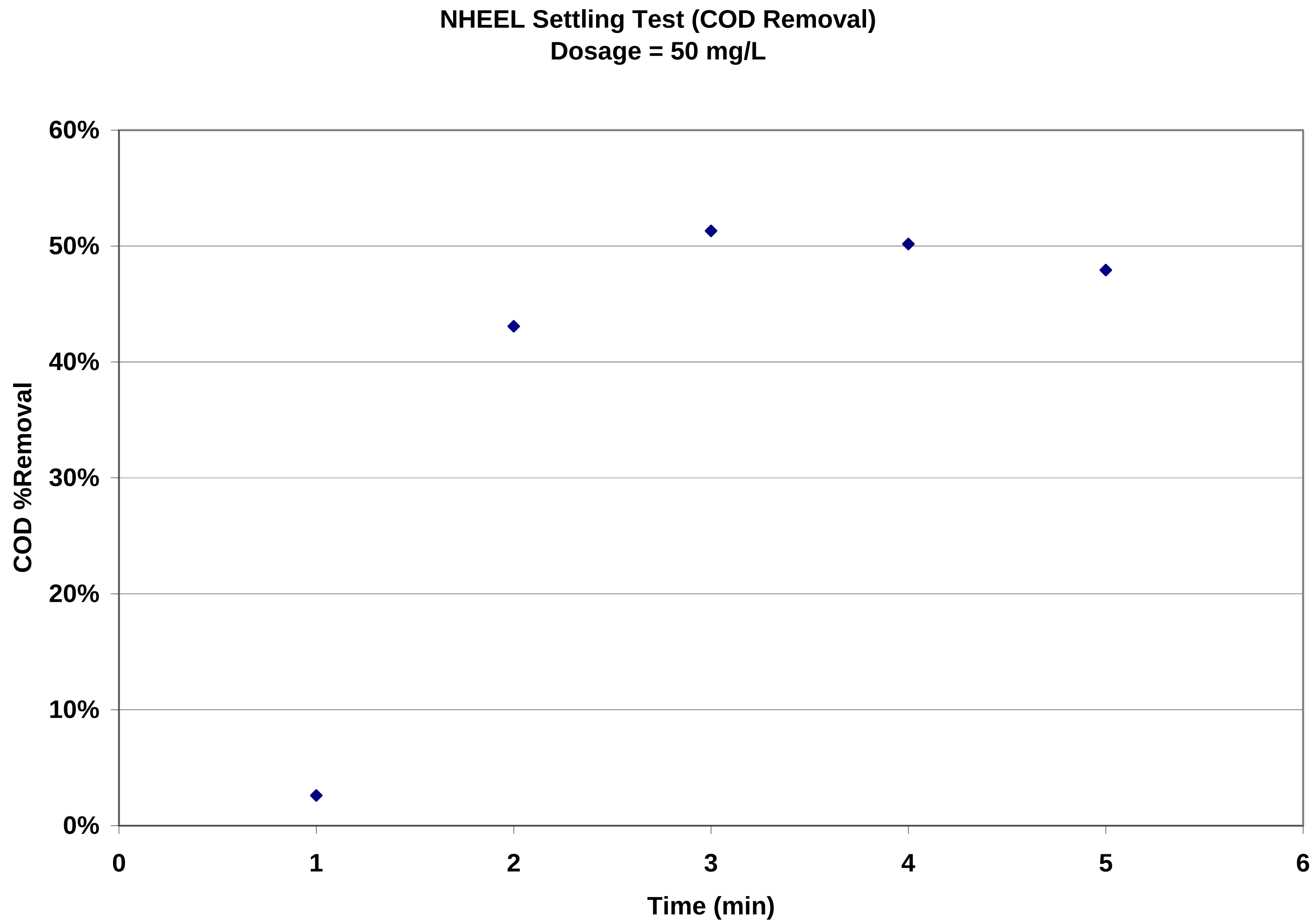
Purpose => Results: The purpose of the jar tests, and the observed results.

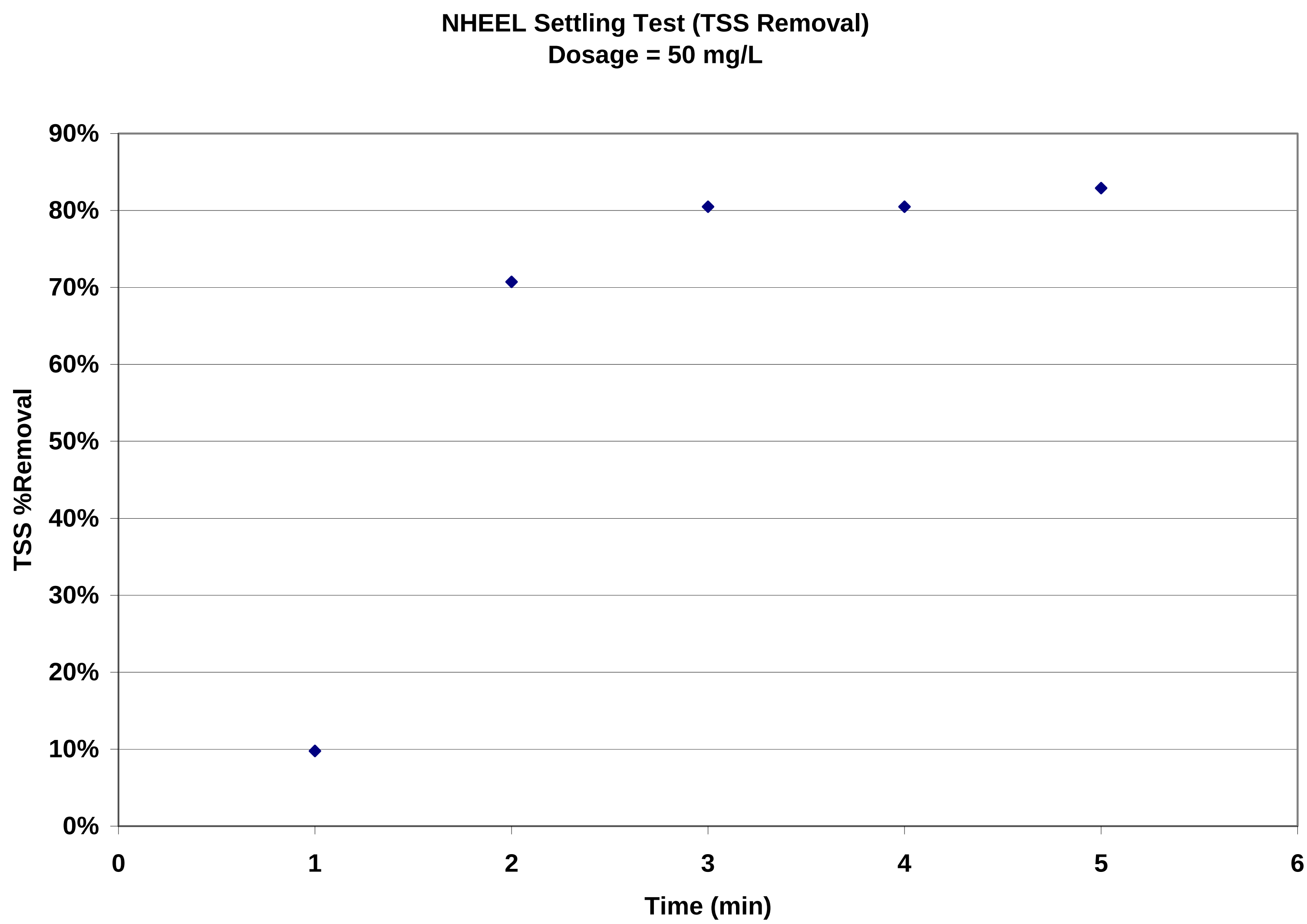
The plots follow and data follow.

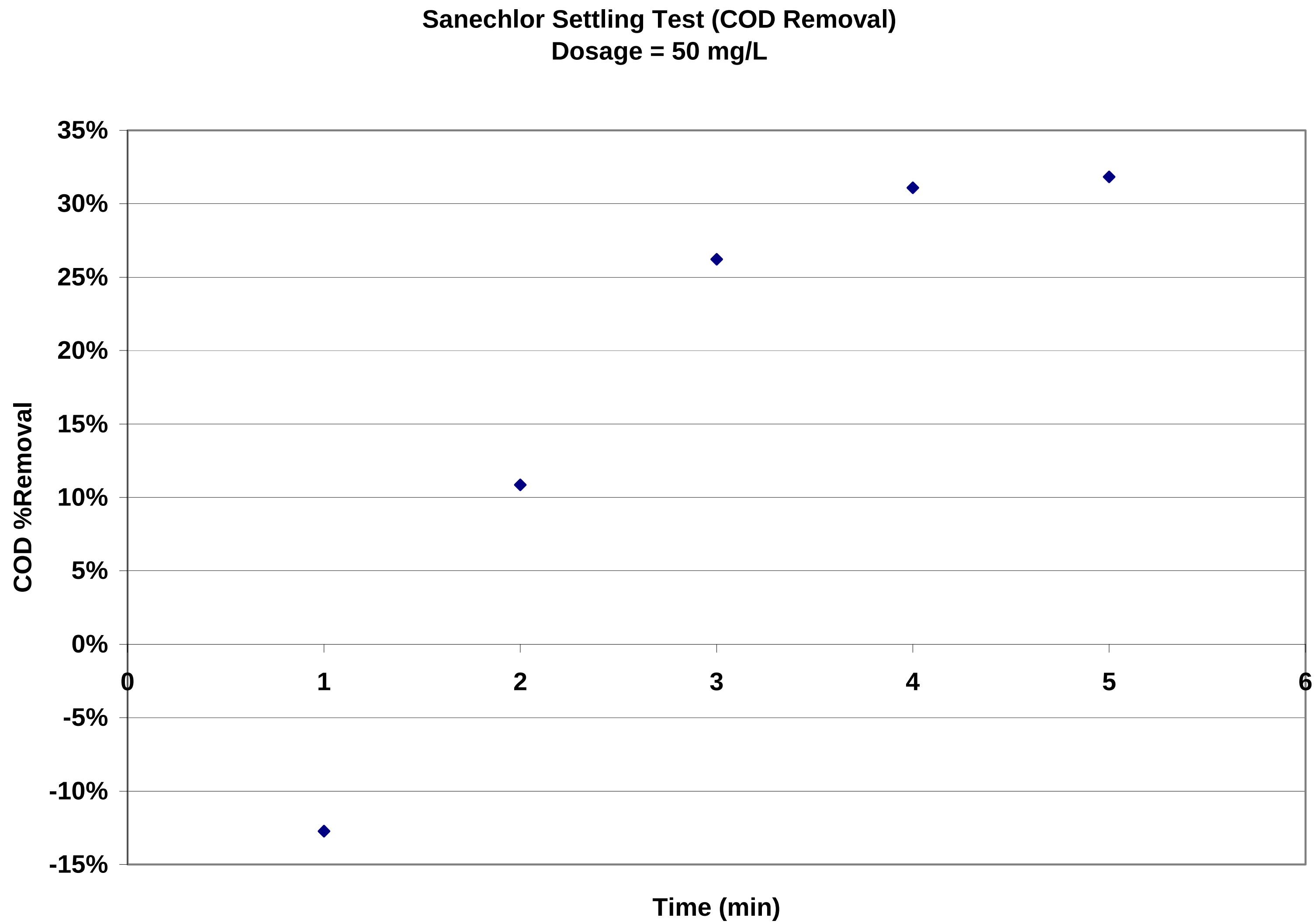

NHEEL
01/16/99
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Removal Efficiencies
0
10
20
30
40
50
60
70
Chemical Dosage (mg/L)
COD
TSS

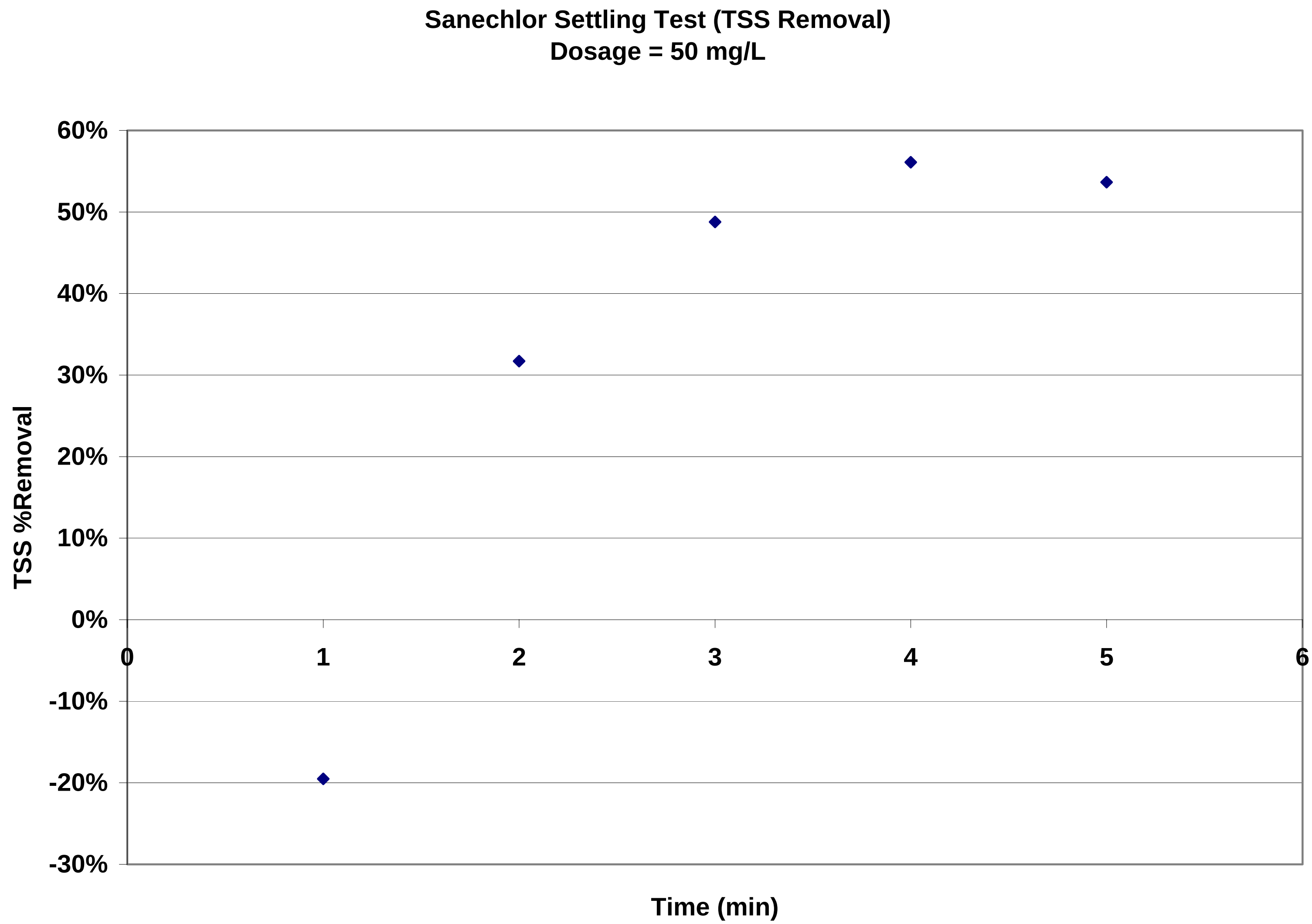

NHEEL
01/17/99
Removal Efficiencies
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
COD
TSS

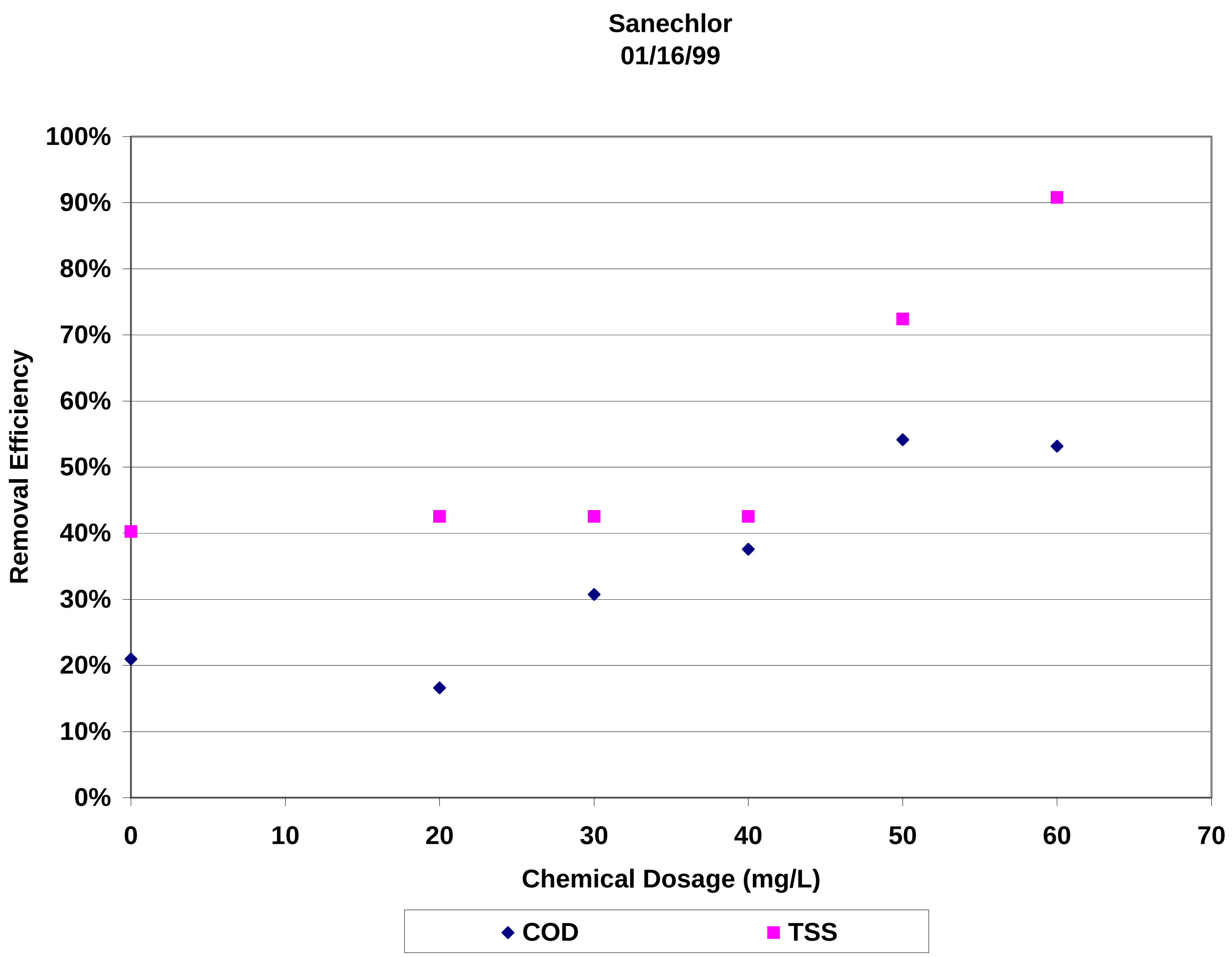

Settling Test (TSS Removal): Comparison of NHEEL @ 50 mg/L and Zero Dosage
TSS %Removal
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)
NHEEL
Zero Chemical

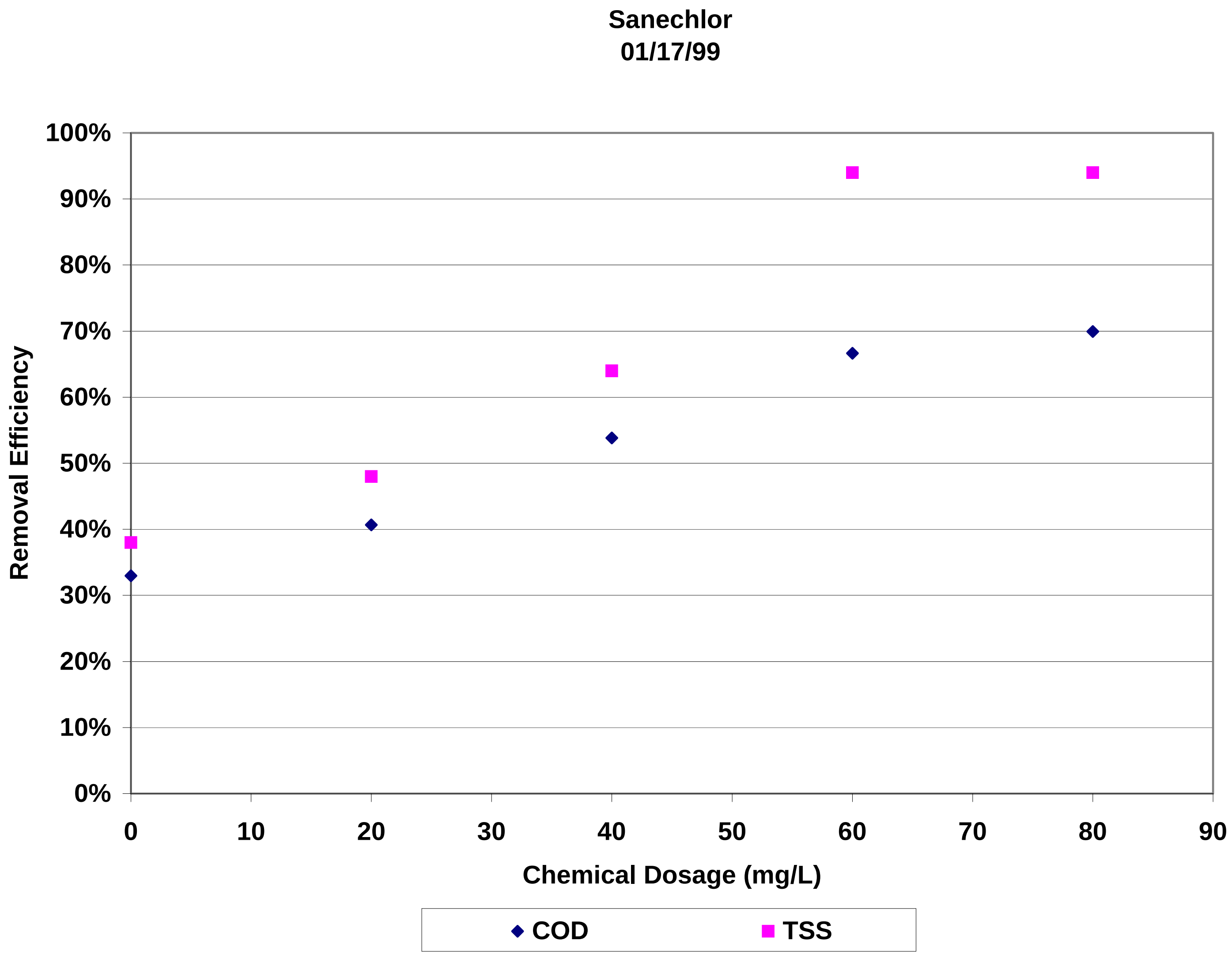

NHEEL Settling Test (COD Removal)
Dosage = 50 mg/L
COD %Removal
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)

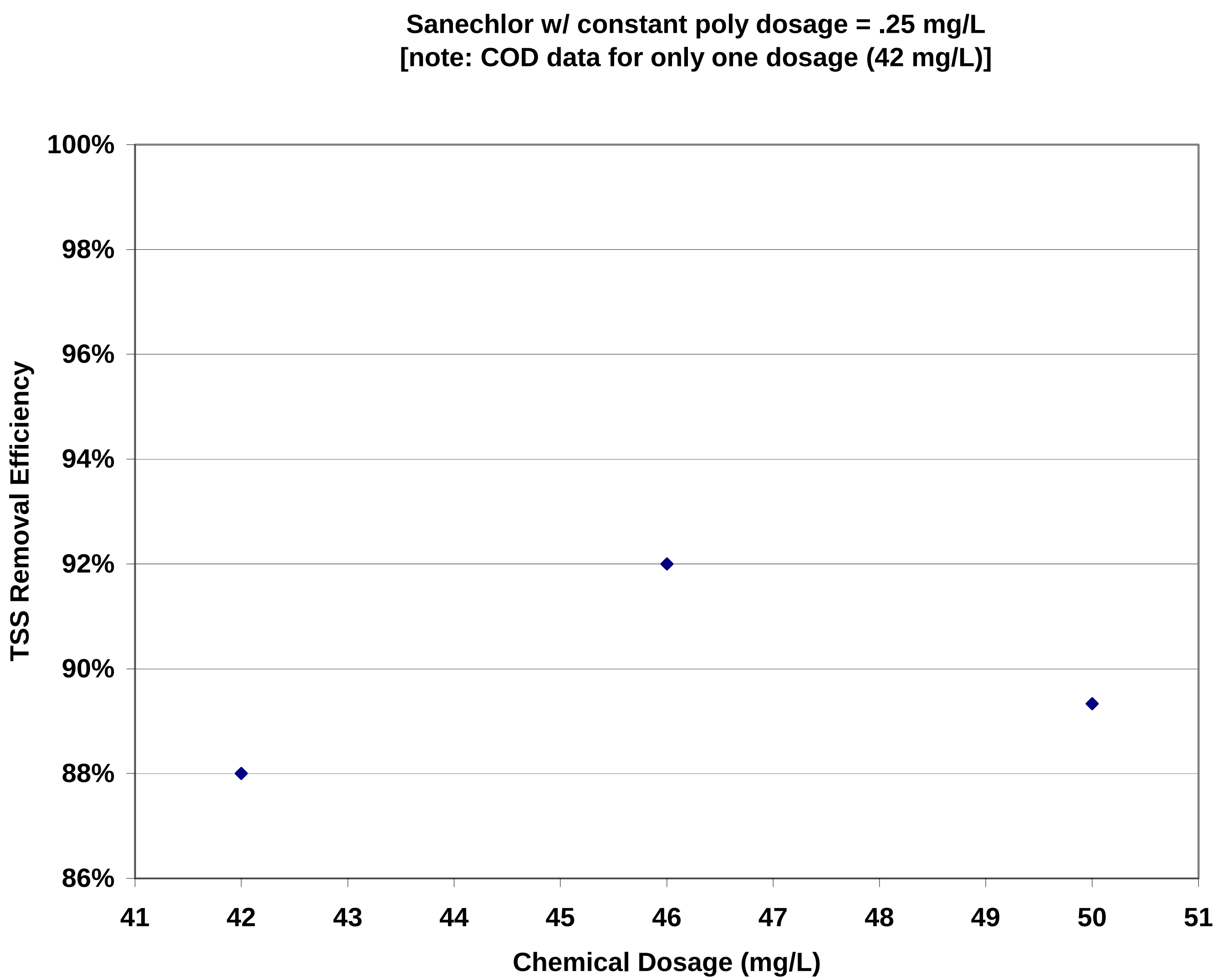

NHEEL Settling Test (TSS Removal)
Dosage = 50 mg/L
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
TSS %Removal
0
1
2
3
4
5
6
Time (min)

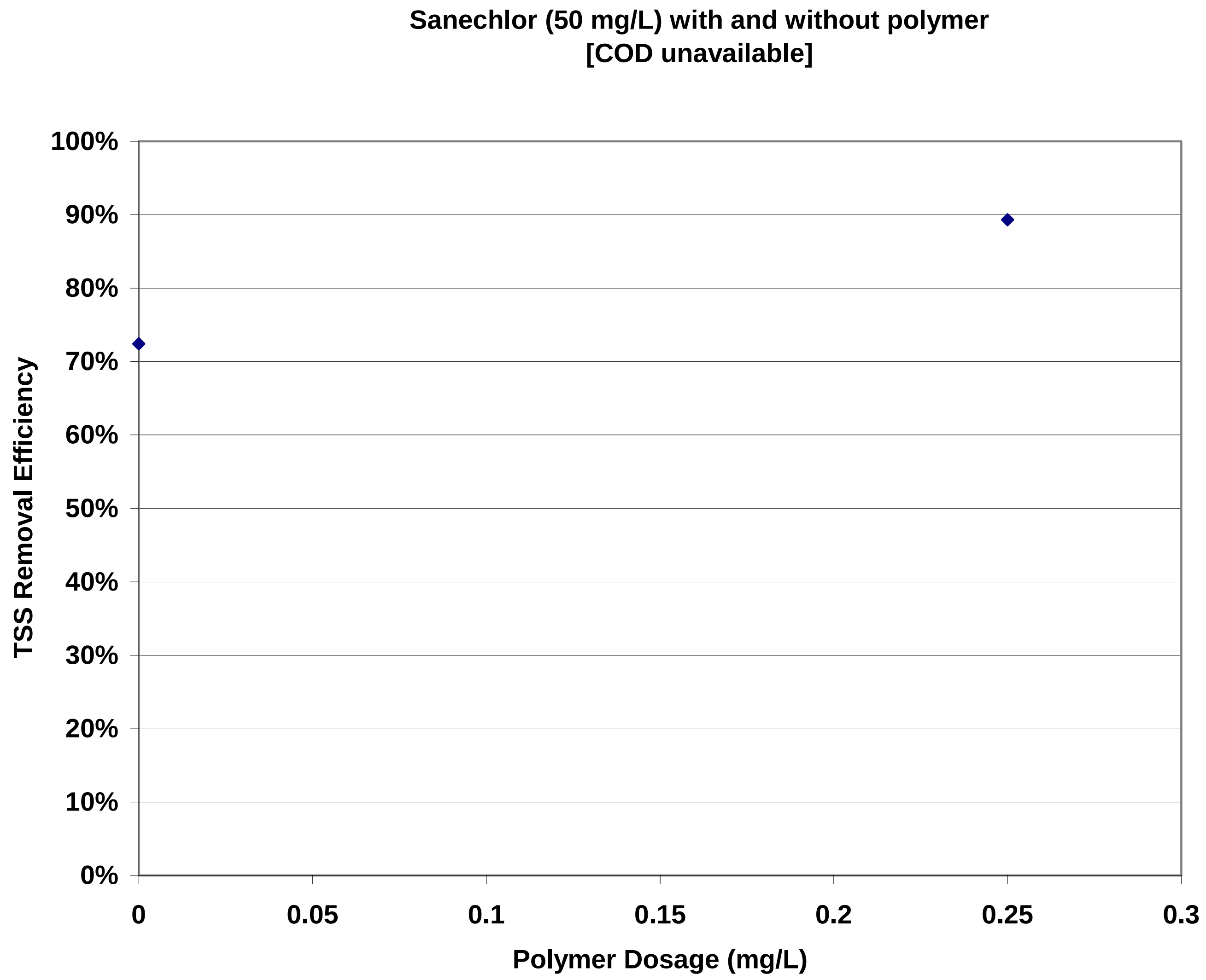

Sanechlor Settling Test (COD Removal)
Dosage = 50 mg/L
35%
30%
25%
20%
15%
10%
5%
0%
-5%
-10%
-15%
COD %Removal
0
1
2
3
4
5
6
Time (min)

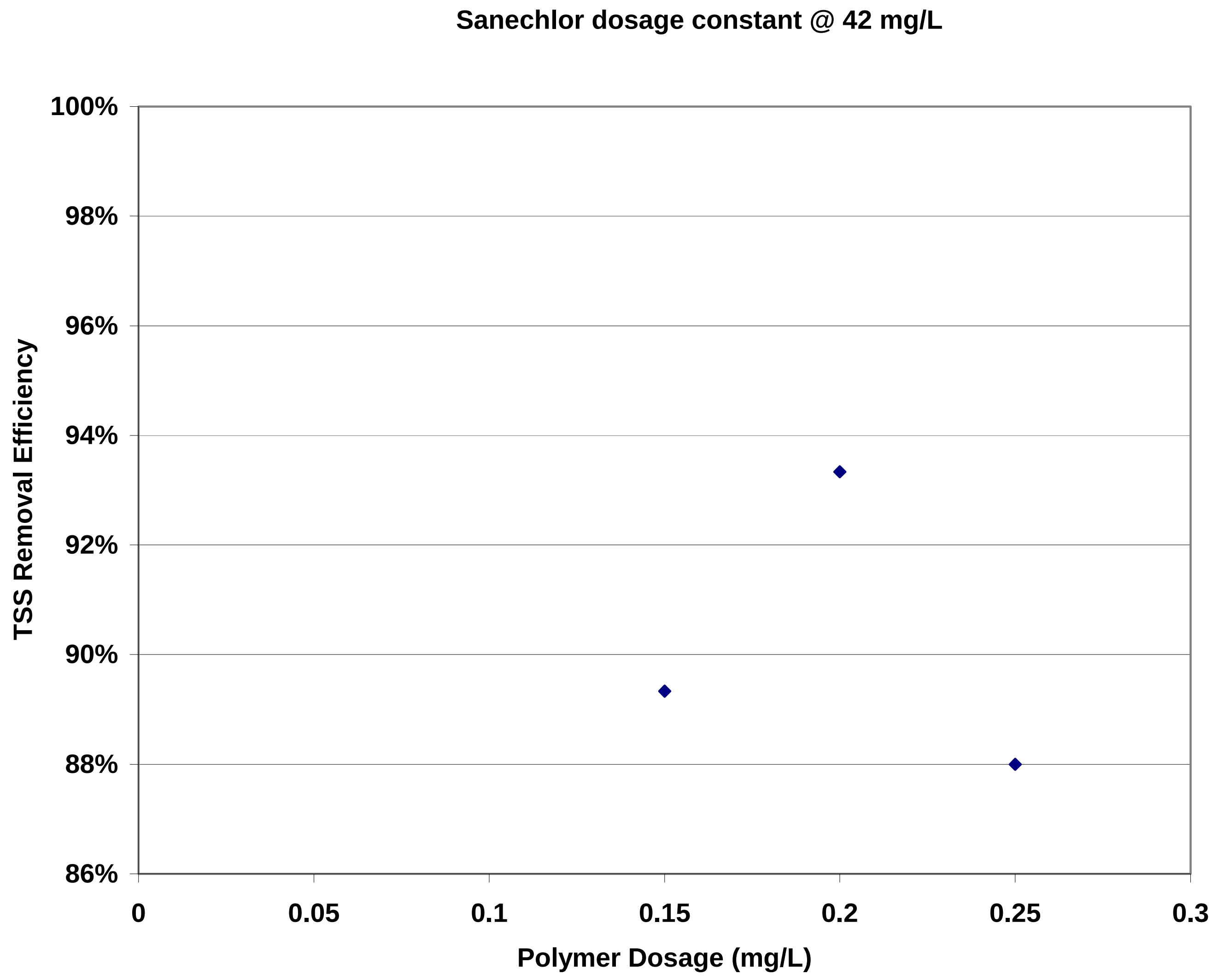

Sanechlor Settling Test (TSS Removal)
Dosage = 50 mg/L
60%
50%
40%
30%
20%
10%
0%
-10%
-20%
-30%
0
1
2
3
4
5
6
TSS %Removal
Time (min)

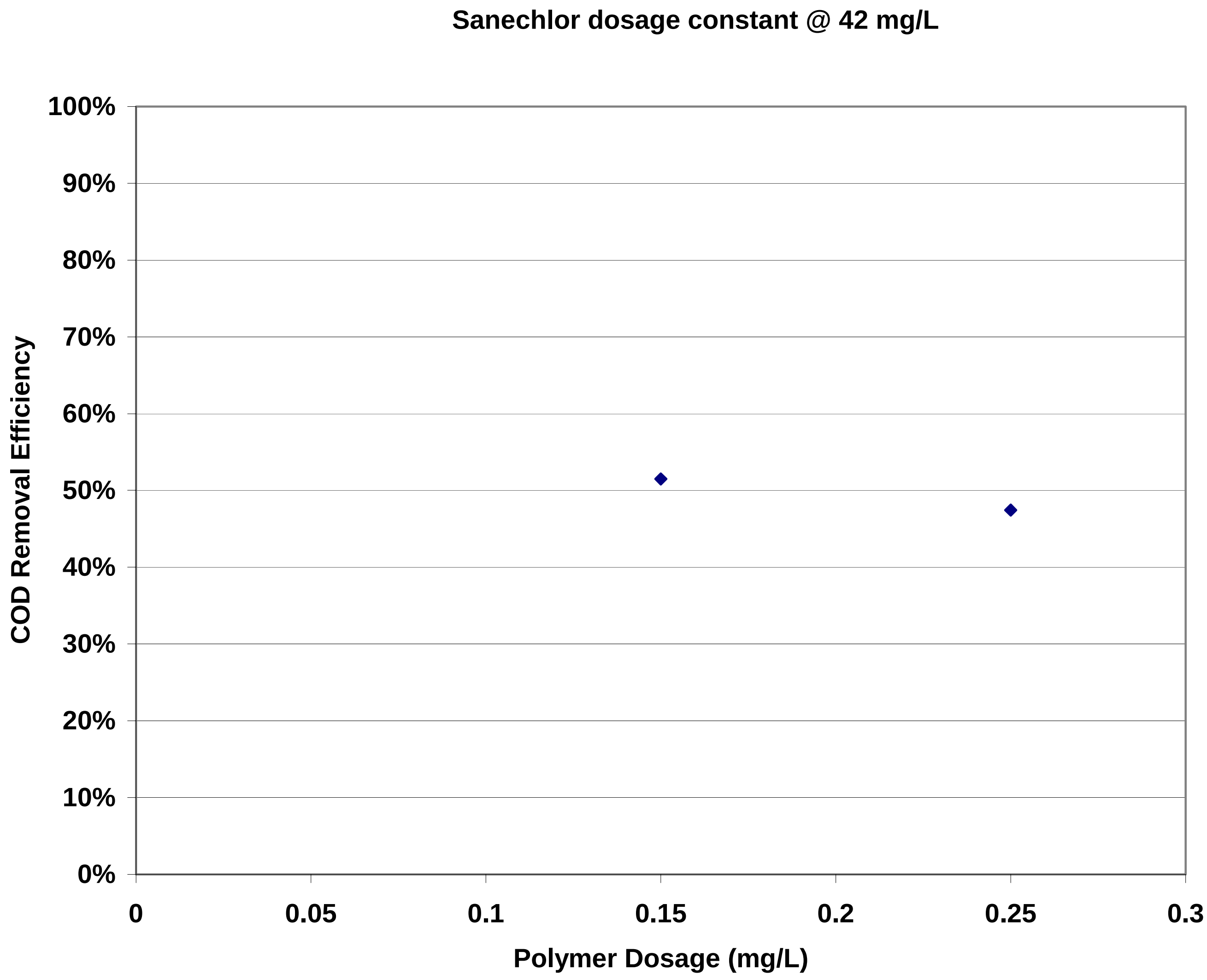

Sanechlor
01/16/99
Removal Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
70
Chemical Dosage (mg/L)
COD
TSS

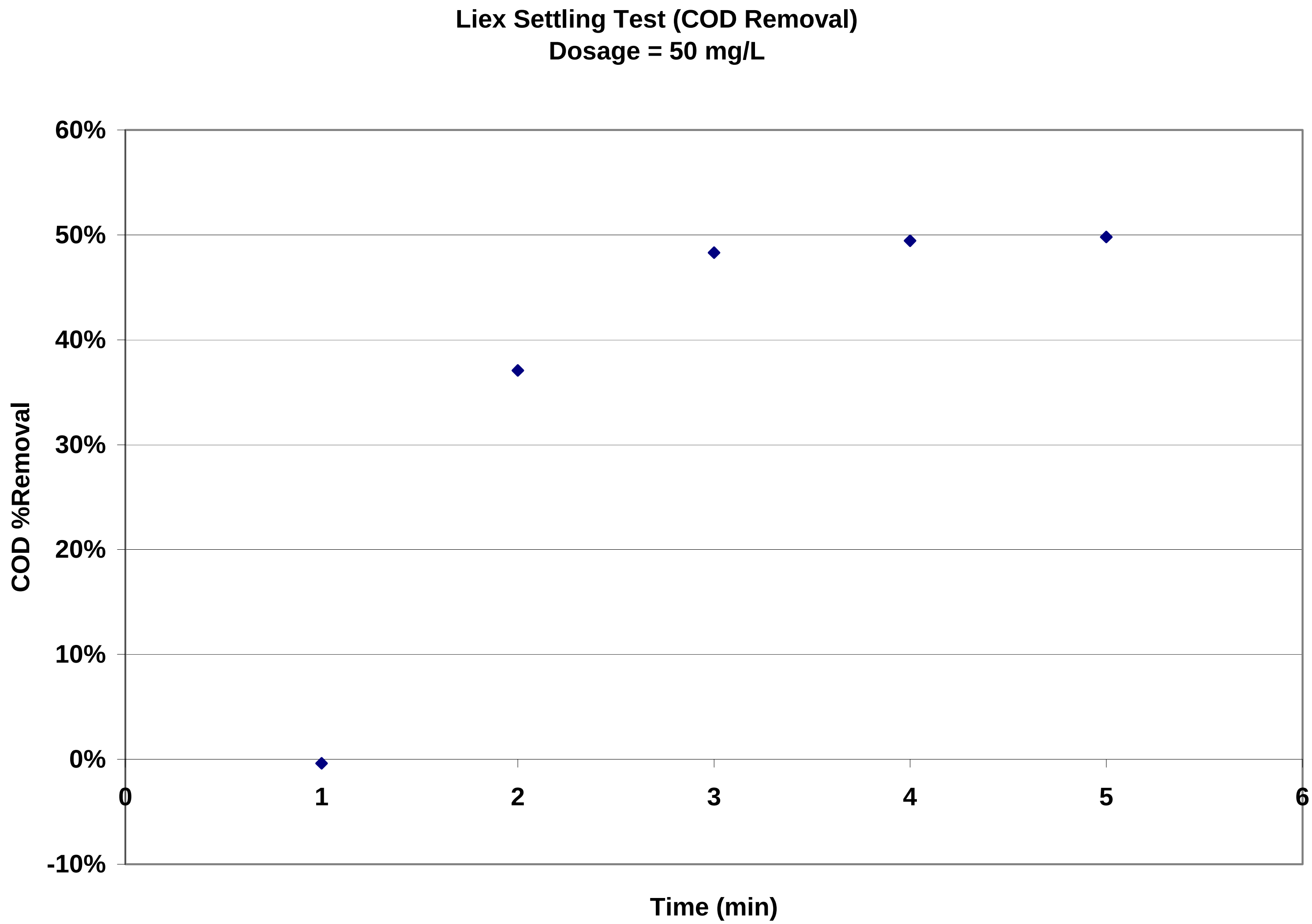

Sanechlor
01/17/99
Removal Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
70
80
90
Chemical Dosage (mg/L)
COD
TSS

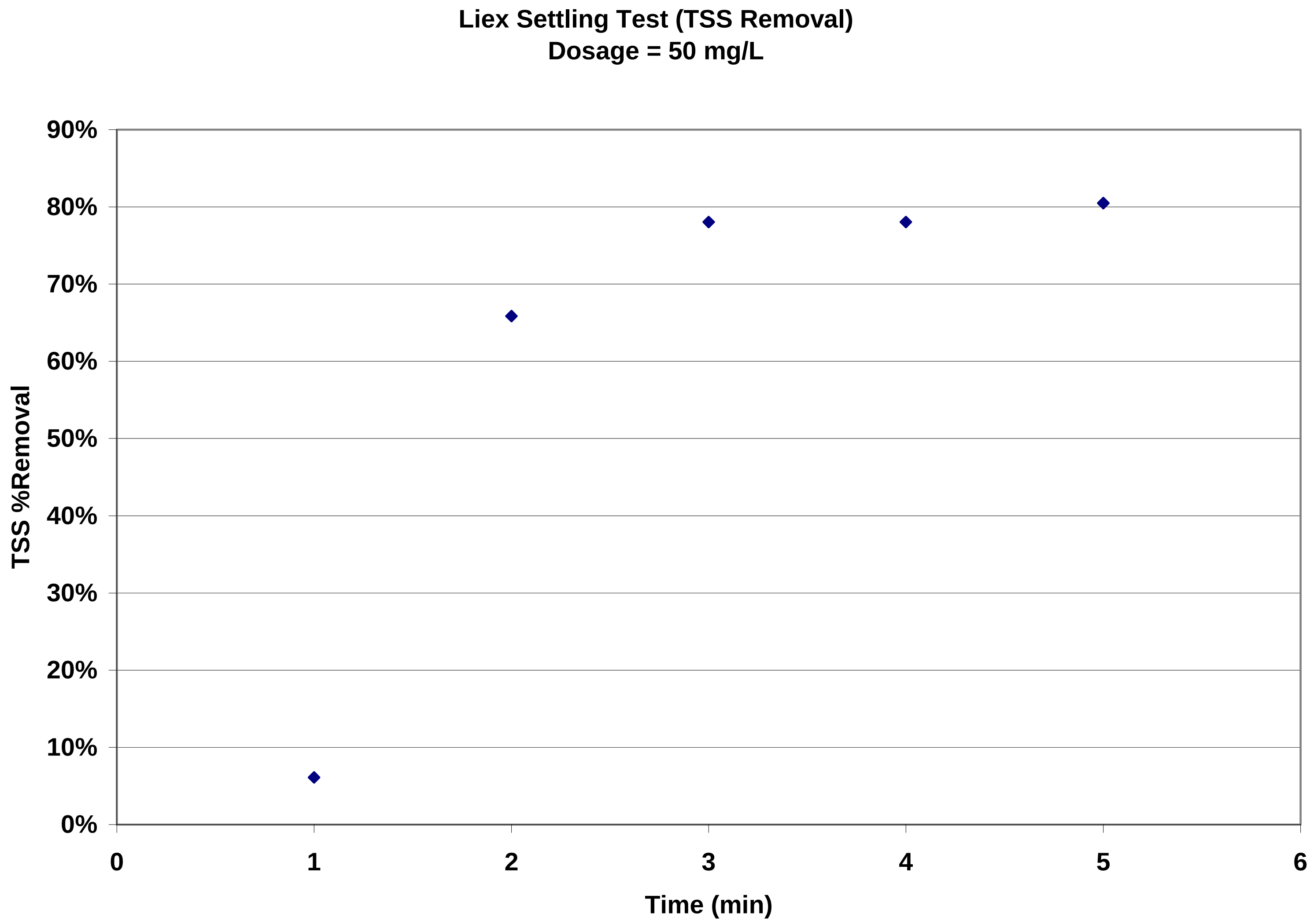

Sanechlor w/ constant poly dosage = .25 mg/L
[note: COD data for only one dosage (42 mg/L)]
TSS Removal Efficiency
100%
98%
96%
94%
92%
90%
88%
86%
41
42
43
44
45
46
47
48
49
50
51
Chemical Dosage (mg/L)

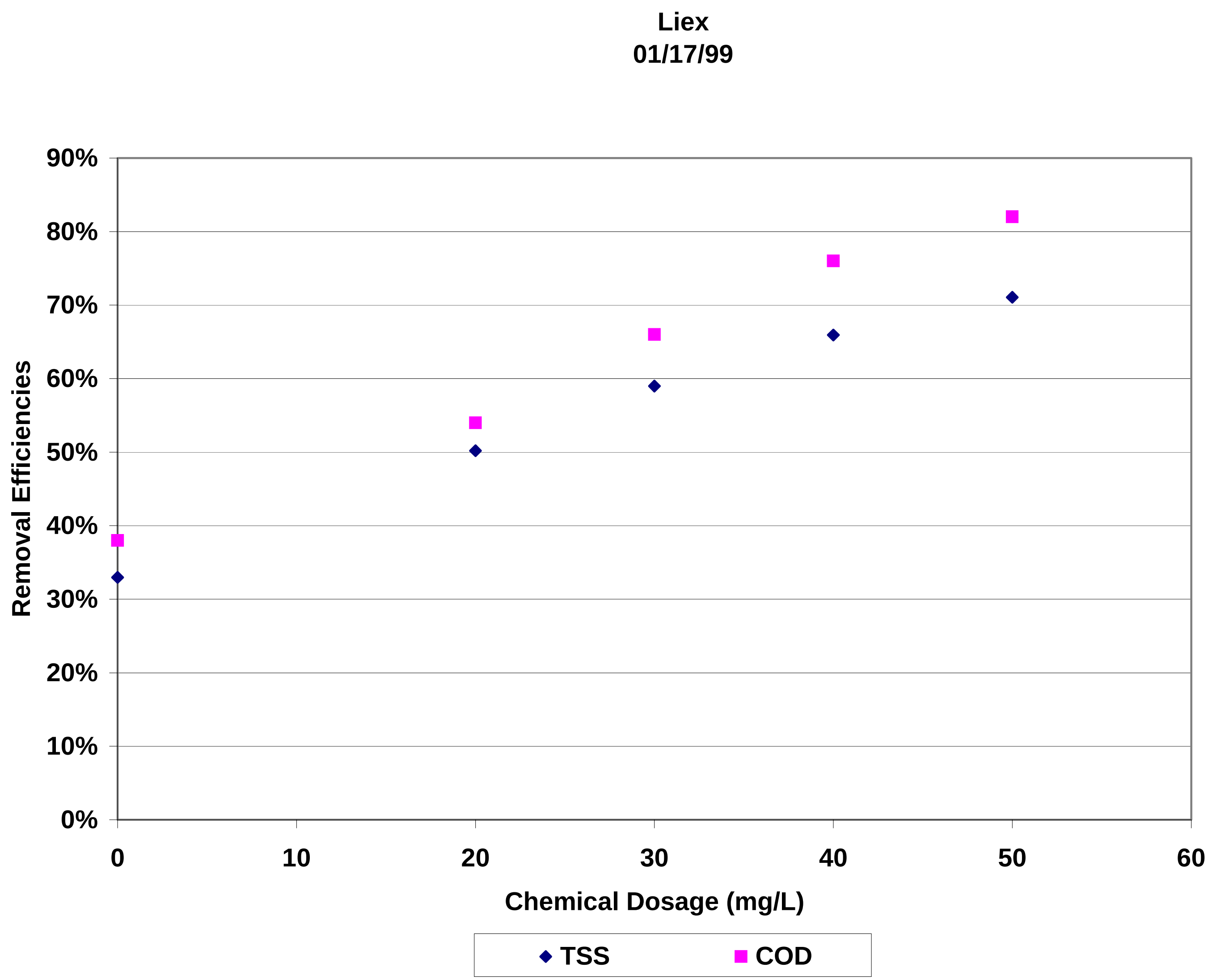

Sanechlor (50 mg/L) with and without polymer
[COD unavailable]
TSS Removal Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
0.05
0.1
0.15
0.2
0.25
0.3
Polymer Dosage (mg/L)

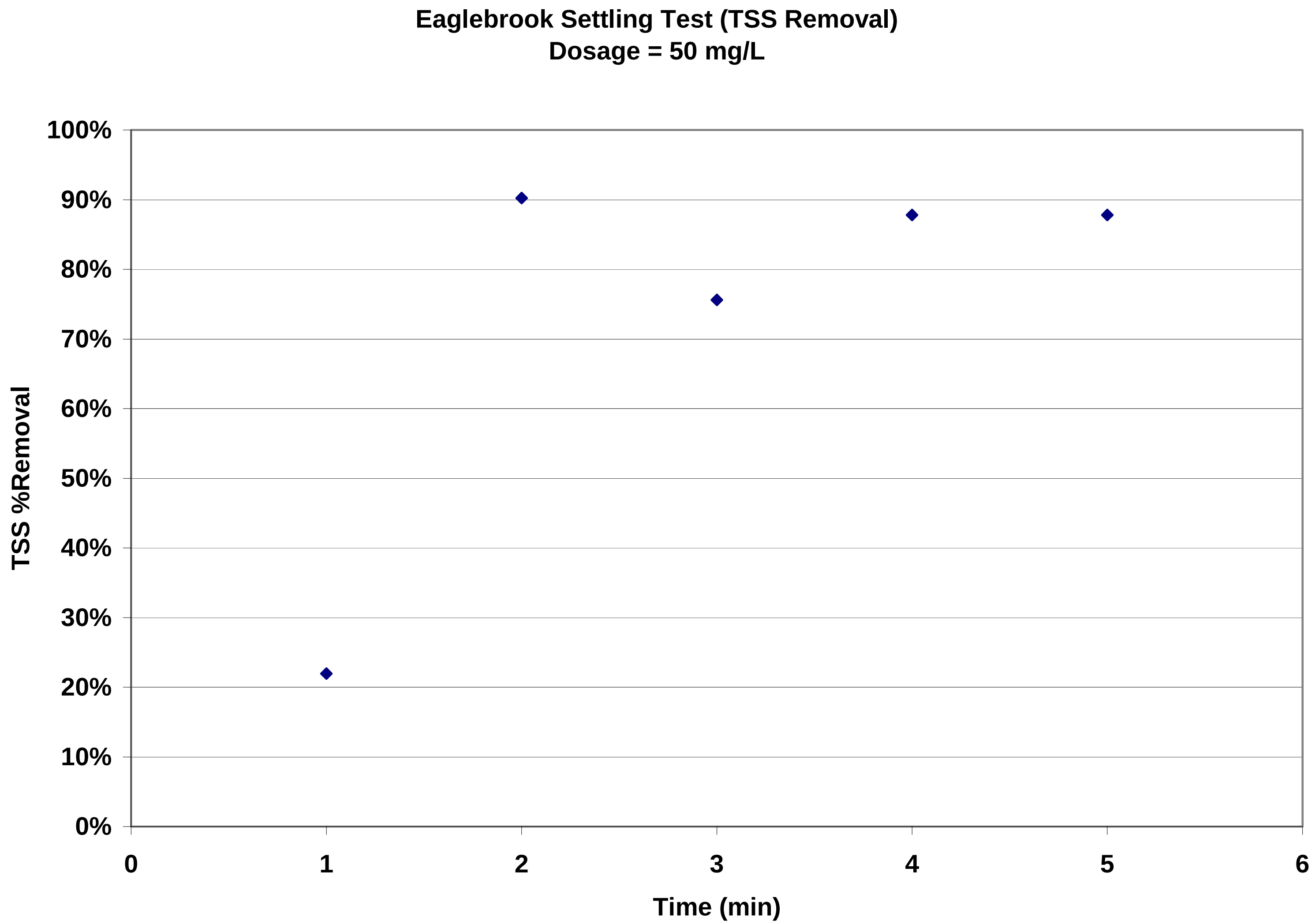

Sanechlor dosage constant @ 42 mg/L
TSS Removal Efficiency
100%
98%
96%
94%
92%
90%
88%
86%
0
0.05
0.1
0.15
0.2
0.25
0.3
Polymer Dosage (mg/L)

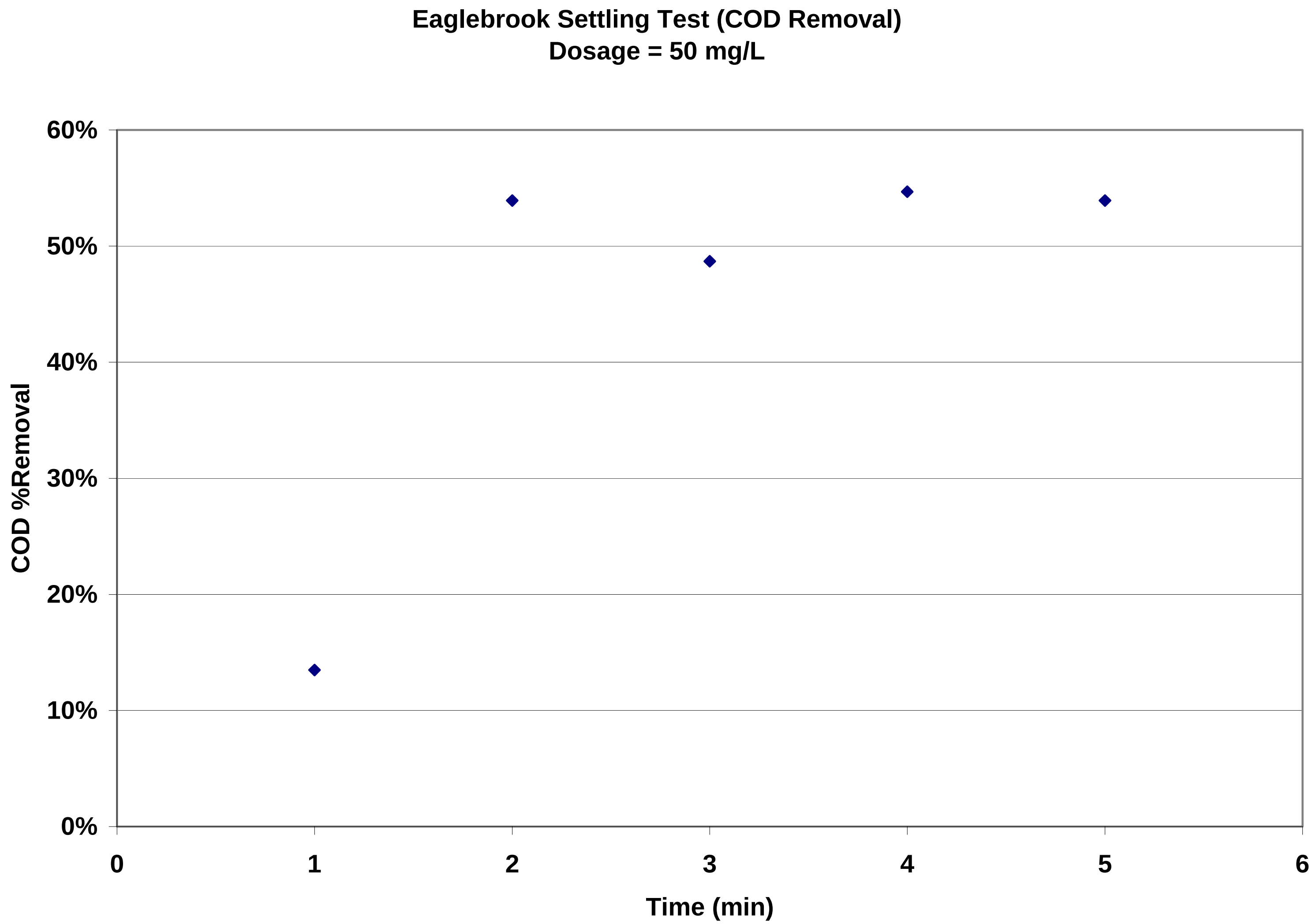

Sanechlor dosage constant @ 42 mg/L
COD Removal Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
0.05
0.1
0.15
0.2
0.25
0.3
Polymer Dosage (mg/L)

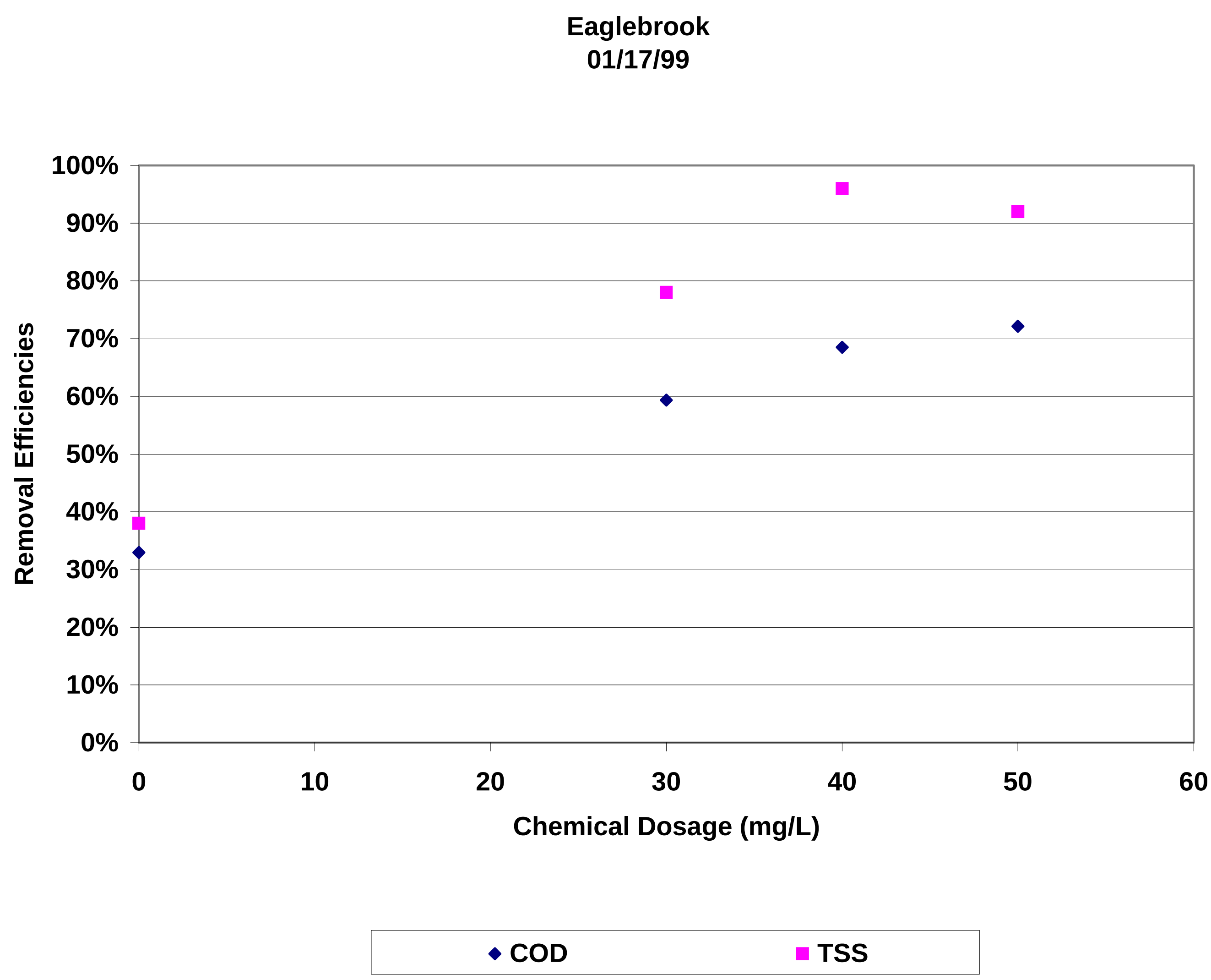

Liex Settling Test (COD Removal)
Dosage = 50 mg/L
COD %Removal
60%
50%
40%
30%
20%
10%
0%
-10%
0
1
2
3
4
5
6
Time (min)


Liex Settling Test (TSS Removal)
Dosage = 50 mg/L
TSS %Removal
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)

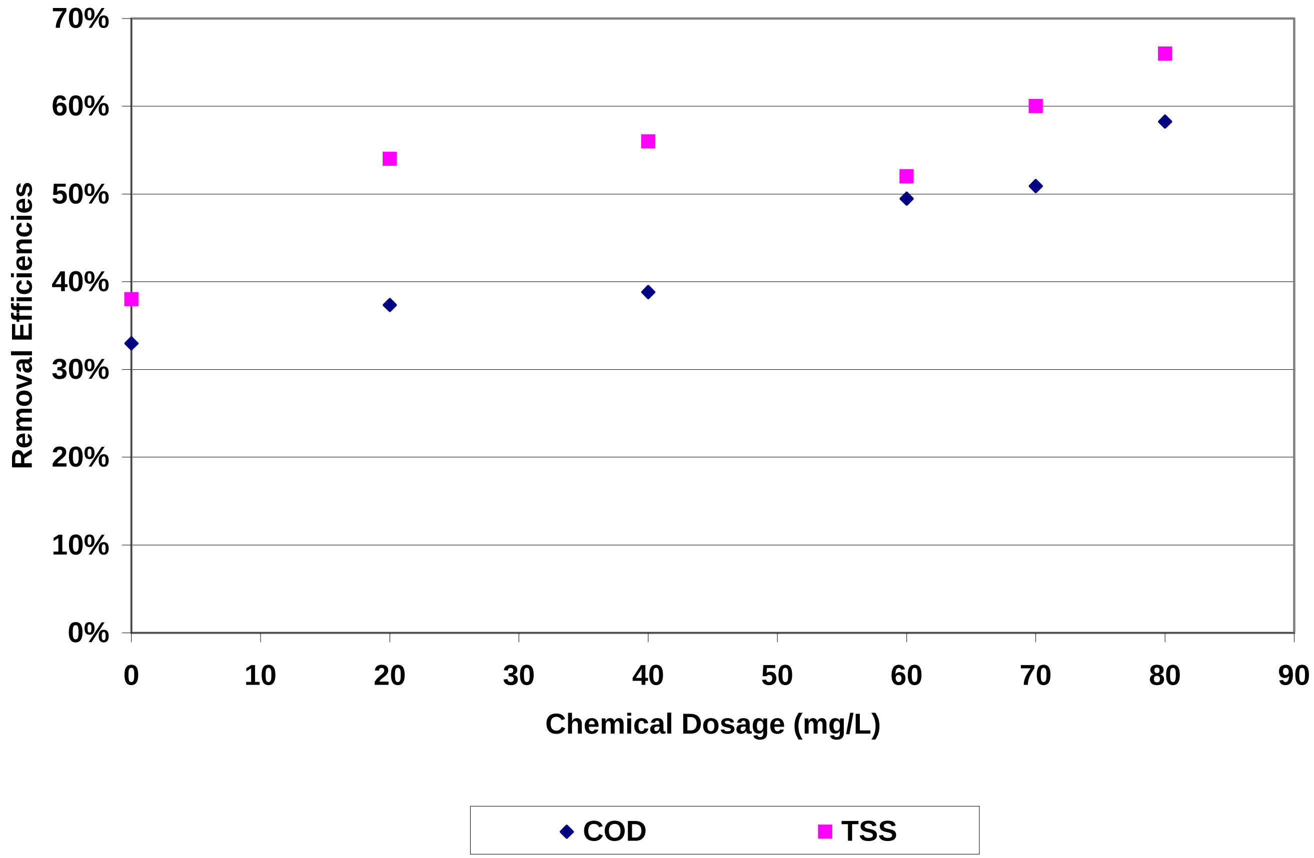

Liex
01/17/99
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Removal Efficiencies
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
TSS
COD

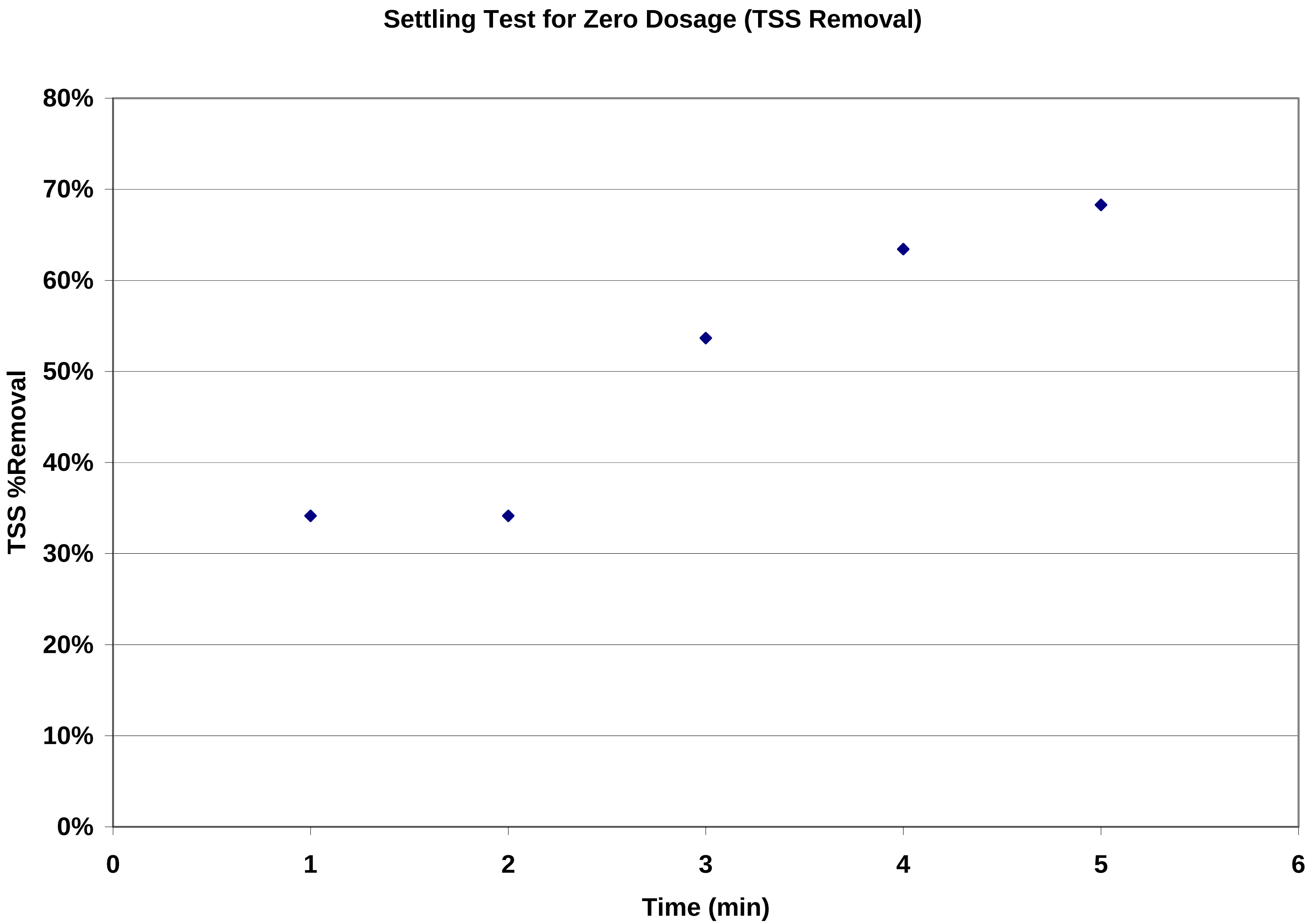

Eaglebrook Settling Test (TSS Removal)
Dosage = 50 mg/L
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
TSS %Removal
0
1
2
3
4
5
6
Time (min)

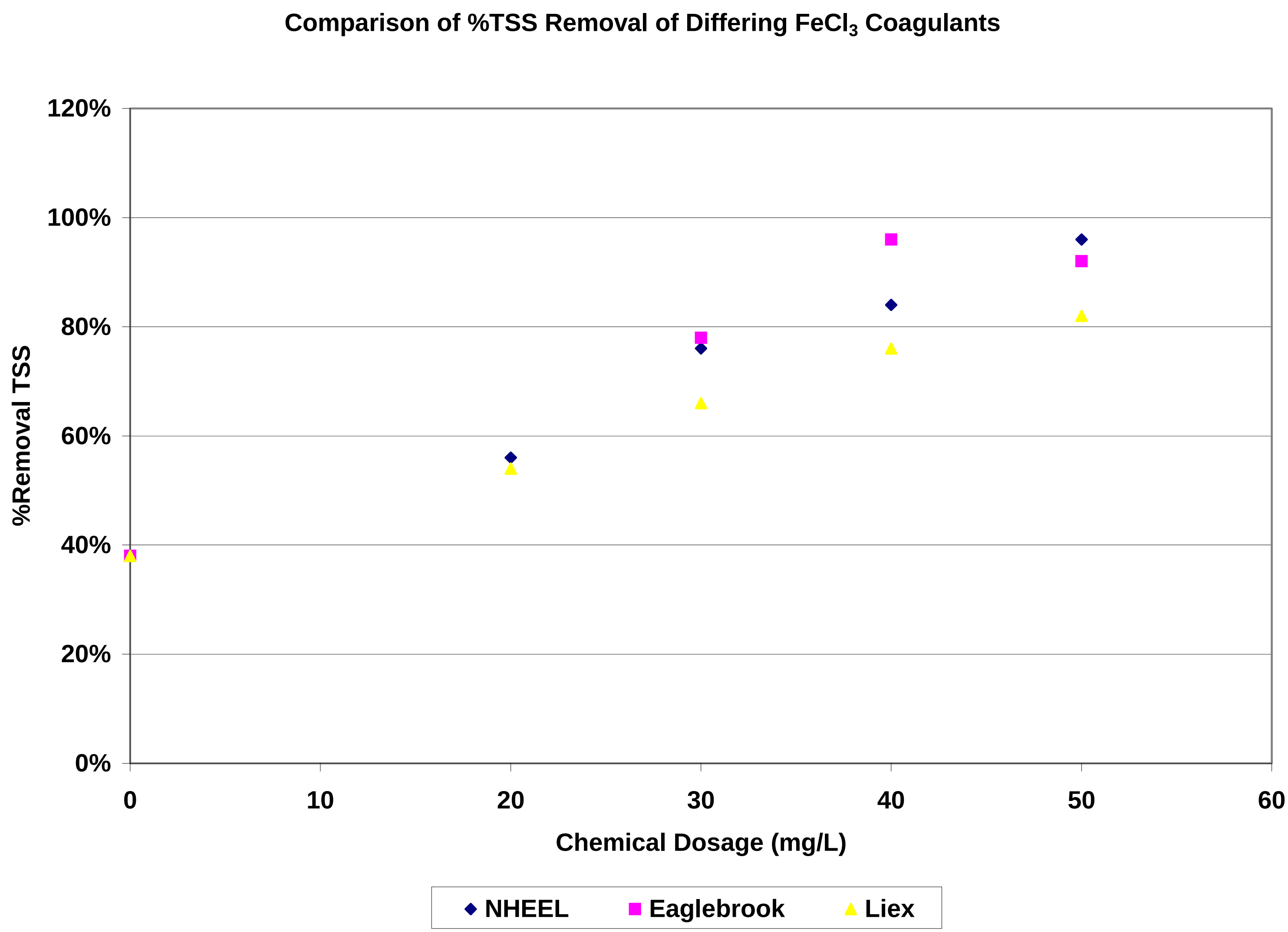

Eaglebrook Settling Test (COD Removal)
Dosage = 50 mg/L
COD %Removal
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)

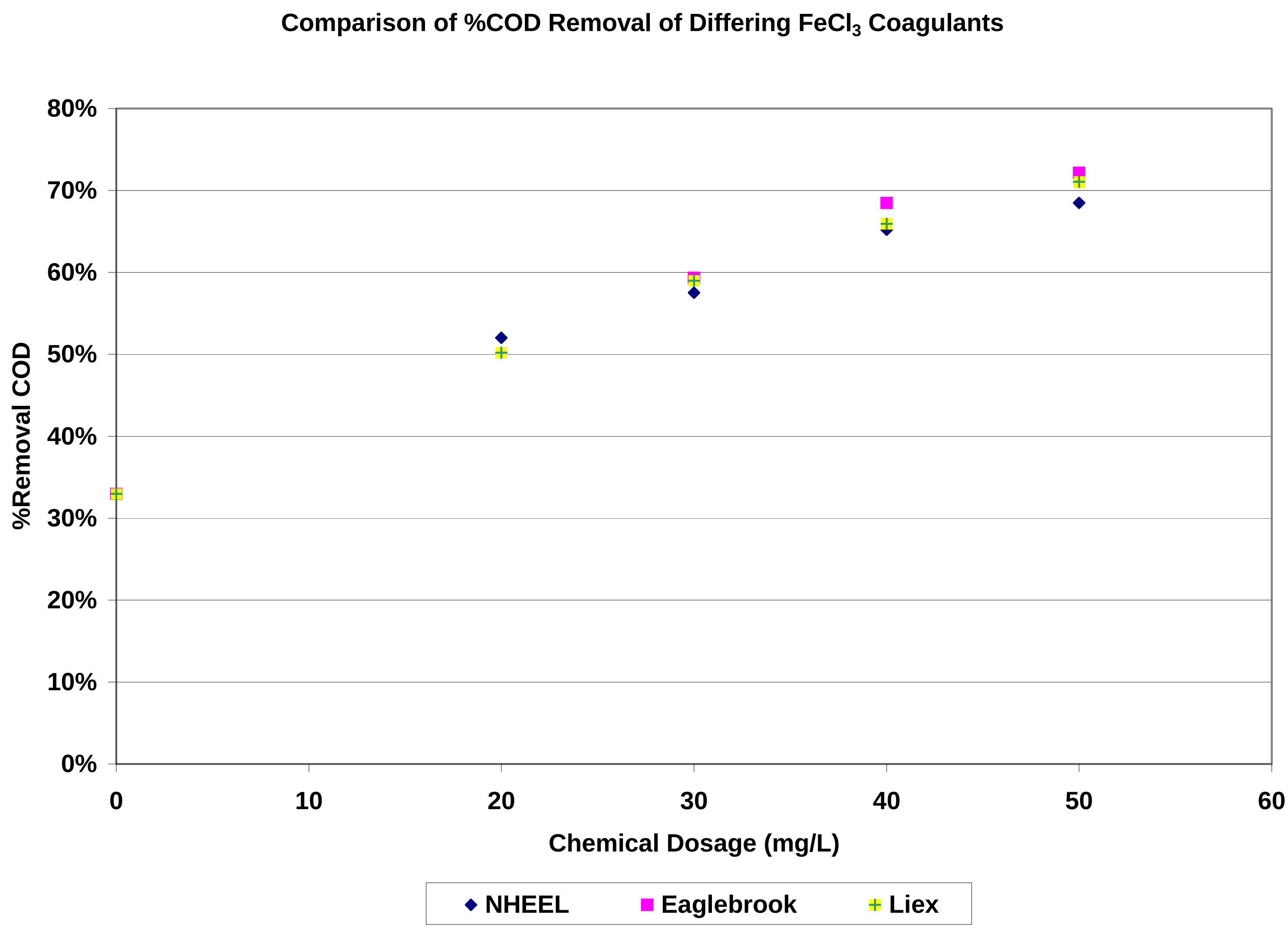

Eaglebrook
01/17/99
Removal Efficiencies
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
COD
TSS

Kemwater
01/17/99




Settling Test for Zero Dosage (TSS Removal)
TSS %Removal
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)


Comparison of %TSS Removal of Differing $FeCl_3$ Coagulants
120%
100%
80%
60%
40%
20%
0%
%Removal TSS
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
NHEEL
Eaglebrook
Liex


Comparison of %COD Removal of Differing $FeCl_3$ Coagulants
80%
70%
60%
50%
40%
30%
20%
10%
0%
%Removal COD
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
NHEEL
Eaglebrook
Liex

ID#	Date	Time	Sampling Location	Sampling Time	Sample (Jar #)	Coagulant
1	15-Jan	15:00	splitter box	11:00	1	Eaglebrook
2	15-Jan	15:00	splitter box	11:00	2	Eaglebrook
3	15-Jan	15:00	splitter box	11:00	3	Eaglebrook
4	15-Jan	15:00	splitter box	11:00	4	Eaglebrook
5	15-Jan	15:00	splitter box	11:00	Raw	None
6	15-Jan	15:30	splitter box	11:00	1	Eaglebrook
7	15-Jan	15:30	splitter box	11:00	2	Eaglebrook
8	15-Jan	15:30	splitter box	11:00	3	Eaglebrook
9	15-Jan	15:30	splitter box	11:00	4	Eaglebrook
10	15-Jan	16:00	splitter box	11:00	1	Zero
11	15-Jan	16:00	splitter box	11:00	2	Liex
12	15-Jan	16:00	splitter box	11:00	3	Liex
13	15-Jan	16:00	splitter box	11:00	4	Liex
14	15-Jan	17:00	splitter box	11:00	1	Liex
15	15-Jan	17:00	splitter box	11:00	2	Liex
16	15-Jan	17:00	splitter box	11:00	3	Liex
17	15-Jan	17:00	splitter box	11:00	4	Liex
18	15-Jan	17:30	splitter box	11:00	1	Kemwater
19	15-Jan	17:30	splitter box	11:00	2	Kemwater
20	15-Jan	17:30	splitter box	11:00	3	Kemwater
21	15-Jan	17:30	splitter box	11:00	4	Kemwater
22	15-Jan	18:00	splitter box	11:00	1	Kemwater
23	15-Jan	18:00	splitter box	11:00	2	Kemwater
24	15-Jan	18:00	splitter box	11:00	3	Kemwater
25	16-Jan	13:00	splitter box	10:00	1	Sanechlor
26	16-Jan	13:00	splitter box	10:00	2	Sanechlor
27	16-Jan	13:00	splitter box	10:00	3	Sanechlor
28	16-Jan	13:00	splitter box	10:00	4	Sanechlor
29	16-Jan	13:00	B1	10:00	B1	None
30	16-Jan	13:00	J1	10:00	J1	None
31	16-Jan	13:30	splitter box	10:00	1	Zero
32	16-Jan	13:30	splitter box	10:00	2	NHEEL
33	16-Jan	13:30	splitter box	10:00	3	NHEEL
34	16-Jan	13:30	splitter box	10:00	4	NHEEL
35	16-Jan	14:00	splitter box	10:00	1	NHEEL
36	16-Jan	14:00	splitter box	10:00	2	Eaglebrook
37	16-Jan	14:00	splitter box	10:00	3	Sanechlor
38	16-Jan	14:00	splitter box	10:00	4	Kemwater
39	16-Jan	14:00	splitter box	10:00	5	Liex
40	16-Jan	16:45	splitter box	10:00	1	Sludge
41	16-Jan	16:45	splitter box	10:00	2	Sludge
42	16-Jan	16:45	splitter box	10:00	3	Sludge
43	16-Jan	16:45	splitter box	10:00	4	Sludge
44	16-Jan	16:45	B1	10:00	B1	None
45	16-Jan	16:45	J1	10:00	J1	None
46	16-Jan	17:15	splitter box	10:00	1	NHEEL + sl.
47	16-Jan	17:15	splitter box	10:00	2	NHEEL + sl.
48	16-Jan	17:15	splitter box	10:00	3	NHEEL + sl.
49	16-Jan	17:15	splitter box	10:00	4	NHEEL + sl.
50	16-Jan	17:15	splitter box	18:00	Raw @18:00	None

51	16-Jan	17:15	J1	10:00	J1	None
52	16-Jan	17:15	splitter box	10:00	Raw	None
53	17-Jan	12:45	splitter box	10:20	Raw	None
54	17-Jan	12:45	splitter box	10:20	1	Liex
55	17-Jan	12:45	splitter box	10:20	2	Liex
56	17-Jan	12:45	splitter box	10:20	3	Liex
57	17-Jan	12:45	splitter box	10:20	4	Liex
58	17-Jan	15:15	splitter box	10:20	1	Zero
59	17-Jan	15:15	splitter box	10:20	2	Eaglebrook
60	17-Jan	15:15	splitter box	10:20	3	Eaglebrook
61	17-Jan	15:15	splitter box	10:20	4	Eaglebrook
62	17-Jan	15:45	splitter box	10:20	1	NHEEL
63	17-Jan	15:45	splitter box	10:20	2	NHEEL
64	17-Jan	15:45	splitter box	10:20	3	NHEEL
65	17-Jan	15:45	splitter box	10:20	4	NHEEL
66	17-Jan	16:10	splitter box	10:20	1	Kemwater
67	17-Jan	16:10	splitter box	10:20	2	Kemwater
68	17-Jan	16:10	splitter box	10:20	3	Kemwater
69	17-Jan	16:10	splitter box	10:20	4	Kemwater
70	17-Jan	17:00	splitter box	10:20	1	Sanechlor
71	17-Jan	17:00	splitter box	10:20	2	Sanechlor
72	17-Jan	17:00	splitter box	10:20	3	Sanechlor
73	17-Jan	17:00	splitter box	10:20	4	Sanechlor
74	17-Jan	17:30	splitter box	10:20	1	Liex
75	17-Jan	17:30	splitter box	10:20	2	Sanechlor
76	17-Jan	17:30	splitter box	10:20	3	NHEEL
77	17-Jan	17:30	splitter box	10:20	4	Kemwater
78	17-Jan	18:00	splitter box	10:20	1	Liex
79	17-Jan	18:00	splitter box	10:20	2	Sanechlor
80	17-Jan	18:00	splitter box	10:20	3	NHEEL
81	17-Jan	18:00	splitter box	10:20	4	Kemwater
82	17-Jan	18:00	splitter box	10:20	1	Zero
83	17-Jan	18:00	splitter box	10:20	Raw-brl btm	None
84	17-Jan	NA	B1	10:20	B1	None
85	17-Jan	NA	J1	10:20	J1	None
86	17-Jan	NA	splitter box	15:00	Raw	None
87	17-Jan	NA	G1	15:00	NA	None
88	17-Jan	NA	J1	15:00	NA	None
89	17-Jan	NA	splitter box	10:20	NA	None
90	18-Jan	11:30	splitter box	9:45	Raw	None
91	18-Jan	11:30	splitter box	9:45	1	GAC Alum
92	18-Jan	11:30	splitter box	9:45	2	GAC Alum
93	18-Jan	11:30	splitter box	9:45	3	GAC Alum
94	18-Jan	11:30	splitter box	9:45	4	GAC Alum
95	18-Jan	13:15	splitter box	9:45	1 min	Liex
96	18-Jan	13:15	splitter box	9:45	2 min	Liex
97	18-Jan	13:15	splitter box	9:45	3 min	Liex
98	18-Jan	13:15	splitter box	9:45	4 min	Liex
99	18-Jan	13:15	splitter box	9:45	5 min	Liex
100	18-Jan	13:45	splitter box	9:45	1 min	NHEEL

101	18-Jan	13:45	splitter box	9:45	2 min	NHEEL
102	18-Jan	13:45	splitter box	9:45	3 min	NHEEL
103	18-Jan	13:45	splitter box	9:45	4 min	NHEEL
104	18-Jan	13:45	splitter box	9:45	5 min	NHEEL
105	18-Jan	14:00	splitter box	9:45	1 min	Eaglebrook
106	18-Jan	14:00	splitter box	9:45	2 min	Eaglebrook
107	18-Jan	14:00	splitter box	9:45	3 min	Eaglebrook
108	18-Jan	14:00	splitter box	9:45	4 min	Eaglebrook
109	18-Jan	14:00	splitter box	9:45	5 min	Eaglebrook
110	18-Jan	15:00	splitter box	9:45	1 min	Sanechlor
111	18-Jan	15:00	splitter box	9:45	2 min	Sanechlor
112	18-Jan	15:00	splitter box	9:45	3 min	Sanechlor
113	18-Jan	15:00	splitter box	9:45	4 min	Sanechlor
114	18-Jan	15:00	splitter box	9:45	5 min	Sanechlor
115	18-Jan	15:00	splitter box	9:45	Raw	None
116	18-Jan	15:00	G1	9:45	G1	None
117	18-Jan	15:00	J1	9:45	J1	None
118	18-Jan	16:30	splitter box	9:45	1 min	None
119	18-Jan	16:30	splitter box	9:45	2 min	None
120	18-Jan	16:30	splitter box	9:45	3 min	None
121	18-Jan	16:30	splitter box	9:45	4 min	None
122	18-Jan	16:30	splitter box	9:45	5 min	None
123	18-Jan	16:30	splitter box	9:45	1	Kemwater (new)
124	18-Jan	16:30	splitter box	9:45	2	Kemwater (old)
125	18-Jan	16:30	splitter box	15:30	Raw	None
126	18-Jan	16:30	G1	15:30	G1	None
127	18-Jan	16:30	J1	15:30	J1	None
128	19-Jan	12:00	splitter box	10:00	1	Zero
129	19-Jan	12:00	splitter box	10:00	2	Alum
130	19-Jan	12:00	splitter box	10:00	3	Alum
131	19-Jan	12:00	splitter box	10:00	4	Alum
132	19-Jan	12:30	splitter box	10:00	1	NHEEL
133	19-Jan	12:30	splitter box	10:00	2	NHEEL
134	19-Jan	12:30	splitter box	10:00	3	NHEEL
135	19-Jan	12:30	splitter box	10:00	4	NHEEL
136	19-Jan	13:00	splitter box	10:00	1	Eaglebrook
137	19-Jan	13:00	splitter box	10:00	2	Eaglebrook
138	19-Jan	13:00	splitter box	10:00	3	Eaglebrook
139	19-Jan	13:00	splitter box	10:00	4	Eaglebrook
140	19-Jan	13:30	splitter box	10:00	1	Kemwater
141	19-Jan	13:30	splitter box	10:00	2	Kemwater
142	19-Jan	13:30	splitter box	10:00	3	Kemwater
143	19-Jan	13:30	splitter box	10:00	4	Kemwater
144	19-Jan	14:10	splitter box	10:00	1	Liex
145	19-Jan	14:10	splitter box	10:00	2	Liex
146	19-Jan	14:10	splitter box	10:00	3	Sanechlor
147	19-Jan	14:10	splitter box	10:00	4	Sanechlor
148	19-Jan	17:00	splitter box	10:00	1	NHEEL
149	19-Jan	17:00	splitter box	10:00	2	NHEEL
150	19-Jan	17:00	splitter box	10:00	3	NHEEL

151	19-Jan	17:00	splitter box	10:00	4	Liex
152	19-Jan	18:00	splitter box	10:00	1	NHEEL
153	19-Jan	18:00	splitter box	10:00	2	NHEEL
154	19-Jan	18:00	splitter box	10:00	3	Liex
155	19-Jan	18:00	splitter box	10:00	4	Eaglebrook
156	19-Jan	18:30	splitter box	10:00	1	NHEEL
157	19-Jan	18:30	splitter box	10:00	2	NHEEL
158	19-Jan	18:30	splitter box	10:00	3	Liex
159	19-Jan	19:00	splitter box	10:00	1	Sanechlor
160	19-Jan	19:00	splitter box	10:00	2	NHEEL
161	19-Jan	19:00	splitter box	10:00	3	Liex
162	19-Jan	19:00	splitter box	10:00	4	Eaglebrook
163	20-Jan	15:20	splitter box	14:30	1	Sanechlor
164	20-Jan	15:20	splitter box	14:30	2	NHEEL
165	20-Jan	15:20	splitter box	14:30	3	Sanechlor
166	20-Jan	15:20	splitter box	14:30	4	NHEEL
167	20-Jan	15:45	splitter box	14:30	1	Sanechlor
168	20-Jan	15:45	splitter box	14:30	2	NHEEL
169	20-Jan	15:45	splitter box	14:30	3	Sanechlor
170	20-Jan	15:45	splitter box	14:30	4	NHEEL
171	20-Jan	16:20	splitter box	14:30	1	Sanechlor
172	20-Jan	16:20	splitter box	14:30	2	NHEEL
173	20-Jan	17:15	splitter box	14:30	1	NHEEL
174	20-Jan	17:15	splitter box	14:30	2	NHEEL
175	20-Jan	17:15	splitter box	14:30	3	NHEEL
176	20-Jan	18:20	splitter box	14:30	1	NHEEL
177	20-Jan	18:20	splitter box	14:30	2	Alum (1.342)
178	21-Jan	11:00	splitter box	10:45	Raw	None
179	21-Jan	11:00	splitter box	10:45	2	Sanechlor
180	21-Jan	11:00	splitter box	10:45	3	Sanechlor
181	21-Jan	11:00	splitter box	10:45	4	Sanechlor
182	21-Jan	12:15	splitter box	10:45	1	Sanechlor
183	21-Jan	12:15	splitter box	10:45	2	Sanechlor
184	21-Jan	12:15	splitter box	10:45	3	Zero
185	21-Jan	13:00	splitter box	10:45	1	Sanechlor + Sl
186	21-Jan	13:00	splitter box	10:45	2	Sanechlor + Sl
187	21-Jan	13:00	splitter box	10:45	3	Sanechlor + Sl

ID#	Chemical Dosage (mg/L)	Polymer	Polymer Dosage (mg/l)	COD (mg/L)	COD %removal	TSS (mg/L)
1	0	None	0	237	24%	46
2	10	None	0	248	21%	44
3	20	None	0	247	21%	44
4	20	None	0	194	38%	50
5	0	None	0	312	NA	104
6	40	None	0	125	60%	6
7	50	None	0	121	61%	0
8	60	None	0	121	61%	12
9	70	None	0	122	61%	14
10	0	None	0	240	23%	34
11	10	None	0	235	25%	46
12	20	None	0	225	28%	46
13	30	None	0	199	36%	50
14	40	None	0	135	57%	16
15	50	None	0	119	62%	-4
16	60	None	0	121	61%	0
17	70	None	0	117	63%	10
18	10	None	0	239	23%	44
19	20	None	0	228	27%	38
20	30	None	0	225	28%	40
21	40	None	0	210	33%	46
22	50	None	0	205	34%	48
23	60	None	0	188	40%	54
24	70	None	0	178	43%	52
25	20	None	0	171	17%	50
26	30	None	0	142	31%	50
27	40	None	0	128	38%	50
28	60	None	0	96	53%	8
29	0	None	0	135	57%	94
30	0	None	0	77	75%	44
31	0	None	0	162	21%	52
32	30	None	0	112	45%	NA
33	40	None	0	88	57%	6
34	60	None	0	72	65%	12
35	50	None	0	68	67%	NA
36	50	None	0	76	63%	2
37	50	None	0	94	54%	24
38	50	None	0	151	26%	50
39	50	None	0	85	59%	14
40	10	None	0	166	19%	30
41	30	None	0	170	17%	36
42	50	None	0	152	26%	30
43	60	None	0	149	27%	24
44	0	None	0	153	51%	94
45	0	None	0	91	71%	52
46	10	None	0	151	26%	40
47	20	None	0	120	41%	32
48	30	None	0	103	50%	26
49	40	None	0	99	52%	18
50	0	None	0	327	NA	130

51	0	None	0	98	69%	144
52	0	None	0	205	NA	87
53	0	None	0	183	NA	62
54	20	None	0	136	50%	46
55	30	None	0	112	59%	34
56	40	None	0	93	66%	24
57	50	None	0	79	71%	18
58	0	None	0	183	33%	62
59	30	None	0	111	59%	22
60	40	None	0	86	68%	4
61	50	None	0	76	72%	8
62	20	None	0	131	52%	44
63	30	None	0	116	58%	24
64	40	None	0	95	65%	16
65	50	None	0	86	68%	4
66	20	None	0	171	37%	46
67	40	None	0	167	39%	44
68	60	None	0	138	49%	48
69	80	None	0	114	58%	34
70	20	None	0	162	41%	52
71	40	None	0	126	54%	36
72	60	None	0	91	67%	6
73	80	None	0	82	70%	6
74	40	None	0	111	59%	14
75	70	None	0	91	67%	8
76	40	None	0	108	60%	14
77	70	None	0	134	51%	40
78	20	None	0	137	50%	30
79	35	None	0	NA	NA	NA
80	20	None	0	173	37%	32
81	35	None	0	NA	NA	NA
82	0	None	0	192	30%	38
83	0	None	0	366	NA	154
84	0	None	0	130	58%	80
85	0	None	0	88	72%	38
86	0	None	0	361	NA	68
87	0	None	0	182	42%	104
88	0	None	0	107	66%	50
89	0	None	0	273	NA	100
90	0	None	0	267	NA	164
91	20	None	0	NA	NA	NA
92	40	None	0	NA	NA	NA
93	60	None	0	NA	NA	NA
94	80	None	0	NA	NA	NA
95	50	None	0	268	0%	154
96	50	None	0	168	37%	56
97	50	None	0	138	48%	36
98	50	None	0	135	49%	36
99	50	None	0	134	50%	32
100	50	None	0	260	3%	148

101	50	None	0	152	43%	48
102	50	None	0	130	51%	32
103	50	None	0	133	50%	32
104	50	None	0	139	48%	28
105	50	None	0	231	13%	128
106	50	None	0	123	54%	16
107	50	None	0	137	49%	40
108	50	None	0	121	55%	20
109	50	None	0	123	54%	20
110	50	None	0	301	-13%	196
111	50	None	0	238	11%	112
112	50	None	0	197	26%	84
113	50	None	0	184	31%	72
114	50	None	0	182	32%	76
115	0	None	0	158	NA	108
116	0	None	0	195	37%	104
117	0	None	0	105	66%	44
118	0	None	0	291	-9%	108
119	0	None	0	320	-20%	108
120	0	None	0	283	-6%	76
121	0	None	0	273	-2%	60
122	0	None	0	261	2%	52
123	60	None	0	NA	NA	NA
124	60	None	0	NA	NA	NA
125	0	None	0	429	NA	152
126	0	None	0	238	24%	108
127	0	None	0	142	54%	64
128	0	None	0	338	NA	36
129	20	None	0	357	NA	40
130	40	None	0	299	NA	44
131	60	None	0	303	NA	44
132	30	#13	0.5	NA	NA	NA
133	30	#15	0.5	NA	NA	NA
134	30	#17	0.5	NA	NA	NA
135	30	#19	0.5	NA	NA	NA
136	30	#13	0.5	NA	NA	NA
137	30	#15	0.5	NA	NA	NA
138	30	#17	0.5	NA	NA	NA
139	30	#19	0.5	NA	NA	NA
140	30	#13	0.5	NA	NA	NA
141	30	#15	0.5	NA	NA	NA
142	30	#17	0.5	NA	NA	NA
143	30	#19	0.5	NA	NA	NA
144	30	#15	0.5	NA	NA	NA
145	30	#17	0.5	NA	NA	NA
146	30	#15	0.5	NA	NA	NA
147	30	#17	0.5	NA	NA	NA
148	30	#17	0.5	NA	NA	NA
149	30	S-non	5	NA	NA	NA
150	30	#17	0.5	NA	NA	NA

151	30	#17	0.5	NA	NA	NA
152	30	S-non	5	NA	NA	NA
153	30	#17	0.5	NA	NA	NA
154	30	#17	0.5	NA	NA	NA
155	30	#17	0.5	NA	NA	NA
156	34	S-non	0.26	NA	NA	NA
157	34	#17	0.26	NA	NA	NA
158	34	#17	0.26	NA	NA	NA
159	50	#17	0.3	NA	NA	NA
160	30	#17	0.3	NA	NA	NA
161	30	#17	0.3	NA	NA	NA
162	30	#17	0.3	NA	NA	NA
163	60	#17	0.3	311	NA	28
164	30	#17	0.2	327	NA	48
165	50	#17	0.2	291	NA	36
166	25	#17	0.15	312	NA	48
167	50	#17	0.25	273	NA	24
168	25	#17	0.25	323	NA	48
169	40	#17	0.3	306	NA	40
170	35	#17	0.15	293	NA	28
171	60	#17	0.15	256	NA	16
172	35	#17	0.25	329	NA	32
173	25	#17	0.5	NA	NA	NA
174	25	S-cat	0.5	NA	NA	NA
175	25	S-an	0.5	NA	NA	NA
176	40	#17	0.3	NA	NA	NA
177	40	#17	0.3	NA	NA	NA
178	0	None	0	565	NA	300
179	42	#17	0.25	297	47%	36
180	46	#17	0.25	NA	NA	24
181	50	#17	0.25	NA	NA	32
182	42	#17	0.15	274	52%	32
183	42	#17	0.2	NA	NA	20
184	0	None	0	374	34%	64
185	40 + 25	#17	0.1	307	46%	44
186	40 + 40	#17	0.1	304	46%	44
187	40 + 0	#17	0.1	288	49%	36

ID#	TSS %removal	SO4	PO4	Floc Size	Visual Observation	Sample Volume
1	55.77%	NA	NA	NA	*Jar 3 had black specks	1L
2	57.69%	NA	NA	NA	that settled to the bottom	1L
3	57.69%	NA	NA	NA	and showed up on the .	1L
4	51.92%	NA	NA	NA	TSS filter.	1L
5	0.00%	NA	NA	NA	NA	NA
6	94.23%	NA	NA	b	NA	1L
7	100.00%	NA	NA	B	NA	1L
8	88.46%	NA	NA	c	NA	1L
9	86.54%	NA	NA	C	NA	1L
10	67.31%	NA	NA	-	NA	1L
11	55.77%	NA	NA	a	NA	1L
12	55.77%	NA	NA	a	NA	1L
13	51.92%	NA	NA	a	NA	1L
14	84.62%	NA	NA	B	NA	1L
15	103.85%	NA	NA	B	NA	1L
16	100.00%	NA	NA	B	NA	1L
17	90.38%	NA	NA	c	NA	1L
18	57.69%	NA	NA	NA	NA	1L
19	63.46%	NA	NA	NA	NA	1L
20	61.54%	NA	NA	NA	NA	1L
21	55.77%	NA	NA	NA	NA	1L
22	53.85%	NA	NA	B	NA	1L
23	48.08%	NA	NA	B	NA	1L
24	50.00%	NA	NA	B	NA	1L
25	43%	NA	NA	A	cloudy-green	1L
26	43%	NA	NA	C	cloudy-green	1L
27	43%	NA	NA	d	less green, less cloudy	1L
28	91%	NA	NA	D	clear supernatant	1L
29	29%	NA	NA	NA	NA	NA
30	67%	NA	NA	NA	NA	NA
31	40%	NA	NA	-	very cloudy	1L
32	NA	NA	NA	C	cloudy	1L
33	93%	NA	NA	d	partially clear	1L
34	86%	NA	NA	D	very clear	1L
35	NA	NA	NA	e	clearest, pin floc	1L
36	98%	NA	NA	e	very clear, no pin floc	1L
37	72%	NA	NA	e	golden/cloudy	1L
38	43%	NA	NA	B	cloudy, worst	1L
39	84%	NA	NA	D	small amount of pin floc	1L
40	66%	NA	NA	b	NA	1L
41	59%	NA	NA	b	NA	1L
42	66%	NA	NA	b	NA	1L
43	72%	NA	NA	c	NA	1L
44	29%	NA	NA	NA	NA	NA
45	61%	NA	NA	NA	NA	NA
46	54%	NA	NA	c	least clear	1L
47	63%	NA	NA	C	NA	1L
48	70%	NA	NA	d	NA	1L
49	79%	NA	NA	D	clearest	1L
50	NA	NA	NA	NA	NA	NA

51	-8%	NA	NA	NA	NA	NA
52	NA	NA	NA	NA	NA	NA
53	NA	NA	NA	NA	NA	NA
54	54%	NA	NA	A	very cloudy, lots of pin floc	1L
55	66%	NA	NA	B	cloudy, some pin floc	1L
56	76%	NA	NA	c	hazy/golden	1L
57	82%	NA	NA	C	clear	1L
58	38%	NA	NA	-	NA	1L
59	78%	NA	NA	d	golden-some pin floc	1L
60	96%	NA	NA	D	slightly golden - little pin floc	1L
61	92%	NA	NA	E	clear	1L
62	56%	NA	NA	c	lots of pin floc	1L
63	76%	NA	NA	C	golden - some pin floc	1L
64	84%	NA	NA	D	clear	1L
65	96%	NA	NA	E	clear	1L
66	54%	NA	NA	A	cloudy	1L
67	56%	NA	NA	b	cloudy	1L
68	52%	NA	NA	c	cloudy	1L
69	66%	NA	NA	d	golden, pin floc	1L
70	48%	NA	NA	b	very cloudy	1L
71	64%	NA	NA	d	cloudy	1L
72	94%	NA	NA	e	cloudy	1L
73	94%	NA	NA	E	clear	1L
74	86%	NA	NA	e	NA	1L
75	92%	NA	NA	E	NA	1L
76	86%	NA	NA	e	NA	1L
77	60%	NA	NA	d	NA	1L
78	70%	NA	NA	e	NA	1L
79	NA	NA	NA	e	NA	1L
80	68%	NA	NA	e	NA	1L
81	NA	NA	NA	C	NA	1L
82	62%	NA	NA	NA	NA	1L
83	NA	NA	NA	NA	NA	NA
84	40%	NA	NA	NA	NA	NA
85	71%	NA	NA	NA	NA	NA
86	NA	NA	NA	NA	NA	NA
87	22%	NA	NA	NA	NA	NA
88	62%	NA	NA	NA	NA	NA
89	NA	NA	NA	NA	NA	NA
90	NA	42	>2.75	NA	NA	NA
91	NA	NA	NA	A	cloudy	1L
92	NA	NA	NA	c	cloudy	1L
93	NA	NA	NA	C	golden, lots of pin floc	1L
94	NA	NA	NA	D	lots of pin floc	1L
95	6%	NA	NA	NA	NA	2L
96	66%	NA	NA	NA	NA	2L
97	78%	NA	NA	NA	NA	2L
98	78%	NA	NA	NA	NA	2L
99	80%	NA	NA	NA	NA	2L
100	10%	NA	NA	NA	NA	2L

101	71%	NA	NA	NA	NA	2L
102	80%	NA	NA	NA	NA	2L
103	80%	NA	NA	NA	NA	2L
104	83%	NA	NA	NA	NA	2L
105	22%	NA	NA	NA	NA	2L
106	90%	NA	NA	NA	NA	2L
107	76%	NA	NA	NA	NA	2L
108	88%	NA	NA	NA	NA	2L
109	88%	NA	NA	NA	NA	2L
110	-20%	NA	NA	NA	NA	2L
111	32%	NA	NA	NA	NA	2L
112	49%	NA	NA	NA	NA	2L
113	56%	NA	NA	NA	NA	2L
114	54%	NA	NA	NA	NA	2L
115	NA	NA	NA	NA	NA	NA
116	22%	NA	NA	NA	NA	NA
117	67%	NA	NA	NA	NA	NA
118	34%	NA	NA	NA	NA	1L
119	34%	NA	NA	NA	NA	1L
120	54%	NA	NA	NA	NA	1L
121	63%	NA	NA	NA	NA	1L
122	68%	NA	NA	NA	NA	1L
123	NA	NA	NA	C	cloudy	1L
124	NA	NA	NA	C	cloudy	1L
125	NA	NA	NA	NA	NA	NA
126	19%	NA	NA	NA	NA	NA
127	52%	NA	NA	NA	NA	NA
128	NA	NA	NA	-	*All were very cloudy with	1L
129	NA	NA	NA	B	lots of pin floc.	1L
130	NA	NA	NA	B		1L
131	NA	NA	NA	c		1L
132	NA	NA	NA	F	worst	1L
133	NA	NA	NA	>>G	best	1L
134	NA	NA	NA	>>G	a close second	1L
135	NA	NA	NA	>>G	third	1L
136	NA	NA	NA	D	last	1L
137	NA	NA	NA	>>G	close second	1L
138	NA	NA	NA	>>G	best	1L
139	NA	NA	NA	>>G	third	1L
140	NA	NA	NA	D	4th	1L
141	NA	NA	NA	>G	1st	1L
142	NA	NA	NA	>G	close 2nd	1L
143	NA	NA	NA	F	3rd	1L
144	NA	NA	NA	>>G	2nd	1L
145	NA	NA	NA	>>G	1st	1L
146	NA	NA	NA	>>G	4th	1L
147	NA	NA	NA	>>G	3rd	1L
148	NA	NA	NA	>>G	NA	1L
149	NA	NA	NA	>>G	NA	1L
150	NA	NA	NA	>>G	NA	1L

151	NA	NA	NA	>>G	NA	1L
152	NA	NA	NA	>>G	*All performed similarly	1L
153	NA	NA	NA	>>G	*All were a bit foggy	1L
154	NA	NA	NA	>>G		1L
155	NA	NA	NA	>>G		1L
156	NA	NA	NA	f	NA	1L
157	NA	NA	NA	G	NA	1L
158	NA	NA	NA	G	NA	1L
159	NA	NA	NA	NA	2nd	1L
160	NA	NA	NA	NA	3rd	1L
161	NA	NA	NA	NA	4th	1L
162	NA	NA	NA	NA	1st	1L
163	NA	NA	NA	NA	NA	1L
164	NA	NA	NA	NA	NA	1L
165	NA	NA	NA	NA	NA	1L
166	NA	NA	NA	NA	NA	1L
167	NA	NA	NA	F	3rd	1L
168	NA	NA	NA	>G	settled fasted	1L
169	NA	NA	NA	>G	2nd	1L
170	NA	NA	NA	d	4th	1L
171	NA	NA	NA	f	smaller flocs	1L
172	NA	NA	NA	G	settled much faster	1L
173	NA	NA	NA	>>G	*The #17 combo was by far	1L
174	NA	NA	NA	c	the best. The two provided	1L
175	NA	NA	NA	c	by SABESP were cloudy.	1L
176	NA	NA	NA	>>G	much better	1L
177	NA	NA	NA	F	not as good	1L
178	NA	75	8.12	NA	NA	NA
179	88%	66	5.88	>G	NA	2L
180	92%	NA	NA	G	NA	2L
181	89%	NA	NA	G	NA	2L
182	89%	65	5.52	F	settled fastest	2L
183	93%	NA	NA	F	NA	2L
184	79%	45	7.72	NA	NA	2L
185	85%	65	NA	F	NA	2L
186	85%	67	NA	F	NA	2L
187	88%	67	NA	F	NA	2L

ID#	Purpose => Results
1	Test Eaglebrook
2	Test Eaglebrook
3	Test Eaglebrook
4	Test Eaglebrook
5	NA
6	Test Eaglebrook
7	Test Eaglebrook
8	Test Eaglebrook
9	Test Eaglebrook
10	Test Liex
11	Test Liex
12	Test Liex
13	Test Liex
14	Test Liex
15	Test Liex
16	Test Liex
17	Test Liex
18	Test Kemwater
19	Test Kemwater
20	Test Kemwater
21	Test Kemwater
22	Test Kemwater
23	Test Kemwater
24	Test Kemwater
25	Test Sanechlor
26	Test Sanechlor
27	Test Sanechlor
28	Test Sanechlor
29	NA
30	NA
31	Test NHEEL
32	Test NHEEL
33	Test NHEEL
34	Test NHEEL
35	A side-by-side comparison of five Iron salts.
36	A side-by-side comparison of five Iron salts.
37	A side-by-side comparison of five Iron salts.
38	A side-by-side comparison of five Iron salts.
39	A side-by-side comparison of five Iron salts.
40	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
41	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
42	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
43	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
44	NA
45	NA
46	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
47	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
48	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
49	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
50	NA

51	NA
52	NA
53	NA
54	Liex series
55	Liex series
56	Liex series
57	Liex series
58	Eaglebrook series
59	Eaglebrook series
60	Eaglebrook series
61	Eaglebrook series
62	NHEEL series
63	NHEEL series
64	NHEEL series
65	NHEEL series
66	Kemwater Series
67	Kemwater Series
68	Kemwater Series
69	Kemwater Series
70	Sanechlor series
71	Sanechlor series
72	Sanechlor series
73	Sanechlor series
74	Side-by-side comparison
75	Side-by-side comparison
76	Side-by-side comparison
77	Side-by-side comparison
78	Side-by-side comparison
79	Side-by-side comparison
80	Side-by-side comparison
81	Side-by-side comparison
82	NA
83	NA
84	NA
85	NA
86	NA
87	NA
88	NA
89	NA
90	NA
91	GAC Alum series
92	GAC Alum series
93	GAC Alum series
94	GAC Alum series
95	Settling test for Liex
96	Settling test for Liex
97	Settling test for Liex
98	Settling test for Liex
99	Settling test for Liex
100	Settling test for NHEEL

101 Settling test for NHEEL
102 Settling test for NHEEL
103 Settling test for NHEEL
104 Settling test for NHEEL
105 Settling test for Eaglebrook
106 Settling test for Eaglebrook
107 Settling test for Eaglebrook
108 Settling test for Eaglebrook
109 Settling test for Eaglebrook
110 Settling test for Sanechlor
111 Settling test for Sanechlor
112 Settling test for Sanechlor
113 Settling test for Sanechlor
114 Settling test for Sanechlor
115 NA
116 NA
117 NA
118 Zero chemical settling test.
119 Zero chemical settling test.
120 Zero chemical settling test.
121 Zero chemical settling test.
122 Zero chemical settling test.
123 Compare old Kemwater to new Kemwater provided by SABESP.
124 Compare old Kemwater to new Kemwater provided by SABESP.
125 NA
126 NA
127 NA
128 Test Alum provided by SABESP => Very poor performance.
129 Test Alum provided by SABESP => Very poor performance.
130 Test Alum provided by SABESP => Very poor performance.
131 Test Alum provided by SABESP => Very poor performance.
132 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
133 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
134 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
135 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
136 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
137 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
138 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
139 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
140 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
141 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
142 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
143 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
144 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
145 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
146 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
147 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
148 *To compare anionic polymer #17 to SABESP's non-ionic (S-non) polymer & compare NHEEL
149 to Liex (both using anionic polymer #17) to find out which will perform better with this specific
150 polymer. => Anionic polymer #17 performed much better than "S-non"; NHEEL was performed

151 better than Liex with anionic polymer #17.
152 *To compare the three best performing Iron (Fe) salts with anionic polymer #17 & to compare
153 anionic polymer #17 to SABESP's non-ionic (S-non). All Fe salts (with polymer #17) performed
154 very similarly, especially in the fact that they all formed very large floc (>>G) that settled very
155 rapidly (in a matter of seconds). S-non had nowhere near as good a performance.
156 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
157 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
158 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
159 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
160 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
161 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
162 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
163 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
164 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
165 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
166 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
167 To compare NHEEL with Sanechlor.
168 To compare NHEEL with Sanechlor.
169 To compare NHEEL with Sanechlor.
170 To compare NHEEL with Sanechlor.
171 To compare Sanechlor and NHEEL.
172 To compare Sanechlor and NHEEL.
173 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
174 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
175 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
176 To compare Aum (1.342) from SABESP to NHEEL.
177 To compare Aum (1.342) from SABESP to NHEEL.
178 NA
179 To find the best Sanechlor and polymer combination
180 To find the best Sanechlor and polymer combination
181 To find the best Sanechlor and polymer combination
182 To find the best Sanechlor and polymer combination
183 To find the best Sanechlor and polymer combination
184 To find the best Sanechlor and polymer combination
185 Test the recycling of chemical sludge at varying dosages.
186 Test the recycling of chemical sludge at varying dosages.
187 Test the recycling of chemical sludge at varying dosages.

APPENDIX-B: LAGOON MODELING

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.

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 B-1.

Table B-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 B-1 presents a schematic of the processes involved in facultative ponds.

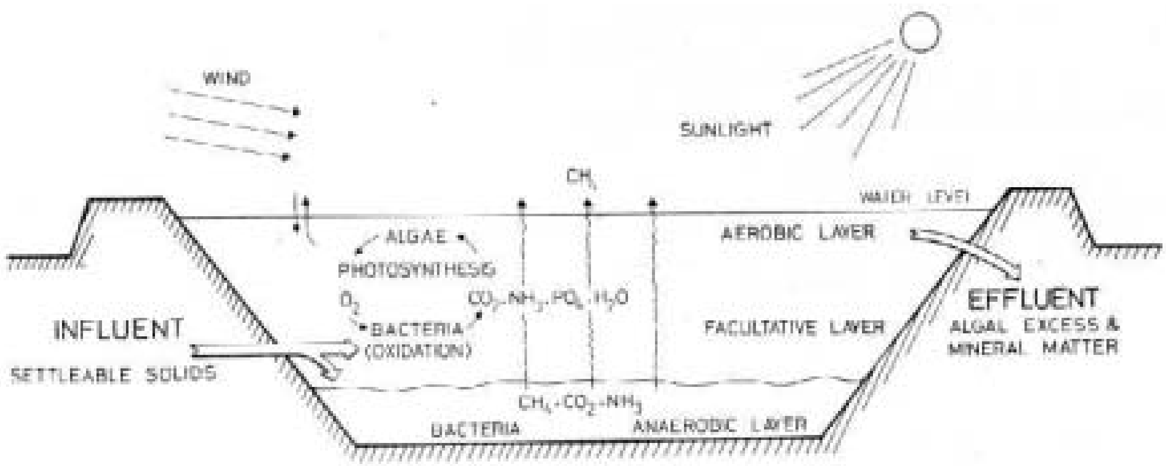


Figure B-1: Processes involved in Facultative Ponds

As can be seen in Figure B-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 B-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 B-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 B-3.

Table B-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

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 B-1 through B-3.

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

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

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

The legend to these equations is presented in Table B-4.

Table B-4: Legend for Equations B-1 to B-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 B-4.

$$R_{XY}^{T} = R_{XY}^{20} \cdot \theta^{(T-20)} \tag{B-4}$$

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 B-2 for Riviera de São Lorenço WWTP schematic.)

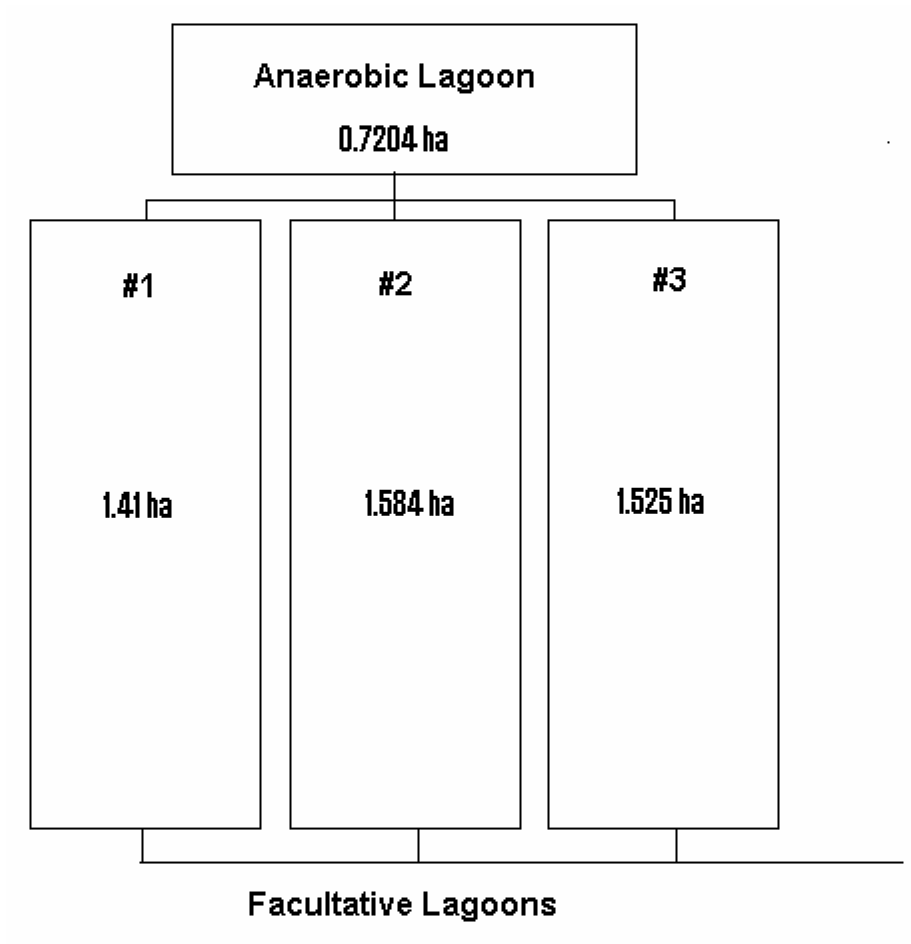


Figure B-2: Riviera de São Lorenço Treatment System Schematic

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 B-3, 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.

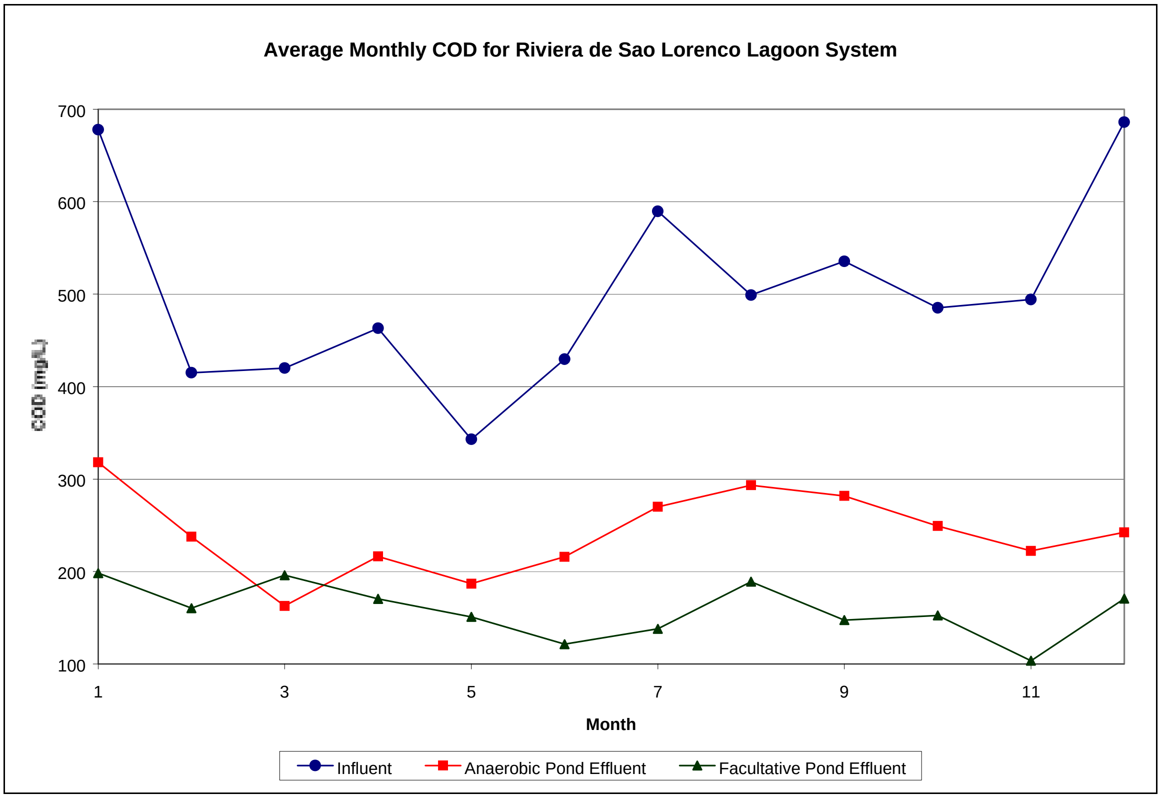


Figure B-3: Monthly Averaged COD values for the Riviera de São Lorenço Lagoon System

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 B-3 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 B-4 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.

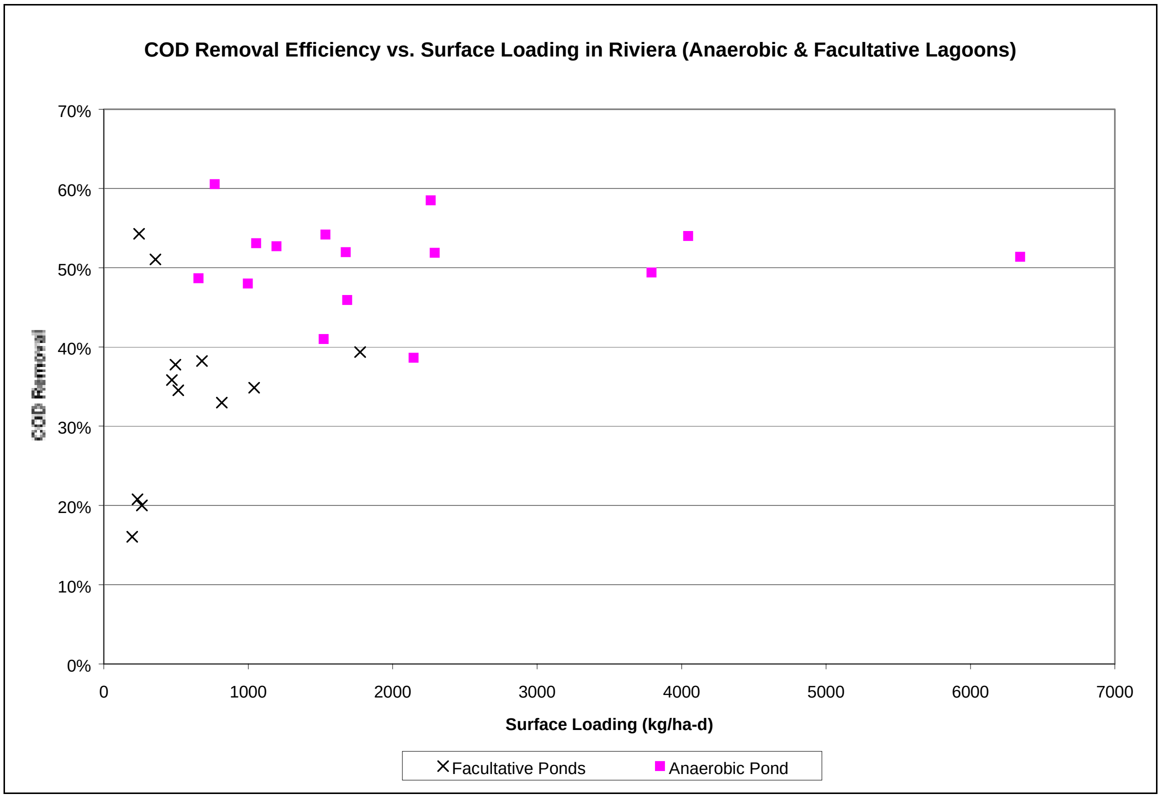


Figure B-4: Removal Efficiency vs. Surface Loading, Riviera de São Lorenço

Due to the very low loading of the anaerobic pond, Figure B-4 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 B-5 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.

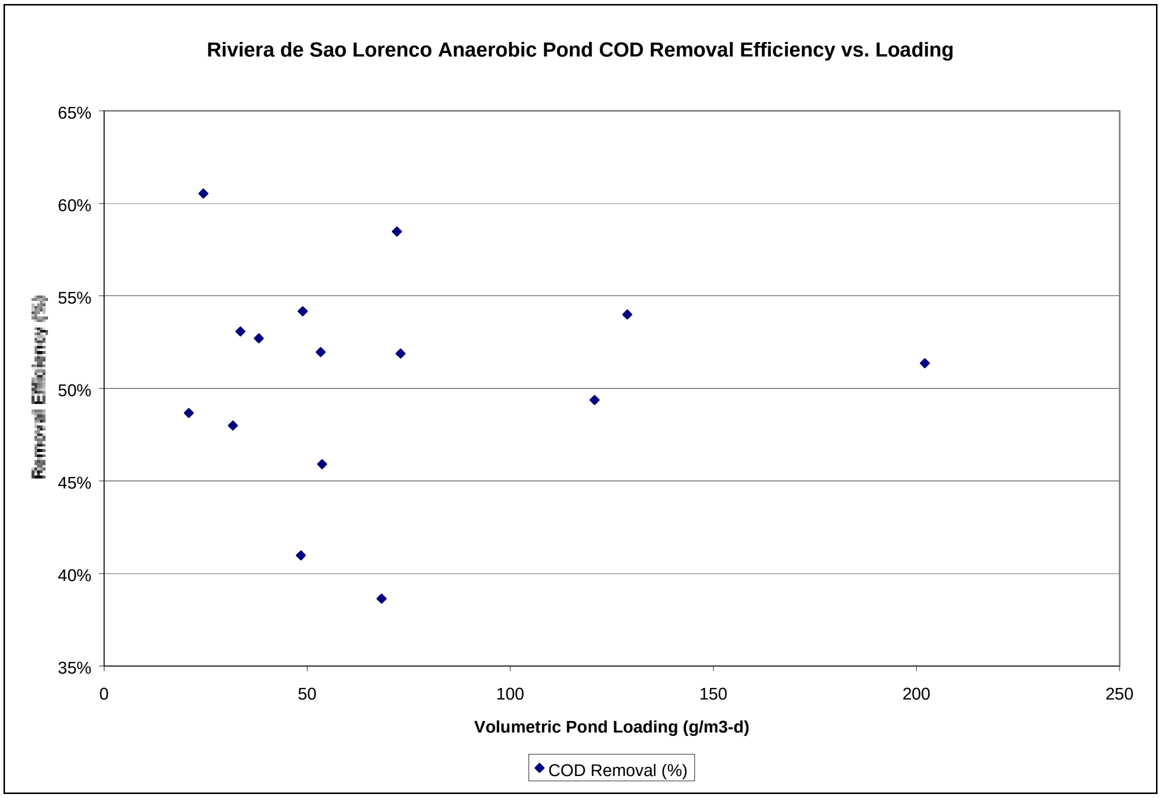


Figure B-5: COD Removal Efficiency vs. Volumetric Loading, Riviera Anaerobic Pond

Figure B-5 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 B-5 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 B-6. 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.

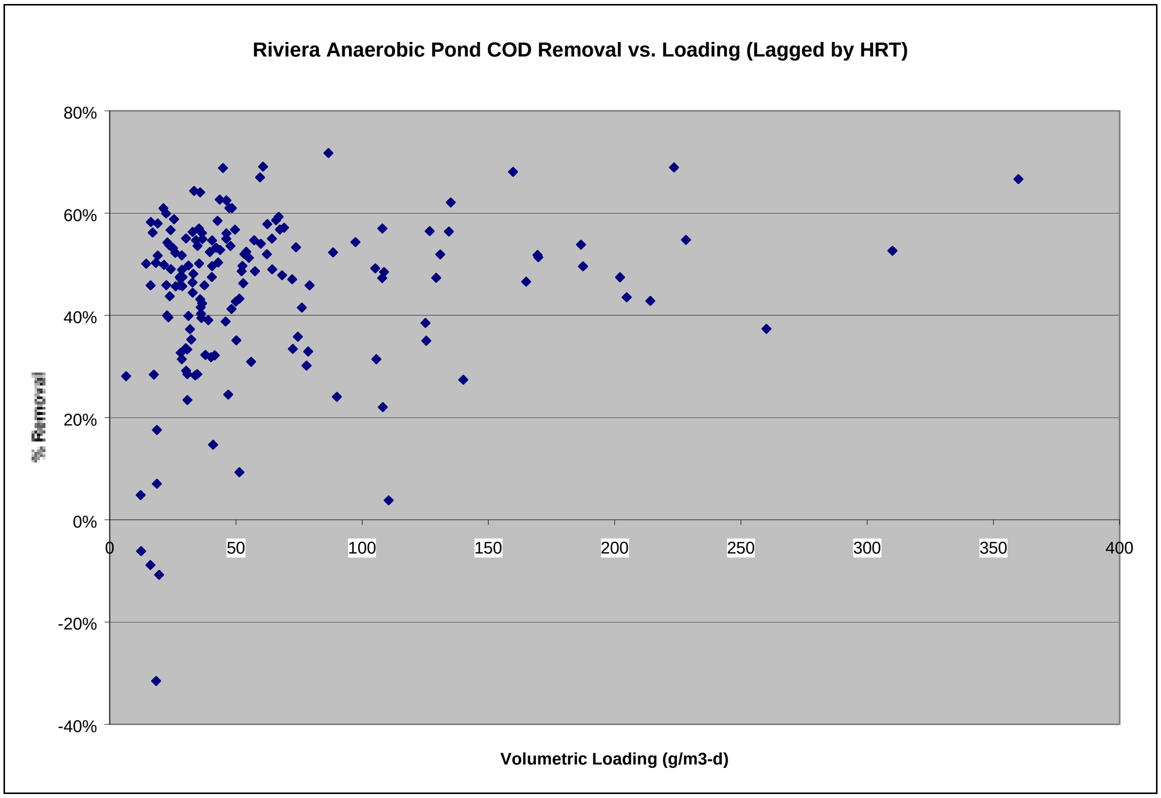


Figure B-6: Anaerobic Pond COD Removal (Effluent Lagged by HRT)

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 B-5.

Accumulation = inflow – outflow ± surface heat exchange (B-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 B-6 and B-7.

$$\text{Inflow} = Q*\rho*C_p*T_{in}(t) \quad \text{(B-6)}$$

$$\text{Outflow} = Q*\rho*C_p*T \quad \text{(B-7)}$$

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^{o}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 B-7 presents a schema of all processes involved in surface heat exchange. These processes, as seen in Figure B-7, 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.

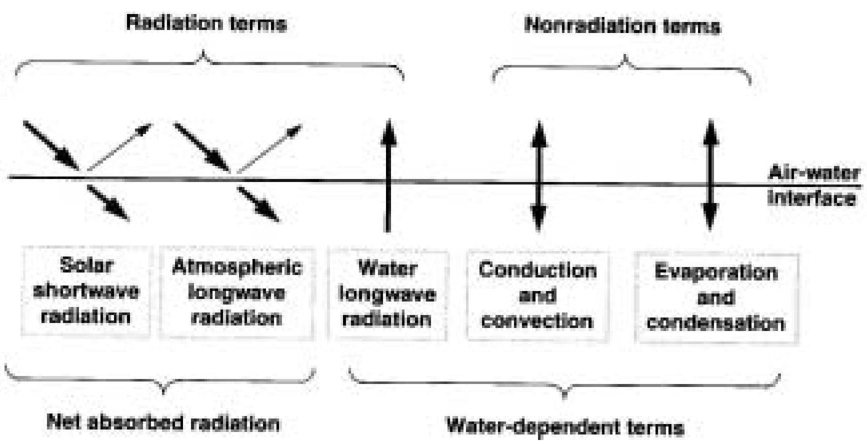


Figure B-7: Schema of Surface Heat Exchange Processes (Chapra, 1997)

The net surface heat exchange can be represented as

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

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 B-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 B-9 through B-12 represent the terms involved in the surface heat exchange. The atmospheric longwave radiation is expressed as

$$J_{an} = \underbrace{\sigma*(T_{air}+273)^4}_{\text{(Stefan-Bolzmann Law)}}*\underbrace{(A+0.031\sqrt{e_{air}})}_{\text{(Atmospheric attenuation)}}*\underbrace{(1-R_L)}_{\text{(Reflection)}} \tag{B-9}$$

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

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

where: ε = emissivity of water (0.97)

T_s = water surface temperature

The conductive heat transfer is expressed as

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

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

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

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

$$e = 4.596 * e^{(17.27*T / 237.3 + T)} \qquad \text{(B-13)}$$

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 B-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 B-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 B-8.

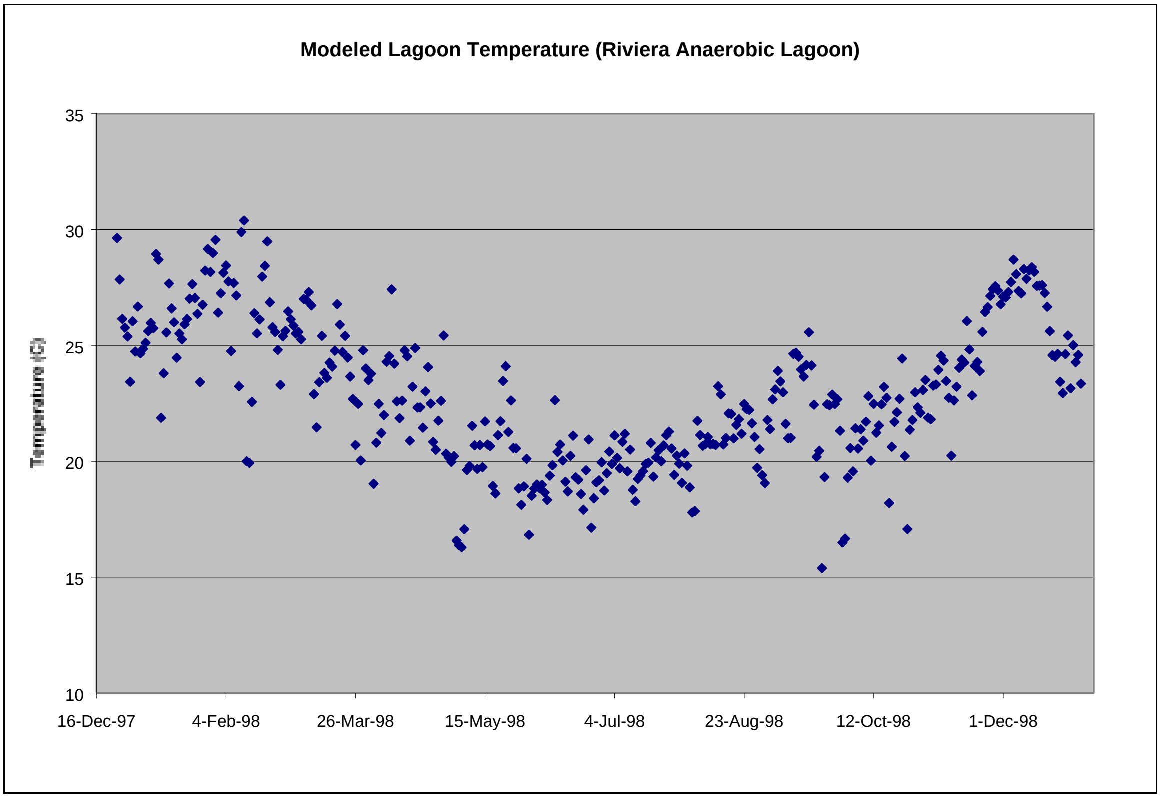


Figure B-8: Modeled Temperature of Riviera Anaerobic Lagoon

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).

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 B-5.

Table B-5: 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.

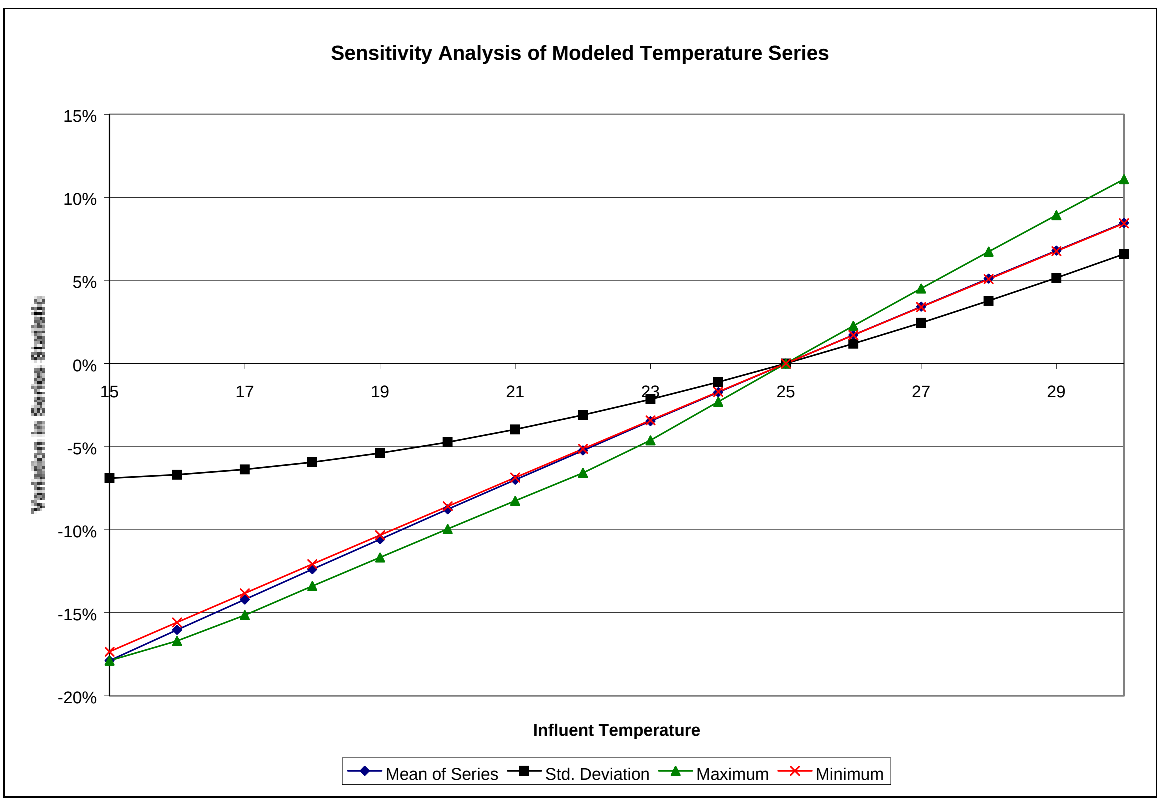


Figure B-9: Plot of Sensitivity Analysis Results

The plot of the sensitivity analysis (Figure B-9) 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^oC). 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^oC 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 B-10.

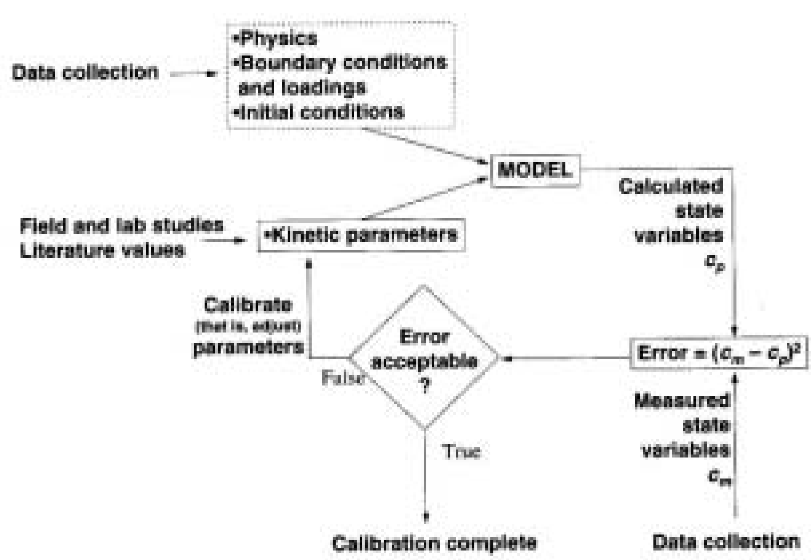


Figure B-10: Schema of Model Calibration Procedure (Chapra, 1998)

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 B-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^oC, 25^oC and 30^oC influent.

The models based on the three different influent temperatures are very close to each other in Figure B-11. 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^oC, an assumption of a constant 25^oC influent is acceptable.

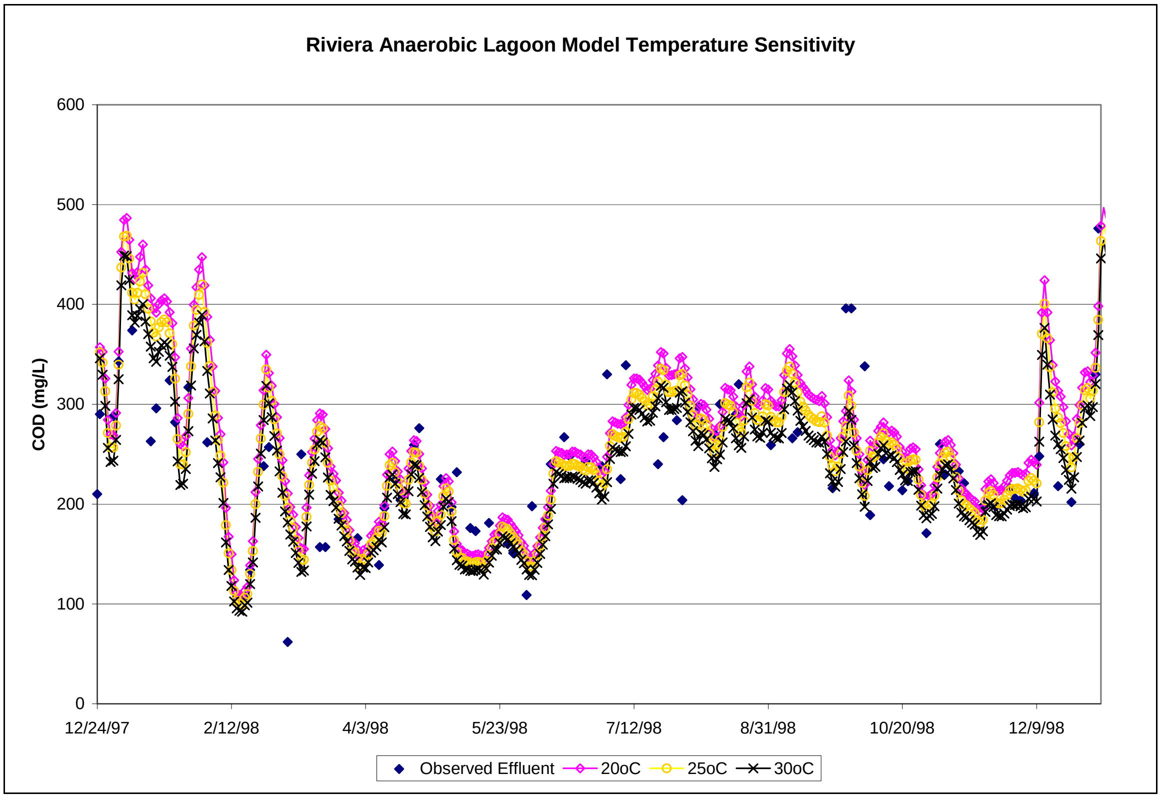


Figure B-11: Riviera Anaerobic Lagoon Model Sensitivity to Lagoon Temperature

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.

Conclusions

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 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. 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 developed on the Brazilian data was used to design a lagoon to follow the CEPT stage for the CEAGESP treatment facility in Tatui, Brazil.

APPENDIX-C: FINANCIAL ANALYSIS

APPENDIX-D: SABESP 1992 REPORT (TSUTIYA & CASSETTARI)

Introduction

This appendix summarizes the Tatui Report from 1992, which contains the most recent information, found in the literature, regarding Tatui main wastewater treatment lagoon, ETE CEAGESP. It was prepared by Milton Tomoyuki Tsutiya and Orlando Zuliani Cassettari as a response to an assessment required by Sabesp in 1992 (the report was translated by Christian Cabral).

The CEAGESP wastewater treatment plant began operating in 1978. At that time the service population was approximately 20,000 inhabitants and the anaerobic and the facultative lagoon were approximately 2.5 meters deep.

Since the treatment plant has no provision for sludge removal, the sludge accumulation in these lagoons has decreased the detention time of the system and, therefore, its efficiency.

The population by the year of 1992 using this facility was around 49,000 and the overall BOD removal efficiency only 60%. At that time, depth of the anaerobic pond was 1.5m and the facultative, only 1.3m.

The major problems related to the maintenance of the lagoons are: short circuiting, due to the irregular sludge settling and overflow rates, and short detention time, related to the sludge accumulation in the bottom of these lagoons.

The collection system

The Tatui Report also evaluated the city's wastewater collection system and its performance. The flow ranged from 88.06 to 176.18 L/sec/inhab in 1992. The sewage return coefficient (the volume of sewage produced divided by amount of water consumed) varied from 52% to 84%, which is considered normal. The averages of maximum flow coefficient and the minimum flow coefficient per hour was 2.69 and 0.37. Regarding the condition of the collection system, the infiltration rate was 0.33 l/sec/km of pipe. Compared to Brazilian standards these results are considered as normal, except for the infiltration rates, which is considered high.

The Lagoons

The report also shows the characteristics of the wastewater and sludge based on this analyzes pH and temperature measurements made every 30 minutes during one week (09/14/92 until 09/21/92):

Table C-1

	Minimum	Maximum	Unit.
Values of pH	4	7	[pH]
Air temperature	12	30	Celsius
Water temperature	17	30	Celsius

Although the variation of pH is unusual (too acid for tropical ponds) 90% of the measurements were around pH 7.

Wastewater Measurements

To represent the influent wastewater conditions several parameters were chosen and the average is shown in the following tables.

Suspended Solids, BOD and COD averages:

Table C-2

	Averages	Unit
BOD	73.5	Grams/inhabitant/day
BOD filtered	37.5	Grams/inhabitant/day
TSS	35.8	Grams/inhabitant/day
VSS	44.0	Grams/inhabitant/day

The average BOD in Brazil is 54 grams/inhabitant/day; therefore these results indicate that these lagoons were already working over their capacity in 1992.

Sludge Analyses

The following data about the anaerobic and facultative lagoons gives the average depth of the sludge accumulated through 14 years:

Anaerobic Lagoon Sludge Accumulation:

Table C-3

Initial Lagoon Volume (1978)	35,326 m3	Final Lagoon Volume (1992)	23,786 m3
Area	23,551 m2	Final Sludge Volume	11,540 m3
Average Sludge Depth	49 cm	Sludge Percentage	31.2 %
Average Sludge Accumulation	Per year	3.9	Cm/year

Facultative Lagoon Sludge Accumulation:

Table C-4

Initial Lagoon Volume (1978)	32,765 m3	Final Lagoon Volume (1992)	26,060 m3
Area	25,204 m2	Final Sludge Volume	67,10 m3
Average Sludge Depth	26.6 cm	Sludge Percentage	17.7 %
Average Sludge Accumulation	per year	2.2	Cm/year

The solids composition of the sludge is shown in the next table:

Table C-5

Total Solids	9.26 %
Fixed Solids	5.28 %
Volatile Solids	3.98 %
Total Suspended Solids	8.54 %
Fixed Suspended Solids	5.53 %
Volatile Suspended Solids	3.01 %

The Biological Analyses of the Sludge:

The report from 1992 also analyses the pathogenic microorganism concentration of the sludge as follows:

Table C-6

Salmonellas	<2	To	9	MPN/100ml	
Total Coliforms	8	To	24	MPN/100ml	10^5
Fecal Coliforms	2.2	To	17	MPN/100ml	10^5
Ascaris Lumbricoides	70	To	110	/100mg of sludge	
Enterobius Vermiclaris	0	To	90	/100mg of sludge	
Trichuris Trichiura	0	To	10	/100mg of sludge	
Hymendepis Nana	0	To	40	/100mg of sludge	
Clonorchis Simensis	10	To	20	/100mg of sludge	
Anacilostomideos	0	To	10	/100mg of sludge	
Balantiduim Coli	0	To	10	/100mg of sludge	

Although the Salmonellas count represents a normal concentration, the concentration of the rest of the pathogenic organisms is considered too high for agricultural application on vegetable crop according to Sabesp's standards.

Present Situation

Nowadays, these lagoons are less efficient than in 1992. From our visual observation we noticed some extra factors that appear to interfere in the wastewater treatment efficiency.

First, the condition of the algae growth on the surface of the lagoons and vegetation all over the margins suggests that there is a lack of proper maintenance (cleaning).

Second, the access to this facility is in bad conditions making it more difficult to maintain the area.

Third, some of the inlets to the anaerobic and facultative lagoons are blocked and the effect of hydraulic short circuiting is aggravated on account of this.

Fourth, the permanent usage of the discharge of river from the first lagoon is damaging the condition of the receiving body (River Manduca) suggesting necessary corrections to clean up the pollution to the river.

Despite these negative aspects of the present situation of the lagoon, there is almost no odor problem even when there is no wind.