Chemically Enhanced Primary Treatment in Alfenas, Brazil

CEPT Plant Design, Sludge Management Strategy, and Reservoir Impact Analysis

by

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Executive Summary

This report presents a solution to the human health, environmental and wastewater management problems of the Furnas Reservoir region of Brazil. Lacking wastewater treatment facilities, Alfenas, and other cities in the Furnas Reservoir region, are polluting the already drought compromised reservoir, which also serves as their drinking water source. Chemically enhanced primary treatment (CEPT) is recommended as a cost effective and feasible wastewater treatment system. Field research results of bench scale testing of the wastewater and laboratory analysis results are presented and analyzed to support design parameters. Two proposed treatments are compared in terms of efficiency in treatment, cost effectiveness and other considerations. A preliminary plant design is presented, along with proposed layout, location and equipment specification guidelines. The CEPT plant, designed for the city of Alfenas, will serve as a model for the Furnas Reservoir region.

This report also presents a financially feasible strategy for the treatment and beneficial use of the sludge produced by the proposed plant. Based on data collected during the field study, and an examination of U.S. and Brazilian regulations on the use of sludge, a sludge treatment system has been designed. Treatment recommendations include disinfection, thickening, and drying the sludge, making it available for use as a fertilizer on local crops. In this study sludge application to coffee crops, the dominant agricultural product in the area, was evaluated as a potential beneficial use strategy. The nutrient value of the sludge was assessed and preliminary land application rates have been calculated. A pilot study at the University of Alfenas coffee farm has been recommended to further study the fertilizer value of the sludge and determine appropriate application rates.

In addition, this report examines how the water quality in the Furnas reservoir might improve due to the use of CEPT in the region. Through testing, the bacterial and nutrient levels in the reservoir are determined and compared with the reduced levels that are expected through CEPT treatment. A mathematical model is developed to quantify the flows to and from the reservoir. The model is used to predict the bacterial and nutrient concentrations in the reservoir using annual water quality data for the reservoir and data obtained from field studies for a section of the reservoir directly adjacent to the city. The results show that substantial decreases in fecal coliform, phosphorus, biochemical oxygen demand (BOD), and phosphorous will result from the implementation CEPT.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. Introduction

The goal of this project is to propose a wastewater treatment and sludge management strategy that is financially and technically feasible for the city of Alfenas, Brazil and analyze its impact on the Furnas Reservoir. The lack of wastewater treatment facilities in the region is exacerbating existing environmental problems. In order to address the region's need for cost effective and technically feasible wastewater treatment a chemically enhanced primary treatment (CEPT) plant is proposed for the city of Alfenas, a 60,000-inhabitant city in the southern portion of the Furnas Reservoir region. This plant is part of a comprehensive regional solution and should serve as a model for other cities surrounding the reservoir. The proposed CEPT plant utilizes the metal salt, ferric chloride, and a locally available organic polymer, Tanfloc, to enhance settling and provide sufficient solids and nutrient removal.

The most expensive phase of most wastewater treatment systems is sludge treatment and disposal. While the effluent leaves the plant relatively clean, the wastewater residuals must be handled carefully in order to prevent the reintroduction of these contaminants into the environment and to minimize health risks to the local community. Without appropriate treatment and disposal, the sludge can be more harmful than the raw sewage and the proposed treatment plant will not have the desired effect of improving the human and environmental health in the city and the region. The existing environmental problems and financial limitations made sludge management a particularly vital part of this regional wastewater treatment project. Furthermore, CEPT produces more sludge than conventional primary treatment, increasing the need for effective sludge management. In order for the CEPT plant to be financially feasible, the sludge must be treated and disposed of in a manner that is cost effective and consistent with the region's environmental goals. This report proposes an effective and feasible sludge treatment system and provides a beneficial use strategy for the city of Alfenas.

Because the Furnas Reservoir is a vital resource to the surrounding communities, it is important to examine the impact of CEPT implementation on this water body. A modeling exercise was performed to predict the water quality impacts of installing CEPT. Predictions were made based on the changing volumes in the reservoir and comparing the existing nutrient and bacterial loads in the reservoir to the reduction of these loads anticipated from implementation of CEPT. Samples were collected from the reservoir and from the waste streams discharged from Alfenas to the reservoir. These samples were analyzed to determine bacterial and nutrient concentrations. Based on laboratory studies performed in Brazil, CEPT implementation with chemical disinfection would result in about a 90% reduction in phosphorus, 60% of BOD, 10^4 of fecal coliforms. A mathematical model based on a well-mixed system is used to predict overall reductions in the ambient concentrations in the reservoir.

1.1 Current Status of the Furnas Reservoir Region

In 1963, the first FURNAS hydroelectric power plant began operation. The construction of this power plant created the Furnas Reservoir, with a surface area of 1,440 km^2. The reservoir has become an important resource for recreation and tourism and also serves as a drinking water source and disposal location for the region's wastewater. Figure 1-1 shows the location of the Furnas Reservoir region.

Figure 1-1: Map of Brazil, FURNAS region highlighted

(Geocities)

At present, this FURNAS power plant provides generates 163 kWh of power per month for 23,000 households. The lake provides 99% of the fresh water supply for the region, and collects 98% of the sewage produced (FURNAS website, www.furnas.com.br).

A combination of severe drought conditions and increased power demand have decreased the reservoir to 11% of its originally volume. The disposal of untreated sewage to the reservoir poses human and environmental health risks. The lower water volume increases the concentrations of contaminants and wastewater treatment is vital to improving reservoir water quality.

1.2 Proposed Objectives for the Region's Wastewater Management

The regional wastewater management solution must be cost-effective and technically viable. CEPT is proposed as a first step towards wastewater treatment in the region. This technology will achieve treatment levels comparable to secondary treatment in terms of total suspended solids and phosphorus removal, but with a lower capital cost. Unlike effluent from conventional primary treatment, CEPT effluent can also effectively be disinfected. Regional implementation of this technology would be a significant step towards preserving the reservoir as an important water resource.

1.3 The City of Alfenas

The city of Alfenas, located in the southeastern area of the lake, was selected for the design and construction of a CEPT plant that could serve as a model for other cities in the Furnas region. Alfenas is a rapidly growing city with a population of 66,000 inhabitants, located in the state of Minas Gerais, about 500 km inland from Rio de Janeiro (see Figure 1-2).

Figure 1-2: Map of Alfenas relative to São Paulo and Rio de Janeiro (Brazilian Institute of Geography and Statistics)

Wastewater in the city is collected in open channel streams and flows into the Furnas Reservoir. The proposed CEPT plant will treat wastewater collected in the Jardim da Boa Esperança, which collects wastewater from 30% of city's population, approximately 20,000 inhabitants. CEPT plants are also used for municipal wastewater treatment in Rio de Janeiro and São Paulo, two of the largest and most economically prominent cities in Brazil.

2. CEPT Plant Design

Upon invitation from José Wurtemberg Manso, mayor of the city of Alfenas, a field study was conducted between January 4 and January 26, 2002. This field study was comprised of bench-scale testing of CEPT and lab analysis of raw wastewater, treated water, sludge and lake water. The objectives of this testing were:

- Determine the optimal combination of chemicals for treatment
- Confirm efficiency of typical overflow rates for CEPT
- Gather chemical analysis data to back up these two findings

For this purpose, the city provided access to the laboratory facilities of the Hydric Resources Environmental Research Laboratory, lead by Prof. Eduardo Tanure, at UNIFENAS (Alfenas University).

The sampling method and location, the laboratory procedures for chemical analysis and the procedures for bench-scale testing (jar testing) are described in Appedix B.

The data presented next highlights the bench-scale jar testing results that are most relevant to the selection of chemical dosing and the confirmation of the appropriate surface overflow rate (SOR) for treatment.

The target SOR was set at 60 m/day, about twice the design value for conventional primary treatment. Additional samples were taken at 30 m/day, to get an idea of the potential of each chemical.

2.1 Raw wastewater characteristics

From the 34 samples of raw wastewater taken from the Jardim de Boa Esperança stream, the average value for the key parameters described in the previous section were:

Table 2-1: Raw wastewater characteristics summary

Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)	Fecal Coliforms (MPN/100 mL)
191	**215**	**494**	**6.9**	**7.6**	**$8\cdot10^6$**

2.2 Discussion of relevant jar testing results

2.2.1 Selection of chemical dosing

First, jar testing explored the use of a single chemical as coagulant. The chemicals tested were: alum, $FeCl_3$, Tanfloc and a neutral synthetic polymer. For the first three coagulants, performance can be assessed in the following graph that compares their COD removal efficiency:

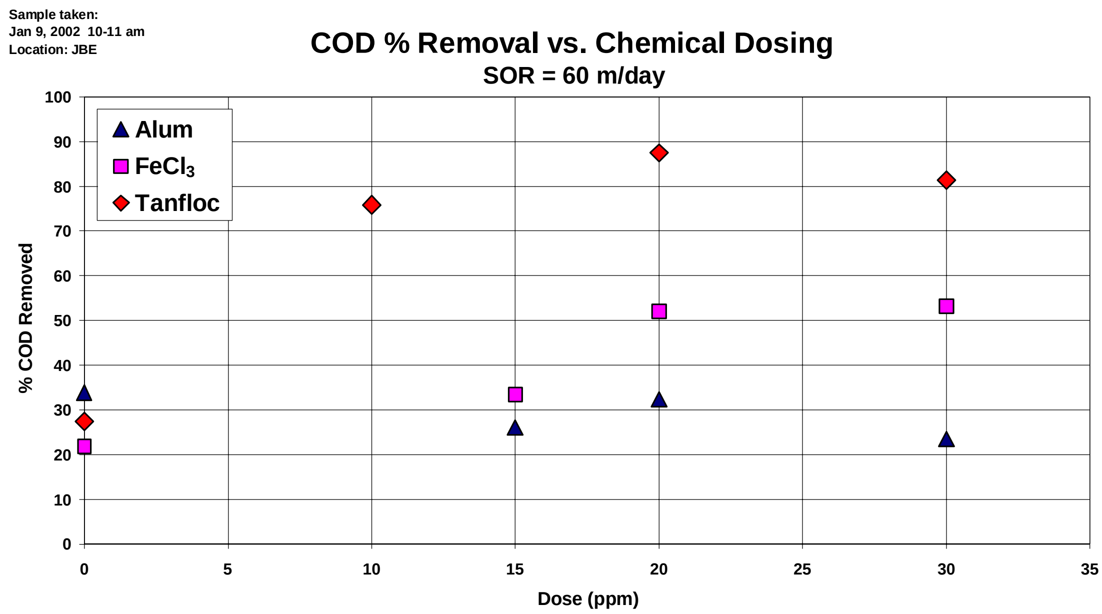

Figure 2-1: Coagulant selection graph

From Figure 2-1, Tanfloc results as the best option for coagulant and alum clearly shows poor performance.

Next, several combinations were tested, using alum, $FeCl_3$ and Tanfloc as coagulants and comparing their performance with several synthetic polymers (anionic, cationic and neutral) and Tanfloc as flocculants. Performance of Tanfloc was comparable to that of synthetic polymers, but for cost reasons, these were dismissed. Average costs of synthetic polymers are around 5 USD per kg, while the cost of Tanfloc is only 0.93 USD per kg. To obtain comparable results, a dosage of 5 ppm for synthetic polymers is required, while only 10 ppm of Tanfloc were needed, thus cost efficiencies remained favorable for the latter.

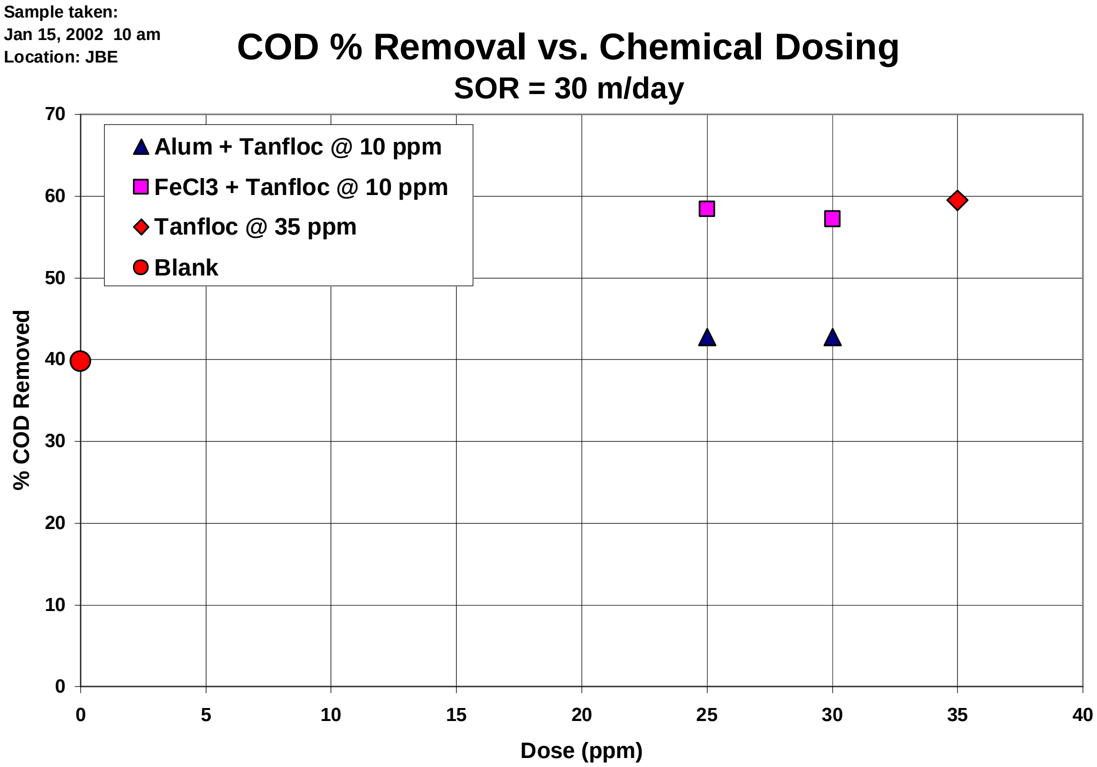

Figure 2-2: Flocculant selection graph

Figure 2-2 shows the high efficiency of Tanfloc, both as a flocculant when using $FeCl_3$ as the coagulant, and as a coagulant on its own. Performance of alum remained poor. From these results, it was concluded that the two best options for treatment are:

- $FeCl_3$ as coagulant (30 ppm) and Tanfloc as flocculant (10 ppm)
- Tanfloc as coagulant (30 ppm)

2.2.2 Results for option 1: $FeCl_3$ 30 ppm and Tanfloc 10 ppm

A summary of jar testing of this chemical combination shows typical results expected for CEPT, with turbidity removal of 60%, TSS removal of 70%, COD removal of 64% and phosphorus removal over 90%.

Table 2-2: Summary of jar testing results for $FeCl_3$ + Tanfloc

30 m/day	Turbidity % Removal	TSS % Removal	COD % Removal
Average	67	77	64
Max	86	89	74
Min	46	70	57
Number of samples: 6			

60 m/day	Turbidity % Removal	TSS % Removal	COD % Removal
Average	55	65	64
Max	57	66	71
Min	53	65	56
Number of samples: 2			

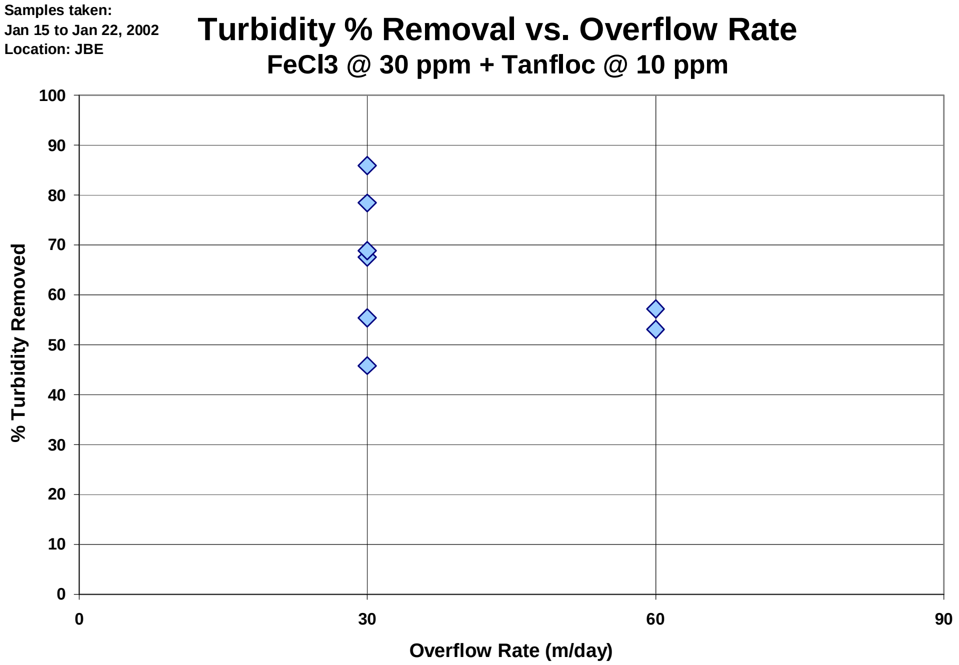

Figure 2-3: Turbidity removal efficiencies for $FeCl_3$ + Tanfloc

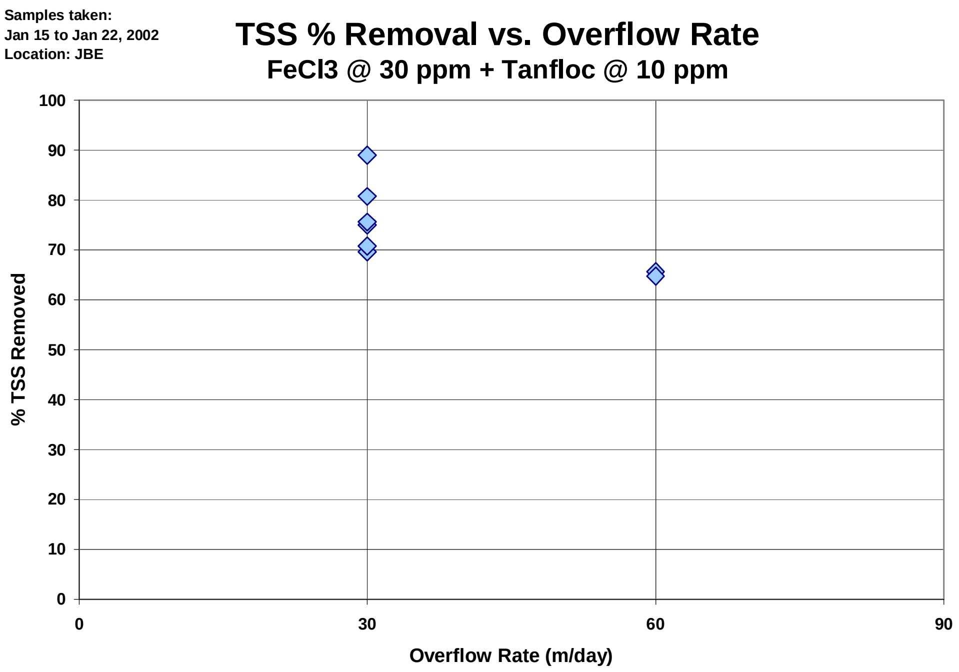

Figure 2-4: TSS removal efficiencies for $FeCl_3$ + Tanfloc

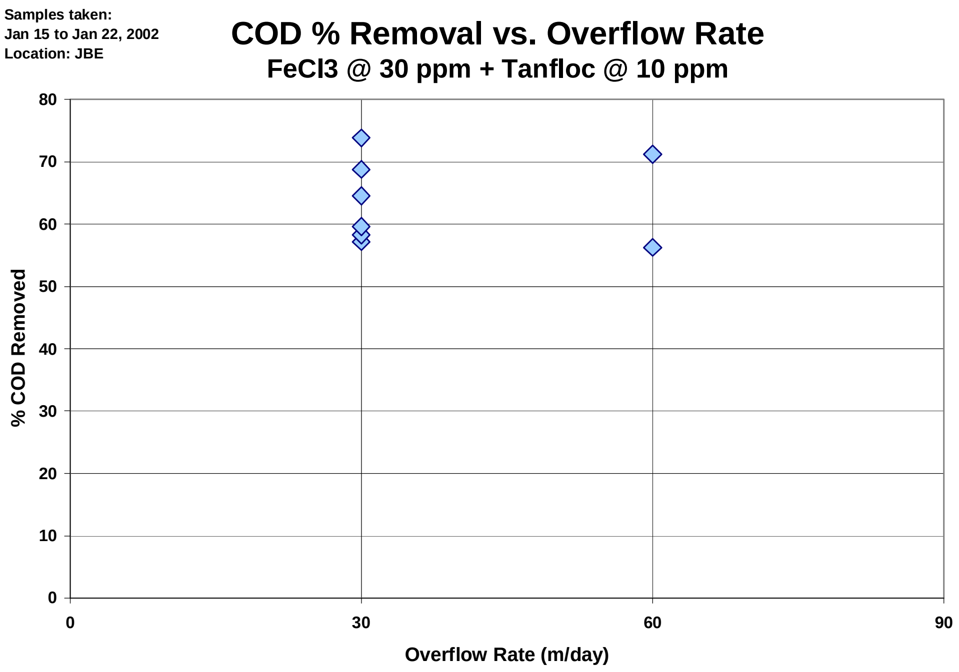

Figure 2-5: COD removal efficiencies for $FeCl_3$ + Tanfloc

2.2.3 *Results for option 2: Tanfloc 30 ppm*

Results of jar testing for this option show removal efficiencies comparable to those of the previous option, with turbidity removal of 75%, TSS removal of 80%, COD removal of 55% and phosphorus removal around 65%.

Table 2-3: Summary of jar testing results for Tanfloc

30 m/day	Turbidity % Removal	TSS % Removal	COD % Removal
Average	80	93	46
Max	94	98	51
Min	59	85	40
Number of samples: 4			

60 m/day	Turbidity % Removal	TSS % Removal	COD % Removal
Average	70	68	67
Max	83	85	81
Min	58	50	54
Number of samples: 2			

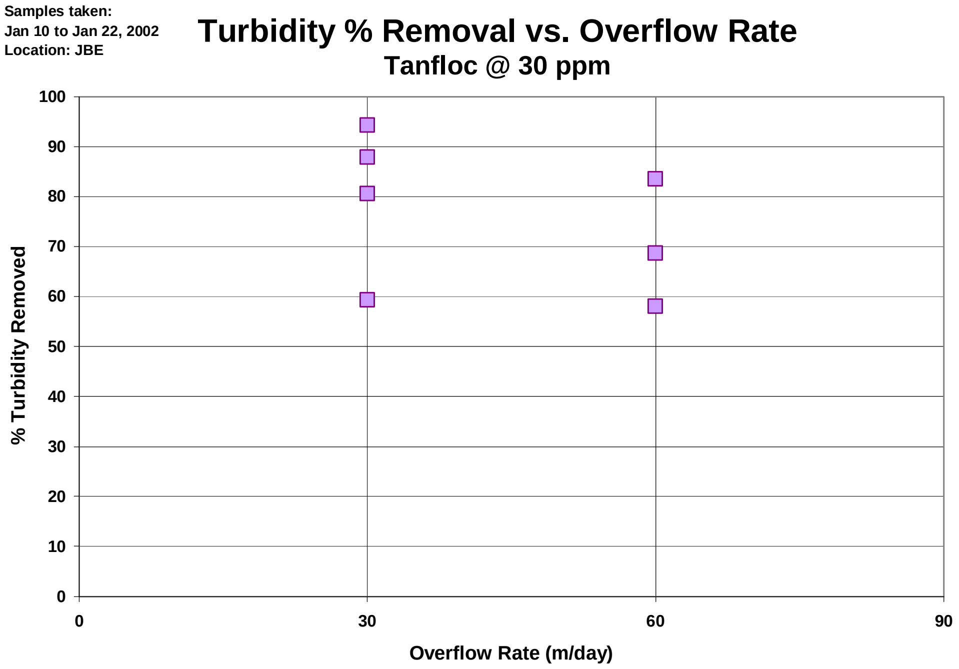

Figure 2-6: Turbidity removal efficiencies for Tanfloc

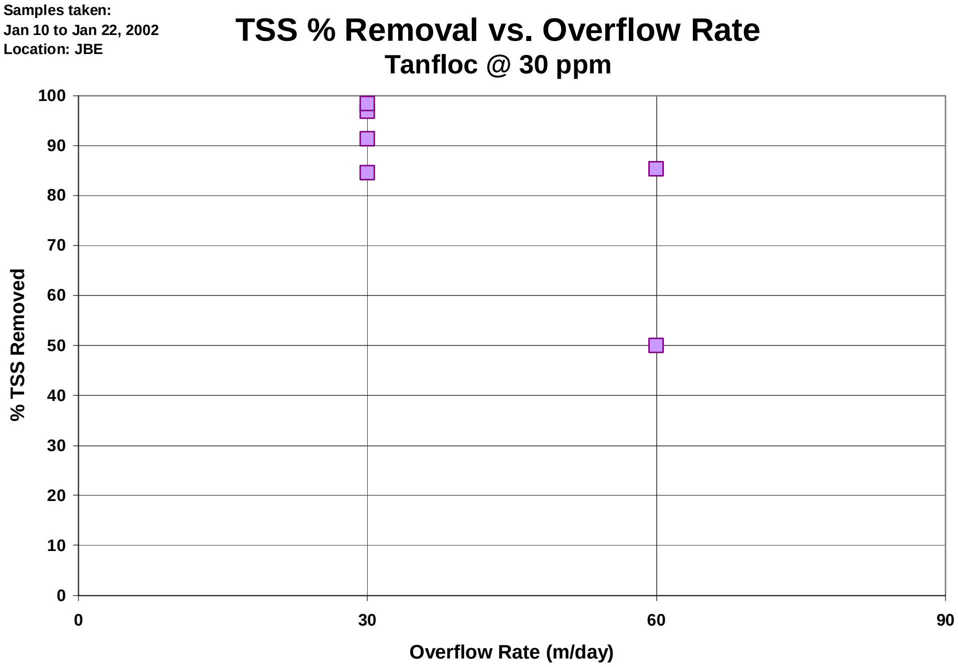
Samples taken:
Jan 10 to Jan 22, 2002
Location: JBE

Figure 2-7: TSS removal efficiencies for Tanfloc

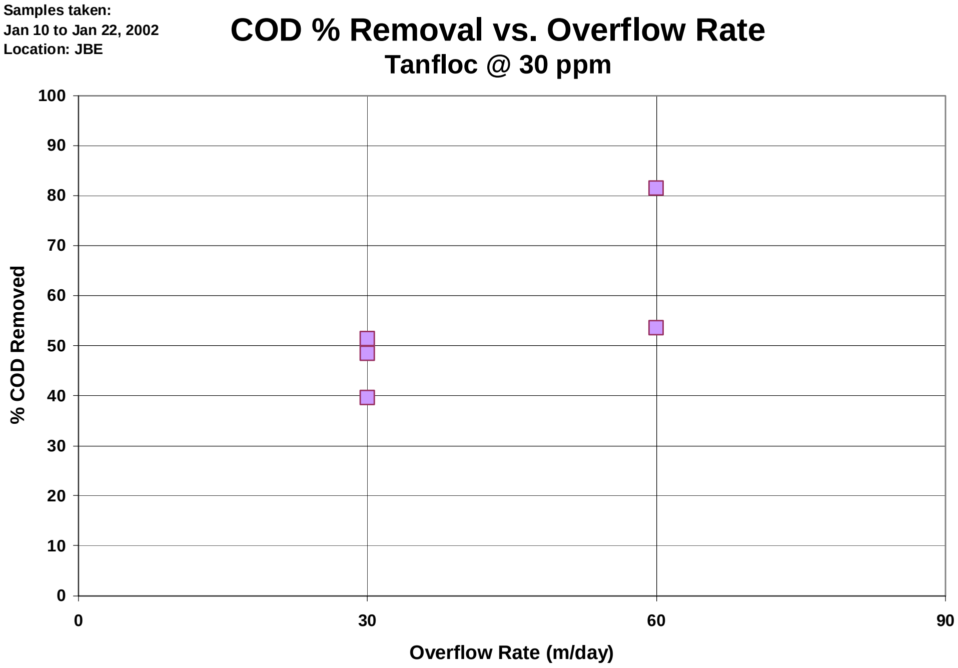
Samples taken:
Jan 10 to Jan 22, 2002
Location: JBE

Figure 2-8: COD removal efficiencies for Tanfloc

2.2.4 Analysis of results

These results validate the selection of 60 m/day as the target overflow rate for the design of the proposed plant. It should be noted that while the data presented offers a good sense of what the expected efficiency of the plant will be, the limited amount of data points obtained indicate that the proposed chemical dosing will require adjustments, which will be a part of the plant's startup procedures.

In general, it is expected that removal rates at higher overflow rates be less than at lower overflow rates, since particles will have more time to settle out when the overflow rate is lower. This proves true in most cases for the data presented, but the COD results for Tanfloc alone show that removal rates at 60 m/day exceeded those at 30 m/day. No strong conclusions can be drawn in this case, since the amount of information is limited to two data points for 60 m/day and three data points for 30 m/day. However, except for the single point indicating 81% removal of COD for Tanfloc at 30 ppm, all others remain around 48% (±8%), indicating that expected removals for COD using Tanfloc alone should be around 50%. Comparing these results with those for $FeCl_3$ + Tanfloc, for which COD removal rates were around 64%, it can be concluded that the combination of chemicals offers better removal efficiency in terms of COD, and thus of BOD.

Total suspended solids removal for $FeCl_3$ + Tanfloc at 60 m/day was 65%, while removal with Tanfloc alone was 68% at the same overflow rate. This means the TSS removal efficiencies for both options are comparable under expected operational conditions. In addition, Tanfloc alone demonstrated higher efficiency in TSS removal at 30 m/day, 93% compared with 77% of the $FeCl_3$ + Tanfloc. Treating wastewaters with $FeCl_3$ produces inorganic precipitates, e.g. ferric hydroxides and ferric phosphates, and thus increases the amount of solids formed in the process, leading to lower TSS removal efficiencies. Tanfloc, being a natural polymer, is not expected to generate as many precipitates. In conclusion, TSS removal efficiencies for the two options are comparable, with a slight advantage towards the Tanfloc alone option.

In the case of turbidity, one important factor to be considered when analyzing results is that $FeCl_3$ not only produces a wider variety of solid precipitates, some of which are not soluble, but also generates a yellow coloring in the water. These two factors contribute to less efficiency in turbidity removal for $FeCl_3$ + Tanfloc, around 55% at 60 m/day, compared to Tanfloc alone, around 70% at the same overflow rate. Visually, effluent treated with Tanfloc was much clearer after 10 minutes of settling than effluent treated with $FeCl_3$. Thus, it can be concluded that turbidity removal efficiencies for Tanfloc alone are higher than for $FeCl_3$+Tanfloc.

Finally, removal of phosphorus, a key parameter for environmental concerns such as eutrophication, was around 90% for $FeCl_3$+Tanfloc and only around 65% for Tanfloc alone. As explained above, $FeCl_3$ produces ferric phosphates as precipitates, which enhances the removal efficiency for phosphorus, while Tanfloc does not possess this quality. Results indicate that $FeCl_3$+Tanfloc is the best option in terms of phosphorus removal.

In conclusion, of the four parameters selected for comparison, the $FeCl_3$+Tanfloc option was shown to perform better in terms of phosphorus and COD removal, while Tanfloc alone was more efficient for turbidity. TSS removal was comparable for both options.

2.2.5 Selection of best option for treatment

Aside from removal efficiencies, a major comparison point between the two options for chemical dosing is that of cost. While using two chemicals entails a higher capital cost, due to the added infrastructure, operational costs for Tanfloc alone are much higher, because it is about three times as expensive as $FeCl_3$. The following table summarizes data for approximate value of plant equipment in USD. Most information was obtained by verbal communication with several manufacturers and design engineers. This data is presented to support the cash flow calculations and to give an idea of the overall costs of a CEPT plant. Labor and other construction costs are neglected.

Table 2-4: Estimate of plant capital costs

Equipment	**Approximate Price (USD)**
Bar screens with manual cleaning	7,200
Grit removal chamber, vortex type	1,600
Parshall flume, prefabricated acrylic	1,500
Magnetic flow meter	700
Programmable logic controller	100
PVC storage tank for $FeCl_3$	400
PVC storage tank for Tanfloc	200
PVC storage tank for NaClO	200
Diaphragm dosing pumps (three)	3x 300
CEPT settling tanks	15,000
Scum/sludge scrapers	25,000
Disinfection chamber	5,000
Piping and accessories	2,200
TOTAL	**60,000**

Per kilogram, $FeCl_3$ costs 0.3 USD while Tanfloc costs 0.93 USD (converted from Brazilian currency at official exchange rates of the Brazilian National Bank during January, 2002). According to the dosing for the $FeCl_3$+Tanfloc option, the daily mass flow of each chemical would be:

$$\text{Daily mass flow of } FeCl_3 = 30\,\tfrac{mg}{L}\cdot 3600\,\tfrac{m^3}{day}\cdot 1000\,\tfrac{L}{m^3}\cdot\tfrac{kg}{10^6\,mg} = 108\,\tfrac{kg}{day}$$
$$\text{Daily mass flow of Tanfloc} = 10\,\tfrac{mg}{L}\cdot 3600\,\tfrac{m^3}{day}\cdot 1000\,\tfrac{L}{m^3}\cdot\tfrac{kg}{10^6\,mg} = 36\,\tfrac{kg}{day}$$

Equation 2-1: Calculation of daily mass flow for the $FeCl_3$+Tanfloc combination

Multiplying by the cost per kg:

$$\text{Daily cost of FeCl}_3 = 108\,\tfrac{\text{kg}}{\text{day}}\cdot 0.3\,\tfrac{\text{USD}}{\text{kg}} = 32\,\tfrac{\text{USD}}{\text{day}}$$
$$\text{Daily cost of Tanfloc} = 36\,\tfrac{\text{kg}}{\text{day}}\cdot 0.93\,\tfrac{\text{USD}}{\text{kg}} = 33\,\tfrac{\text{USD}}{\text{day}}$$

Equation 2-2: Calculation of daily operational costs for the $FeCl_3$+Tanfloc combination

The total daily cost of the $FeCl_3$+Tanfloc option is thus 65 USD. Following the same reasoning, the cost of using Tanfloc alone is:

$$\text{Daily mass flow of Tanfloc} = 30\,\tfrac{\text{mg}}{\text{L}}\cdot 3600\,\tfrac{\text{m}^3}{\text{day}}\cdot 1000\,\tfrac{\text{L}}{\text{m}^3}\cdot\tfrac{\text{kg}}{10^6\,\text{mg}} = 108\,\tfrac{\text{kg}}{\text{day}}$$
$$\text{Daily cost of Tanfloc} = 108\,\tfrac{\text{kg}}{\text{day}}\cdot 0.93\,\tfrac{\text{USD}}{\text{kg}} = 116\,\tfrac{\text{USD}}{\text{day}}$$

Equation 2-3: Calculation of daily mass flow and cost for Tanfloc

As estimated above, the proposed CEPT plant will cost 60,000 USD, of which approximately 1200 USD can be allocated for chemical dosing tanks, piping and pumps. For the Tanfloc option, this value decreases to approximately 700 USD, which does not represent a significant difference. It can be concluded that capital costs of equipment are comparable for both options, since the fixed costs of all the rest of the equipment are much greater. Furthermore, operation and maintenance costs such as labor and parts were not factored into the cash flow estimate, as they will also be comparable for both options.

Using a discount rate of 10%, typical value for this type of project, and a project life of 10 years, the net present value of the cost of the $FeCl_3$+Tanfloc option is approximately 215,000 USD, while that of Tanfloc alone is approximately 320,000 USD. This points to $FeCl_3$+Tanfloc as the best option for treatment, in terms of cost.

With respect to ease of operation, using only one chemical is more efficient as it requires less maintenance. However, the cost efficiency of using two chemicals, $FeCl_3$ as coagulant and Tanfloc as flocculant, is much higher and relevant in this case, thus will constitute the best option for treatment in this case. One major objective of this proposed plant is to be cost-effective for a developing country, thus further supporting the decision to use $FeCl_3$+Tanfloc. Furthermore, this option offers the highest versatility, since having two chemicals with which to adjust the treatment makes it easier to regulate its effectiveness and control operational costs.

2.2.6 Analysis of relevant regulations

According to Brazilian regulation nº 010/86, issued by the Environmental Policy Commission on September 8, 1980, treated wastewater that is to be discharged into natural bodies of water should meet, among others, the following specifications:

Table 2-5: Summary of relevant regulation requirements for treated wastewater discharge

Parameter	**Value**
PH	6.5 to 8.5 (±0.5)
COD	90 mg/L max.
BOD_5	60 mg/L max. (or 85% removal)
TSS	100 mg/L max.
Phosphorus	0.1 mg/L max.
Fecal coliforms	1000 per 100 mL max.

The level of pH required will be achieved through CEPT, as will the TSS requirement. Disinfection with NaClO will effectively kill most pathogens in the effluent, complying with this portion of the regulatory requirements. However, phosphorus levels after CEPT will remain above regulation standards, as will COD levels. Using the average raw wastewater characteristics presented in section 2.1, the corresponding removal rates discussed above and the correlation between COD and BOD established in Appendix B, the expected levels of BOD for each treatment option are:

Table 2-6: Expected BOD for treated water

Treatment Option	**Wastewater COD**	**Wastewater BOD**	**Removal of COD & BOD**	**Expected Treated Water COD**	**Expected Treated Water BOD**
$FeCl_3$ + Tanfloc	494	296	54 %	227	136
Tanfloc	494	296	64 %	178	107

An increase in dosing can achieve removal rates that will allow the effluent to reach regulation standards, but since removal rates of BOD for CEPT usually do not exceed 70%, unless the incoming wastewater's BOD remains below 200 mg/L, this will not ensure that the effluent will meet the standard. To meet the regulation in full, later use of secondary treatment will be necessary. Having applied CEPT, this treatment will be less costly than having implemented conventional primary treatment. Stabilization ponds or lagoons are strongly recommended for their ease of operation.

2.3 Process description

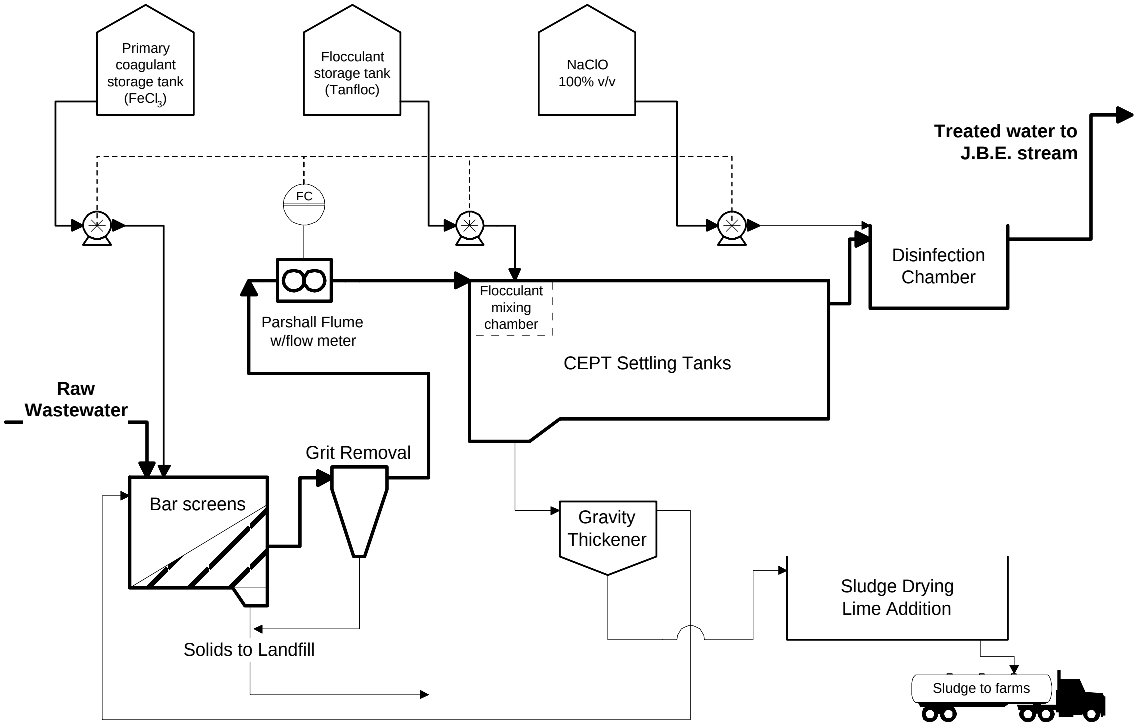

Figure 2-9: Process Flow Diagram (with Instrumentation)

Raw wastewater, collected through the sewer system, arrives at the plant and flows first through bar screens, where coarse solids, such as rags, twigs and rocks, are separated from the stream. At this point, the coagulant is injected, the dosing for which will be determined by a feedback control system tied into the flow meter located in the Parshall flume (downstream). Injection of coagulant at this point will ensure proper and full mixing.

Next, the water flows through the grit removal chamber, where finer solids, such as sand, are separated. The stream then flows through a Parshall flume, where volumetric flow is constantly measured and used to control the dosing of coagulant.

The flocculant is injected at this point, just before the water enters the CEPT settling tanks. Water then flows through the CEPT tank to let solids settle out of it. Finally, water passes through the disinfection contact chamber, where NaClO in liquid solution is mixed with the water, the dosing of which is also controlled by the flow meter in the Parshall flume. As an option, the dosing of the disinfectant could be controlled by an online chlorine analyzer. Finally, the treated water is discharged into the Jardim de Boa Esperança stream.

Sludge is taken from the bottom of the CEPT tank into a gravity thickener, and the thickened sludge flows into the sludge drying beds, where lime is added for disinfection and the sludge is left to dry (Stout, 2002).

2.4 *Dimensioning of CEPT settling tank*

This CEPT plant will serve a population of 20,000 inhabitants that discharge their wastewater into the Jardim de Boa Esperança stream. Based on the typical flow rates of wastewater for Latin American countries (Metcalf & Eddy, 1991), it will be assumed that each inhabitant will produce 180 liters of wastewater per day. Therefore, the incoming flow of wastewater will be:

$$\text{Incoming Flowrate} = \frac{20{,}000\ \text{inhab.} \times 180\ \text{liters / inhab.}}{1000\ \text{liters / m}^3} = 3600\ \text{m}^3\text{/day}$$

Equation 2-4: Calculation of incoming wastewater flow rate

Operating overflow rate will be set at 60 m/day a typical value for CEPT (Morrisey and Harleman, 1992), which also provided adequate COD, TSS and turbidity removal rates during jar testing. Thus, the required footprint (area) for the CEPT tank will be:

$$\text{Footprint (Area)} = \frac{3600\ \text{m}^3\text{/day}}{60\ \text{m/day}} = 60\ \text{m}^2$$

Equation 2-5: Calculation of footprint for CEPT tank

Tank depth will be set at 3 m, which is a typical value for CEPT tanks, and it takes into account the difficulty of building deeper tanks. Thus, the tank volume will be:

$$\text{Volume} = 60\ \text{m}^2 \times 3\ \text{m} = 180\ \text{m}^3$$

Equation 2-6: Calculation of CEPT tank volume

Tank dimensions for CEPT are typically such that the tank has a rectangular shape, to allow space for longitudinal mixing and proper settling. For this reason, a width of 3 m is set. Thus, the total required length of the CEPT tank would be:

$$\text{Length} = \frac{180\ \text{m}^3}{3\ \text{m} \times 3\ \text{m}} = 20\ \text{m}$$

Equation 2-7: Calculation of CEPT tank length

For construction, this length will be separated into two 10 m long tanks, with approximately three additional meters for inlet and outlet space in each tank. The first tank will also have a baffle 4 meters after the inlet to allow for flocculant mixing. The residence time in the CEPT tanks will be:

$$\text{Residence Time} = \frac{180\ \text{m}^3}{3600\ \text{m}^3/\text{day}} \times 24\ \tfrac{\text{hours}}{\text{day}} = 1.2\ \text{hours}$$

Equation 2-8: Calculation of CEPT tank residence time

This residence time fits within the suggested standard for CEPT settling tanks (Metcalf & Eddy, 1991), thus confirming the choice of assumed parameters. Although typical values are closer to one hour, the 20% of excess residence time will be used to buffer peak flows.

2.5 Dimensioning of disinfection chamber

To achieve the desired disinfection, which will yield an effluent with 1000 or less fecal coliforms per 100 mL, as required by Brazilian regulations (see page 31), contact time with NaClO will be 30 minutes and under peak conditions, contact time can lower to 20 minutes while maintaining disinfection requirements (ASCE, 1998, page 14-106). A plug flow is preferred for disinfection, in order to enable extensive and intimate contact between the disinfectant and the water. For a volumetric flow of 3600 m^3/day, the required volume for the disinfection chamber is:

$$\text{Vol} = 3600\,\frac{\text{m}^3}{\text{day}}\cdot 30\,\text{min}\cdot\frac{\text{day}}{1440\,\text{min}} = 75\text{m}^3$$

Equation 2-9: Calculation of disinfection chamber volume

Maintaining the geometry of 3 m deep and 3 m wide used for the CEPT tank, the disinfection chamber requires a total length of:

$$\text{Length} = \frac{75\,\text{m}^3}{3\,\text{m}\times 3\,\text{m}} = 8.3\,\text{m}$$

Equation 2-10: Calculation of disinfection chamber length

2.6 Plant location and layout

Figure 2-10 shows where the CEPT plant is projected to be built. For the layout, a simple process-oriented distribution is shown in Figure 2-11.

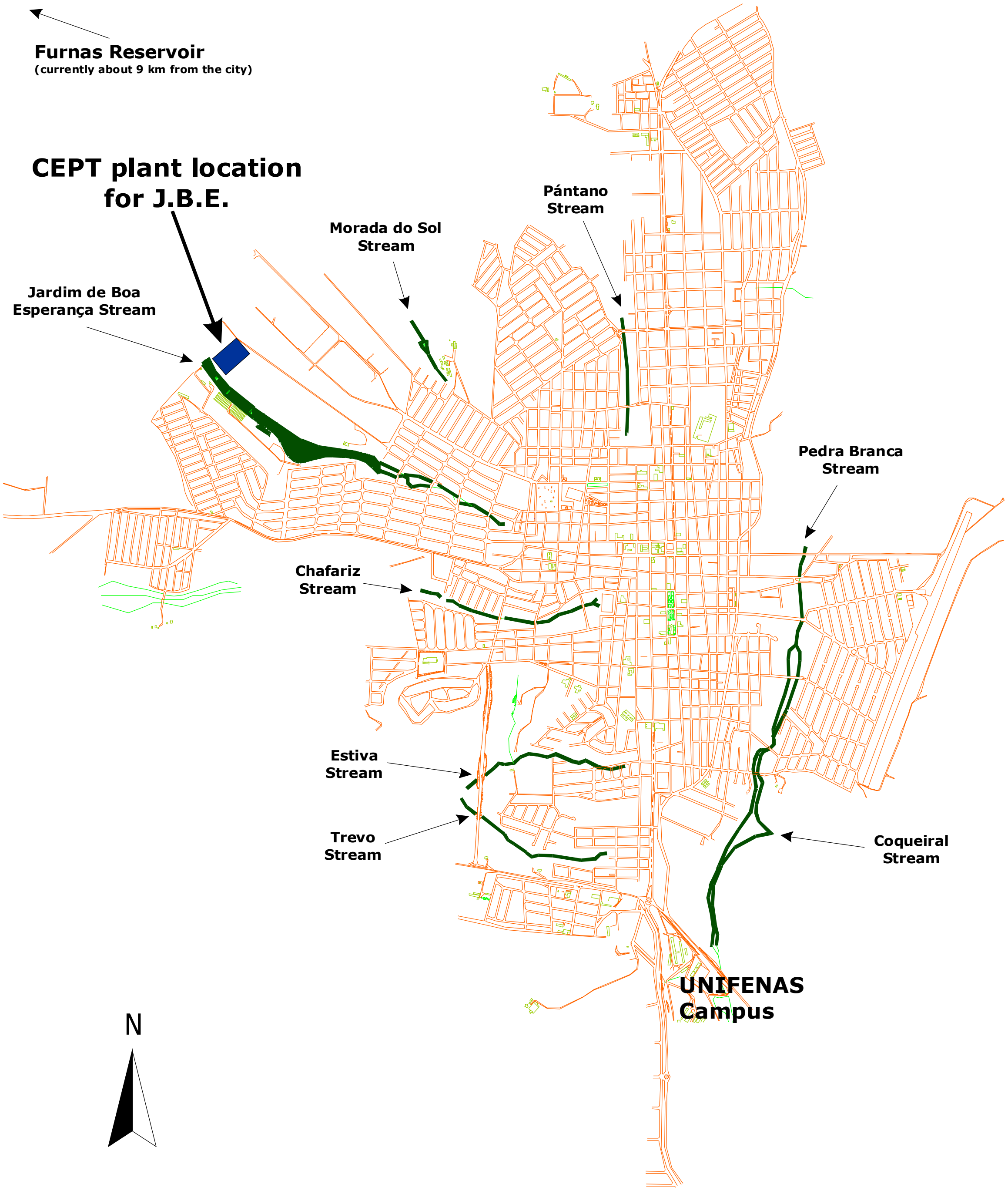

Figure 2-10: CEPT plant location for the Jardim de Boa Esperança stream (Source: Alfenas City Hall, Office of Cartography)

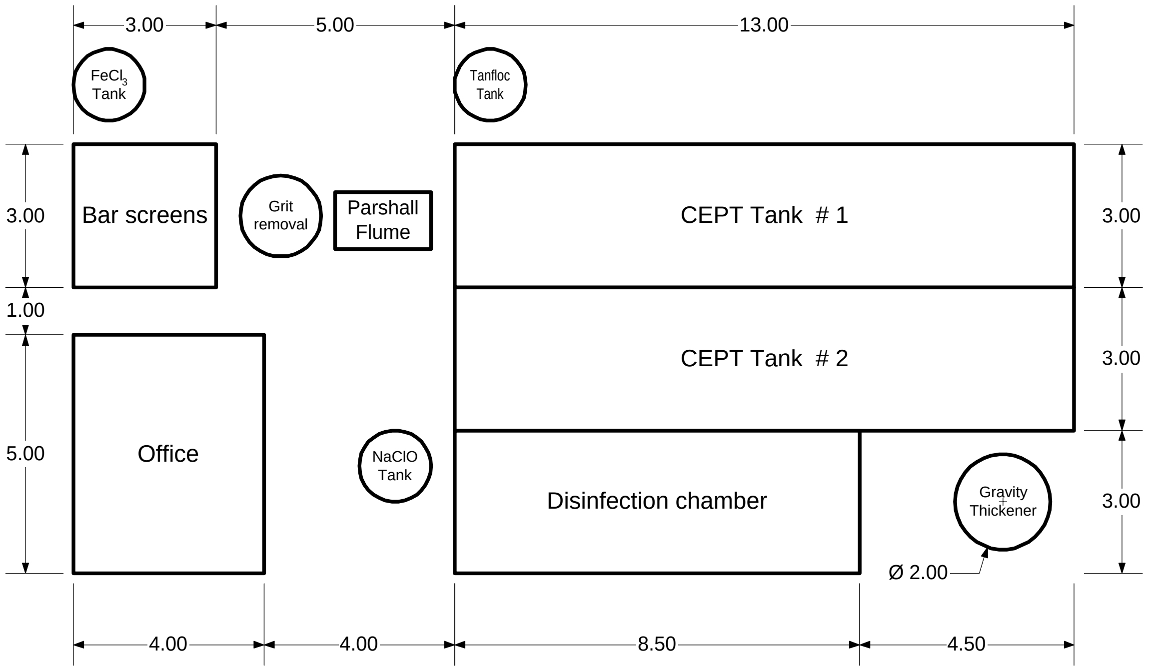

Figure 2-11: Proposed plant layout, distances in meters

2.7 Equipment specifications

2.7.1 Bar screens and grit chamber

To remove coarse solids that usually flow together with wastewaters, two unit operations of pre-treatment will be used: bar screens and grit removal.

Bar screens will be 3 meters wide and comprised of sixty 10 mm wide by 30 mm deep stainless steel bars, with a spacing of 40 mm between them and a slope of 45º. The method for cleaning will be manual.

For grit removal, a vortex-type grit chamber will be used (Metcalf & Eddy, 1991). The detention time in the grit chamber will be 30 seconds. Diameter will be set at 1.2 m, and height will be 1.5 m for the cylindrical portion of the chamber; the conical bottom will have a total height of 35 cm.

2.7.2 Parshall flume with flow meter

A vinyl pre-fabricated Parshall flume will be used to measure the incoming flow of raw wastewater. A magnetic flow meter will be included to provide volumetric flow data for the control system. A four-way programmable logic controller (PLC) will gather the signal from the Parshall flume and emit signals to control the flow of the three dosing pumps.

2.7.3 Chemical storage tanks and dosing system

Roofed PVC tanks will be used to store a stock of 8 days of both CEPT chemicals and disinfectant. Diaphragm pumps will be used to dose these into the proper section of the process.

For $FeCl_3$, the required volume to store 8 days will be:

$$\text{Daily } FeCl_3 \text{ consumed} = 30\,\tfrac{mg}{L} \times 3600\,\tfrac{m^3}{day} \times 1000\,\tfrac{L}{m^3} \times 10^{-6}\,\tfrac{kg}{mg} = 108\,\tfrac{kg}{day}$$

$$\rho_{FeCl_3 \text{ aqueous solution}} \approx 1\tfrac{g}{mL} \Rightarrow 108\,\tfrac{kg}{day} \times 8 \text{ days} = 864 \text{ kg} \approx 864 \text{ L}$$

Equation 2-11: Calculation of $FeCl_3$ storage tank

To ensure proper storage capacity, the $FeCl_3$ tank will be specified at 1000 L, to allow for unexpected problems with supply.

For Tanfloc and NaClO, the required volume to store 8 days will be:

$$\text{Daily Tanfloc or NaClO consumed} = 10\,\tfrac{\text{mg}}{\text{L}} \times 3600\,\tfrac{\text{m}^3}{\text{day}} \times 1000\,\tfrac{\text{L}}{\text{m}^3} \times 10^{-6}\,\tfrac{\text{kg}}{\text{mg}} = 36\,\tfrac{\text{kg}}{\text{day}}$$

$$\rho_{\text{Tanfloc or NaClO aqueous solution}} \approx 1\tfrac{\text{g}}{\text{mL}} \Rightarrow 36\,\tfrac{\text{kg}}{\text{day}} \times 8\ \text{days} = 288\ \text{kg} \approx 288\ \text{L}$$

Equation 2-12: Calculation of Tanfloc and NaClO storage tanks

To ensure proper storage capacity, the Tanfloc tank will be specified at 350 L, to allow for unexpected problems with supply. Following this same reasoning, the NaClO storage tank should also have this volume, as its consumption is the same.

2.7.4 CEPT settling tanks and scraper system

Concrete tanks will be used, with the typical sump at the head of the tank, which will allow sludge collection. The tanks will be connected by a 30 cm wide weir, which will allow water to flow from one to the next.

Continuous moving sludge and scum scrapers will be used, which will assist in gathering the sludge as it settles and in removing lipids and other scum from the surface of the water. An option in this case would be to construct this mechanism using locally available technology, but it could also be imported directly from a manufacturer, for instance Finnchain (http://www.finnchain.fi)

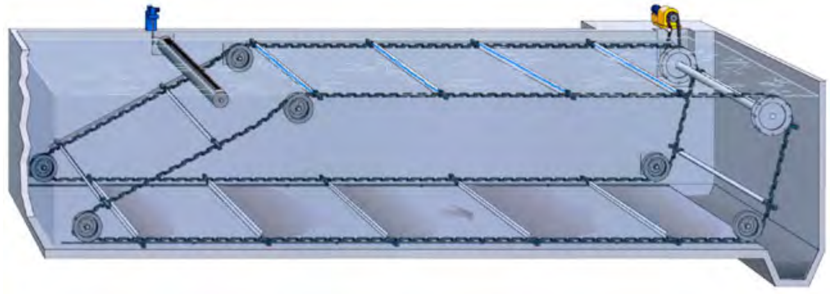

Figure 2-12: Illustration of a sludge and scum scraper
(Source: Finnchain, http://www.finnchain.fi)

2.7.5 Disinfection chamber

Concrete will also be used for the disinfection chamber. Two longitudinal baffles will be added, 1 meter apart, to promote plug flow.

3. Sludge Treatment Design

This section proposes a sludge management strategy that is financially and technically feasible for the city of Alfenas, while providing a level of treatment that allows the sludge to be beneficially used by the community. An important goal of this project was to design a sludge management system that was sustainable and could be maintained for many years without being dependent on the economic status of the region. The treatment technologies proposed were selected with an effort to minimize operational costs. Technically complex equipment was also avoided in order to limit opportunities for equipment failures that could require costly replacement parts and skilled mechanics. Because of the proximity of agricultural land to Alfenas and the importance of agricultural in the region land application of the sludge was selected as an appropriate and beneficial sludge disposal method. By applying the sludge on agricultural land the operation of the treatment plant and the sludge management strategy are not dependent on available landfill or storage space, or incinerator operation.

The sludge treatment techniques recommended in this report were selected based on technical simplicity, minimal operational costs, and compliance with the regulatory requirements for the application of sludge to agricultural land. The U.S. EPA has outlined the planning steps for a sludge land application program (U.S. EPA 1995). These steps are shown in Figure 3-1.

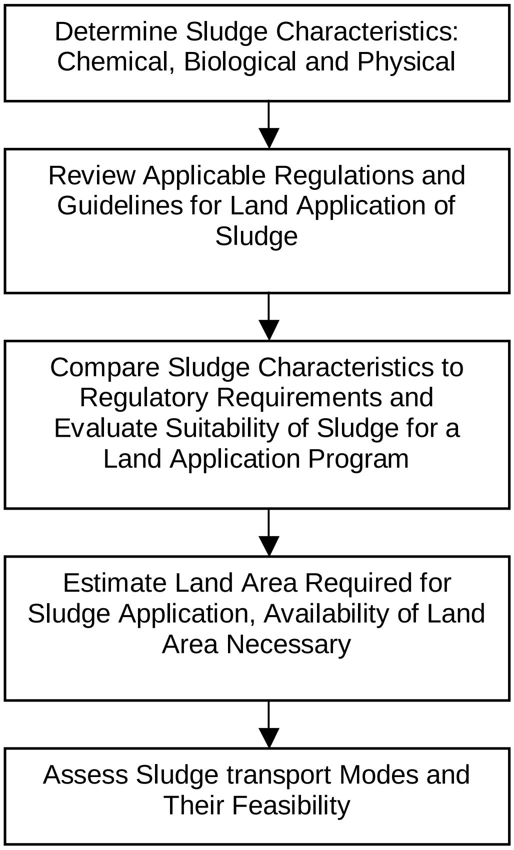

Figure 3-1: Planning Steps for a Sludge Land Application Program

(U.S. EPA 1995)

In developing the following recommendations these basic steps were followed and are addressed in this document. In order to study the local wastewater conditions and test the proposed wastewater and sludge treatment techniques a 3-week field study period was conducted in Alfenas in January 2002. During this period, the chemical additives for the wastewater treatment were selected based on availability and treatment efficiency. Sludge samples were also collected for chemical, physical and biological analysis and treatment techniques were evaluated for effectiveness and regulatory compliance. The results of these field tests are discussed in this section. Based on these results and the U.S. EPA standards for land applied sludges (also adopted by the Brazilian government) the proposed treatment strategy and land application plan were developed.

3.1 Experimental Results

3.1.1 Sludge Production

The results presented here were collected in January of 2002 at Unifenas University in Alfenas, Brazil. Raw wastewater samples were collected from the Jardim da Boa Esperança, a wastewater and storm water collection stream, in Alfenas. Sludge was produced from the wastewater samples after no more than eight hours of dark storage. In order to obtain reliable sludge data it was necessary to generate sludge that would be representative of the sludge produced by the proposed CEPT plant. In order to accomplish this goal the chemical addition and mixing regime utilized in the bench scale analyses for chemical selection and plant design (Olive 2002) were also implemented in these experiments. Sludge was produced, for the purpose of these analyses, in the following manner:

1. A volume of well-mixed, raw wastewater was transferred to a 20 liter cylindrical mixing tank.
2. The sample was stirred rapidly for 30 seconds.

3. The coagulant chemical was added at the appropriate dosage.
4. The sample was stirred rapidly for 30 seconds.
5. The flocculent chemical was added at the appropriate dosage.
6. The sample was stirred slowly for 5 minutes.
7. The sample was allowed to settle for approximately 20 minutes.
8. Supernatant was decanted by pouring of excess water, with efforts to leave settled sludge undisturbed.
9. Well-mixed sludge samples were collected for individual analyses.

All stirring was done by hand, using a glass stirring rod two feet in length. The samples were allowed to settle for 20 minutes in order to collect the maximum quantity of sludge for analysis while maintaining time efficiency during the short, 3 week, field study period.

Two types of sludge were produced in order to reflect to the two proposed wastewater treatment options (Olive 2002). Sludge A was produced from the addition of ferric chloride at 30mg/l, as a coagulant, followed by the addition of Tanfloc, a cationic polymer, at 10mg/l, as a flocculant. The sludge B was produced from the addition of Tanfloc at 40mg/l, as a coagulant. Table 3-1 lists the chemicals and concentrations added to the wastewater samples to create each of the sludge types. The characteristics of both sludges are discussed in this section.

Table 3-1: Chemical Additives and Dosages for each Sludge Type

Sludge Type	**Coagulant**	**Dosage** (mg/l)	**Flocculant**	**Dose** (mg/l)
Sludge A	Ferric Chloride ($FeCl_3$)	30	Tanfloc	10
Sludge B	Tanfloc	40	none	-

Although the characteristics of the raw wastewater varied slightly with sampling time or day, all of the sludge characteristics measured in this study remained fairly consistent throughout the three-week testing period. For most of the tests presented here more samples of Sludge A than Sludge B were analyzed because the addition of ferric chloride in combination with Tanfloc is the primary treatment recommendation (Olive 2002)

3.1.2 pH

The pH of sludge can be an important parameter, especially if the sludge is to be eventually utilized as a fertilizer on agricultural land. Because the applied biosolids can influence the pH of the soil, impacting soil chemistry and plant productivity, the pH of the biosolids should not exceed 6.5 (U.S. EPA 1983). The initial pH of the sludge can also influence the downstream treatment process. When utilizing the addition of lime for the purpose of disinfection, as proposed in this report, the pH must be raised above 12.

The pH of the sludge entering the lime addition process affects the dosage of lime required for pH elevation and, as a result impacts operating costs.

The pH of the sludge samples was consistent throughout the testing period and did not vary significantly with chemical additive. Table 3-2 gives the pH values of the sludge samples.

Table 3-2: pH of Sludge Samples

Sludge Type A	pH	Sludge Type B	pH
Sample 1	6.6	Sample 1	6.7
Sample 2	6.9	Sample 2	7.0
Sample 3	6.8	Sample 3	6.6
Sludge A Average	**6.8**	**Sludge B Average**	**6.8**
		Average of All Samples	**6.8**

Both Sludge A and Sludge B were found to have an average pH of 6.8. Untreated primary sludge typically has a pH between 5 and 8 (Metcalf & Eddy 1991). The pH values measured during this test are consistent with raw sludge pH data collected at the Point Loma CEPT plant in San Diego, California. The average pH of raw sludge at the Point Loma plant for the year 2000 was 6.27 (Point Loma Ocean Outfall Annual Monitoring Report 2000).

3.1.3 Total Solids

Total solids data was collected in order to evaluate the concentration of solid material in the sludge. The percent total solids can also be used in calculations of sludge volume and lime requirements. Total solids content was measured by drying the sample at 105 degrees Celsius for one hour according to Standard Methods for Water and Wastewater Examination, procedure 2540B

(Standard Methods 1991). The results of the total solids testing are given in Table 3-3.

The average percent total solids of Sludge A was 0.36 and the average percent total solids of Sludge B was 0.43. The difference in percent total solids of the two sludge types is 0.7%. This data suggests that there is no significant difference in the total solids content of the two sludges.

Untreated primary sludge ranges from 2% to 8% total solids, with a typical value of 5% (U.S. EPA 1979). The solids content often depends on the influent wastewater composition and can also be affected by the addition of chemicals and the dose. Raw sludge produced at the Point Loma plant averaged 4.5% total solids in 2000 (Point Loma Ocean Outfall Annual Monitoring Report 2000).

Table 3-3: Percent Total Solids

Sludge Type A	%TS	Sludge Type B	%TS
Sample 1	0.36	Sample 1	0.6
Sample 2	0.37	Sample 2	0.42
Sample 3	0.36	Sample 5	0.41
Sample 4	0.39	Sample 6	0.29
Sample 5	0.42	**Sludge B Average**	**0.43**
Sample 6	0.29		
Sample 7	0.29	**Average of All Samples**	**0.38**
Sample 8	0.45		
Sample 9	0.38		
Sample 10	0.29		
Sludge A Average	**0.36**		

There is an order of magnitude difference between the data reported at the Point Loma plant and the percent total solids of the sludge collected in this study. It should be noted that the percent total solids content is influenced by the method of supernatant removal. In wastewater treatment plants settled sludge is pumped from the bottom of the settling tank. For the purpose of this study the excess water was poured out of the top of the mixing tank. This decanting process, while time and resource efficient, did not allow for the effective removal of all the excess water without disturbing the settled sludge. Pumping methods utilized in treatment plants for sludge removal are superior to this decanting process as sludge integrity is better preserved and less effluent water is captured in the sludge flow. The method of sludge collection used in this study resulted in lower percent total solids values.

3.1.4 Volatile Suspended Solids (VSS)

It is essential to consider the organic fraction of sludge that is to be reused for agricultural proposes. The organic content of the sludge samples was evaluated by measuring the volatile suspended solids concentration. The VSS concentration was measured by baking the total solids samples at 550 degrees Celsius for one hour according to Standard Method procedure (Standard Methods 1991). The volatile suspended solids data is given in Table 3-4 as a percentage of the total solids.

Table 3-4: Volatile Suspended Solids as a Percentage of the Total Solids

Sludge Type A	**% VSS**	**Sludge Type B**	**% VSS**
Sample 1	67	Sample 1	73
Sample 2	65	Sample 2	70
Sample 3	69	Sample 3	57
Sample 4	58	Sample 4	76
Sample 5	72	**Sludge B Average**	**69**
Sludge A Average	**66**	**Average of All Samples**	**67**

The volatile suspended solids content of untreated primary sludge, as a percentage of total solids, ranges from 60% to 80% total solids, with a typical value of 65% (U.S. EPA 1979). Raw sludge produced at the Point Loma plant averaged 75.6% volatile solids in 2000 (Point Loma Ocean Outfall Annual Monitoring Report 2000). The results for volatile solids produced in this study are consistent with untreated primary sludge and slightly lower than sludge produced at the Point Loma plant.

3.1.5 Volume

In order to estimate the quantity of sludge that will be produced by the proposed CEPT plant the volume of sludge produced from each of the tests was measured by pouring the sample into a 1000ml beaker. The volume of raw wastewater and the volume of sludge produced from it are given in Table 3-5.

Table 3-5: Sludge Volumes as a Percentage of Wastewater Sample Volume

Sludge Type A	% Sludge Volume	Sludge Type B	% Sludge Volume
Sample 1	7	Sample 1	8
Sample 2	7	Sample 2	7
Sample 3	7	Sample 3	10
Sample 4	11	**Sludge B Average**	**8**
Sample 5	10		
Sludge A Average	**8**	**Average of All Samples**	**8**

The proposed CEPT plant will receive wastewater from approximately 20,000 inhabitants of Alfenas. The total volume of wastewater produced by this population, assuming that 180 liters is produced per person per day, is 3.6 million liters per day (Metcalf & Eddy 1991). Using the experimental data presented above, an average of 8% of the influent wastewater flow becoming sludge flow, the plant will produce 290,000 liters of sludge per day. It is important to note, however, that the volume of sludge calculated above would contain on average 0.38% totals solids, as reported in section 3.1.3. This volume estimate is compared with calculated estimates in section 3.4. Based on the data from the Point Loma CEPT plant, the sludge produced at the proposed plant is expected to have approximately 4% total solids (Point Loma Ocean Outfall Annual Monitoring Report 2000). As discussed in section 3.1.3, the sludge collection techniques employed in this study do not reflect true plant conditions and the 0.38% total solids figure is not an accurate design value. The expected total solids content is approximately ten times greater than this experimental value. The above calculation of sludge volume predicts that 290,000 liters of sludge will be produced, at approximately 0.4% solids. Increasing the solids concentration to 4% requires a ten-fold decrease in sludge volume to account for the same mass of solids. Therefore the sludge volume of 290,000 l/d, at 0.4% solids predicts a design sludge volume of 29,000l/d at 4% solids.

3.1.6 Fecal Coliform

If sludge is to be beneficially reused for agricultural purposes, as proposed by this report, it must meet the standards outlined in the EPA 40 CFR part 503 rule: Land Application of Biosolids (U.S. EPA 1993). The Brazilian government has also adopted these standards. Fecal coliforms are used as an indicator organism to assess the health safety of sludge. The presence of fecal coliforms is used as evidence that other pathogenic organisms are also present. In order to meet Class B biosolids standards the sludge must have a fecal coliform count of less than 2,000,000 MPN per gram of dry sludge or be disinfected through one of the approved methods outlined in the legislation. In order to evaluate the treatment steps necessary to make the sludge available for beneficial use, samples were tested for fecal coliform levels. Fecal coliform analysis was done using the most probable number technique, Standard Methods procedure 9221 (Standard Methods 1991). Sample dilutions are incubated in lauryl tryptose broth for 48 hours to test for the presence of total coliform. Positive samples are reinoculated in EC medium and incubated for 24 hours to determine fecal coliform counts. Table 3-6 gives the fecal coliform counts as the most probable number (MPN) per gram of dry sludge.

Table 3-6: Fecal Coliform Counts as most probable number (MPN) per gram of dry sludge

Sludge Type A	MPN (per g dry sludge)	Sludge Type B	MPN (per g dry sludge)
Sample 1	1,000,000	Sample 1	150,000,000
Sample 2	20,000,000	Sample 2	24,000,000
Sample 3	13,000,000	Sample 3	24,000,000
Sample 4	9,000,000	Sample 4	80,000,000
Sample 5	68,000,000	Sample 5	270,000,000
Sludge A Average	**24,000,000**	**Sludge B Average**	**110,000,000**
		Average of All Samples	**67,000,000**

Typical fecal coliform concentrations in unstabilized liquid biosolids are given as 1 x 10^9 MPN per 100ml (McFarland 2001). Converting the average fecal coliform counts for the two types of sludges to these units gives 9.6 x 10^7 MPN per 100ml in Sludge A and 4.4 x 10^8 MPN per 100ml in Sludge B. Therefore, both sludges have fecal coliform concentrations below the typical concentrations. However the fecal coliform concentrations of the two sludges seem to be considerable different, with Sludge B concentrations being much higher than Sludge A concentrations. This may be a result of the characteristics of the chemical additives or the limited number of samples. A larger scale analysis could determine if the fecal coliform counts of the two sludge types are statistically different.

The most probable number counts found in this study indicate that the neither sludge type will meet the quality standards set by the legislation for fecal coliform counts. As a result disinfection methods must be considered if reuse strategies are to be pursued.

3.1.7 Lime Addition

Lime addition is a commonly used and cost effective disinfection technique (WEF Manual of Practice No. 8). According to the EPA 40 CFR part 503 lime addition is an approved method to significantly reduce pathogens (U.S.EPA 1993). To achieve sufficient disinfection and meet Class B biosolids standards through lime addition the pH of the sludge must be raised to 12 and remain at or above 12 for a least 2 hours. The pH must then remain above 11.5 for at least 24 hours (U.S. EPA 1993).

Commercial grade lime, $Ca(OH)_2$ in dry form, was added to the sludge until a pH of 12 was reached. In this study it was preferable to use locally available products for the purpose of assessing treatment strategies to ensure that the proposed design would be financially and technically feasible. The lime used in these tests was obtained from the drinking water plant at the University of Alfenas. In order to analyze the feasibility of this disinfection technique the quantities of lime necessary to raise the pH of the sample to just above 12 were recorded. This data is given in Table 3-7 as the milligrams of lime added per milligram of solids.

Table 3-7: Quantity of Lime Required to Raise Sample pH to 12

Sludge Type A	Lime (mg/mg of solids)	Sludge Type B	Lime (mg/mg of solids)
Sample 1	0.9	Sample 1	1
Sample 2	0.9	Sample 2	0.8
Sludge A Average	**0.9**	**Sludge B Average**	**0.9**
		Average of All Samples	**0.9**

These samples were monitored for 24 hours and met the time requirements for the desired pH levels. Fecal coliform tests were performed on four of the lime treated sludge samples in order to demonstrate disinfection and ensure the effectiveness of the lime addition. These samples all contained less than 3500 MPN per gram of dry solid. The fecal coliform counts were decreased by a minimum of four orders of magnitude by the addition of lime. This data demonstrates that a lime dosage of 0.9 milligrams (per milligram of dry solids) provides adequate disinfection and reduces the fecal coliform counts in the sludge to well below the 2,000,000 MPN level required by the legislation.

Typical lime dosages for primary sludge are between 0.06 and 0.17 grams of lime per gram of solids (U.S. EPA 1979). However these typical values are for sludges with 2-5% solids, considerable higher solids content than sludge analyzed in this study. The higher quantity of lime required for pH adjustment in this test may reflect the additional volume of water that had to be treated given the high solids dilution (WEF 1995). Sludges with solids content below 2% typically require high lime dosing (WEF 1995). The dosage required in this study may also indicate that the lime used was of low quality. Because lime reacts with iron to form iron hydroxide species, the presence of iron in the Sludge A may also account for some of the lime requirement (McFarland 2001).

Because the sludge studied in these tests had a considerable lower solids content (~0.4) than the sludge that will be produced at the proposed plant (4%) the lime dosage required in these tests (0.9 mg/mg of dry solids) is not an appropriate design value. The actual amount of lime necessary for disinfection will be considerable lower and is expected to be more consistent with typical dosages for primary sludges, between 0.6 and .17 grams of lime per gram of dry solids (U.S. EPA 1979). In order to ensure disinfection and take into account the effect of ferric chloride a design value of 0.2 grams of lime per gram of dry solids will be used.

The addition of lime also impacts the total solids content of the sludge. By mixing lime with the sludge the amount of solids in the sludge, and the final weight of solids to be disposed of, is increased.

3.1.8 Nutrients

If sludge is to be applied to agricultural land the nutrient content of the sludge must be known. The nutrient concentrations are used to compare the sludge to conventional fertilizers and to calculate sludge application rates. The percentages of nitrogen, phosphorus, and potassium of the total solids in raw and lime-treated sludge samples are given in Table 3-8 through 3-11. Nitrate nitrogen and potassium were measured using Hach methods 8038 and 8049, respectively (Hach 1997). The methods for ammonia nitrogen and phosphorus are described in Appendix D.

Table 3-8: Nitrate Nitrogen Concentrations of Raw and Lime-Treated Sludge and the Typical Concentration Range and Mean as a Percentage of Total Solids

(McFarland 2001)

Sludge Type	Lime to pH = 12	Nitrate N	Typical Range	Mean
Sludge A	NO	0.003	0.0002 - 0.49	0.05
Sludge B	NO	0.011	0.0002 - 0.49	0.05
Sludge A	YES	0.002	0.0002 - 0.49	0.05
Sludge B	YES	0.006	0.0002 - 0.49	0.05

Table 3-9: Ammonia Nitrogen Concentrations of Raw and Lime-Treated Sludge and the Typical Concentration Range and Mean as a Percentage of Total Solids

(McFarland 2001)

Sludge Type	Lime to pH = 12	Ammonia N	Typical Range	Mean
Sludge A	NO	0.443	0.0005 - 6.76	0.65
Sludge B	NO	0.400	0.0005 - 6.76	0.65
Sludge A	YES	0.445	0.0005 - 6.76	0.65
Sludge B	YES	0.224	0.0005 - 6.76	0.65

Table 3-10: Phosphorus Concentrations of Raw and Lime-Treated Sludge and the Typical Concentration Range and Mean as a Percentage of Total Solids

(McFarland 2001)

Sludge Type	Lime to pH = 12	P	Typical Range	Mean
Sludge A	NO	0.433	<0.1 - 14.3	2.3
Sludge B	NO	0.407	<0.1 - 14.3	2.3
Sludge A	YES	0.160	<0.1 - 14.3	2.3
Sludge B	YES	ND	<0.1 - 14.3	2.3

ND - No Data

Table 3-11: Potassium Concentrations of Raw and Lime-Treated Sludge and the Typical Concentration Range and Mean as a Percentage of Total Solids

(McFarland 2001)

Sludge Type	Lime to pH = 12	K	Typical Range	Mean
Sludge A	NO	0.600	0.02 - 2.64	0.4
Sludge B	NO	0.300	0.02 - 2.64	0.4
Sludge A	YES	0.117	0.02 - 2.64	0.4
Sludge B	YES	0.063	0.02 - 2.64	0.4

Comparing the typical values to the experimental results indicates that the nutrient levels of the sludge samples were within the typical ranges and fell below the mean value for all nutrients examined. Application rate calculations of the sludge to crops and an assessment of the feasibility of agricultural sludge usage, are discussed in section 4.

3.2 Recommended Sludge Treatment

Based on the experimental results presented in section 3.1 and the U.S. EPA standards for land applied sludges (also adopted by the Brazilian government) the proposed treatment strategy was developed. The proposed sludge treatment system is shown in Figure 3-2.

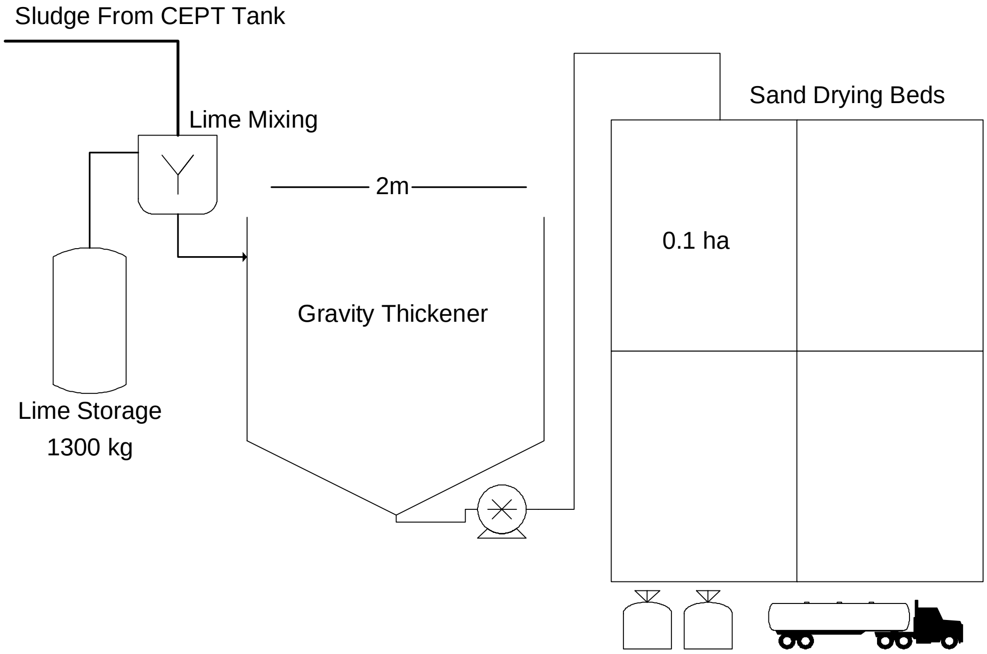

Figure 3-2: Proposed Sludge Treatment System

The sludge collected from the CEPT is pumped into a lime mixing tank where lime is added to a pH of 12 for the purpose of disinfection. After exiting the lime mixing tank sludge enters the gravity thickener, where the solids content of the sludge is increased. The liquid is removed from the top of the gravity thickener and returned to the head of the plant. The thickened sludge is pumped out to sand drying beds where the sludge is dried for a period of 1 to 2 weeks. From these drying beds the sludge can be removed and transported off site to agricultural locations.

3.3 Sludge Production

Calculations of sludge production are vital to wastewater treatment plant design as sludge treatment and handling can account for a large portion of the construction and maintenance costs of the plant. The volume of sludge produced depends on the influent wastewater quality and the type of wastewater treatment process used (WEF Manual of Practice 1998). CEPT plants typically create more sludge than primary treatment plants. This is due, in part, to the enhanced settling of particles, and the chemicals that are added during the CEPT process, that eventually become part of the sludge.

Several methods have been employed to calculate the volume of sludge flow and dry weight of sludge that will be produced by the proposed CEPT plant. The analysis of these methods and the estimates they provide ensures that the sludge management facilities will be appropriately sized.

3.3.1 Method 1: Mass Balance

Step 1: Calculate the mass of solids entering the plant

The influent TSS concentration ranged from 96 to 320mg/l, with a mean of 200mg/l (Olive 2002). The maximum value of 320mg/l will be used in these calculations to ensure that the sludge handling facilities are appropriately sized for maximum loading conditions. The proposed plant will receive wastewater from 20,000 inhabitants of Alfenas. The volume of influent wastewater is calculated based on a daily usage of 180 liters per person (Metcalf & Eddy 1991). The expected daily influent is calculated to be 3.6 million liters per day (Olive 2002).

$$S_{in} = TSS \times Q_{in}$$

Where: S_{in} = Influent solids mass (mg/d)

TSS = Influent total suspended solids concentration (mg/l)

Q_{in} = Influent wastewater volume (l/d)

Using this equation the mass of solids entering the plant is found to be 1,150 kilograms per day.

Step 2: Calculate the mass of solids exiting the plant

The calculation of the mass of solids exiting the plant is based on the 75% removal efficiency of both of the proposed CEPT treatment options (Olive 2002).

$$S_{out} = .25(TSS) \times (Q_{in} - Q_{sludge})$$

Where: S_{out} = Exiting solids mass (mg/d)

TSS = Influent total suspended solids concentration (mg/l)

Q_{in} = Influent wastewater volume (l/d)

Q_{sludge} = Sludge volume (l/d)

Step 3: Calculate the mass of Sludge

The mass of sludge is based on the assumption that the sludge will have 4% total solids. Because the proposed plant will utilize technology similar to that in place at the Point Loma plant the sludge produced is expected to have similar solids content. Sludge produced at the Point Loma CEPT plant has an average of 4.5% total solids.

$$S_{sludge} = TS \times Q_{sludge} = (S_{in} - S_{out})$$

Where: S_{sludge} = Dry mass of sludge (mg/d)

S_{in} = Influent solids mass (mg/d)

S_{out} = Exiting solids mass (mg/d)

TS = Total solids concentration (mg/l)

Q_{sludge} = Sludge volume (l/d)

Step 4: Solve for the volume and mass of sludge

By combing the above equations the volume and mass of sludge can be calculated.

$$S_{out} = S_{in} - S_{sludge}$$

Where: S_{sludge} = Dry mass of sludge (mg/d

S_{in} = Influent solids mass (mg/d)

S_{out} = Exiting solids mass (mg/d)

Using this mass balance the mass of sludge produced is calculated as 863 kilograms per day. The corresponding sludge volume is calculated to be 22,000 liters per day. This calculation predicts that the sludge flow will be approximately 0.6% of the daily influent flow. However, this method neglects the additional sludge resulting from chemical addition.

3.3.2 Method 2: Murcott Equation

Murcott developed this equation for calculating CEPT sludge production (1992). This method accounts for TSS removal and for additional sludge produced from chemical addition.

$$S_p = Q \times [TSS_{rem} + F(P_{rem}) + K(C_c)] \times 10^{-3}$$

Where: S_p = Dry mass of sludge (kg/d)

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

TSS_{rem} = Concentration of total suspended solids removed (mg/l)

F = Stoichiometric factor; 1.42 for mono and trivalent metals, 2.48 for divalent metals

P_{rem} = Concentration of phosphorus removed (mg/l)

K = constant (.66 for $FeCl_3$)

C_c = Concentration of metal salt added (mg/l)

This equation calculates the sludge production based on the total suspended solids removal, as in the mass balance method, but also calculates sludge mass produced by metal salt precipitation ($Fe(OH)_3$) and phosphorus removal. Assuming that Tanfloc does not react chemically in the wastewater and no chemical precipitates are formed, the addition of Tanfloc does not increase the amount of sludge produced.

The total suspended solids removed by the ferric chloride and Tanfloc treatment is 240 milligrams per liter and 7 milligrams per liter of phosphorus are removed. Using this data in the Murcott equation gives a predicted sludge mass of 970 kilograms per day. If the sludge is assumed to be 4% solids the volume of sludge can be estimated as 24,000 liters per day. These calculations are consistent with method 1. Using this type of calculation to determine sludge production from primary treatment without chemical addition gives a sludge mass of 860 kilograms per day and a sludge volume of 22,000 liters per day. Therefore, CEPT produces approximately 10% more sludge than conventional primary treatment. According to these calculations 7% of the sludge produced by the proposed plant will be due to ferric chloride precipitation and 4% will be due to phosphorus removal.

3.3.3 Method 3: Typical production rates

The ASCE manual “Design of Municipal Wastewater Treatment Plants” reports that sludge production rates at municipal plants typically fall between .2 and .3 kg/m^3 and recommends .25 kg/m^3 as an approximation (1998). This method predicts the mass of sludge produced to be 900 kg/d, with a sludge volume of 23,000 l/d (assuming 4% solids).

This is the least accurate of the sludge production estimations, however it is in agreement with the values produced by the more reliable methods.

3.3.4 Design Sludge Volume

The sludge mass and volumes obtained from each of the calculation methods is summarized in Table 3-12.

Table 3-12: Sludge Mass and Volume for each Calculation Method

Calculation Method	Sludge Mass (kg/day)	Sludge Volume (L/d)
Method 1	863	22,000
Method 2	970	24,000
Method 3	900	23,000

The sludge volume and mass used to design the sludge treatment facilities is selected based on the three estimations presented above. These three approximations were relatively in agreement, with the highest estimate of sludge mass of 970 kg/d being only 12% greater than the lowest estimate of 863 kg/d. In order to size the plant and the necessary equipment appropriately, and to accommodate for seasonal peak loadings, the highest estimate of sludge production, 970 kg/d, will be used as the design value. Assuming that the sludge will be 4% solids the design sludge volume is 24,000 l/d. The sludge flow is approximately 0.7% of the influent wastewater flow.

3.4 Lime Stabilization

Lime addition is recommended for the purpose of sludge disinfection. In order for sludge to be utilized on agricultural land, as proposed in this report, it must be effectively disinfected. The U.S. EPA has developed the *Standards for the Use and Disposal of Sewage Sludge* regulations to ensure that sludge applied to land is not a threat to human or environmental health (U.S. EPA 1993). The Brazilian government has also adopted these standards. In order to meet

Class B biosolids standards the sludge must have a fecal coliform count of less than 2,000,000 MPN per gram of dry sludge or be disinfected through one of the approved methods outlined in the legislation. As discussed in section 3.1.6 the sludge samples did not meet the fecal coliform standards and one of the disinfection methods must be employed. Complying with Class B biosolids requirements using lime disinfection requires that the pH of the sludge be raised to 12 for a minimum of 2 hours and remain above 11.5 for 22 hours (U.S. EPA 1993). Other methods could be employed for the reduction of pathogens that would also comply with regulatory standards for agricultural use of biosolids. The advantages of lime treatment, and the motives for recommending it here, are its low capital cost and simplicity of operation (McFarland 2001, WEF 1995). Lime is one of the least expensive and the most widely used alkaline additives available for wastewater treatment (WEF 1995). In addition, lime treatment is feasible because of its availability in Brazil. It is currently used at the University of Alfenas drinking water plant.

In order to comply with the regulatory requirements outlined above it is recommended that the sludge be treated with calcium hydroxide, or hydrated lime ($Ca\ (OH)_2$). There are several types of lime that could be used effectively in this process, including quicklime, which is often selected for its heat generating benefits (WEF 1995). Hydrated lime has been chosen for this plant because it holds several advantages over quicklime. Although hydrated lime costs approximately 30% more than quicklime, it requires significantly less operating equipment. Because quicklime must be converted to hydrated lime, a process called slaking, before it can be added to sludge, additional equipment is required. The use of hydrated lime is economically feasible for small facilities where usage does not exceed 3.5 million grams per day (WEF 1995). The calculations below for lime requirements at the proposed plant indicate that lime usage will be below this limit, confirming the appropriateness of using hydrated lime.

3.4.1 Lime Quantity

In order to calculate the quantity of lime necessary to raise the pH of the sludge above 12 bench scale tests were conducted during the field study period, January 2002. The results of these tests, discussed in section 3.1.5, indicate that 0.9 gram of lime must be added per gram of dry solids in the sludge. However this quantity of lime was required for samples with 0.4% solids, considerable more dilute sludge than will be limed treated at the proposed plant. The actual amount of lime necessary for disinfection will be considerable lower and is expected to be more consistent with typical dosages for primary sludges, between 0.6 and 0.17 grams of lime per gram of dry solids (U.S. EPA 1979). In order to ensure disinfection and take into account the effect of ferric chloride a design value of 0.2 grams of lime per gram of dry solids will be used. Based on the mass of sludge produced by the plant, calculated in section 3.3 as 970 kg/d, approximately 190kg/d of lime are necessary to stabilize the sludge. Lime, in the form of a 10% liquid solution, will be added to the sludge in a lime mixing tank. The volume of liquid solution required is approximately 1900 liters per day.

3.4.2 Level of Disinfection

The lime treated samples used for lime quantity analysis were also tested for fecal coliforms to verify appropriate disinfection. The results of these tests presented section 3.5, show a decrease in fecal coliform counts by four orders of magnitude when compared to samples without lime treatment. The treated samples all contained less than 3500 MPN per gram of dry solid. Monitoring the pH of these samples indicated that they stay at or above the necessary levels to comply with the 40 CFR 503. The tests confirm that disinfection can be attained through the addition of hydrated lime.

It is important to note that if a fecal coliform monitoring program was instituted it may be possible to utilize less lime while still producing Class B biosolids. Sludge can meet the Class B standards if the fecal coliform count is below 2 million MPN/g solid and adding lime decreases the count to well below this level. This suggests that decreasing the fecal coliform count below 2 million MPN/g solid would require less lime than the amount used in this study. However in order to comply with the regulations, if the pH is not raised to 12, the fecal coliform concentrations in the sludge must be monitored to confirm adequate disinfection. While cost savings could be accrued by reducing the amount of lime required, regular fecal coliform testing will require financial resources and a reliable testing location or trained staff. This may be infeasible and challenging to maintain, and monitoring does not eliminate the need for the lime addition system. However, further investigation could determine if the lime cost savings is more significant than the cost of fecal coliform monitoring. This report recommends lime addition to a pH of 12 in order to comply with Class B standards.

3.4.3 Equipment Requirements

The lime mixing tank should allow for a contact time of two hours to ensure that the sludge remains at a pH above 12 for this time period. Therefore the size of the lime mixing tank depends on how often the sludge is pumped from the CEPT tank. Assuming the sludge is pumped into the lime mixing tank only once a day, the tank must hold both the 24,000 liters of sludge and the 1,900 liters of lime solution. The lime mixing tank should therefore have an effective volume of 26,000 liters. This tank must also be equipped with a device for mixing, either mechanical mixing or aeration can be used. Further equipment requirements for this procedure include:

- A storage facility for dry lime with a capacity equal to at least a one-week supply of lime, or approximately 1,300 kg (WEF 1995).

- A tank for lime solution preparation, with a volume equal to a one-day lime solution demand or 1,900 liters.
- A chemical addition system to convey the dry lime from the storage facility to the solution mixing tank and appropriately dose the lime.
- A pump to inject the lime solution into the lime mixing tank.
- A pH meter to ensure adequate disinfection.

The hydrated lime will react with bicarbonate alkalinity in the water and atmospheric carbon dioxide producing calcium carbonate that can clog pipelines (WEF 1995). As a result, the facilities listed above should be located in close proximity to one another to decrease the distance the lime slurry has to be transported.

3.4.4 Mass Balance and Solids Content

The addition of lime increases the solids content of the sludge. Because lime is being added at a ratio of 0.2 grams of lime per gram of solids, the total solids content of the sludge is expected to increase. However, the water added to the sludge with the lime also has a dilution effect and increases the volume of sludge flow. A mass balance can be used to determine the volume of sludge and the concentration of solids exiting the lime-mixing tank.

$$(Q_{in})(C_{sin}) + (Q_{lime})(C_{lime}) = (Q_{out})(C_{sout})$$

Where: Q_{in} = Volume of sludge entering the lime mixing tank (l/d)

C_{sin} = Total solids concentration of sludge entering the lime mixing tank (g/l)

Q_{lime} = Volume of lime solution entering the lime mixing tank (l/d)

C_{lime} = Concentration of the lime solution entering the lime mixing tank (g/l)

Q_{out} = Volume of sludge exiting the lime mixing tank (l/d)

C_{sout}= Total solids concentration of the sludge exiting the lime mixing tank (g/l), equal to $Q_{in} + Q_{lime}$

The above mass balance calculates the total solids of the lime tank effluent to be 4.5%. The volume of sludge exiting the lime mixing tank is the sum of the volume of sludge entering the tank and the volume of lime added, equal to 26,000 liters. The total mass of sludge exiting the lime mixing tank, at 4.5% solids, is therefore 1170 kg/d.

3.5 Thickening

A gravity thickener is recommended, following the lime addition process, to improve the sludge treatment process efficiency and reduce sludge drying costs. Thickening decreases the volume of sludge to be transported to the drying beds and minimizes the sludge drying time, resulting in financial benefits. A gravity thickener operates similar to a settling tank. Sludge accumulates in the bottom of the tank, by gravity, and the water is removed from the top and pumped back to the head of the treatment plant (WEF 1998). The removal of liquid from the sludge stream increases the solids content of the sludge and reduces the volume. The increased solids percentage of thickened sludge allows for faster drying, resulting in reduced acreage requirement for the drying beds, as well as land acquisition and equipment cost savings.

There are a variety of techniques used to thicken sludges, including gravity, flotation, centrifugal, gravity belt, and rotary drum thickeners (WEF 1998). Gravity thickening has been selected for its low capital cost and technical simplicity.

3.5.1 Size

Typically gravity thickeners are designed as circular tanks with a depth of 3 to 4m (WEF 1998). The bottom of the tank is cone shaped with a slope of 2:12 to 3:12 (WEF 1998). A gravity thickener depth of 3m and a floor slope of 2:12 should be adequate for this relatively small treatment plant.

The necessary surface area of the gravity thickeners is often calculated using a method based on bench scale testing and the solids flux theory. An array of settling column tests is conducted to determine the settling velocity of the sludge particles at various solids concentrations (McFarland 2001). The settling velocities are then used to compute the surface area of the thickener. This method is not completely valid because it assumes that the settling velocity of the sludge solids is only a function of the concentration (WEF 1998). Conducting the bench scale tests as required by this method is time consuming and was infeasible for this study. However, gravity thickeners can also be sized based on the extensive existing data on gravity thickener performance (WEF 1998). The *Process Design Manual for Sludge Treatment and Disposal* published by the U.S. EPA gives typical gravity thickener data for various types of sludges (1979). For primary sludge receiving high lime dosing the typical feed solids concentration entering the thickener is 7.5% and the typical concentration of solids exiting the thickener is 12% (U.S. EPA 1979). The typical unit solids loading, or the quantity of sludge that can be applied to the thickener per unit area per time, is given as 120 kg/m$^{3/}$d (U.S. EPA 1979). The concentration of solids exiting the lime mixing tank and entering the gravity thickener was calculated, in section 3.4.1, to be 4.5%. Although this concentration is lower than the typical value of 7.5% given in the EPA guidance document, it is assumed that the unit solids loading rate of 120 kg/ m$^{3/}$d is a valid design value. This value can used to calculate the area of the thickener using the following equation (WEF 1998):

$$A = (S \div U_s) / h$$

Where: A = Surface area of the gravity thickener (m^2)

S = Expected daily solids loading (kg/d)

U_s = Unit solids loading (kg/ m3/d)

h = Height of the gravity thickener (m)

The expected solids loading rate, 1160 kg/d, is the sum of the mass of solids entering the lime mixing tank, approximately 970 kg/d, and the mass of lime required, approximately 190 kg/d. Using the equation above the surface area of the gravity thickener is calculated to be 3.3 m^2, giving a tank diameter of 2m. The overflow rate, based on a sludge volume of 26 m^3/d, as calculated in section 3.3.4, is 8m^3/m^2/d. Maximum overflow rates for primary sludge are typically 15.5 to 31.0 m^3/m^2/d.

3.5.2 Equipment Requirements

If possible the lime treated sludge exiting the lime mixing take will be fed by gravity into the gravity thickener, eliminating the need for a pump. The thickener must contain a rake mechanism for sludge collection and a skimming mechanism and baffle to remove scum and other floating material (WEF 1998). A pump is necessary to transfer solids from the gravity thickener to the sand drying beds and a second pump is required to transfer the overflow liquid back to the head of the plant.

3.5.3 Operational Procedures

The retention time of the thickened sludge can be up to 2 to 4 days, however 1 to 2 days is ideal. The sludge depth within the tank should be kept between 1 and 2m to minimize dilution. If possible the sludge should be removed continuously to ensure consistent and effective thickening. Removal on an intermittent bases should be frequent, once every few hours, rather than once or twice per day (WEF 1998).

3.5.4 Mass Balance and Solids Content

As previously calculated, the sludge entering the gravity thickener will be approximately 4.5% solids and the daily flow rate will be 26,000 liters. As described above for primary sludges, treated with high dosages of lime, the typical solids concentration, of sludge exiting the gravity thickener, is 12%. This value is an appropriate assumption because the sludge entering the gravity thickener is expected to have enhanced settling ability due to its chemical content. The sludge contains lime and, for the primary treatment option, ferric chloride. These chemicals are the most commonly used inorganic conditioning agents, chemicals added to sludge to aid in water removal during thickening and dewatering processes (WEF 1998). The addition of lime introduces calcium carbonate to the sludge, which is dense and porous and creates pathways for rapid water removal (WEF 1998). Ferric chloride aids in thickening in the same manner it enhances settling, through coagulation. The presence of these chemicals in the sludge suggests that it will thicken to at least the 12% solids concentration recommended in the EPA manual.

The volume of sludge exiting the gravity thickener can be calculated assuming a thickened solids content of 12%, and that all the solids entering the thickener exit in the sludge. Using 1160 kg/d as the total solids mass entering and exiting the gravity thickener, the volume of sludge exiting the thickener is approximately 10,000 liters, at 12% solids.

3.6 Sand Drying Beds

Dry sludge is considerable less expensive and more convenient to handle and transport than liquid sludge. Sand drying beds provide a cost effective method for dewatering sludge and is recommended as the final sludge treatment step. The beds allow for dewatering through two processes, evaporation and drainage. Conventional sand drying beds are rectangular and contain layers of sand and gravel which overlay an under drain system for leachate collection.

A variety of mechanical systems are available for sludge dewatering. However the high capital and operating costs of these systems make them inappropriate for this design (WEF 1998). Furthermore mechanical techniques are often employed when space constraints exist and land is not available for the construction of drying beds. The city of Alfenas has an abundance of open land along the periphery of the city and in the area proposed for the treatment plant. The availability of land and favorable climatic conditions indicate that sand drying beds are appropriate and feasible.

3.6.1 Bed Design

The floor of the beds can be constructed of concrete with a slight slope towards the center of the bed to a culvert drain and a slight slope towards one end of the bed for fluid collection. A gravel layer, between 20 and 46 cm deep, should be placed below the sand (WEF 1998). The sand layer should be between 20 and 46cm deep and the sand should be of good quality, free from clay and foreign matter (WEF 1998). Bricks can be layed on top of the sand with some space left between bricks for drainage. The sidewalls and dividers between the beds can also be constructed of concrete and should rise 0.5 to 0.9 meters above the top of the sand (WEF 1998). A diagram of a sand drying bed is shown in Figure 3-3.

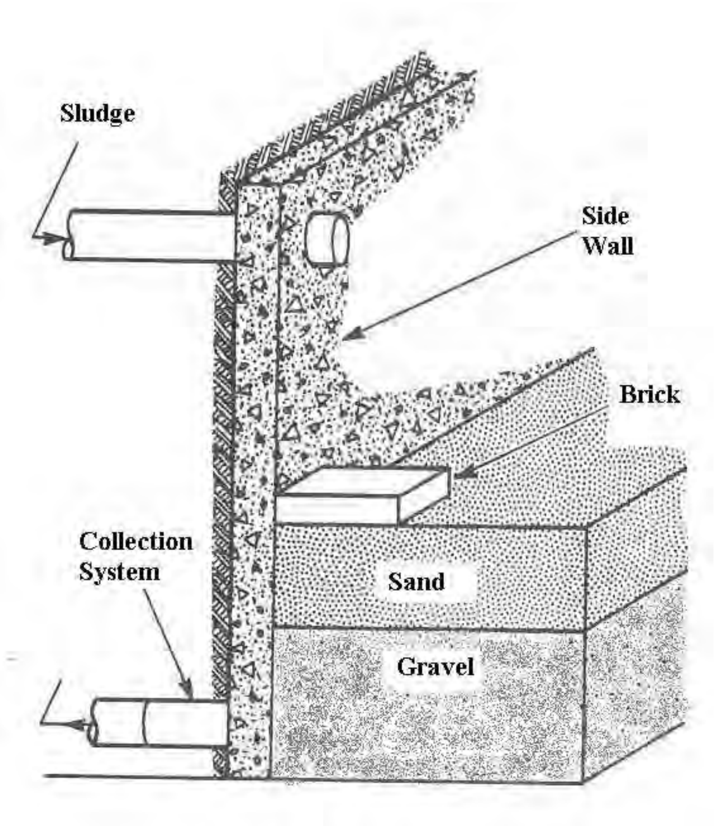

Figure 3-3: Side View of a Sand Drying Bed

(McFarland 2001)

3.6.2 Size

The area of drying beds required is based on the length of time the sludge will require to dry. According to plant operators at a municipal wastewater treatment plant in Serenia, Brazil, where sand drying beds are used, sludge drying requires approximately one week in dry weather conditions and 2 weeks in wet weather conditions. Assuming a 2 week drying period, the drying beds must be capable of containing a 2 week volume of sludge. The volume of sludge entering the drying beds is 10,000 liters per day, requiring a drying bed volume of 140 m^3. Sludge is typically applied to drying beds at a thickness of 20 to 23 cm

(McFarland 2001). Using a design depth of 20cm, the sludge drying bed area required is 700m^2. A safety factor of 1.5 or higher is typically used in the design of sand drying beds, increasing the area requirement to 1050 m^2, or approximately 0.1 hectares.

4. Sludge Disposal Recommendations – Agricultural Use

Disposing of the sludge in an efficient and inexpensive manner will increase the feasibility and effectiveness of the proposed CEPT plant. The disposal technique recommended for the city of Alfenas is agricultural land application. This recommendation is based on analyses of local land usage, recommended wastewater and sludge treatment strategies, sludge characteristics, and financial considerations.

Because of the limited time available for the field study, approximately 3 weeks, the extensive sample collection and analysis that would be required to accurately design a land application system was not conducted. While some sludge nutrient testing was completed, a much larger sample size and a more in depth analysis would provide the data necessary to confidently recommend a land application strategy. The experimental results collected during the field study are used here to obtain a preliminary estimate of the appropriate sludge application rate. A number of locally grown crops may be appropriate for sludge application, however in this study application rates are calculated for coffee, as it is an important and abundant crop in Alfenas and the Furnas Reservoir region. It is recommended that the application rate estimate calculated here be used as the starting point for the land application pilot study outlined below. Before sludge application to agricultural land begins the effect of the sludge on soil and crops should be carefully evaluated. Because the sludge will most likely not contain the precise nutrient ratio required for optimum plant growth and production, maximum benefit may result from the combined application of sludge and supplemental chemical fertilizers. The calculations presented below provide evidence of the value of the sludge as a fertilizer. However, by conducting a pilot study at the University of Alfenas coffee farm optimal application rates and supplemental fertilizer requirements can be determined and the sludge characteristics can be more thoroughly investigated.

4.1 Advantages of Utilizing Sludge as a Fertilizer

Land application is a cost effective sludge disposal method that holds significant advantages for the community and local agricultural production. Sludge can be an effective fertilizer because of its rich nutrient content. Sludge from municipal wastewater treatment plants contains the plant macronutrients nitrogen and phosphorus, as well as the micronutrients boron, manganese, copper, molybdenum, and zinc (U.S. EPA 1995). While the nutrient content of sludge will not match plant needs as well as a carefully formulated commercial fertilizer, most agronomic crops respond favorably to sludge nutrients (U.S. EPA 1995). The nutrients in sludge are released and made plant available at a rate better suited to crop growth and harvesting. The rate of nitrogen release from biosolids is more similar to nitrogen uptake of corn plants than the nitrogen release from commercial fertilizers, which typically create excess nitrogen conditions at the beginning of the growing season and depleted nitrogen conditions near the end (WEF 1998). This excessive nitrogen is a potential pollutant that can be transported to ground or surface water. The comparison of nitrogen release from biosolids and commercial fertilizers to the nitrogen requirements of corn is shown in Figure 4-1.

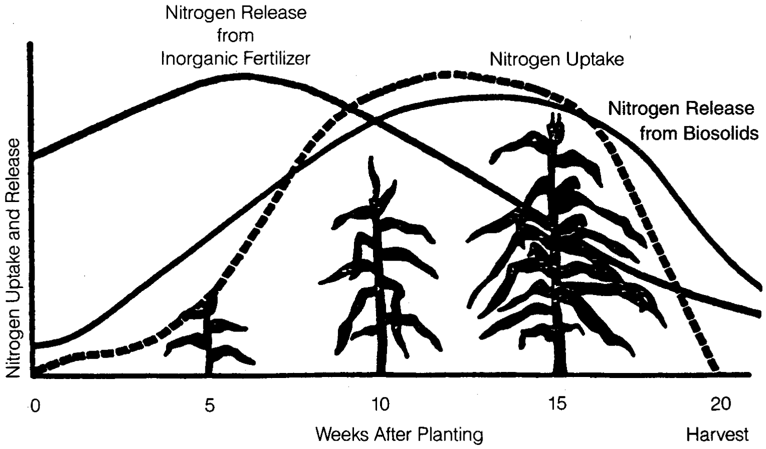

Figure 4-1: Nitrogen Release from Sludge and Commercial Fertilizers and Nitrogen Uptake by Corn Plants

(WEF 1998)

The physical properties of the soil can also be improved through the application of sludge. Fine clays can be made loser and the porosity can be increased, creating space for root growth and water flow. The addition of sludge to sandy soil can increase its water holding capacity and provide chemical sites for nutrient exchange and absorption (U.S. EPA 1995).

Other advantages of sludge application to agricultural land are financial benefits to the community. The municipality may reduce the operational costs of the wastewater treatment plant as agricultural usage is often less expensive than other sludge disposal techniques (U.S. EPA 1995). Agricultural land application eliminates the need for land acquisition which results in further costs savings. This disposal technique also saves valuable landfill space and is an effective method of nutrient recycling. Since the sludge is often provided to the farmers free of charge, farmers can also experience significant financial benefit from the application of sludge to their crops (Matthews 1996).

In addition to being economically favorable, the application of biosolids to agricultural land is relatively low-risk. This practice is considered safe and acceptable, and is encouraged by the U.S. EPA. Nitrogen contamination of groundwater and surface water is the most likely type of contamination resulting from biosolids application (WEF 1998). However, soil microbes release the nitrate-nitrogen in sludge slowly as the crop grows and takes up nitrogen, whereas the nitrogen in commercial fertilizers is released more quickly and is less soluble. As a result nitrogen in commercial fertilizers is more available for movement into the groundwater and presents a greater risk of contamination (WEF 1998). Furthermore, excessive nitrogen loading is avoided by calculating sludge application rates based on the nitrogen needs of the specific crop receiving the biosolids (WEF 1998).

The risk to human health by sludge-born pathogens is negligible when the applied sludge has been treated by lime stabilization, the disinfections technique recommended by this report. Concentrations of disease causing organisms are decreased to levels that do not present a health risk. Furthermore, there has never been a documented case of disease caused by the application of biosolids, when applied according to the EPA regulations (WEF 1998).

4.2 Disadvantages of Utilizing Sludge as a Fertilizer

Sludge can contain chemicals and metals that may be harmful to the plants it is applied to and the eventual end consumers, animals or humans (U.S. EPA 1995). In order to avoid potential negative health effects to humans, livestock, and the environment, regulations have been developed to ensure safe application techniques and rates. The U.S. EPA's 40 CFR part 503 regulations set limits on the quantity of sludge that can be applied per unit area on an annual and cumulative basis (U.S. EPA 1995). The land application of municipal sludge can be carried out safely and effectively by following the management practices outlined by the legislation. Calculations of appropriate sludge application rates, based on the U.S. EPA standards are presented in section 4.6.

4.3 Availability of Coffee Crops

Brazil is the world's largest coffee producer and the second largest consumer (Romero 1999). The coffee industry in Brazil produces over 20 million bags per year and employs 3% of the population. The map in Figure 4-2 indicates the large areas of Brazil where coffee is cultivated.

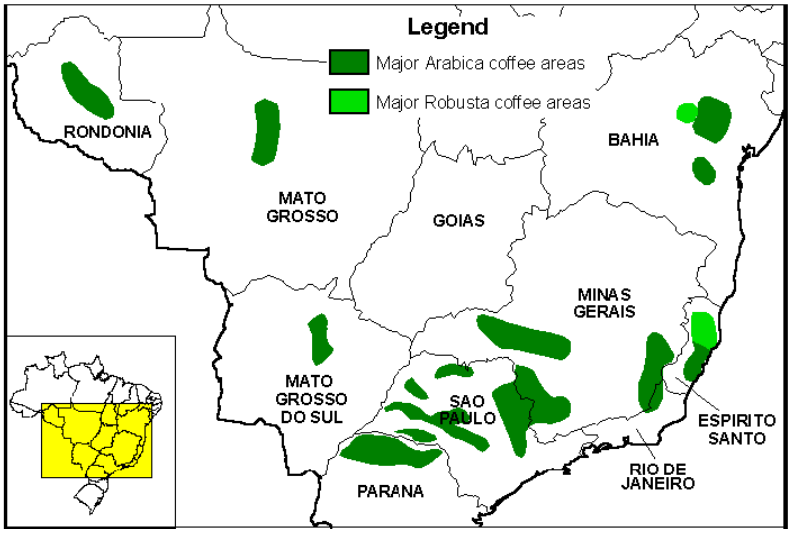

Figure 4-2: Coffee Cultivation in Brazil

(U.S Department of Agriculture, www.usda.gov/agency/oce/waob/jawf/profiles/html/brz/brzcoff.html)

The abundance and importance of coffee in Brazil, as well as the specific characteristics of the crop, make it an appropriate crop for the application of biosolids.

Sludge transportation costs can be considerable and the feasibility of land application as a disposal technique is highly dependent upon cost considerations. Coffee farms have been recommended as potential sludge application sites because of their presence in and around Alfenas and the Furnas Reservoir region. Because of the abundance of coffee plantations in Brazil the techniques recommended here may be applicable in other regions.

4.3.1 Alfenas

Coffee is the primary agricultural crop in and around the city of Alfenas, with 14,100 hectares devoted to coffee cultivation (personal conversation with Renata Santos de Mandonca 2002). The city is home to 360 coffee producers and the annual production of coffee from Alfenas is approximately 330,000 bags or 20 million kilograms(personal conversation with Renata Santos de Mandonca 2002). Approximately the 3% of the coffee crop is consumed locally and 97% is sold commercially.

Small Brazilian cities, such as Alfenas, generally do not have suburbs and as a result the agricultural land directly abuts the city limits. As a result sludge produced at the proposed CEPT plant would most likely only travel a short distance to the final disposal site, minimizing transport costs. The proximity of coffee farms to the city and the proposed treatment plant, as well as the abundance of the crop in the area, indicate that sludge application would be both feasible and sustainable for Alfenas.

4.3.2 Minas Gerais and the Furnas Reservoir Region

The state of Minas Gerais produces 40% of Brazil’s coffee, and, as shown in Figure 4-2, most of this coffee is grown in the southeastern region of the state. This region contains the Furnas Reservoir and the surrounding area. The abundance of coffee throughout the Furnas region indicates that the land application techniques recommended in this report may be feasible for implementation in others cities developing wastewater treatment strategies.

4.4 Coffee Fertilization with Class B Biosolids

Coffee crops provide a more feasible and sustainable application site, as compared to other food crops. Because the coffee plant's cherries, which contain one to three beans, are not in direct contact with the applied biosolids, regulatory compliance is more easily attained and site restrictions and management practices are less stringent.

The sludge treatment techniques outlined in this document meet the Class B biosolids standards of the U.S. EPA's 40 CFR part 503 rule. According to these regulations when Class B biosolids are applied to food crops with harvested parts that touch the biosolids and soil mixture (such as melons, cucumbers, squash, etc.) the crops should not be harvested for 14 months after application (U.S. EPA 1995). Food crops with harvested parts below the soil surface (such as potatoes, carrots, radishes) should not be harvested for 20 months after the application of Class B biosolids (U.S. EPA 1995). However food crops, feed crops, and fiber crops (that do not touch the soil or applied sludge) can be harvested as early as 30 days following the Class B biosolids application (U.S. EPA 1995).

The cherries, containing the coffee beans, are produced above the land surface and therefore have limited contact with the soil or applied biosolids. As a result coffee plants are subject to the least stringent harvesting requirements following sludge application. Figure 4-3 gives the yearly schedule for coffee blooming and harvesting in Brazil.

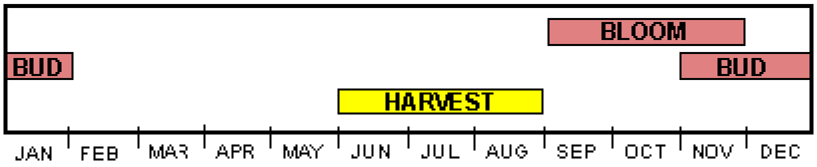

Figure 4-3: Coffee Blooming and Harvesting Schedule in Brazil

(From the U.S Department of Agriculture,
www.usda.gov/agency/oce/waob/jawf/profiles/html/brz/brzcoff.html)

Depending on the harvesting schedule, it may be possible to apply sludge on a regular bases. If sludge application to coffee crops is not possible during the 3-month harvest period, it may be possible to apply to sludge to other local crops. The region also produces fruit, rice, beans, and potato crops that have not been analyzed for sludge application potential in this study. The pilot study, recommended in section 4.11, may provide an opportunity to examine sludge application on other locally available crops. For 9 months of the year coffee is not harvested and sludge can be applied without harvesting time constraints. As a result sludge application on coffee crops is both a feasible and sustainable disposal technique for the city of Alfenas and the surrounding region.

4.5 Site Selection

4.5.1 Site Selection Process

Before the proposed plant is operational specific coffee farms and possibly specific fields will have to be selected for sludge application. The physical and hydrological characteristics of the application sites must be evaluated to ensure that sludge application will be effective and will not impose environmental or human health risks. Economic feasibility and social acceptance issues must also be considered during the site selection process (U.S. EPA 1995). The U.S. recommends a five-step method for evaluating potential application sites (U.S. EPA 1995):

1. Initial site screening
2. Field site survey
3. Field investigations and testing
4. Economic feasibility
5. Final site selection

The details of each of these steps are described in the U.S. EPA's Process Design Manual for the Land Application of Municipal Sludge (1995). This procedure is recommended for the identification of coffee farms in and around the city of Alfenas for the application of sludge produced at the proposed CEPT plant. Conducting this type of assessment allows for maximum benefit to the community as the site chosen will be one that is environmentally, financially, and socially appropriate.

In the United States site selection requirements for the land application of biosolids are set by each state and state permits must be obtained before the application program can begin. It may be necessary to obtain permits for the application of biosolids at a specific sight from the appropriate agency in Brazil.

4.5.2 Site Characteristics of Coffee Farms in Alfenas

As suggested by the five-step site selection procedure there are important physical site characteristic that must be investigated and considered when planning a land application program. The physical characteristics of concern, as identified by the U.S. EPA are (U.S. EPA 1995):

- Topography
- Soil permeability, infiltration, and drainage patterns
- Depth to groundwater
- Proximity to surface water

During the field investigation period in January 2002 several coffee farms were visited and visual observations of physical characteristics were made. However, specific soil investigations were not conducted at the potential applications sites. General information about the soil and topography in the area of Alfenas was available and can be used, in combination with visual observations, to evaluate the appropriateness of the application of biosolids on coffee farms in Alfenas.

4.5.2.1 Topography

Topography affects the surface water and groundwater flow, which can impact the rate of erosion and runoff at the site. Runoff is of concern when considering biosolids application because rapid overland flow can transport applied biosolids offsite into areas of increased risk, for examples surface water bodies (U.S. EPA 1995). The steepness and length of the slope, as well as the overall shape of the landsurface determine the rate of runoff (U.S. EPA 1995). The U.S. EPA does not recommend the application of biosolids on sites with slopes greater than 15% (U.S. EPA 1995). It was noted, during the field study period, that many of the coffee farms around the city of Alfenas are on hillsides and other uneven or sloped terrain. While no slope measurements were taken, the slope and resulting runoff at some potential sites may be of concern.

4.5.2.2 Soil Permeability and Infiltration

The permeability of the soil and the rate of infiltration through the soil column influences how well and how quickly the sludge will be incorporated into the soil and become available for absorption through plant roots. These parameters also affect the time necessary for rainwater and applied sludge to reach the water table. The U.S. EPA states that with proper design and operation, sludge can be successfully applied to virtually any soil (U.S. EPA 1995). Sites with moderate soil permeability, between 0.24 and 2.4 cm/hr, are preferable to areas with very slow or very rapid permeability (U.S. EPA 1995).

The soil studies in localities around Alfenas have reported the soils to have predominantly sand-clay texture (Silva 1997). The soil is further described as mud to very clayey, with granular texture and having good drainage (Silva 1997). While soil studies at specific potential application sites have not been conducted, these observations of local soil characteristics suggest that land application of biosolids is feasible and appropriate.

4.5.2.3 Depth to Groundwater

The important groundwater parameters that should be considered during the site selection process are the depth to the water table, the existing groundwater quality and the type of usage (U.S. EPA 1995). The U.S. EPA recommends that depth to the groundwater, at an agricultural biosolids application site, be no less than 1 meter if the aquifer is used for drinking water and no less than 0.5 meters if it is an excluded aquifer (U.S. EPA 1995). Generally sites with deeper water tables are preferable to those located above shallow aquifers (U.S. EPA 1995). It is recommended that the water table depth at potential application sites in Alfenas be identified during the site screening process for this proposed project.

4.5.2.4 Proximity to Surface Water

The U.S. EPA recommends examining surface water bodies that may receive runoff from the proposed site, in order to minimize the potential environmental and human health risks of contaminating these water bodies with the wastewater residuals that have been applied at the site (U.S. EPA 1995). It is recommended that surface water bodies in the vicinity of the agricultural sites receiving biosolids from the proposed CEPT plant be identified and an evaluation of the risk of contamination be conducted.

4.6 Nutrient Comparison – CEPT Sludge v. Coffee Plant Requirements

In order to evaluate the effectiveness of sludge from the proposed CEPT plant as a fertilizer for local coffee crops the nutrient content of the sludge must be compared with the nutrient requirements of coffee plants. It is also important to evaluate the content and application schedule of commercial fertilizers currently being used on coffee farms.

4.6.1 Nutrient Requirements of Coffee Plants

The recommended method for determining the fertilizer needs of coffee plants requires the measurement of nutrient concentrations in the soil and plant leaves. The procedure, as outlined by the Brazil Department of Agriculture, suggests that fertilizer application should begin after the coffee trees are three years old (Thomaziella 1999). During the three-year maturation period, the trees grow and adjust to existing soil conditions. The nitrogen concentration of the leaves and the phosphorus and potassium concentrations in the soil are then determined in order to assess the fertilizer requirements. Application rates of nitrogen, phosphorus, and potassium are recommended based on this testing and the expected crop yield. Table 4-1 gives the fertilizer requirements of coffee trees based on these criteria.

Table 4-1: Coffee Crop Fertilizer Requirements, in kg/ha, based on Leaf and Soil Testing and the Expected Yield

(Thomaziella 1999)

Expected Yield (kg/Ha)	**N in leaves (g/kg)**			**P in Soil (mg/dm^3)**				**K in Soil (mg/dm^3)**			
	<25	**26-30**	**>30**	**0.5**	**6-12**	**13-30**	**>30**	**0-0.7**	**0.8-1.5**	**1.6-3.0**	**>3.0**
<600	150	100	50	40	20	20	0	150	100	50	20
600-1200	180	120	70	50	30	20	0	180	120	70	30
1200-1800	210	**140**	90	60	40	**20**	0	240	140	**90**	40
1800-2400	240	160	110	70	50	30	0	240	160	110	50
2400-3600	300	200	140	80	60	40	20	300	200	140	80
3600-4800	360	250	170	90	70	50	30	360	250	170	100
>4800	450	300	200	100	80	60	40	450	300	200	120

As discussed in section 4.3.1 Alfenas produces 20 million kilograms of coffee per year on 14,100 hectares. Given these figures the expected yield can then be estimated as 1400 kg per hectare. In general the coffee farms selected for biosolids application would be composed of mature trees already receiving fertilizer. The fertilizer requirements have, therefore, already been determined and fertilizer is applied at an appropriate rate. The amount of nitrogen, phosphorus, and potassium fertilizer utilized at a particular farm could provide the nutrient requirement information needed to calculate sludge application rates.

In order to estimate typical coffee crop fertilizer needs, for the purpose of this study, a median value for nitrogen leaf concentrations and soil phosphorus and potassium concentrations is chosen from Table 4-1. The nitrogen leaf concentration can be estimated as 26 to 30 g/kg, indicating a nitrogen fertilizer requirement of 140kg/ha, for the calculated crop yield of 1400kg/ha. The soil phosphorus concentration of 13-30 is selected because it is a large range of concentrations and is the most conservative estimate that still permits for phosphorus application. Utilizing this estimate gives a phosphorus requirement of 20kg/ha. The soil potassium concentration is also conservatively estimated to be 1.6-3.0 mg/dm^3, giving a potassium requirement of 90kg/ha. These approximations of the nutrient requirements of coffee trees will be compared to the nutrient concentrations of sludge samples in order to calculate sludge application rates.

4.6.2 CEPT Sludge

The results of nutrient analysis conducted on sludge samples produced during the field study period are presented in section 3.1.8. Nutrient concentrations were measured in both untreated and lime treated samples. The proposed sludge treatment method includes lime treatment of the sludge. As a result, it is appropriate to use the data collected for lime treated samples. Because of the limited sample number and the relative similarity between the two sludge types, the Sludge A and Sludge B concentrations have been averaged. These average nutrient concentrations shown in Table 4-2, serve as approximations that can be utilized to calculate sludge application rates.

Table 4-2: Average Nutrient Concentrations of Lime-Treated Sludge Samples

(as % of total solids)

Nitrate N	Ammonia N	Phosphorus	Potassium
0.004	0.335	0.160	0.090

4.7 Approximation of Biosolids Application Rate

The amount of biosolids applied to a specific site and the rate of application can be determined based on the nutrient requirements for the crop selected or on the limiting metals concentrations (U.S. EPA 1995). Either the nitrogen requirements or the phosphorus requirements of the crop can be used to obtain biosolids loading rates. The legislative limits for annual cadmium addition can also be used to determine appropriate application quantities. The method selected for these calculations is generally chosen based on the sludge composition and on specific site characteristics and concerns, such as existing soil condition.

4.7.1 Calculations Based on Nitrogen Requirements

Because nitrate does not absorb onto soil particles, nitrate contamination of groundwater is a concern whenever nitrogen is applied to soils (U.S. EPA 1995). Calculations of biosolids application rates are often based on the nitrogen requirements of the selected crop to ensure that excessive nitrogen loading does not occur. The organic nitrogen in biosolids, unlike ammonia nitrogen, NH_4^+, and nitrate nitrogen, NO_3^-, is not immediately available for plant uptake (U.S. EPA 1995). Because it is released slowly, for several years after application, residual organic nitrogen from previous years must be considered in calculating biosolids application quantities. The following equation is used to estimate the sludge application rate, in metric tones per hectare, for the first year (U.S. EPA 1993).

$$S = Np / \{[(NO_3) + K_v(NH_4) + F_{(\text{year } 0\text{-}1)}(N_o)]*10\}$$

Where: N_p = Plant available nitrogen (kg/ha)

S = Sludge application rate (mt/ha)

NO_3 = Percent nitrate nitrogen in the sludge

K_v = Volatilization factor

NH_4 = Percent ammonia nitrogen in the sludge

$F_{(year\ 0\text{-}1)}$ = Mineralization factor for organic nitrogen in the sludge in the first Year

N_0 = Percent organic nitrogen in the sludge

The plant available nitrogen provided by the applied sludge must not exceed the crop nitrogen requirement, estimated above as 140kg/ha. The volatilization factor for dewatered sludge is 1. The percentage of the organic nitrogen applied that is mineralized in a given year is represented as the mineralization factor and is dependent on the type of sludge treatment and the years since the application. In the first year following application 40% of the organic nitrogen in unstabilized primary sludge is made plant available (U.S. EPA 1995). The percentages of nitrate nitrogen and ammonia nitrogen in the sludge samples are listed in Table 4-2, however the organic nitrogen content of the samples was not measured. Typical percentages of organic nitrogen in municipal sludge are between <0.1 and 17.6, with a mean of approximately 3 (WPCF 1989). Using these assumptions, the sludge application rate for the first year can be estimated as 9 metric tons per hectare.

For the years following the first year, sludge application rates must take into account residual organic nitrogen from previous years application that becomes plant available during the current year. The organic nitrogen that is mineralized in subsequent years can be calculated using the following equation (U.S. EPA 1993).

$$N_m = (K_m)(N_o)(S)$$

Where: N_m = Quantity of N_0 mineralized in the year under consideration (kg/ha)

Km = Mineralization factor for the year under consideration (kg/mt/%N0)

N_o = Percent organic nitrogen in the sludge

S = Sludge application rate (mt/ha)

The sludge application rate for the second year can then be calculated by combining the two above equations, so that the plant nitrogen needs are met by the plant available nitrogen added in the second year and the residual nitrogen from year one which is mineralized.

$$N_p = N_p \text{ (from second year)} + N_m \text{ (from first year)}$$

As mentioned above, the Np, plant available nitrogen, must equal the plant nitrogen needs of 140kg/ha. This equation can be rewritten in order to solve for the application rate for year two.

$$S = N_p/[(NO_3 + (K_v)(NH_4) + (F_{(year\ 0\text{-}1)})(N_o))(10) + (K_m)(N_o)]$$

Where: K_m = Mineralization factor for the second year (kg/mt/%N0)

N_o = Percent organic nitrogen in the sludge

S = Sludge application rate in year 2(mt/ha)

N_p = Plant available nitrogen (kg/ha)

NO_3 = Percent nitrate nitrogen in the sludge

K_v = Volatilization factor

NH_4 = Percent ammonia nitrogen in the sludge

$F_{(year\ 0\text{-}1)}$ = Mineralization factor for organic nitrogen in the sludge in the first Year

Using U.S EPA recommended values of K_m for unstabilized primary sludge, the sludge application rate for year two and subsequent years can be calculated (1993). The application rates for the first 5 years of sludge application are given in Table 4-3.

Table 4-3: Sludge Application Rates for the First Five Years of Application

Application Year	**Sludge Application Rate** (mt/ha)
1	7.4
2	6.9
3	6.6
4	6.5
5	6.4

4.7.2 Calculations Based on Phosphorus Requirement

Another method for calculating sludge application rates utilizes the crop phosphorus requirement. This alternate sludge application rate based on plant phosphorus needs can be calculated, using the following equation (U.S. EPA 1993):

$$S_p = (C_p/P_p) * (1,000\ \text{kg/mt})$$

Where: S_p = Sludge application rate (mt/kg)

C_p = Crop phosphorus requirements (kg/ha)

P_p = Phosphorus concentration of the sludge (mg/kg)

Most sludges contain relatively equal concentrations of nitrogen and phosphorus, however crop nitrogen needs are often much greater than phosphorus needs. As a result application rates based on phosphorus requirements can eliminate the potential for the over application of phosphorus. This may be particularly important for CEPT sludge because of the increased phosphorus removal from the waste stream, as compared to primary treatment. However, the nitrogen concentration of the sludge analyzed here was roughly twice the phosphorus concentration; as a result sludge application rates based on phosphorus concentrations will be significantly greater than the rates calculated for nitrogen requirements. Only approximately half of the phosphorus contained in the sludge can be considered available for plant uptake (U.S. EPA 1993). Using the experimental values for sludge phosphorus concentrations, the sludge application rate can be calculated as 25mt/ha. Because this rate is much greater than the nitrogen based rate, there is potential for the over application of nitrogen.

4.8 Metals - U.S. EPA Maximum Loading Restraints

When sludge is to be land applied the potential for the contamination of soil and groundwater with heavy metals is a major concern. The EPA has addressed this potential hazard, in the 40 CFR Part 503 rule, by establishing maximum metals concentration limits in sludge and cumulative metals loading rate for agricultural sites (Crites et al. 2000). The first type of standards limits the concentrations of pollutants in the sludge and the second set of standards limit the rate at which sludge can be applied to land (McFarland 2001). These regulations can also limit the number of years that sludge can be applied to the same agricultural location (U.S. EPA 1995). The specific metals concentration limits are outlined in the legislation and are also summarized in a numbered of texts (McFarland 2001, Crites et al. 2000).

Metals analyses were not conducted on the sludge samples produced during the field study period. However, in order to comply with the Brazilian regulations and ensure that metals contamination will not occur, metals testing of sludge samples produced at the proposed plant will have to be conducted before land application can proceed. If metals concentrations of the sludge are of concern, the sludge application rate can be calculated based metal limitations set by the legislation (U.S. EPA 1995).

4.9 Final Recommendation for Disposal

The recommendations presented here are preliminary estimates of the appropriate sludge application rates. The proposed pilot test is a comprehensive study that will provide more extensive and accurate data for determining the value of sludge as a fertilizer and effective application rates. The application rates calculated here can be used as the initial rates for beginning the pilot study.

The calculations of land application rates reveal that the nitrogen based rate is considerably more conservative than the phosphorus based rate. In order to minimize unnecessary nutrient application, and prevent nitrogen, metals, or pathogen contamination of the soil, groundwater, or nearby surface water bodies it is recommended that the lower nitrogen based application rates be used as the design values. The quantity of sludge required to meet the nitrogen needs of coffee crops in the first year of application was calculated as 9mt/ha. Based on the calculations of sludge production presented in section 3.3 the proposed plant will produce approximately 3.5×10^5 kilograms of sludge per year, or 350 metric tons per year. Using the recommended sludge application rate of 9mt/ha, the sludge from the proposed plant could be used to fertilize approximately 40 hectares of coffee crops in the first year of application.

The nitrogen based calculations show that the sludge application rate decreases over the subsequent five years due to the presence of residual nitrogen and the calculations indicate that an appropriate long-term sludge application rate would be approximately 6mt/ha (See Table 4-3). As a result, after the first year the sludge from the proposed plant could be used to fertilize approximately 60 hectares of coffee crops. Sludge applied at the recommended rates to these approximated land areas is meant to meet the nitrogen requirements of coffee trees. However, the sludge may not meet the phosphorus, potassium, or other micronutrient needs of the crop and it may be necessary to apply supplemental fertilizer in order to ensure the expected production and crop yields.

4.10 Feasibility, Transportation, and Cost

The major advantage of drying the sludge in the sand drying beds at the treatment plant site is the ease with which sludge can then be transported and land applied. Removing the sludge from the site by truck is considered an appropriate mode of transporting dry sludge by the EPA (U.S. EPA 1995). Because the sludge is does not require specialized equipment for handling and transport, the feasibility and cost effectiveness of land applying the sludge is improved. An evaluation of sludge transportation modes by the U.S. EPA finds that truck transport is the most reliable and least complex and requires low capital investment and operator skill (U.S. EPA 1995).

4.11 Pilot Study at the University of Alfenas Coffee Farm

4.11.1 The University of Alfenas Coffee Farm

The University of Alfenas (Unifenas) has several farms that are used for educational and experimental purposes. The largest farm, Sociedade Agricola Vitoria, is over 1800 hectares and has both agricultural crops and animals. Coffee trees are grown on 120 hectares of the university owned farm, and animals are kept on 35 hectares, with the largest area of the farm, 900 hectares being devoted to orange trees. (Personal Conversation with Renata Santos de Mendanca January 2002.)

4.11.2 Experimental Setup

The purpose of the pilot study would be to evaluate the effectiveness of CEPT sludge as a fertilizer on local coffee trees. The feasibility of the land application of sludge could be evaluated by comparing the effects of sludge from the proposed plant and commercial fertilizers on the tree characteristics and soil conditions. The area of the Sociedade Agricola Vitoria planted with coffee trees is very large,120 hectares and is larger than the potential land area that could be fertilized by sludge from the proposed plant. An experimental study of CEPT sludge as a fertilizer for local coffee trees would require only a portion of the universities coffee farm. As calculated above if all the sludge produced at the proposed plant were to be used in the experiment, approximately 40 hectares of coffee trees could be fertilized. Assuming that the coffee farm would be available, it is recommended that 80 hectares be devoted to this study. This 80 hectares can be divided in half so that 40 hectares receives sludge fertilization and 40 hectares receives commercial fertilizer, serving as a control.

The sludge from the proposed plant may not meet the phosphorus, potassium, or other micronutrient needs of the crop and it may be necessary to apply chemical fertilizer in combination with the sludge to obtain the expected production and crop yields. Furthermore, because the sludge production is relatively small compared to the available coffee acreage, it is unlikely that a farm will depend solely on sludge for fertilizer. Farmers may choose to apply sludge in combination with commercial fertilizers and the field study should also address this possibility. Varying combinations of sludge and chemical fertilizer can be experimented with and an ideal mixture and application schedule can be developed.

4.11.3 Proposed Tests

4.11.3.1 Metals Uptake

In order to comply with regulations sludge must be tested to determine theconcentration of the following metals: arsenic, cadmium, copper, lead, mercury, molybdenum, nickel, selenium, zinc (McFarland 2001). As discussed in section 4.8. concentration limits and maximum loading rates exist for these metals and monitoring is required. Before the pilot study begins the concentrations of these metals in the sludge should be determined and a metals analysis should be conducted periodically during the study. Because the wastewater stream, Jardim da Boa Esperança, being treated by the proposed plant does not include any industrial outputs, it is not anticipated that metals concentrations will be of concern. However metals testing must still be carried out as naturally occurring metals may be present.

Metals uptake by plants is also a concern when land applying biosolids. While evidence suggests that metals accumulation in plants is minimal, especially in the fruits of trees (like the coffe cherry), the pilot study should conduct some analysis of the metal content of the coffee cherries and leaves (U.S. EPA 1993).

4.11.3.2 Soil Quality and Crop Productivity

The quality of the soil determines the plant productivity and should be monitored closely to determine if optimum crop yields can be obtained using sludge as a fertilizer. To assess the effectiveness of sludge fertilization both the soil parameters and the crop productivity must be monitored closely and compared. Monitoring the nutrient content of the soil is particularly important, as sludge nutrients may not be as available for plant uptake as nutrients contained in commercial fertilizers. The nutrient content of the coffee plant leaves can also be tested to quantify the availability of the nutrients in the two fertilizer types. Sludge application may also require the addition of supplemental fertilizers and the quantities necessary should also be recorded during the pilot study. Soil pH should be monitored to ensure that it remains above 6.5 to minimize metals uptake. The recommended sludge treatment system raises the pH of the sludge above 12. The addition of this sludge will raise the pH of the soil and, as a result, low soil pH is not expected to be a concern. Productivity can be assessed by counting or weighing the cherry or coffee bean production of the trees.

4.12 Community Acceptance

The addition of sludge to food crops is a controversial issue because the general public, the end users of the crops, and environmentalists often have concerns about human and environmental health. This recommended pilot study should provide evidence that sludge application to coffee crops is not only a financially feasible sludge disposal solution, but is safe, effective and beneficial to local farmers, and the community as whole. Confidence in the safety of the sludge application program can be increased by presenting data and information about the sludge characteristics and the pilot study results to the local community.

5. Furnas Reservoir Water Quality Modeling

The purpose of this project is to predict how the water quality in the Furnas Reservoir will improve as a result of Chemically Enhanced Primary Treatment (CEPT) implementation at Alfenas. This project examines the reduction in bacterial and nutrient concentrations that is expected in the reservoir due to the use of CEPT at wastewater streams discharging from Alfenas to the Furnas Reservoir.

The objective of this project is accomplished in two ways. First, the existing nutrient and bacterial concentrations are determined in the reservoir. Using the expected reductions of these concentrations due to CEPT use at Alfenas, as well as historical data about the water quality in the reservoir, the improvements in contaminant concentrations are predicted using a mathematical model. The model is applied to a well-mixed section of the reservoir where wastewater from Alfenas is discharged. Reductions in BOD, fecal coliform, and phosphorus were examined. Since a reduction in BOD concentration can lead to increases in dissolved oxygen (DO) concentration, improvements in DO concentrations are also examined. Decreases in DO concentrations could potentially lead to anoxic conditions in the reservoir, causing eutrophication to occur.

Reductions in BOD, fecal coliforms, and phosphorus were examined. Since a reduction in BOD concentration can lead to increases in DO concentration, improvements in DO concentrations are also examined. Decreases in DO concentrations can potentially lead to anoxic conditions in the reservoir, causing eutrophication to occur.

A combination of options is examined. The mathematical model is used to predict concentrations when the reservoir is at its highest and lowest volume, as well as situations where CEPT was or was not implemented. The data used for the model is based on reservoir measurements from onsite analyses in Brazil. Annual water quality data from the University of Alfenas is also used.

Raw sewage discharged from Alfenas is collected in open-channel streams. Open-channel flow occurs in these streams, where the surface of the flowing liquid within the streams is at atmospheric pressure. There are eight wastewater streams emanating from Alfenas, as shown in Figure 5-1, seven of which directly discharge into the reservoir and are the subject of this study.

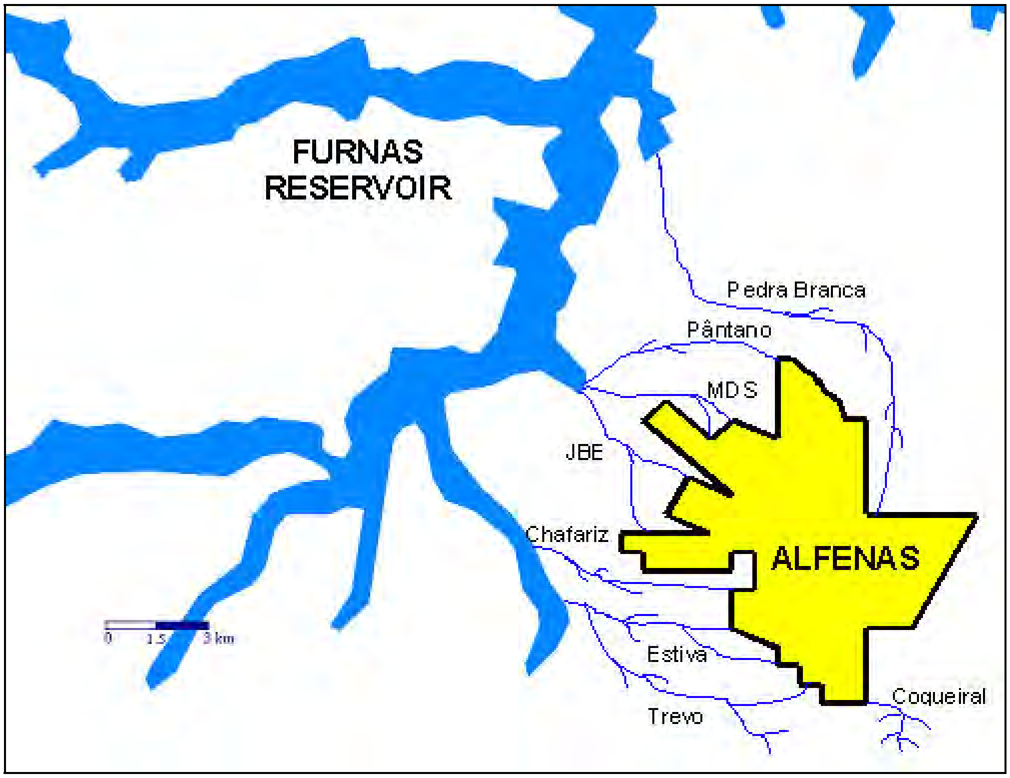

Figure 5-1: Open-channel streams from Alfenas discharging into the Furnas reservoir

Seven of these streams flow to the reservoir. Pedra Branca, Trevo, Estiva, Chafariz, Jardim Boa Esperança (JBE), Morada do Sol (MDS), and Pântano flow directly to the reservoir. Each stream contributes a percentage of the total wastewater flowing from the city, according to Table 5-1.

Table 5-1: Wastewater percentages from open-channel streams

Stream	Percent of Total Wastewater from Alfenas (%)
Pedra Branca	32
Trevo	4.1
Estiva	11
Chafariz	16
JBE	11
MDS	5
Pântano	16

Source: Engesolos Report, 2001

CEPT implementation is expected to result in a 90% reduction in phosphorus and a 60% reduction in BOD in wastewater. (Olive, 2002). Following CEPT treatment with disinfection, effluent concentrations of fecal coliform are expected to be reduced from approximately 10^7 MPN/100 ml to 10^2 MPN/100 ml.

5.1 Water Quality Analysis

5.1.1 Variability in Water Quality of Furnas

Variations in the concentrations in the reservoir can only be determined using historical data. These variations were determined using water quality data from the Projecto Furnas reports obtained from the University of Alfenas. (Fateen, 2002).

In the sewage streams, BOD appears to have increased while DO concentrations decreased during the sampling period between 1996 and 2001. This appeared to be the case for most of the sampling points. There appeared to have been no significant change in fecal coliform concentrations for many of these areas that were sampled.

Some of the in-lake sampling points in the reservoir show that BOD concentrations increased, while DO concentrations decreased. However, these changes do not appear significant between 1996 and 2001. The data show a rise in turbidity and a general decline in fecal coliform concentrations.

5.1.2 Experimental Results

5.1.2.1 DO and Temperature

Depth profiles for temperature and DO concentrations in the reservoir are shown graphically Appendix E for each sampling day. These profiles show little to no stratification with respect to DO and temperature. Also, the measurements of DO concentrations and temperature were consistent because the concentrations measured at different locations in the reservoir were approximately the same. (Fateen, 2002). This means that the section of the reservoir studied could be described as a well-mixed system.

DO concentrations ranged from 4 mg/L to 8 mg/L. The lowest DO concentrations occurred at the lowest depths in the reservoir and the higher values were found at the reservoir surface. The temperature ranged between 24°C and 30°C. If fact, there was little variation in temperature throughout the depth of the reservoir. Based on the depth profiles, the section of the reservoir that was studied appeared to be weakly stratified with respect to temperature and DO concentration.

The DO concentration appeared to be highest at the surface of the reservoir and lowest at the reservoir bottom. The concentration was above saturation at times due to the daytime algal production of oxygen. The highest concentration was 123% more than the saturation value. DO saturation concentrations appeared to drop at deeper levels in the reservoir. At the lowest depths, the concentrations are about 50% of the saturation concentration.

5.1.2.2 pH

The pH of the reservoir samples for the first two days of sampling ranged between 7.5 and 7.8, as shown in Table 5-2.

Table 5-2: pH Measurements for 1st and 2nd Day of Reservoir Sampling

Sample number	Depth of Sample	pH Measurement	
		January 17, 2002	January 18, 2002
Point 1	Near surface	7.5	7.4
Point 2	Near surface	8.2	7.6
Point 3	Near surface	7.6	7.5
Point 4	Near surface	7.7	7.8

On the third day, more sampling points were selected and more water samples were collected. Samples were collected at discrete depths, as well as near the surface of the reservoir. All of the samples were collected in Whirl-Pak bags containing a sodium thiosulfate capsule to keep the sample sterile. The pH values ranged between 8.8 and 9.3, as shown in Table 5-3.

Table 5-3: pH Measurements for 3rd Day of Reservoir Sampling

January 21, 2002		
Sample number	pH	
	Near surface	7 ft below surface
Point 1	9.2	--
Point 2	9.3	9.0
Point 3	9.3	9.1
Point 4	9.2	9.2
Point 5	9.3	--
Point 6	9.2	--
Point 7	8.8	--

The pH values for the fourth, and final, day of sampling ranged from 7.8 and 9.1, as shown in Table 5-4. Water samples were taken at various discrete depths of 1, 2, 4, and 7 meters.

Table 5-4: pH Measurements for 4th Day of Reservoir Sampling

January 24, 2002							
Sampling Point	**Depth (m)**	**pH Measure #1**	**pH Measure #2**	**pH Measure #3**	**Average [H+]**	**Average of pH Measurements**	**Average pH**
Point 1	1	8.8	8.9	--	1.43E-09	8.8	8.7
	2	9.1	9.0	--	8.37E-10	9.1	
	4	8.8	9.0	--	1.33E-09	8.9	
	7	7.8	7.9	7.75	1.60E-08	7.8	
Point 2	1	8.9	8.7	--	1.51E-09	8.8	8.8
	2	8.9	9.1	--	9.64E-10	9.0	
	4	9.0	8.7	--	1.59E-09	8.8	
	7	8.7	8.7	8.67	2.08E-09	8.7	

The average pH was calculated by converting the pH values to hydrogen concentrations ($[H^+]$), averaging the hydrogen concentrations, and taking the mathematical log of the average hydrogen concentration. The pH is converted to the hydrogen concentration using the equation: $pH= -\log[H^+]$.

The pH was not reported for water samples collected from Pedra Branca on January 17th. Samples were not collected from either waste stream on January 24th due to time constraints. The pH values are listed in Table 5-5 for the streams.

Table 5-5: pH for Waste Streams

Stream pH Measurements			
Waste Stream	**January 17, 2002**	**January 18, 2002**	**January 21, 2002**
Pedra Branca	Not Reported	7.5	7.0
JBE	7.7	7.5	7.1

5.1.2.3 Orthophosphate

The orthophosphate concentrations in the reservoir and waste streams are listed in Table 5-6 and Table 5-7, respectively. Although water samples were collected from the reservoir on January 24th, nutrient analyses were not conducted on these samples. The average phosphorus concentration for January 17th was 0.008 mg/L, 0.02 mg/L for January 18th, and 0.01 for January 21st. The average phosphorus concentration of the three days was approximately 0.01 mg/L.

Table 5-6: Orthophosphate concentrations in the Reservoir

Sampling Point	Orthophosphate (mg Phosphorus/L)			
	January 17, 2002	**January 18, 2002**	**January 21, 2002**	**January 24, 2002**
Point 1	0.007	0.06	0.02	Not Reported
Point 2	0.007	0.008	0.016	Not Reported
Point 3	0.008	0.008	0.01	--
Point 4	0.008	0.01	0.02	--
Point 5	--	--	0.01	--
Point 6	--	--	0.01	--
Point 7	--	--	0.017	--

Table 5-7: Orthophosphate Concentrations in Waste Streams

Sampling Point	Orthophosphate (mg Phosphorus/L)		
	January 17, 2002	January 18, 2002	January 21, 2002
Pedra Branca	0.06	0.02	0.08
JBE	0.06	0.11	0.15

5.1.2.4 Nitrate and Ammonia

All nitrate concentrations measured for the water samples taken from the reservoir were less than 0.01 mg/L. Also, ammonia concentrations were not detected in any of water samples collected from the reservoir. The ammonia and nitrate concentrations in the open-channel waste streams are listed in Table 5-8.

Table 5-8: Ammonia and Nitrate Concentrations in the Waste Streams

Sampling Point	January 17, 2002		January 18, 2002		January 21, 2002	
	Ammonia (mg/L)	Nitrate (mg/L)	Ammonia (mg/L)	Nitrate (mg/L)	Ammonia (mg/L)	Nitrate (mg/L)
Pedra Branca	1.3	0.001	1.3	0.001	2.4	0.001
JBE	2.5	0.001	2.5	0.002	2.5	0.002

5.1.2.5 COD

The COD concentrations in the reservoir are listed in the tables below. The samples collected from the reservoir on January 21st were collected in Whirl-Pak sampling bags that contained a sodium thiosulfate capsule, which was used to keep the sample sterile. All other samples taken from the reservoir, as well as samples taken from the streams were collected in heat-sterilized bottles provided by the University of Alfenas.

Table 5-9: COD in the Reservoir for January 17th and 18th

Sample number	COD (mg/L)	
	January 17, 2002	January 18, 2002
Point 1	23.3	23.10
Point 2	15.5	7.80
Point 3	25.8	22.70
Point 4	21.6	23.70

The reservoir samples show elevated COD levels, which may have been due to the chemical capsule in the sampling bag. Thus, due to the uncertainty in these results, the COD measurements for the samples collected on January 21st were not used in any further analysis.

The average COD concentration in the reservoir for January 17th is 28.8 mg/L, 19.33 mg/L for January 18th, and 7.2 mg/L for January 24th. The average COD concentration of the three days is 18.4 mg/L.

Table 5-10: COD Measurements for January 21st

Sample number	COD (mg/L)	
	Near surface	7 ft below surface
Point 1	36.50	--
Point 2	21.90	78.30
Point 3	125.30	78.40
Point 4	101.10	30.90
Point 5	39.50	--
Point 6	88.10	--
Point 7	33.40	--

Table 5-11: COD Measurements for January 24th

Sampling Point	Depth (m)	COD (mg/L)
Point 1	1	10.8
	2	14.4
	4	7.2
	7	3.6
Point 2	1	7.2
	2	3.6
	4	7.2
	7	3.6

Table 5-12: COD Concentrations in Waste Streams

Waste Stream	January 17, 2002	January 18, 2002	January 21, 2002
Pedra Branca	12.2	54.4	11.8
JBE	37.8	148.4	36.5

5.1.2.6 Fecal Coliform

Table 5-13 shows the fecal coliform concentrations in the reservoir and the waste streams. Points 1 through 7 represent the fecal coliform concentrations in the reservoir. The average fecal coliform concentration in the reservoir for January 21st was 335.8 MPN/100 ml. The average concentration over January 17th, January 18th, and January 21st was 568.6 MPN/100 ml in the reservoir.

Table 5-13: Fecal Coliform Concentrations in MPN per 100 ml

Sampling Point	January 17, 2002	January 18, 2002	January 21, 2002
Point 1	--	--	--
Point 2	--	--	--
Point 3	--	--	45
Point 4	170	1200	93
Point 5	--	--	700
Point 6	--	--	--
Point 7	--	--	460
Pedra Branca	1.7×10^4	2.2×10^5	2.0×10^3
JBE	2.2×10^6	1.6×10^8	4.5×10^5

5.2 Water Quality Improvement in the Reservoir

In this section, a mathematical model is used to predict the level of improvements of the ambient concentrations in the reservoir due to CEPT implementation. This study will focus on two cases: (1) predicting the reservoir concentrations at high and low water volume in the reservoir, and (2) predicting concentrations in the reservoir due to the use of CEPT and the case in which CEPT is not used to treat the wastewater in Alfenas. Only BOD, fecal coliform, phosphorus, and dissolved oxygen concentrations are predicted.

For examining concentrations at high and low reservoir volumes, annual water quality data gathered by the University of Alfenas was used. At high volume, 1996 data is used to estimate concentrations. At low volume, 2001 data was used, since the reservoir dropped to its lowest level in this year. This examination looks at the impact of how the changing reservoir levels will impact the concentrations in the reservoir and how effective CEPT will be in reducing these concentrations.

The concentrations in the reservoir are predicted using a well-mixed model for the section of the reservoir studied during January 2002. There is little variation in this section based on depth profiles conducted on the reservoir, in which the sampling points are shown in Appendix D. Therefore, a well-mixed model is used to predict the ambient concentration for local use of CEPT in Alfenas.

5.2.1 Well-Mixed Model

5.2.1.1 Theory

The well-mixed model is based on a completely mixed system, in which the concentrations are the same throughout the volume of the system. It is also assumed that the volume of fluid within the system is completely mixed. Figure 5-2 shows an example of a well-mixed system.

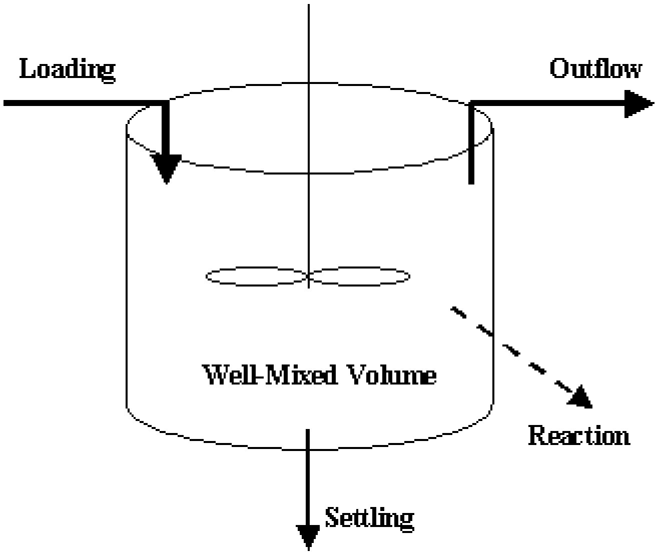

Figure 5-2: Well-Mixed System

The arrows in the figure represent the major sources and sinks of contaminant in the system. The dashed arrow represents a sink due to reaction. It is used to distinguish it from the other sources and sinks which are transport mechanisms. For this study, a section of the reservoir is taken to represent a well-mixed system. The inputs to the system include loading from Alfenas, as well as other tributaries. Losses in the system include contaminant decay due to reaction and settling.

The mass balance for the system is expressed as the following:

Accumulation = Mass in – Mass out + Sources – Sinks

The accumulation term represents the change in mass over time. If mass is being removed from the system, the accumulation term is negative. If the system gains mass, the accumulation term is positive.

$$\text{Accumulation} = \frac{dM}{dt}$$

In the above equation, M represents mass. M can be expressed as C*V, where C is concentration in mg/L and V is the volume in Liters. At constant volume,

$$\text{Accumulation} = V\frac{dC}{dt}$$

The mass entering the system represents the loading. The total loading is the sum all the loadings. For a specific input, the loading is represented by the product of the concentration of contaminant in the input and its flowrate. The loading can be expressed as the following, where C_i is the concentration in mg/L of input i and Q_i is its flowrate in L/d:

$$\text{Mass in} = \sum C_i Q_i$$

The mass exiting the system is the concentration of the contaminant in the outflow multiplied by its flowrate. The concentration at the output is also the same as the average concentration within the system, by definition of a well-mixed system. The outflow is represented by the following equation in which Q_{out} is the total flow leaving the well-mixed system in L/d:

$$\text{Mass out} = CQ_{out}$$

Other losses in the system include reaction decay and settling. These losses are normally characterized by the way in which they occur. For example, the reaction decay is normally represented by mass multiplied by a decay constant. Settling depends on the settling velocity of the particle and the surface area available for falling particles. Reaction decay and losses due to settling can be expressed by the following, where k_d is the decomposition rate in day^{-1}, u is the apparent settling velocity in m/s, and A_s is the surface area of the sediments in m^2:

$$\text{Reaction} = k_d * V * C$$

$$\text{Settling} = u * A_s * C$$

The most common representation for reaction decay, which purge contaminants from natural waters, is kVC. Settling losses can be formulated as a flux of mass across the surface of the sediment-water interface, as shown in Figure 5-3. (Chapra, 1997).

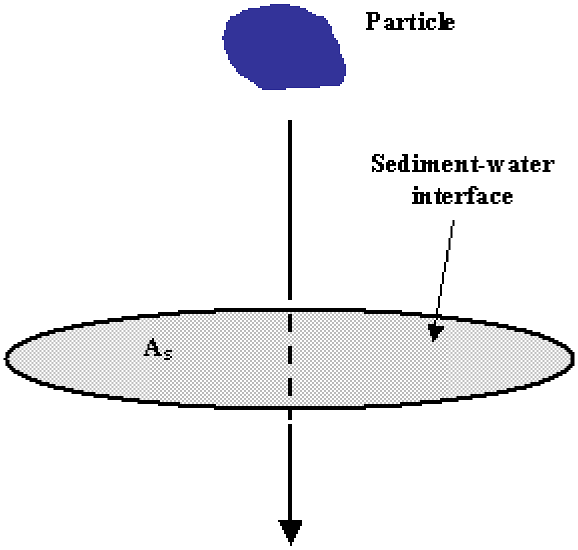

Figure 5-3: Particle falling to a sediment surface

The product of the flux and the area gives the settling term in the mass balance. Since the volume is equal to the average depth, H, and the reservoir surface area, the settling equation can be formulated into the first-order reaction:

$$\text{Settling} = k_s VC$$

where k_S is the settling rate of the contaminant, which is equal to $\frac{u}{H}$. Often, the settling rate and the reaction decay rate are considered together. Thus, the terms k_S and k_d are combined as shown in the following:

$$k=k_S+k_d$$

in which k represents the total removal rate (day^{-1}) of the contaminant, which includes both decay and settling.

Therefore, the complete equation is $V\frac{dC}{dt} = \sum C_i Q_i - CQ_{out} - kVC$. At steady state $\frac{dC}{dt} = 0$ and rearranging the equation to solve for C gives:

$$C = \frac{\sum C_i Q_i}{Q_{out} + kV}$$

The concentration C within the well-mixed volume represents the average concentration. Therefore, $\overline{C}$ is used to represent C. So, the equation becomes:

$$\overline{C} = \frac{\sum C_i Q_i}{Q_{out} + kV}$$

where, $Q_{out} = Q_{Alf} + Q_{in}$

C_i is the concentration input in mg/Liter; $\overline{C}$ is average concentration in the control volume in mg/Liter; Q_i is the flowrate of input stream i in Liters/day; Q_{out} is total the flowrate leaving the control volume in Liters/day; k is the decay constant in day^{-1}; and V is the volume of the section in Liters. The above equation applies to BOD, phosphorus, and fecal coliform concentrations.

Dissolved oxygen concentrations in the reservoir decrease as BOD concentrations increase in the reservoir. This can lead to detrimental anoxic conditions in the water body. CEPT may actually improve DO concentrations in the reservoir because it reduces BOD concentrations. The calculations for predicting changes in DO concentrations in the reservoir are shown in Figure 5-4.

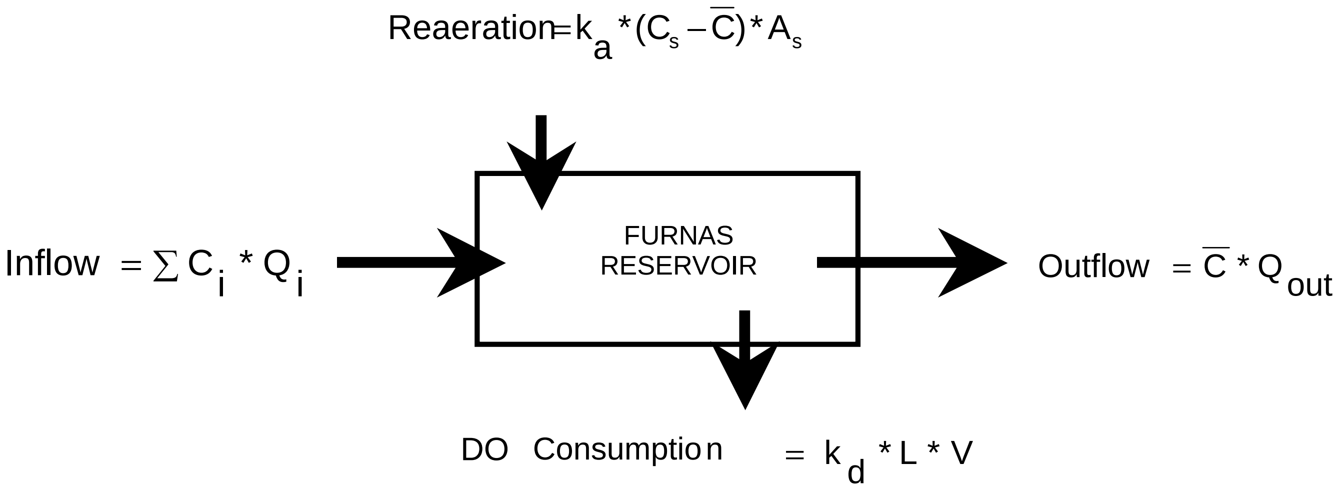

Figure 5-4: Schematic of DO Concentrations in the Reservoir

The parameters C_i and $\overline{C}$ are the inflowing and average DO concentrations in the reservoir; k_a is the reaeration constant; C_s is the DO concentration at saturation; V is the reservoir volume; A_s is the surface area; Q_i and Q_{out} are the inflow and outflow flowrates; and k_d and L are the decay constant and concentration of BOD. The mass balance around the reservoir section leads to the following equation:

$$\overline{C} = \frac{C_{in}Q_{in} + k_a C_s A_s - kLV}{Q_{out} + k_a A_s}$$

The removal rate k for BOD is used in place of the decay rate k_d for BOD, although this may result in an underestimation in the predicted DO concentration.

5.2.2 Application to Furnas Reservoir

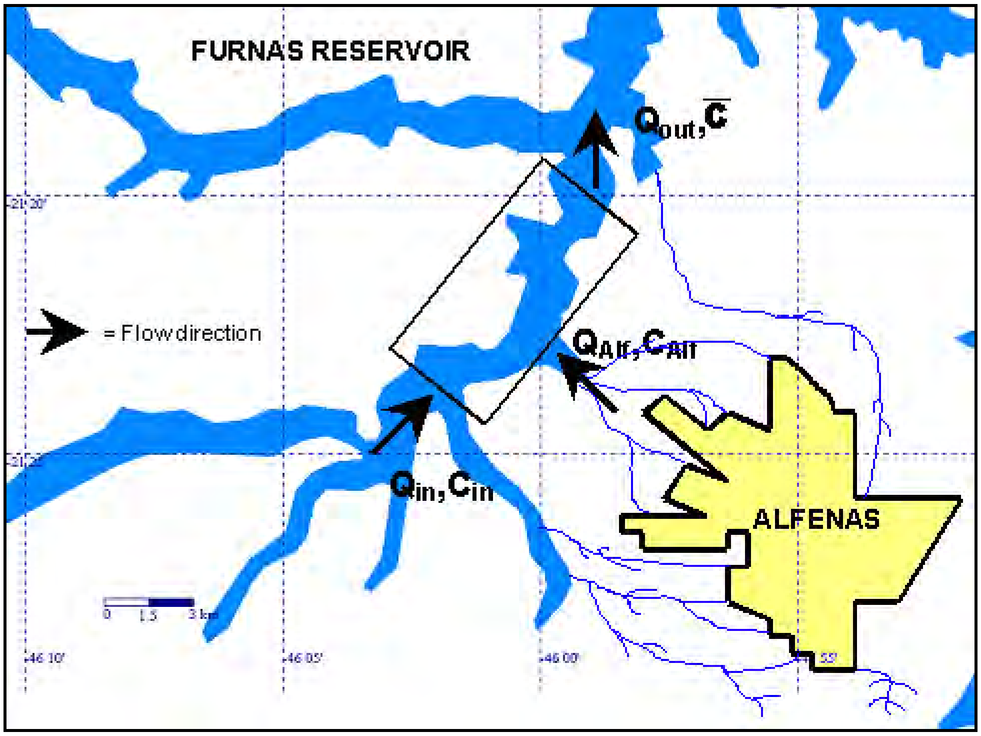

Figure 5-5: Well-Mixed Section of Furnas

The principles for the well-mixed model, discussed previously, were applied to a section of the Furnas Reservoir to predict BOD, fecal coliform, and phosphorus. Concentration predictions for DO are handled somewhat differently, also discussed previously. Due to the drop in the reservoir volume between 1996 and 2001, data from these years are used to predict ambient concentrations in the reservoir at high volume (1996) and at low volume (2001).

The loading entering the section from Alfenas is based on a rate of sewage production from the city of 180 L/day-inhabitant and the percentage of the total population that the wastewater streams serves. (Metcalf and Eddy, 2002).

A population of 58,963 inhabitants for the city is used for 1996. (IBGE website). The census population for 2000 is used for the concentration prediction of 2001. The 2000 population is reported to be 66,957 inhabitants. (IBGE website). To simplify the calculations, a rounded number of 67,000 is used for 2001, and 59,000 is used for 1996. There are three wastewater streams flowing from Alfenas directly into the control volume in the reservoir. These streams are called Pântano, MDS, and JBE. Altogether, these streams represent raw sewage from a total of 32 % of the total population of Alfenas. (Furnas Lake Users Association, 2001).

COD is used as a surrogate for BOD. Based on the BOD to COD correlation for the raw sewage, the ratio of BOD to COD is estimated to be 0.6. The COD concentration in of the raw sewage, based on field studies, is 494 mg/L. Thus, the BOD concentration is 60 percent of the COD concentration or and 296 mg/L. (Olive, 2002)

The fecal coliform concentration in the raw sewage was measured. This concentration was determined to be 8 x 10^6 MPN per 100 ml. This value varies in comparison to the fecal coliform concentrations in the two streams reported in Chapter 5. The fecal coliform concentration in the raw sewage will fluctuate.

The fecal coliform and BOD concentrations are based on field studies conducted by the project team at the JBE wastewater stream in Alfenas. For this study, these concentrations are assumed to be the same as the concentrations in the other streams.

5.2.3 Reservoir and Stream Flowrates

The flowrates can be estimated using rainfall data or using information about the total flow in the reservoir as shown in Equation 5-1 and Equation 5-2.

Equation 5-1 is used to estimate the flowrates using rainfall data. In the equation, P is the precipitation, ET is the evapotranspiration, and DA is the area of the drainage basin. Q_{Alf} is the flow contribution from Alfenas, and Q_{in} is the other stream flows. Q_{out} is the total flow from the system, as shown in Figure 5-5. Groundwater flow into and out of the system is assumed to be the same. Thus, groundwater flow is assumed to be negligible or zero. The evapotranspiration is estimated using the Thornwaite and Mather method for estimating potential evapotranspiration. (Thornwaite and Mather, 1957). Estimates for evapotranspiration assume a soil moisture storage of 50 mm. This value may be higher. Higher values of soil moisture storage result in decreased flow calculation. The total rainfall was 1445 mm/yr for 2001 and 1473 mm/yr for 1996.

(Furnas Lake Users Association, 2002). The estimated evapotranspiration is 882.7 mm/yr for 2001 and 916.2 mm/yr for 1996. (Thornwaite and Mather, 1957).

$$Q_{out} = (P - ET) * (DA)$$
$$Q_{out} = Q_{Alf} + Q_1$$

Equation 5-1: Flowrate Calculation using Rainfall Data

Equation 5-2 is used to estimate the flowrates using total flow in the reservoir at the Dam. In the equation, Q is the total flow in the reservoir, DA_i is the drainage basin area of our control volume, and DA is the area of the entire drainage basin for the Furnas region. The total flow, Q, into the reservoir can be described by the following equation, where Q_{dam} is the total flow through the turbines at the dam and $\frac{dV}{dt}$ is the change in water storage over time in the reservoir.

$$Q = Q_{dam} + \frac{dV}{dt}$$

Both Q_{dam} and $\frac{dV}{dt}$ are known. (Furnas Lake Users Association, 2002). Due to unavailability of some data, 2000 data is used for calculations for dates beyond 2000. So, the total average reservoir flows for both years 876 m^3/s for 2001 and 957 m^3/s for 1996. (Furnas Lake Users Association, 2002). The area for the drainage basin for the well-mixed system is 1,537 km^2 and the drainage basin area for the Furnas region is 46,450 km^2. (IBGE, 1997).

$$Q_{out} = Q * \frac{DA_i}{DA}$$

Equation 5-2: Flow calculation using Total Reservoir Flow

The loading from Alfenas is based on the raw sewage discharged from the city. The flow from Alfenas for both years is calculated below.

Q_{Alf}=180 L/day-inhab * 67,000 inhab * 0.32 ≈ 4 mil L/d (2001)

Q_{Alf}=180 L/day-inhab * 59,000 inhab * 0.32 ≈ 3.4 mil L/d (1996)

The flowrate for Q_{out} is estimated using both methods and the average is used in further calculations of the average concentrations in the reservoir.

Estimated Q_{out} for 2001:

Rainfall:

$$(1445 - 883)\frac{mm}{yr} * \frac{1}{365\ days} * \frac{m}{1000\ mm} * 1537 * 10^6 m^2 * \frac{1000\ L}{m^3} = 2367\ million \frac{L}{day}$$

Total Flow:

$$876 \frac{m^3}{sec} * \frac{1000}{m^3} * \frac{86400\ sec}{day} * \frac{1537\ km^2}{46450\ \ km^2} = 2504\ million \frac{L}{day}$$

Average: $\frac{(2367 + 2504)\ million}{2} = 2436\ million \frac{L}{day}$

Estimated Q_{out} for 1996:

Rainfall:

$$(1473-916)\frac{\text{mm}}{\text{yr}}*\frac{1}{365\,\text{days}}*\frac{\text{m}}{1000\,\text{mm}}*1537*10^{6}\,\text{m}^{2}*\frac{1000\,\text{L}}{\text{m}^{3}}=2194\,\text{million}\frac{\text{L}}{\text{day}}$$

Total Flow:

$$957\frac{\text{m}^{3}}{\text{sec}}*\frac{1000}{\text{m}^{3}}*\frac{86400\,\text{sec}}{\text{day}}*\frac{1537\,\text{km}^{2}}{46450\ \ \text{km}^{2}}=2736\,\text{million}\frac{\text{L}}{\text{day}}$$

Average: $\dfrac{(2194+2736)\,\text{million}}{2}=2465\,\text{million}\dfrac{\text{L}}{\text{day}}$

Since the 1996 and 2001 flows are so close, we can assume that the total flow for both years is the average of the flows for both years, or 2450 mil L/d. The flowrate Q_{in} is simply the difference between Q_{out} and Q_{Alf}.

5.2.4 Calibration of Existing Conditions in 2002

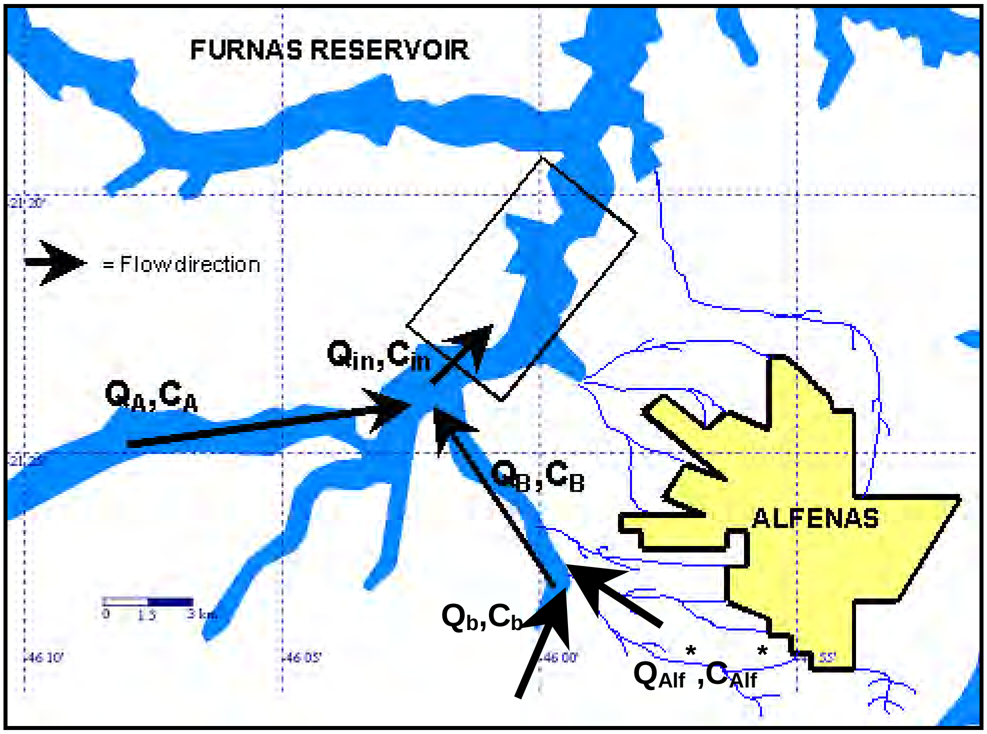

Figure 5-6: Schematic of flows contributing to C_{in}

Calibration of k involves using three equations. Equation 5-4 and Equation 5-5 are used to estimate upstream concentrations. Equation 5-6 is used to perform the final calculation of the removal rate, k, for BOD, fecal coliform, and phosphorus. k is estimated using data from the field studies during January 2002 in Brazil, and water quality data obtained from the reservoir by the Hydric Resources Research Laboratory at the University of Alfenas.

$$k = \frac{C_{Alf} Q_{Alf} + C_{in} Q_{in}}{\overline{C} V} - \frac{Q_{out}}{V}$$

Equation 5-3: Equation for estimating the removal rate, k

Figure 5-6 describes the flow regime for used for the well-mixed system. Concentration C_{in} is found by the following method:

$$C_{in} = \frac{C_A Q_A + C_B Q_B}{Q_A + Q_B}$$

Equation 5-4: Equation for C_{in}

C_A, C_B, Q_A, and Q_B are shown in Figure 6-5. C_A and C_B are based on water quality data collected during January 2002 by the University of Alfenas. Q_A and Q_B are based on flow estimates based on the local watershed. C_B requires estimation using Equation 5-5, where C_{Alf}^* and Q_{Alf}^* represent contaminant concentrations and flow in the waste streams discharged from Alfenas. C_b represents the water quality concentrations measured in the reservoir by the University of Alfenas, and Q_b represents the water flow rate, which is derived from watershed data. C_b and Q_b are upstream of inputs C_{Alf}^* and Q_{Alf}^*, as indicated in Figure 5-6.

$$C_B = \frac{C_{Alf} Q_{Alf} + C_b Q_b}{Q_{Alf} + Q_b}$$

Equation 5-5: Equation for upstream concentration

The specific flow rates are listed in the table below. The volume of the well-mixed section is approximately 36 billion liters, also derived from watershed data and an average depth of 8 meters. These values are based on 2002 estimates.

Table 5-14: Flow rates in Furnas

Parameter	Flow rates (Liters per day)
Q_{out}	2450 million
Q_{Alf} , $Q_{Alf}{}^{*}$ (1996)	3.4 million
Q_{Alf} , $Q_{Alf}{}^{*}$ (2001)	4 million
Q_{in}	2446 million
Q_A	2203 million
Q_B	243 million
Q_b	239 million

The reaeration constant determines the rate in which dissolved oxygen is replenished in the reservoir. The reaeration constant, k_a, is predicted using Equation 5-6.

$$k_a = \frac{\overline{C}Q_{out} + kLV - C_{in}Q_{in}}{(C_s - \overline{C}) * SA}$$

Equation 5-6: DO Reaeration Equation

Neglecting the impact of the BOD concentrations from the waste streams discharging from Alfenas, the DO concentrations in C_B and C_{in} are estimated below.

$$C_B = \frac{C_bQ_b + k_aC_sSA_B - kLV_B}{Q_B + k_aSA_B} \text{ and } Q_{in} = \frac{C_AQ_A + C_BQ_B}{Q_A + Q_B}$$

The surface area for section B, SA_B, is 6 million m^2; C_b is 8 mg O_2/L; C_s is 7.4 mg O_2/L; k is 0.043/day, as derived in the Section 6.3.1; V_B is 30 billion liters; and C_A is 4.6 mg O_2/L. Plugging these values into the above equations, and solving for k_a by trial-and-error leads to a value of 0.85 meters per day.

5.2.4.1 Removal Rate Constant for BOD

C_{Alf} and C_{Alf}^* are 296 mg/L, assuming that the raw waste in all of the waste streams in Alfenas has the same BOD concentration. (Olive, 2002). C_b is 3.8 mg/L, based on water quality measurements taken by the University of Alfenas in January 2002. (Tanure, 2002).

$$C_B = \frac{C_{Alf}^* Q_{Alf}^* + C_b Q_b}{Q_{Alf}^* + Q_b} = \frac{296 * 4 * 10^6 + 3.8 * 239 * 10^6}{243 * 10^6} = 8.6 \text{ mg/L}$$

C_A was assumed to be the average of water quality measurements taken between the months of December 1998 and March 1999, since water quality data was not taken after this time. Therefore, C_A is 1.8 mg/L. (Tanure, 1999).

$$C_{in} = \frac{C_A Q_A + C_B Q_B}{Q_A + Q_B} = \frac{1.8 * 2203 * 10^6 + 8.6 * 243 * 10^6}{2446 * 10^6} = 2.5 \quad \text{mg/L}$$

COD concentrations measured in the reservoir are used as a surrogate for estimating BOD concentrations. Using historical water quality data from the University of Alfenas, a BOD to COD ratio of 0.1 is used to estimate BOD concentrations using COD measurements. BOD to COD ratio of treated wastewater, which is comparable to ambient conditions in water bodies, is between 0.1 and 0.3. (Metcalf & Eddy, 2002).

The average concentration for $\overline{C}$ measured in the reservoir was 18.4 mg/L COD (See Chapter 5). Therefore, the BOD concentration is 0.1 multiplied by 18.4 or 1.84 mg/L.

$$k = \frac{C_{Alf}Q_{Alf} + C_{in}Q_{in}}{\overline{C}V} - \frac{Q_{out}}{V} = \frac{296 * 4 * 10^6 + 2.5 * 2446 * 10^6}{1.84 * 36000 * 10^6} - \frac{2450 * 10^6}{36000 * 10^6} = 0.043/\text{day}$$

5.2.4.2 Removal Rate Constant for Fecal Coliform

C_{Alf} is 8 million MPN/100 ml, with the same raw waste assumption as that for BOD concentrations. (Olive, 2002). C_b is 27 MPN/100 ml, based on water quality measurements taken by the University of Alfenas in January 2002. (Tanure, 2002).

$$C_B = \frac{C_{Alf}{}^{*}Q_{Alf}{}^{*} + C_bQ_b}{Q_{Alf}{}^{*} + Q_b} = \frac{8 * 10^6 * 4 * 10^6 + 27 * 239 * 10^6}{243 * 10^6} \approx 131{,}700 \text{ MPN/100 ml}$$

The same assumptions for BOD above also apply here for C_A, which was assumed to be the average of water quality measurements taken between the months of December 1998 and March 1999, since water quality data was not taken after this time. Therefore, C_A is 81 MPN/100 ml. (Tanure, 1999).

$$C_{in} = \frac{C_A Q_A + C_B Q_B}{Q_A + Q_B} = \frac{81 * 2203 * 10^6 + 131714 * 243 * 10^6}{2446 * 10^6} \approx 13{,}200 \text{ MPN/100 ml}$$

The average concentration for $\overline{C}$ in the reservoir was 568 MPN/100 ml.

$$k = \frac{C_{Alf} Q_{Alf} + C_{in} Q_{in}}{\overline{C} V} - \frac{Q_{out}}{V} = \frac{8 * 10^6 * 4 * 10^6 + 13158 * 2446 * 10^6}{568 * 36000 * 10^6} - \frac{2450 * 10^6}{36000 * 10^6} = 3.1/\text{day}$$

5.2.4.3 Removal Rate Constant for Phosphorus

C_{Alf} and C_{Alf}^* are 7.5 mg/L, with the same raw waste assumptions as that for BOD and fecal coliform concentrations. (Olive, 2002). Since no nutrient water quality measurements were taken in the reservoir by the University of Alfenas, C_b and C_A were assumed to be negligible or zero.

$$C_B = \frac{C_{Alf}^* Q_{Alf}^* + C_b Q_b}{Q_{Alf}^* + Q_b} = \frac{7.5 * 4 * 10^6 + 0 * 239 * 10^6}{243 * 10^6} = 0.12 \text{ mg/L}$$

$$C_{in} = \frac{C_A Q_A + C_B Q_B}{Q_A + Q_B} = \frac{0 * 2203 * 10^6 + 0.12 * 243 * 10^6}{2446 * 10^6} = 0.012 \text{ mg/L}$$

The average concentration for $\overline{C}$ in the reservoir was 0.01 mg/L.

$$k = \frac{C_{Alf} Q_{Alf} + C_{in} Q_{in}}{\overline{C} V} - \frac{Q_{out}}{V} = \frac{7.5 * 4 * 10^6 + 0.012 * 2446 * 10^6}{0.01 * 36000 * 10^6} - \frac{2450 * 10^6}{36000 * 10^6} = 0.015/\text{day}$$

5.2.4.4 Rate Constants

Table 6-2 shows the removal rate constants for BOD, fecal coliform, and phosphorus, and the reaeration constant for DO. The reaeration constant is 0.85 meters per day or 0.11 day^{-1} for the control volume, which has an average depth of 8 meters. The removal rate constant for BOD is expected to be much higher than the estimated value of 0.043 day^{-1}, especially since the reservoir is located in a tropical climate. BOD decay occurs at a rate that increases with increasing temperature. (Bowie et al, 1985). Phosphorus and fecal coliform rate constants appear to be within the range of expected values.

The estimated rate constants neglect temperature variations. These values are used to predict the concentrations for high volume (1996) and low volume (2001) in the reservoir.

Table 5-15: Rate Constants

Parameter	Constant	Value	Units	Effect
BOD	k	0.043	day^{-1}	Total removal rate
DO	k_a	0.11	day^{-1}	Reaeration constant
Fecal Coliform	k	3.1	day^{-1}	Total removal rate
Phosphorus	k	0.015	day^{-1}	Total removal rate

5.3 CEPT Implementation at 3 Streams

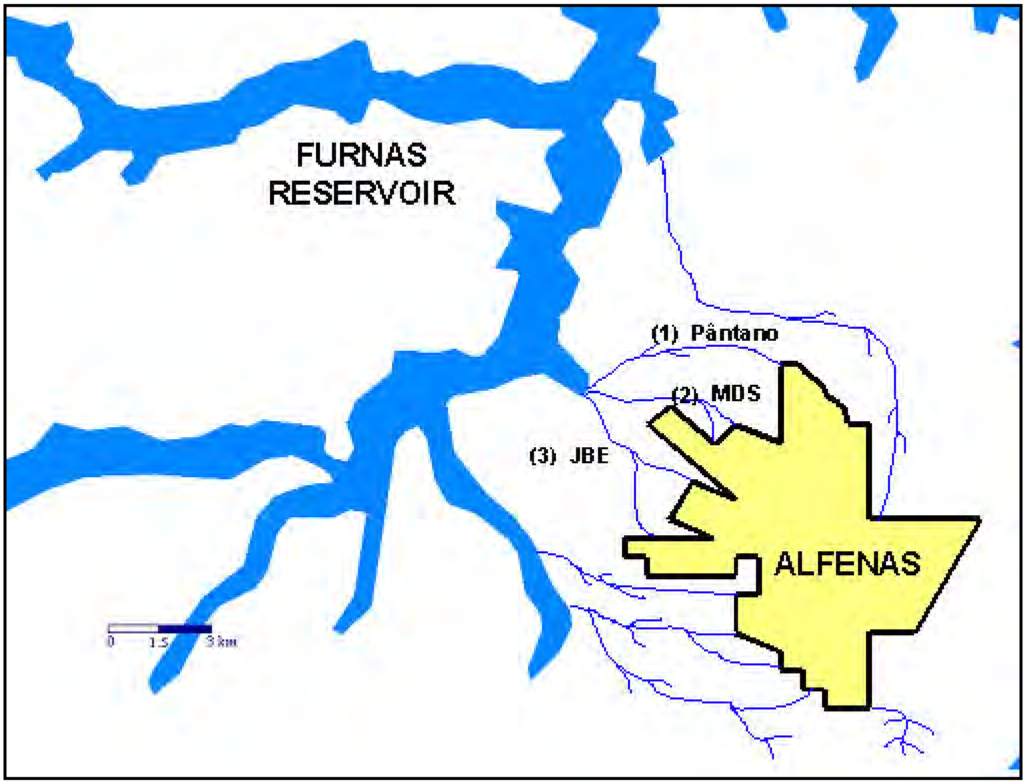

Figure 5-7: CEPT use at 3 waste streams

Assuming CEPT use on the wastewater streams JBE, MDS, and Pântano, as shown in Figure 5-7, the BOD, phosphorus, and fecal coliform concentrations are found using the equations discussed previously and the decay rates, shown in Table 5-15.

The average concentration $\overline{C}$ is predicted for low (2001) and high (1996) water volumes in the reservoir. Table 5-16 through Table 5-19 show the values for C_A, C_B and C_{in} for BOD, fecal coliform, phosphorus, and DO concentrations.

Table 5-16: BOD Concentrations for C_{in} Considering CEPT at 3 Streams

BOD (mg/L)			
Year	C_A	C_B	C_{in}
High Volume (1996)	0.9[a]	5.6	1.4
Low Volume (2001)	1.6 [a]	6.3	2.1
January 2002	1.84 [a]	8.6	2.5

[a] Source: University of Alfenas, Projecto Furnas I, II, and III

Table 5-17: Fecal Coliform Concentrations for C_{in} Considering CEPT at 3 Streams

Fecal Coliform (MPN/100 ml)			
Year	C_A	C_B	C_{in}
High Volume (1996)	356 [a]	112047	11543
Low Volume (2001)	510 [a]	131712	13544
January 2002	81 [a]	131714	13158

[a] Source: University of Alfenas, Projecto Furnas I, II, and III

Table 5-18: Phosphorus Concentrations for C_{in} Considering CEPT at 3 Streams

Phosphorus (mg/L)			
Year	C_A	C_B	C_{in}
High Volume (1996)	0.0	0.105	0.01
Low Volume (2001)	0.0	0.124	0.012
January 2002	0.0	0.124	0.012

Table 5-19: DO Concentrations for C_{in} Considering CEPT at 3 Streams

DO (mg/L)			
Year	**C_A**	**C_B**	**C_{in}**
High Volume (1996)	7.1 [a]	6.6	7.1
Low Volume (2001)	5.7 [a]	6.2	5.8
January 2002	4.6 [a]	5.3	4.7

[a] Source: University of Alfenas, Projecto Furnas I, II, and III

Using the values in Table 5-16 through Table 5-19, and the values inTable 5-15 the average concentrations for BOD, fecal coliform, phosphorus, and DO are predicted for the situation in which CEPT is used at the highlighted waste streams shown in Figure 5-7. These predictions are compared to the situation in which CEPT is not used to determine the level of reduction expected in the reservoir due to CEPT use on the waste streams discharging from Alfenas. The expected reductions are 60% for BOD, 90% for phosphorus, and a 10^4 to 10^5 reduction in fecal coliform if CEPT is followed by chemical disinfection.

Table 5-20: Volumes of Well-Mixed Section of Reservoir for Various Years

Year	Volume (Liters)
High volume (1996)	112.5 billion
Low volume (2001)	11.25 billion[a]
January 2002	36 billion[b]

[a] Estimate based on reservoir losing 8 meters of height between 1996 and 2001
[b] Based on reservoir gaining 5 meters (Furnas Lake Users Association, 2002)

The results for BOD, fecal coliform, DO, and phosphorus are shown in Table 5-21 through Table 5-24.

Table 5-21: Average Reservoir BOD Concentrations, CEPT at 3 Streams

BOD (mg/L)			
Year	Without CEPT	With CEPT	Percent Reduction
High Volume (1996)	0.6	0.5	17%
Low Volume (2001)	2.1	1.9	10%
January 2002	1.8	1.7	9%

Table 5-22: Average Reservoir Fecal Coliform Concentrations, CEPT at 3 Streams

Fecal Coliform (MPN/100 ml)			
Year	Without CEPT	With CEPT	Percent Reduction
High Volume (1996)	159	81	49%
Low Volume (2001)	1762	896	49%
January 2002	568	285	50%

Table 5-23: Average Reservoir Phosphorus Concentrations, CEPT at 3 Streams

Phosphorus (mg/L)			
Year	Without CEPT	With CEPT	Percent Reduction
High Volume (1996)	0.021	0.011	45%
Low Volume (2001)	0.024	0.014	45%
January 2002	0.024	0.014	45%

Table 5-24: Average Reservoir DO Concentrations, CEPT at 3 Streams

DO (mg/L)			
Year	Without CEPT	With CEPT	Percent Improvement
High Volume (1996)	7.41	7.45	0.6%
Low Volume (2001)	6.73	6.75	0.3%
January 2002	5.88	5.92	0.7%

5.4 *CEPT Implementation at 6 Streams*

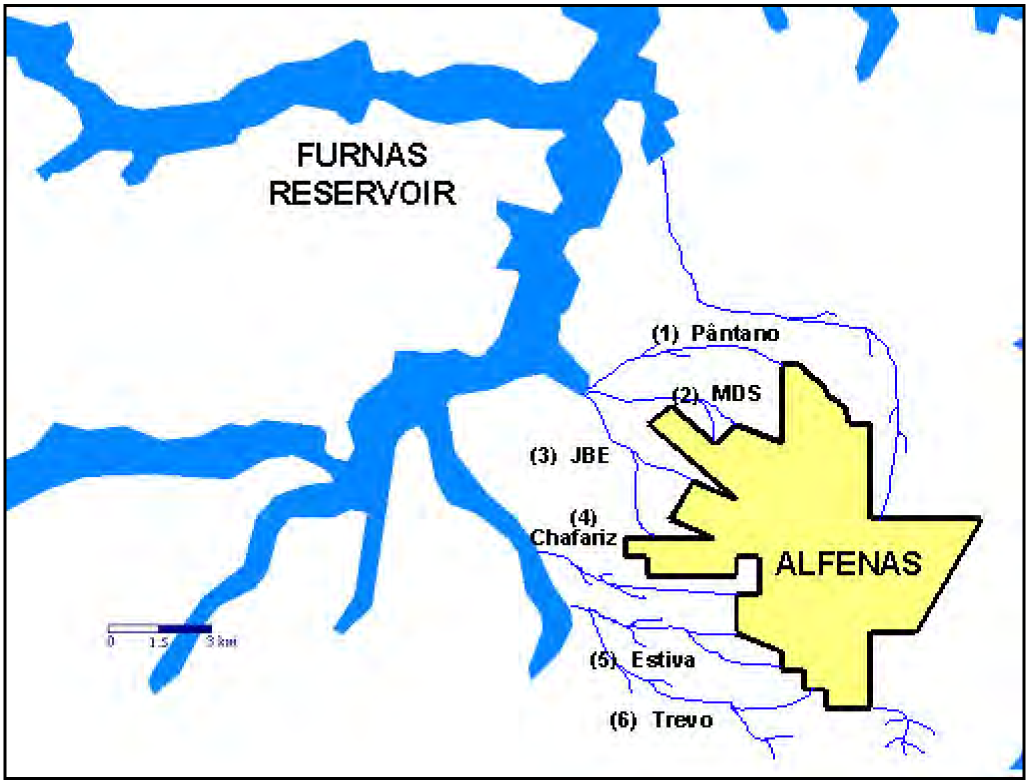

Figure 5-8: CEPT use at 6 waste streams

CEPT becomes somewhat more effective in reducing bacterial and nutrient concentrations if used at more than three waste streams. It can also be more effective in raising DO concentrations in the reservoir. The concentration predictions for expanding CEPT use to a total of 6 wastewater streams are listed below. The streams include Trevo, Estiva, and Chafariz, as well as JBE, MDS, and Pântano, all of which are shown in Figure 5-8. In obtaining the average concentrations in the reservoir, the reductions discussed previously for BOD, phosphorus, and fecal coliform are applied to all six streams, instead of three.

The same assumptions and calculations for 3 streams are used for predicting the concentrations for 6 streams, but an additional 32% (64% total) of the wastewater from the city is assumed to be treated with CEPT. The estimates for C_A, C_B and C_{in} are in Table 5-25 through Table 5-28.

Table 5-25: BOD Concentrations for C_{in} Considering CEPT at 6 Streams

BOD (mg/L)			
Year	**C_A**	**C_B**	**C_{in}**
High Volume (1996)	0.9[a]	3.1	1.1
Low Volume (2001)	1.6[a]	3.4	1.8
January 2002	1.8[a]	4.4	2.1

[a] Source: University of Alfenas, Projecto Furnas I, II, and III

Table 5-26: Coliform Concentrations for C_{in} Considering CEPT at 6 Streams

Fecal Coliform (MPN/100 ml)			
Year	**C_A**	**C_B**	**C_{in}**
High Volume (1996)	356[a]	115	332
Low Volume (2001)	510[a]	26	462
January 2002	81[a]	28	76

[a] Source: University of Alfenas, Projecto Furnas I, II, and III

Table 5-27: Phosphorus Concentrations for C_{in} Considering CEPT at 6 Streams

Phosphorus (mg/L)			
Year	**C_A**	**C_B**	**C_{in}**
High Volume (1996)	0	0.011	0.001
Low Volume (2001)	0	0.012	0.0012
January 2002	0	0.012	0.0012

Table 5-28: DO Concentrations for C_{in} Considering CEPT at 6 Streams

DO (mg/L)			
Year	**C_A**	**C_B**	**C_{in}**
High Volume (1996)	7.1 [a]	7.4	7.1
Low Volume (2001)	5.7 [a]	7.2	5.8
January 2002	4.6 [a]	6.6	4.7

Source: University of Alfenas, Projecto Furnas **I**, **II**, and **III**

The results for BOD, fecal coliform, DO, and phosphorus are shown in Table 5-29 through Table 5-32.

Table 5-29: Average Reservoir BOD Concentrations, CEPT at 6 Streams

BOD (mg/L)			
Year	**Without CEPT**	**With CEPT**	**Percent Reduction**
High Volume (1996)	0.60	0.43	28%
Low Volume (2001)	2.13	1.65	23%
January 2002	1.83	1.40	24%

Table 5-30: Average Reservoir Fecal Coliform Concentrations, CEPT at 6 Streams

Fecal Coliform (MPN/100 ml)			
Year	Without CEPT	With CEPT	Percent Reduction
High Volume (1996)	159	2	98.5%
Low Volume (2001)	1762	31	98.3%
January 2002	568	2	99.7%

Table 5-31: Average Reservoir Phosphorus Concentrations, CEPT at 6 Streams

Phosphorus (mg/L)			
Year	**Without CEPT**	**With CEPT**	**Percent Reduction**
High Volume (1996)	0.021	0.0021	90%
Low Volume (2001)	0.024	0.0024	90%
January 2002	0.024	0.0024	90%

Table 5-32: Average Reservoir DO Concentrations, CEPT at 6 Streams

DO (mg/L)			
Year	**Without CEPT**	**With CEPT**	**Percent Reduction**
High Volume (1996)	7.41	7.50	1.2%
Low Volume (2001)	6.77	6.77	0.6%
January 2002	5.88	5.99	1.8%

5.5 Discussion of Results - Lake Study

The results show that CEPT use at Alfenas results in reductions in BOD, fecal coliform, and phosphorus. It can also potentially increase DO concentrations in the reservoir. Reductions in nitrate and ammonia in the reservoir are not considered in this study, although these concentrations were measured in the water body. Fecal coliform concentration reductions will be achieved if CEPT is followed by chemical disinfection. Chemical disinfection would also kill other bacteria and pathogens as well.

BOD levels can be reduced by 17% at high water volume (1996) and 10% at low water volume (2001) in the reservoir for CEPT use at three streams. Fecal coliform concentrations can be reduced by 49% if CEPT is used at three streams for high and low water volume. Phosphorus concentrations can be reduced by 45% at high and low water volume also for CEPT use at three waste streams. Dissolved oxygen concentrations are predicted to increase from 0.3% at low volume to 0.6% at high volume 25% through CEPT use. Thus, it appears that significant reductions, especially for BOD and DO concentrations, can be attained at higher reservoir volumes. Reductions in all contaminant concentrations at higher water volume are due to the higher flows in the reservoir in 1996 than in 2001.

CEPT will to be more effective in reducing bacterial and nutrient concentrations if it is used at six waste streams. CEPT use can reduce BOD concentrations by 28% at high water volume and 23% at low water volume in the reservoir. It also increases DO concentrations by 1.2% at high water volume and 0.6% at low water volume. Reductions in phosphorus and fecal coliform concentrations are 90% and 98%, respectively, at low and high volumes. These reductions appear likely due to increased flows in the reservoir in 1996 than in 2001. As expected, CEPT is twice as effective at six waste streams than if it were used at only three waste streams.

5.6 Recommendations – CEPT Impact on Furnas Reservoir

The purpose of predicting the concentrations for CEPT use at three and six streams is to show the level of improvement expected in the Furnas Reservoir due to the use of CEPT. It is twice as effective at six streams as at three streams. Thus, by this reasoning, use of this technology at one stream would result in reductions that are a sixth of the reductions at sixth streams. However, the decision to use CEPT and the number of plants to be installed will ultimately depend on the decision-making authorities in Alfenas and on available funds for financing such a project.

CEPT implementation at six wastewater streams (Trevo, Estiva, Chafariz, JBE, MDS, and Pântano) would result in higher reductions of fecal coliform, BOD, and phosphorus concentrations discharged to the reservoir. This would lead to better water quality in the reservoir. However, these results apply only to the well-mixed section of the reservoir studied for this project. Concentration variations downstream of the control volume or for other cities in the region are not considered.

CEPT is more effective and efficient in reducing BOD concentrations in the reservoir at higher water volumes in the water body, due to increased water flow. Since reductions in BOD concentrations can lead to increases in DO concentrations in the reservoir, the improvement or increase in the DO concentrations were predicted. CEPT can increase DO concentrations from 0.6%

to 1.2%. The greatest improvements in DO concentrations are likely to occur at higher water volumes. This improvement does not appear to be significant in the short-term, but may be in the long-term.

CEPT will improve the water quality in the reservoir. CEPT should be followed by chemical disinfection of the effluent to reduce fecal coliform concentrations. Based on the results of this study, CEPT is recommended for treating wastewater at Alfenas. CEPT technology is a viable option for reducing contaminant concentrations in reservoir for Alfenas, as well as for any city along the Furnas Reservoir to sustain good water quality in the water body.

6. Conclusions

This report outlines a sustainable, financially feasible, and effective wastewater management strategy for the city of Alfenas, Brazil. CEPT is recommended is recommended as the initial wastewater treatment step, followed by chemical disinfection. CEPT has been chosen based on its ability to effectively reduce BOD, phosphorus and fecal coliform concentrations, while minimizing capital and operational costs. A treatment dosage of 30 mg/L of ferric chloride and 10 mg/L of Tanfloc is recommended as it will provide optimal removal efficiencies with locally available and affordable chemicals. Further disinfection is required to meet Brazilian regulations for effluent discharge to surface water bodies. Bacterial concentrations can be significantly reduced and human health can be protected by chemically disinfecting the CEPT effluent. Results from the reservoir analysis suggest that reservoir quality can be improved through the implementation of CEPT and chemical disinfection.

The goal of the proposed sludge treatment system is to convert the waste products produced by the CEPT plant into a valuable resource for the local community, in a financially and ecologically sustainable manner. By utilizing inexpensive and locally available technologies the sludge can be treated to compliance with U.S. EPA and Brazilian standards for land applied sludges, ensuring the health of the community and environment. The recommended treatment system includes lime addition for disinfection, thickening by gravity settling, and takes advantage of the warm climate by dewatering the sludge in sand drying beds. Land application is an ideal sludge disposal method for Alfenas because the city is surrounded by an abundance of agricultural land and, as a result, the nutrient rich sludge can be easily and cheaply transported to the crops. Furthermore, the city minimizes costs by eliminating the need for landfill space for the sludge and farmers can cut costs by supplementing chemical fertilizers with sludge. These financial benefits are particularly important for Brazilian communities and increase the feasibility of the project for a developing country.

Coffee production is a primary source of income for Alfenas and the Furnas Reservoir region and coffee crops are well suited for sludge application due to the plant characteristics and favorable harvesting schedule. A pilot study at the University of Alfenas experimental farm, testing sludge and chemical fertilizer application would provide valuable data on the effectiveness of sludge as a fertilizer and appropriate application rates to coffee and other crops.

The wastewater treatment and sludge disposal strategies recommended in this report are a vital part of a regional approach to the preservation of the Furnas Reservoir as a valuable resource, and provide a model solution for other cities in the region.

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Appendix A – CEPT process theory

Coagulation and Flocculation

Low-dose Chemically Enhanced Primary Treatment entails the use of additives in the treatment of wastewaters to aid the settling of solid particles suspended in water. This takes place by two physicochemical processes: coagulation and flocculation.

Coagulation is achieved by adding multi-valent cationic metals, preferably in the form of salts, such as $Al_2(SO_4)_3$ and $FeCl_3$, or low molecular weight cationic polymers. The purpose is forming denser, more compact, solid masses gathered by electrostatic forces. In the case of metallic salts, typical concentrations range from 5 to 40 mg per liter (ppm) of water to be treated (Ødegaard, 1998), while cationic polymers are usually dosed in ranges from 0.1 to 5 ppm. Energetic mixing is needed for the cationic additive to bind to the suspended solids in the wastewater. Therefore, the cationic coagulant is usually added as far upstream in the process as possible or dosed in a contact chamber equipped with mechanical mixers.

Flocculation takes place after adding high molecular weight anionic polymers, which, again by electrostatic forces, group the coagulated particles into larger structures. Flocs, being much larger particles, settle faster by gravity than suspended solids alone, as governed by Stokes' Law. This law states that particles will settle through any given fluid by gravity forces with a speed that is directly proportional to the square of their size. Slow mixing is typically used to assist in the flocculation process.

The exact combination of salts and polymers is different for each stream of wastewater, requiring detailed field-testing to determine the appropriate dosage in each case. Figure A-1 schematically shows the processes of coagulation and flocculation.

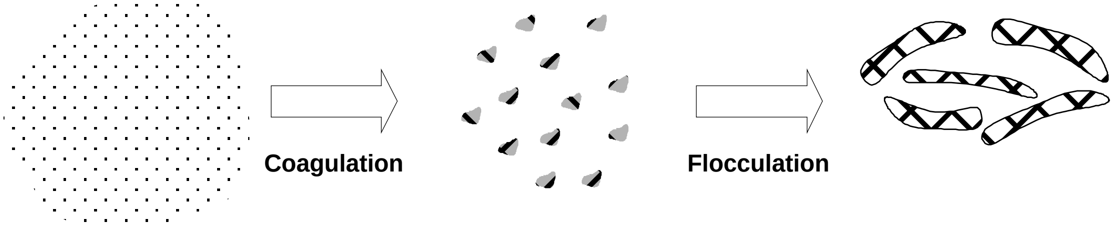


Figure A-1: Graphical Depiction of the Coagulation and Flocculation Processes

Process efficiency

Contrasting with secondary treatment, CEPT yields comparable Total suspended solids (TSS) removal rates. Biochemical Oxygen Demand (BOD) removal is lower, but efficient in terms of cost. Phosphorus (P) removal rates are remarkably higher when using $FeCl_3$, due to its precipitation as $Fe_2(PO_4)_3$. All of this is achieved while generating low volumes of sludge. These results for CEPT are shown in Table A-1, which compares removal efficiencies and sludge production for primary treatment, secondary treatment and CEPT.

Table A-1: Comparison of Removal Rates and Sludge Production

Treatment Type	TSS Removed	BOD Removed	P Removed	Sludge from TSS	Sludge from Chemicals or Biomass	Total Sludge
Primary	60 %	35 %	20 %	X	0	X
Chemically Enhanced Primary ($FeCl_3$ + anionic polymer)	80 %	57 %	85 %	1.33·X	0.12·X	1.45·X
Primary + Biological Secondary	85 %	85 %	30 %	1.42·X	0.48·X	1.90·X

Source: CEPT results from San Diego, CA – Pt. Loma plant operational data (Langworthy, 1990), Secondary treatment results from Black & Veatch, Inc., Boston, MA. January 1998. Residual Management Facilities Plan: Draft Characterization of Residuals, Suppl. Rep. No. 1. Prepared for MWRA.

From the table, it is clear that CEPT offers optimal removal rates for TSS and P per unit of sludge produced where "X" is the standard raw sludge production for conventional primary treatment. Another important factor is that after CEPT treatment, water can be effectively disinfected to produce an effluent suitable for discharge into natural bodies of water.

Typical CEPT process flow

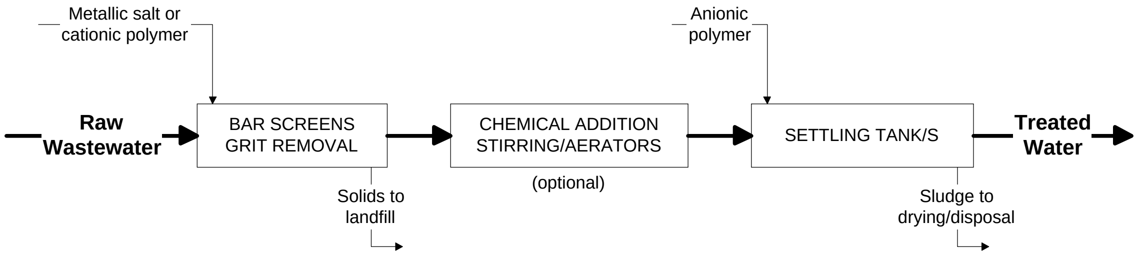


Figure A-2: CEPT Process Flow Diagram

Figure A-2 depicts typical unit operations and processes for CEPT. Larger particles are removed first by letting water flow through bar screens and a grit removal chamber. For chemical mixing, there are two options. The first is to inject the appropriate dosage of metallic salt (usually $FeCl_3$) or cationic polymer at the head of the plant, before the flow passes through the bar screens. The second option is to use a chemical mixing chamber, assisted with mechanical mixers or aerators. Water then flows over to the settling tank, where the anionic polymer, if necessary, will be injected, and as the flow progresses through the tank, flocs will settle out of the water column. Residence times are in the range of 5-10 minutes for chemical mixing and 1 hour for settling, depending on chemical dosage, flow rate and water constituents. Sludge is removed from the settling tanks, and the supernatant is ready for disinfection, secondary treatment or final disposal.

Advantages of CEPT

The foremost advantage of using CEPT instead of conventional primary treatment is that settling tanks required for the first are approximately half the size of those required for the second. Since surface overflow rates for CEPT can double those used for conventional primary treatment, for the same volumetric flow of wastewater, the required surface area for CEPT will be approximately half that of conventional primary treatment. This translates into significant capital cost savings.

Furthermore, a CEPT system can be more effectively operated and maintained than an activated sludge system because it allows for greater resilience, and reliability. CEPT systems remain functional and can maintain optimal removal efficiencies in the presence of a broad range of waste stream compositions and temperatures, avoiding biological upsets due to the formation of toxic materials, a characteristic issue with biological secondary treatment units. Chemical dosages can conveniently be altered to match changes in loading and composition, allowing for greater reliability and flexibility.

A CEPT plant can also be easily expanded to process larger flow volumes, if necessary, by increasing chemical dosing and adding additional tanks. Such upgrades in a CEPT plant have minimal negative impacts on system performance, as it was demonstrated in the Riviera de São Lourenço project (Harleman, et al., 1999). Moreover, conventional primary treatment plants can be retrofitted with CEPT technology, effectively doubling the plant’s previous capacity. CEPT tanks can also be easily added to any existing facility, as they tend to be small and easy to accommodate.

Appedix B – Plant design field testing procedures

Sampling method and location

Samples were taken from a sewer runoff at the Jardim de Boa Esperança stream (see Figure B-1). Since the sewer system is not yet completed, the sampling point was selected to be at the place where currently built sewers meet with the stream. This is also the point where the storm water causeway ends for this stream (see Figure B-2).

The location of the sampling point (see Figure B-3) was downstream enough to contain a representative composition of the wastewater that would reach the end of the stream, at the point where the proposed plant would be constructed. In addition, accessibility was considered, as the sampling point was located in public property and easily accessible from the road. Sampling took place usually during the morning, typical time of collection ranging from 8 to 11:30 am.

Two 20-liter plastic bottles were filled with wastewater at this source, and carried to the lab covered in black plastic paper bags, to avoid adverse biological and chemical reactions that might occur upon exposition to UV radiation.

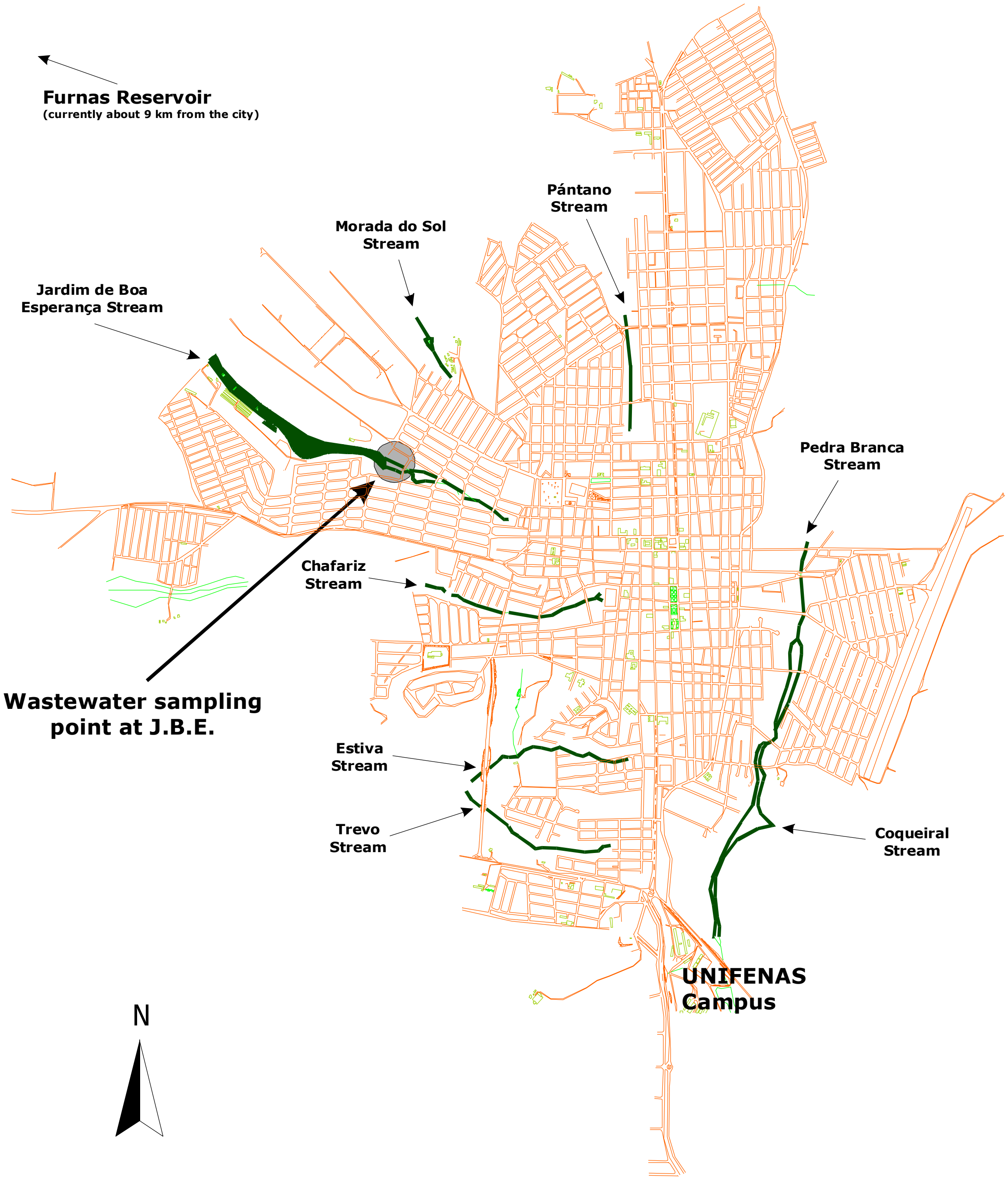


Figure B-1: Map of sampling area, sampling site enclosed in circle

(Source: Alfenas City Hall, Office of Cartography)

Figure B-2: Storm water channel, image taken at the source of the Jardim de Boa Esperança stream

Figure B-3: Sampling point at the Jardim de Boa Esperança stream, wastewater was collected from underneath the bamboo branches

Jar testing procedures

Jar testing was conducted using a Kemwater Flocculator 2000 kit (see Figure B-4). The kit consists of six cylindrical 1 L jars with agitators that are controlled from a central computerized unit. Full programming capabilities allow the establishment of four treatment stages:

- High-speed mixing (60 seconds)
- Low-speed mixing (5 minutes)
- Settling with no mixing (varied according to desired overflow rate)
- Secondary high-speed mixing (not used)

For the purposes of CEPT jar testing, the rapid mixing stage was set at 100 RPM for 60 seconds and slow mixing was set at 40 RPM for 5 minutes. Settling time varied from 1,5 to 10 minutes, according to the overflow rate desired. The secondary rapid mixing was not used.

For jar tests using only one chemical as coagulant, injection occurred after 30 seconds of high-speed mixing. For combined coagulant plus flocculant tests, the coagulant was injected at 30 seconds of rapid mixing and the flocculant at 60 seconds, when the mixing changed from rapid to slow.

Figure B-4: Jar-testing equipment used on the field study

The basis for relating batch jar-testing results to a continuous flow treatment system is that the overflow rate for both processes is the same. The efficiency of the coagulation and flocculation processes are proportional to the time the chemicals are in contact with the water, so it is possible to extrapolate data from jar tests and apply it to plant design. For a continuous-flow settling tank, the residence time can be calculated as the ratio of its volume to the flow rate of water:

$$t_R = \frac{L \cdot W \cdot H}{Q}$$

Equation B-1: Residence time in a CEPT tank

Where t_R is the residence time, L is the length, W is the width, H is the water depth and Q is the volumetric flow rate. The surface overflow rate (SOR) is correlated with the percent removal of particulate material in a settling tank, and it can be expressed as:

$$\mathrm{SOR} = \frac{Q}{L \cdot W} = \frac{H}{t_R}$$

Equation B-2: Surface overflow rate for a CEPT tank

From the jar tests, we define a value for settling depth and time within the jar, h and t_J respectively, from which we can express:

$$\mathrm{SOR} = \frac{h}{t_J}$$

Equation B-3: Surface overflow rate for jar test

All samples were taken from an outlet located 6 cm below the surface of the water, so h = 6 cm. Residence time in the jar, t_J, was varied to obtain different SOR. For instance, for a t_J = 1.5 min, the corresponding SOR would be:

$$\mathrm{SOR}_{1.5\,\mathrm{min}} = \frac{6\mathrm{cm} \cdot {}^{0{,}01\mathrm{m}}\!/_{\mathrm{cm}}}{1.5\,\mathrm{min} \cdot {}^{\mathrm{day}}\!/_{24 \cdot 60\,\mathrm{min}}} = 57.6\,\frac{\mathrm{m}}{\mathrm{day}} \approx 60\,\frac{\mathrm{m}}{\mathrm{day}}$$

Equation B-4: Surface overflow rate for jar test at t_J = 1.5 min

During the test, observations were recorded as to the floc size, change in color or turbidity of water and speed of settling. These observations were used as support data together with lab analysis results.

Samples of supernatant treated water were collected in clean, clear plastic bottles, properly labeled so they could be unequivocally identified. Bottles were immediately stored in a Styrofoam cooler, to avoid temperature and sunlight exposure from promoting adverse reactions in the water.

The chemicals used for jar testing included alum (aluminum sulfate), ferric chloride, synthetic cationic, anionic and neutral polymers, and Tanfloc, a locally available organic cationic polymer made from Acacia Mearnsii bark extracts. Tanfloc is a product that has been extensively used for water treatment, with very satisfactory results (http://www.tanac.com.br/ingles/index.html).

Lab analysis procedures

The following section describes the chemical analysis procedures used during the field study in Alfenas.

Total suspended solids

Total suspended solids were measured according to the procedures indicated in Standard Methods # 2540D. The vacuum apparatus used was composed of a membrane filter funnel and a suction flask connected to an electric air pump. Glass fiber filters, 5 cm in diameter with a pore size of 1 μm, were used. An electric oven was used to dry the samples. During the first week, between Jan 9 and 11, the oven used for this purpose was malfunctioning, and maintained temperatures varying from 60 to 110 ºC. At the beginning of the second week, the oven was replaced for another that was kept constantly at 105 ºC, according to the procedure. For storage and transportation, samples were placed in aluminum weighing dishes and kept in a dessicator.

Glass fiber filters were cleaned before use by filtering three 20 mL portions of distilled and deionized water through them. They were then placed in aluminum weighing dishes and put to dry in the oven for 60 minutes. After cooling to room temperature in a dessicator, the ready-to-use, also referred to as “blank,” filters were weighed. The weight of each filter plus the weighing dish was recorded.

To carry out the measurement, a blank filter was placed in the apparatus and one 20 mL volume of distilled and deionized water was run through. Then, a well-mixed volume of sample water, ranging from 10 to 40 mL, was extracted using a pipette and let flow through the filter. Two 20 mL volumes of distilled and deionized water followed to ensure all particles were properly washed from the flask’s walls. The filters were then placed back into their aluminum weighing dishes and in the oven for drying. After 60 minutes of drying in the oven, samples were put in the dessicator to cool down and were then weighed. Again, weight of both the filter and the weighing dish were recorded.

To calculate the total suspended solids in a sample, the following formula was used:

$$\text{mg total suspended solids / L} = \frac{(\text{sample weight - blank weight}) \cdot 1000}{\text{sample volume, mL}}$$

Equation B-5: Calculation of total suspended solids

Chemical oxygen demand

Chemical oxygen demand (COD) was measured using the dichromate Hach Method number 8000, which is approved by the U.S. Environmental Protection agency. A Hach model DR/4000 spectrophotometer was used to read the samples. Standard Hach COD digestion vials for the 0-1500 mg/L range were used (Cat. No. 21279-15).

Samples were well mixed and a 2 mL portion was taken using an automatic pipette and injected into the COD vial. Samples were then placed in the pre-heated COD reactor and were left to digest for 120 minutes. After cooling, the COD content was measured using the spectrophotometer.

BOD-COD correlation

COD was chosen over the lengthy biochemical oxygen demand (BOD) analysis because of time constraints. BOD analyses require three or five days of digestion while COD analyses require only two hours. However, regulations are always referred to BOD levels and a proper correlation needs to be established between the two.

To obtain this relationship, the values of COD and BOD from wastewater samples from Alfenas were used. These samples were taken as part of the Furnas II project, led by Professor Eduardo Tanure of UNIFENAS (Alfenas University) from four key points around the city where wastewater streams are mixed with fresh water natural springs. Seventy samples, taken between 1996 and 1999, were used to obtain the correlation.

The following graph (Figure B-5) shows a scatter plot for the data and the regression line traced over them. Correlation was very high, with an $R^2 = .96$, confirming the relationship and providing a link between the two parameters.

For samples that had a COD value of less than 200 mg/L, the regression line shows a lower slope, but still within the expected BOD/COD ratio of 0.4 to 0.8 (Metcalf & Eddy, 1991). Therefore, the correlation is proper for values of COD ranging from 250 to 1100 mg/L, which are typical for the raw wastewater found in Alfenas.

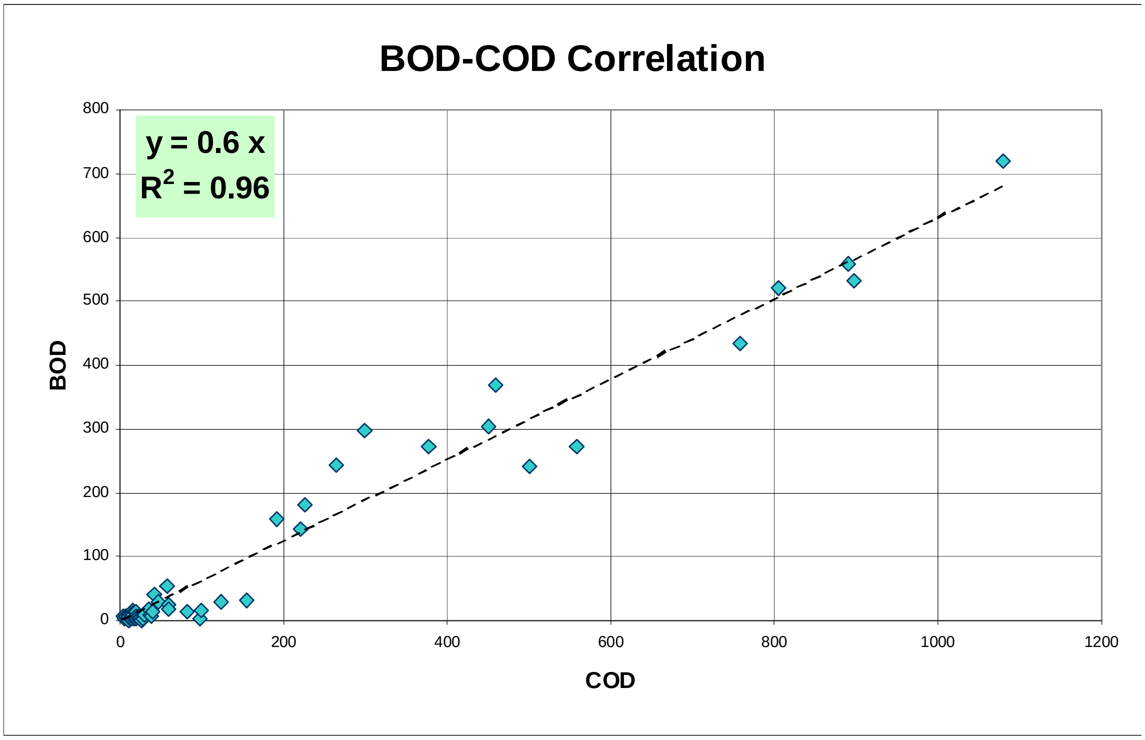


Figure B-5: COD-BOD correlation scatter plot

From the regression curve, it is found that BOD could be calculated from COD data by applying a factor of 0.6 to the COD value. To confirm this relationship, two raw wastewater samples were analyzed for both COD and BOD, using Dr. Tanure’s methods. These values, shown in the table below, confirm the relationship within reasonable analysis error.

Table B-1: BOD and COD results for two wastewater samples

Sample	BOD_3 (mg/L)	COD (mg/L)	BOD/COD
1	164	282	0.57
2	175	257	0.68

It will be assumed that removal rates for COD and BOD will also have a linear relationship, thus treatment efficiencies for COD removal discussed in section 2.2, will also apply to BOD removal.

Turbidity

Turbidity for water samples was measured using a Hach 2100 series turbidimeter. Standard Hach 20 mL vials were filled with the sample and measured using the NTU scale.

Total and fecal coliforms

To measure total and fecal coliforms, the multiple-tube method 9221 of the Standard Methods was used. Digestion mediums were inoculated with a drop of sample, with dilution ranging from 10^{-3} to 10^{-7} and left to digest in an oven set at 35 ºC for 48 hours. Tubes showing positive reaction, evidenced by bubbling, were re-inoculated in fecal coliform mediums and heated in water bath at 40 ºC for 24 hours, after which a second reading was taken.

Phosphorus

To measure phosphorus levels, the Hach disc colorimeter method for orthophosphate was used in the 0-50 mg/L range. 10 mL of sample were mixed with one reaction packet (Cat. No. 25080-50) and left to react for 5 minutes, then the coloring was compared with the standardized disc to obtain the reading.

All raw wastewater samples showed ortho-phosphorus content of 10 mg/L or less. Upon treatment, the supernatant showed values below detectable levels, i.e. less than 2 mg/L, in the cases where $FeCl_3$ was used. For other chemicals, treated water contained less than 4 mg/L. Most jar testing samples were not tested for phosphorus, see Appedix C for details on the ones that were tested.

Appendix C – Field data: jar testing results

Below are presented the results of all jar testing and laboratory analysis data obtained during the field study. In all cases, the raw wastewater used for the jar test is typified by the date and time it was collected, together with its Turbidity (NTU), Total suspended solids (mg/L), Chemical Oxygen Demand (mg/L) phosphorus (mg/L) and pH. Then, each jar test is typified by the type and dosing of coagulant and flocculant that were used and the overflow rates sampled. Each sample of treated water is typified by the values obtained for Turbidity, TSS, COD and phosphorus.

Date/Time:	Raw wastewater characteristics				
Jan 9, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**230**	**n/a**	**387**	**n/a**	**10**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
Alum	15			60	175	50	286	
Alum	20			60	115	180	262	
Alum	25			60	108			
Alum	30			60	106	100	296	
Alum	40			60	99.7			
No chemicals				60	148	50	256	
Alum	15			8.64	74.8			
Alum	20			8.64	121			
Alum	25			8.64	68.6			
Alum	30			8.64	50.8			
Alum	40			8.64	52.7			
No chemicals				8.64	105			

Date/Time:	Raw wastewater characteristics				
Jan 9, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
11:30 am	**173**	**n/a**	**659**	**n/a**	**10**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
$FeCl_3$	10			60	140			
$FeCl_3$	15			60	120	120	439	
$FeCl_3$	20			60	77.5	120	316	
$FeCl_3$	25			60	86			
$FeCl_3$	30			60	47	50	308	
No chemicals				60	160	170	515	
$FeCl_3$	10			8.64	77.5			
$FeCl_3$	15			8.64	54.1			
$FeCl_3$	20			8.64	57.7			
$FeCl_3$	25			8.64	50.2			
$FeCl_3$	30			8.64	34.1			
No chemicals				8.64	88.9			

Date/Time:	Raw wastewater characteristics				
Jan 9, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
11:30 am	**173**	**n/a**	**659**	**n/a**	**10**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Tanfloc	10			60	84.5	130	365	
Tanfloc	20			60	44.2	120	320	
Tanfloc	30			60	54.2	160	344	
Tanfloc	50			60	16.2			
Tanfloc	70			60	13.7			
No chemicals				60	119	100	553	
Tanfloc	10			8.64	44			
Tanfloc	20			8.64	35.9			
Tanfloc	30			8.64	22.8			
Tanfloc	50			8.64	8.46			
Tanfloc	70			8.64	6			
No chemicals				8.64	55.9			

Date/Time:	Raw wastewater characteristics				
Jan 10, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**279**	**320**	**n/a**	**n/a**	**9**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Tanfloc	10			60	165	120		6
Tanfloc	15			60	151	220		5
Tanfloc	20			60	144	220		4
Tanfloc	25			60	128	190		2.5
Tanfloc	30			60	117	160		1.5
No chemicals				60	182	220		5
Tanfloc	10			30	105	120		4
Tanfloc	15			30	98.7	50		1.5
Tanfloc	20			30	80.2	40		1.5
Tanfloc	25			30	82	100		2
Tanfloc	30			30	54.3	10		1.5
No chemicals				30	121	170		4

Date/Time:	Raw wastewater characteristics				
Jan 10, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**279**	**320**	**n/a**	**n/a**	**9**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Neutral polymer	0.1			60	174	200		5
Neutral polymer	0.5			60	125	170		2
Neutral polymer	1			60	116	110		2
Neutral polymer	2			60	74.6	70		1.5
Neutral polymer	5			60	75	67		1.5
No chemicals				60	164	140		4
Neutral polymer	0.1			30	75.1	730		2
Neutral polymer	0.5			30	63.5	80		1.5
Neutral polymer	1			30	70	49		1.5
Neutral polymer	2			30	56.2	46		1.5
Neutral polymer	5			30	60.2	42		1.5
No chemicals				30	96.1	85		2.5

Date/Time:	Raw wastewater characteristics				
Jan 11, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**173**	**n/a**	**n/a**	**7.1**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Alum	25	Tanfloc	25	60	18.7			
Alum	25	Anionic #20	0.5	60	97.5			
Alum	25	Anionic #5	0.5	60	28.1			
Alum	25	Cationic #14	0.5	60	48.6			
Alum	25	Cationic #36	0.5	60	35.3			
No chemicals				60	125			
Alum	25	Tanfloc	25	30	9			
Alum	25	Anionic #20	0.5	30	32.5			
Alum	25	Anionic #5	0.5	30	25			
Alum	25	Cationic #14	0.5	30	18.1			
Alum	25	Cationic #36	0.5	30	17.8			
No chemicals				30	48.2			

Date/Time:	Raw wastewater characteristics				
Jan 11, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**173**	**n/a**	**n/a**	**7.1**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
$FeCl_3$	30	Anionic #20	0.5	60	8.32			
$FeCl_3$	30	Anionic #5	0.5	60	4.51			
$FeCl_3$	30	Cationic #36	0.5	60	6.67			
Alum	25	Anionic #20	0.5	60	80			
Alum	25	Anionic #5	0.5	60	8.95			
Alum	25	Cationic #36	0.5	60	35.3			
$FeCl_3$	30	Anionic #20	0.5	30	4.37			
$FeCl_3$	30	Anionic #5	0.5	30	4.42			
$FeCl_3$	30	Cationic #36	0.5	30	3.53			
Alum	25	Anionic #20	0.5	30	14			
Alum	25	Anionic #5	0.5	30	8.33			
Alum	25	Cationic #36	0.5	30	34.1			

Date/Time:	Raw wastewater characteristics				
Jan 11, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**173**	**n/a**	**n/a**	**7.1**	**n/a**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
$FeCl_3$	12	Anionic #20	0.5	60	39.9	54		
$FeCl_3$	12	Anionic #5	0.5	60	17.6	103		
$FeCl_3$	12	Cationic #36	0.5	60	29	5		
Alum	15	Anionic #20	0.5	60	108	97		
Alum	15	Anionic #5	0.5	60	72.5	25		
Alum	15	Cationic #36	0.5	60	84	68		
$FeCl_3$	12	Anionic #20	0.5	30	16.7	73		
$FeCl_3$	12	Anionic #5	0.5	30	14.4	24		
$FeCl_3$	12	Cationic #36	0.5	30	13.3	17		
Alum	15	Anionic #20	0.5	30	78.3	69		
Alum	15	Anionic #5	0.5	30	55.7	34		
Alum	15	Cationic #36	0.5	30	50.4	23		

Date/Time:	Raw wastewater characteristics				
Jan 14, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**162**	**142**	**409**	**6.9**	**10**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
Alum	20			60	152		413	
$FeCl_3$	15			60	60.9		288	
$FeCl_3$	10			60	76.4		303	
Tanfloc	15			60	98.9		326	
Tanfloc	10			60	111		331	
No chemicals				60	120		368	
Alum	20			30	55.9		299	
$FeCl_3$	15			30	52.7		273	
$FeCl_3$	10			30	60.5		270	
Tanfloc	15			30	75.3		289	
Tanfloc	10			30	77.9		298	
No chemicals				30	106		290	

Date/Time:	Raw wastewater characteristics				
Jan 14, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**151**	**147**	**396**	**7.1**	**5.5**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Alum	20			30	91.5	115	340	
Alum	15	Tanfloc	5	30	34.6	0	257	
$FeCl_3$	15			30	25.4	35	242	
Tanfloc	15			30	44.4	25	270	
Tanfloc	20			30	29.8	80	256	
No chemicals				30	60.6	25	275	

Date/Time:	Raw wastewater characteristics				
Jan 14, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**156**	**172**	**362**	**7.2**	**6**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Alum	20			30	50.8	35	268	
$FeCl_3$	20			30	18.4	15	222	
$FeCl_3$	30			30	16.1	-5	230	
Tanfloc	20			30	21.1	10	229	
Tanfloc	30			30	9.02	15	219	
No chemicals				30	49.9	30	255	

Date/Time:	Raw wastewater characteristics				
Jan 15, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**168**	**227**	**575**	**6.9**	**4.5**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
Alum	30	Tanfloc	10	30	84.6	0	329	
Alum	25	Tanfloc	10	30	87	118	329	
$FeCl_3$	30	Tanfloc	10	30	36.3	0	246	
$FeCl_3$	25	Tanfloc	10	30	39.2	57	239	
Tanfloc	35			30	27.4	48	233	
No chemicals				30	99.1	109	346	

Date/Time:	Raw wastewater characteristics				
Jan 15, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**255**	**263**	**606**	**7.0**	**4.5**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
FeCl3	20	Tanfloc	10	17.28	53.5	95	201	
FeCl3	25	Tanfloc	10	17.28	40.75	70	190	
FeCl3	25	Tanfloc	5	17.28	47.35	40	187	
Tanfloc	30			17.28	37.8	25	187	
Tanfloc	35			17.28	37	10	191	
No chemicals				17.28	101	109	232	

Date/Time:	Raw wastewater characteristics				
Jan 16, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**209**	**113**	**269**	**6.8**	**n/a**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
$FeCl_3$	25	Tanfloc	5	30	88.1	33	146	
$FeCl_3$	25	Tanfloc	5	17.28	84.6	17	139	
$FeCl_3$	30	Tanfloc	5	30	78.9	35	138	
$FeCl_3$	30	Tanfloc	5	17.28	80.3	23	133	
$FeCl_3$	30	Tanfloc	2	30	73.7	20	141	
$FeCl_3$	30	Tanfloc	2	17.28	72.9	25	141	
Tanfloc	20			30	86.4	38	145	
Tanfloc	20			17.28	89.6	35	141	
Tanfloc	30			30	85	17	139	
Tanfloc	30			17.28	85.2	27	124	
No chemicals				30	174	62	175	
No chemicals				17.28	154	48	147	

Date/Time:	Raw wastewater characteristics				
Jan 16, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**192**	**n/a**	**282**	**6.9**	**n/a**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
$FeCl_3$	25	Tanfloc	5	60	48.6	33	138	
$FeCl_3$	25	Tanfloc	5	30	29.3	10	125	
Tanfloc	30			60	31.8	22	131	
Tanfloc	30			30	23.4	3	137	
				60	138	75	191	
				30	93.6	45	145	

Date/Time:	Raw wastewater characteristics				
Jan 17, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**169**	**176**	**448**	**6.7**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
$FeCl_3$	25	Tanfloc	5	30	72.5	45	191	
$FeCl_3$	30	Tanfloc	5	30	79.5	55	183	
$FeCl_3$	30	Tanfloc	10	30	54.8	38	159	
$FeCl_3$	40	Tanfloc	5	30	68.7	55	189	
$FeCl_3$	40	Tanfloc	10	30	55.7	48	190	
No chemicals				30	154	117	281	

Date/Time:	Raw wastewater characteristics				
Jan 17, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
10:00 am	**183**	**234**	**506**	**6.7**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
Alum	50	Tanfloc	10	30	71.6	43	219	
Tanfloc	40			30	40.7	30	143	
$FeCl_3$	30	Tanfloc	5	30	54.2	73	152	
$FeCl_3$	30	Tanfloc	10	30	56.9	17	158	
$FeCl_3$	30	Tanfloc	15	30	50.9	40	141	
No chemicals				30	122	85	217	

Date/Time:	Raw wastewater characteristics				
Jan 18, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**171**	**255**	**537**	**6.6**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
$FeCl_3$	30	Tanfloc	10	60	80.3	88	235	
$FeCl_3$	30	Tanfloc	10	30	92.7	77	224	
Tanfloc	40			60	81.4	40	226	
Tanfloc	40			30	67.1	48	198	
$FeCl_3$	30	Tanfloc	5	60	93.3	86	222	
No chemicals				60			347	
$FeCl_3$	30	Tanfloc	10	60 (*)	82	105	224	
$FeCl_3$	30	Tanfloc	10	30 (*)	92.7		206	
Tanfloc	40			60 (*)	79.5	48	200	
Tanfloc	40			30 (*)	72.3		188	
$FeCl_3$	30	Tanfloc	5	60 (*)	96.9		225	
No chemicals				60 (*)	137	157	338	

(*) Disinfected with 0.01 ppm of NaClO

Date/Time:	Raw wastewater characteristics				
Jan 18, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
2:00 pm	**166**	**248**	**604**	**6.5**	**n/a**

Coagulant	Coag. Dose (ppm)	Flocculant	Floc. Dose (ppm)	SOR (m/day)	Turb. (NTU)	TSS (mg/L)	COD (mg/L)	Phos. (mg/L)
FeCl3	30	Tanfloc	10	60	71.3	88	174	
FeCl3	30	Tanfloc	10	43.2	67.3	69	149	
FeCl3	30	Tanfloc	10	34.56	62.3	63	146	
FeCl3	30	Tanfloc	10	30	74.2	73	158	
FeCl3	30	Tanfloc	10	24.68	67.1	53	181	
No chemicals				60	173	183	414	
FeCl3	30	Tanfloc	10	60 (*)	71.6	85	189	
FeCl3	30	Tanfloc	10	43.2 (*)	64.7	0	203	
FeCl3	30	Tanfloc	10	34.56 (*)	56.6	0	150	
FeCl3	30	Tanfloc	10	30 (*)	69.4	75	173	
FeCl3	30	Tanfloc	10	24.68 (*)	58	0	144	
No chemicals				60 (*)	163	213	389	

(*) Disinfected with 0.1 ppm of NaClO

Date/Time:	Raw wastewater characteristics				
Jan 21, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
9:00 am	**239**	**437**	**980**	**6.7**	**n/a**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
$FeCl_3$	30	Tanfloc	10	60	178	203	570	
$FeCl_3$	30	Tanfloc	10	43.2	128	163	436	
$FeCl_3$	30	Tanfloc	10	30	110	107	396	
Tanfloc	40			60	113	160	490	
Tanfloc	35			60	126	180	479	
No chemicals				60	255	415	806	
$FeCl_3$	30	Tanfloc	10	60 (*)	132	263	434	
$FeCl_3$	30	Tanfloc	10	43.2 (*)	107	150	456	
$FeCl_3$	30	Tanfloc	10	30 (*)	79.7	100	648	
Tanfloc	40			60 (*)	101	153	443	
Tanfloc	35			60 (*)	117	170	407	
No chemicals				60 (*)	253	420	679	

(*) Disinfected with 10 ppm of NaClO

Date/Time:	Raw wastewater characteristics				
Jan 22, 2002	Turbidity (NTU)	TSS (mg/L)	COD (mg/L)	pH	Phosphorus (mg/L)
8:00 am	**n/a**	**117**	**339**	**7.0**	**n/a**

Coagulant	**Coag. Dose (ppm)**	**Flocculant**	**Floc. Dose (ppm)**	**SOR (m/day)**	**Turb. (NTU)**	**TSS (mg/L)**	**COD (mg/L)**	**Phos. (mg/L)**
$FeCl_3$	30	Tanfloc	10	30			169	
$FeCl_3$	30	Tanfloc	10	17.28			183	
$FeCl_3$	30	Tanfloc	10	30			155	
$FeCl_3$	30	Tanfloc	10	15.7			142	
Tanfloc	35			30			168	
Tanfloc	35			17.28			154	
$FeCl_3$	30	Tanfloc	10	30 (*)		44	198	
$FeCl_3$	30	Tanfloc	10	17.28 (*)		46	183	
$FeCl_3$	30	Tanfloc	10	30 (*)		45	127	
$FeCl_3$	30	Tanfloc	10	15.7 (*)		33	123	
Tanfloc	35			30 (*)		16	165	
Tanfloc	35			17.28 (*)		19	143	

(*) Disinfected with 10 ppm of NaClO

Appendix D – Nutrient Testing Techniques

Ammonia Nitrogen Procedure

Courtesy of Professor Eduardo Luis Tanure, University of Alfenas, Brazil

1. Use 500ml of sample
2. Pour into 500ml beaker
3. Add 25ml of buffer solution
4. Add 6N sodium hydroxide to a pH of 9.5
5. Pour all of the sample into a flask
6. Add 50ml of boric acid at a concentration of 20g/l to an flask that will collect the distillate
7. Distill the sample until 200-220ml have been condensed. Adjust the volume to 250ml with distilled water
8. Remove 100ml and add 1.5ml of 6N sodium hydroxide and 2ml of Messler reagent.
9. Measure using program 2400 on the Hach spectrophotometer. Adjust to 425 nm and calibrate with a blank.

Total Phosphorus Procedure

Courtesy of Professor Eduardo Luis Tanure, University of Alfenas, Brazil

Solutions:

1. Phenolphaline indicator solution

2. Suluric solution, 30%: slowly add 300ml of concentrated H_2SO_4 to 600ml of distilled water, complete the volume to 1000ml.

3. Potassium persulfate solution (prepare within an hour of use): 5g $K_2S_2O_8$ in distilled water and complete the volume to 100ml.

4. Sodium hydroxide solution, 1N: 40g NaOH in distilled water and complete the volume to 1000ml.

5. Combined mixture: dissolve .13g of $KsbOC_4H_4O_6 * ½ H_2O$ in 700ml of distilled water, add 5.6 g of $(NH_4)_6Mo_7O_{24} * 4H_2O$ and dissolve, add 70ml of concentrated H_2SO_4, cool and dilute to 1000ml in a volumetric flask.

6. Combined Reagent (1-week stability): add .5g of ascorbic acid to 100ml of the combined mixture. If the solution is muddy, let it sit for a few minutes and store in a refrigerator.

7. Phosphorus stock solution: dissolve 219.5 mg of KH2PO4 in distilled water and complete the volume to 1000ml in a volumetric flask. 1ml = 50ug PO_4^{-3} as P.

8. Phosphorus standard solution: dilute 50ml of the phosphorus stock solution in 1000ml of distilled water in a flask. 1ml = 2.5ug PO_4^{-3} as P.

Procedure:

1. Collect 100ml of sample in a 250ml flask

2. Add 1 drop of phenophaline solution (if the sample becomes colored, discolor it with 30% sulfuric acid, adding 1ml at a time)

3. Add 15ml of potassium persulfate (5g per 100ml – prepare before use)

4. Boil for 30 minutes, maintaining the a volume of 25-50ml with distilled water.

5. Cool and add 1 drop of phenophaline and add sodium hydroxide until the sample turns pink.

6. Transfer the mixture to a 100ml flask and complete the volume with distilled water.

7. Pipette 50ml of sample to a 125ml test tube.

8. Add 10ml of the combined reagent, shake well and let it sit for at least 10 minutes, but not more than 30 minutes.

9. Read transmittance at 880nm.

10. Prepare a 100ml blank with steps 1 through 9.

Construction of the standard curve:

Prepare standard solutions of varying phosphorus concentrations, amking dilutions of the standard solutions in volumetric flasks according to the table:

Concentration of PO_4 as P (mg/l)	**ml of Standard Solution**
0	0
0.005	20
0.1	40
0.2	80
0.35	140
0.5	200

Complete the volume of each solution to 1000ml with distilled water. Treat 100ml of each of the standard solutions according to steps 1 through 5.

Appendix D – Reservoir Field Studies

During the month of January 2002, samples were collected from the Furnas Reservoir and in two open-channel waste streams which flowed from Alfenas. The specific sampling points are indicated in Figure D-1. The specific sampling coordinates, which were obtained using a global positioning system (GPS) device, are listed in Table D-1. Water samples were collected on January 17th, January 18th, January 21st, and January 24th. The particular section of the reservoir was selected because the open-channel waste streams from Alfenas flowed directly into this section. This location appeared to be the best area where representative samples could be collected. Various sampling points were selected within the reservoir. Generally, samples were collected in the centerline of the reservoir section, which is usually the deepest point in the section.

Measurements of temperature and DO concentration were made on-site at each sampling point. Depth measurements were also made at each sampling point. A “grab” water sample was taken at each sampling point in either a heat-sterilized bottle or a sampling bag containing a sterilization capsule. Bacterial and nutrient analysis was performed in a laboratory. These analyses included COD, nitrate, ammonia, orthophosphate and total and fecal coliforms.

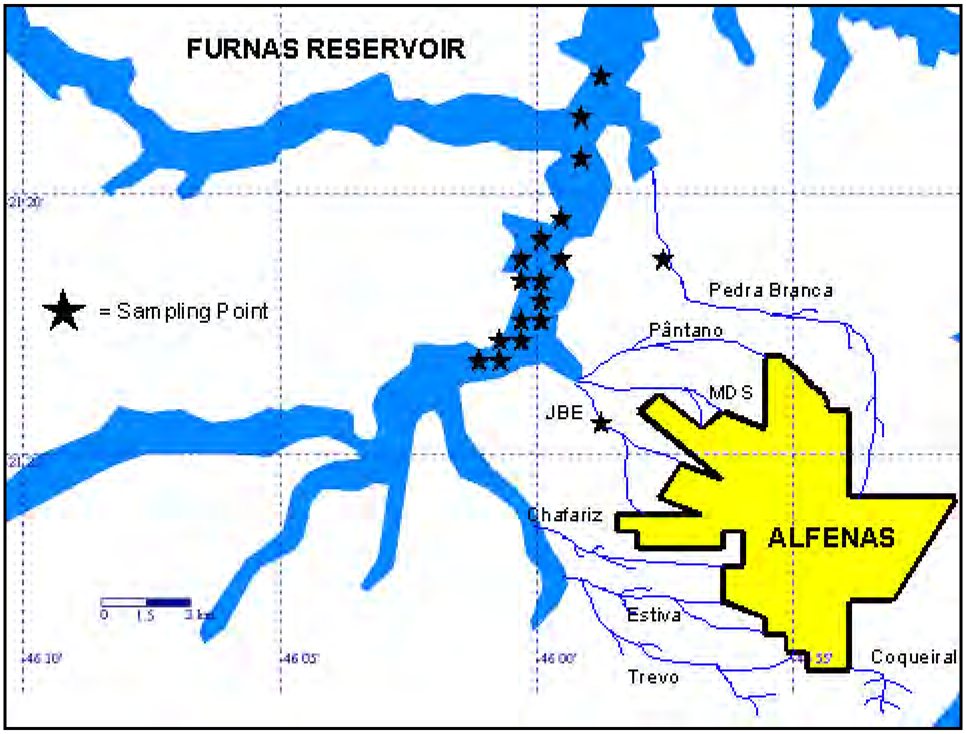


Figure D-1: Sampling Points in the Furnas Reservoir

On January 17th and January 18th, water samples were collected from the surface of the reservoir. On January 21st and January 24th, water samples were collected from the surface and at various depths below the surface. The samples taken at various depths below the reservoir surface were first collected using a Van Dorn horizontal water bottle and then transferred to a heat-sterilized bottle or sterile Whirl-Pak bag, as shown in Figure D-2. A Van Dorn water bottle was used to collect water samples from discrete depths. Usually 2 to 3 samples were collected in one water column, at various depths, and analyzed to see differences in water quality between surface and deep water. The samples collected in the sterilized bags had elevated pH and COD levels in the samples, which may have been due to a sodium thiosulfate preservative.

Table D-1: Reservoir Sampling Point Coordinates

Sample Point	January 17, 2002		January 18, 2002	
	Latitude, S	Longitude, W	Latitude, S	Longitude, W
Point 1	21°23.213'	46°00.937'	21°22.912'	46°00.937'
Point 2	21°22.819'	46°00.532'	21°21.816'	46°00.532'
Point 3	21°21.730'	46°00.184'	21°21.506'	46°00.184'
Point 4	21°20.786'	45°59.940'	21°18.593'	45°59.940'

Sample Point	January 21, 2002		January 24, 2002	
	Latitude, S	Longitude, W	Latitude, S	Longitude, W
Point 1	21°23.051'	46°00.163'	21°22.715'	46°00.409'
Point 2	21°21.716'	46°00.651'	21°22.021'	46°00.320'
Point 3	21°19.506'	45°59.508'	---------	---------
Point 4	21°18.115'	45°58.946'	---------	---------
Point 5	21°22.687'	46°00.103'	---------	---------
Point 6	21°23.589'	46°00.892'	---------	---------
Point 7	21°23.729'	46°01.133'	---------	---------

Simultaneous field measurements of DO and temperature were taken at different depths in the reservoir using a portable YSI model 57 DO meter, equipped with a YSI model 5329 DO probe which are shown in Figure D-3 and Figure D-4. The probe was attached to a 50-foot cable, which was very practical for taking depth profile measurements of DO and water temperature. The DO meter was calibrated by Winkler titration.

The probe was immersed directly into the reservoir at depths between 1 meter and 11 meters depending on the depth of the part of the reservoir being measured. Measurements were taken at 1- and 2-meter intervals. Depth profiles of temperature and DO were taken over the course of the 4-day period.

Figure D-2: Whirl-Pak Bag with Sodium Thiosulfate Preservative

Source: Nasco website

Figure D-3: Portable DO and Temperature meter

Source: YSI website

Figure D-4: Dissolved Oxygen Field Probe

Source: YSI website

Stream Sampling

Water samples collected from the open-channel wastewater streams required the use of water-proof boots and gloves, as well as the use of a safety mask for sample collection. The streams that were sampled include Pedra Branca and JBE, as shown in Figure D-1. The specific sampling coordinates, using a GPS device, were located at 21°21.531' South, 45°57.472' West and 21°23.924' South, 45°58.900' West, respectively.

The streams which contained various domestic wastes, consisted of mostly raw sewage. The water in the streams clearly had rapid velocities, and the water looked murky and emitted a sewage odor. Water samples were collected from two wastewater streams discharging from Alfenas to the Furnas Reservoir.

Samples were collected over a 3-day period, in which no samples were collected on January 24th due to time constraints. Some samples were collected during the day before 11:00 am and others after 2:00 pm.

Testing Methods

DO, Temperature, pH, COD, orthophosphate, ammonia and nitrate were measured. HACH test kits were used for the chemical analyses to measure the amount of nitrate, orthophosphate, ammonia, and COD.

DO and water temperature was measured in the field as noted in the section above. pH was measured in the laboratory using a pH meter provided by the University of Alfenas.

HACH color disc test kits were used for measuring nitrate, ammonia, and orthophosphate concentrations. Each kit provided two vials, one for a blank sample with no reaction and the other for mixing a reagent into the sample. Depending on whichever test kit was used, each vial for a kit required 5 to 20 ml of sample. When the applicable reagent was mixed into a sample, there may have been a color change. This color change was compared to the blank non-reacting sample, as a reference, using the color disk to as shown in Figure D-5. The value on the scale of the color disc, where the color change in the reacting vial matched the color on the color disc using the blank sample, was used. This value was multiplied by 4.4 to obtain mg nitrate per liter for nitrate concentrations and divided by 50 to obtain mg phosphate per liter for orthophosphate concentrations. The ammonia concentration was read directly from the scale on the color disc for the ammonia kit. Accuracy for color disc kits is typically ± 10% or ± the smallest increment, subject to individual color perception. (HACH website).

Figure D-5: HACH Color-Based Test Kits

Source: HACH website

Figure D-6: COD reagent test tubes

Source: HACH website

COD was also measured using a HACH COD test kit. The kit provided prepared COD reagents in test tubes as shown in Figure D-6. However, the reagents consisted of hazardous material, so the test tubes were always capped. A 2-ml volume of sample was added to a COD vial. Then the vial was capped and shaken to allow the sample to react with the reagent. After this mixing step, the vial was added to a COD reactor to allow the reagent to digest the sample contents at 150 °C for two hours.

Total and fecal coliforms, were measured using the standard method for Multiple-Tube Fermentation Analysis. In this process, digestion mediums were inoculated with a drop of sample, with dilution ranging from 10^{-3} to 10^{-7} and left to digest in an oven set at 34.5 ºC for 48 hours. Tubes showing positive reaction, evidenced by bubbling, were re-inoculated in fecal coliform mediums and heated in a water bath at 44.5 ºC for 24 hours, after which a second reading was taken.

Appendix E – Furnas Reservoir Depth Profiles

January 17, 2002 Field Measurements

Figure E-1: Temperature and DO Concentration Profiles within Reservoir

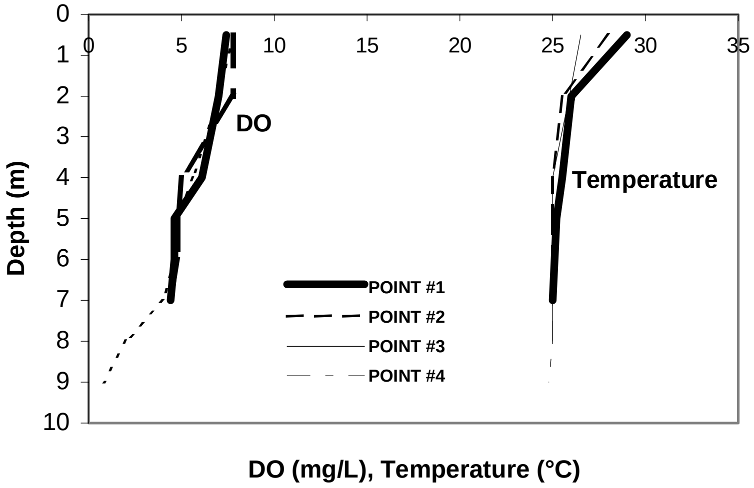
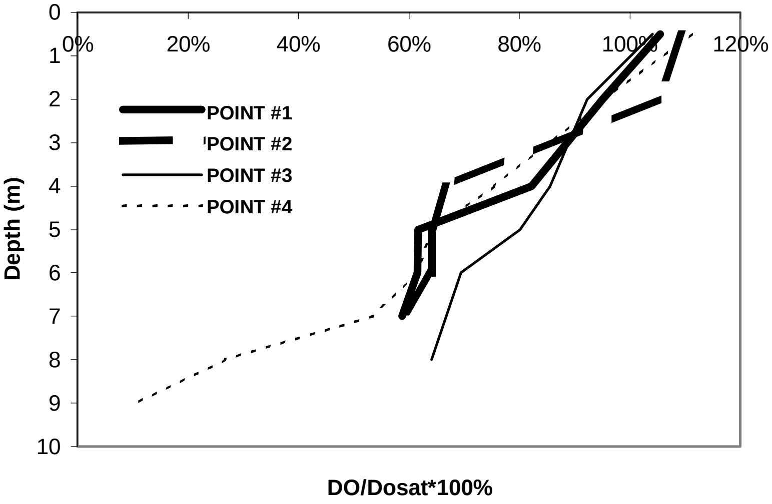


Figure E-2: Concentration Profiles for % Saturation of DO Concentrations



January 18, 2002 Field Measurements

Figure E-3: Temperature and DO Concentration Profiles

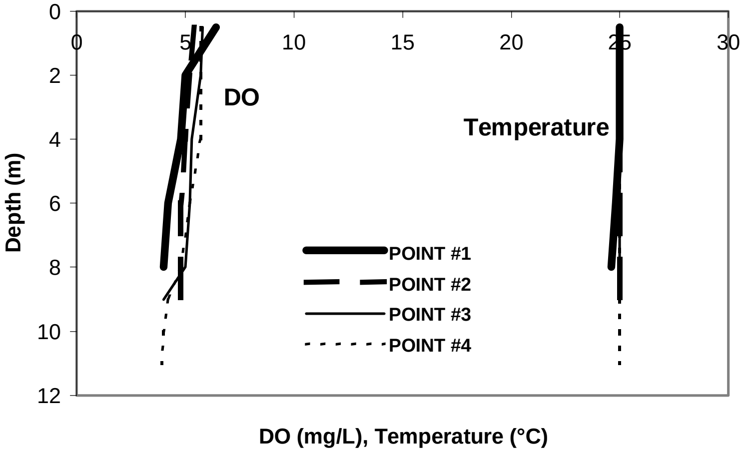
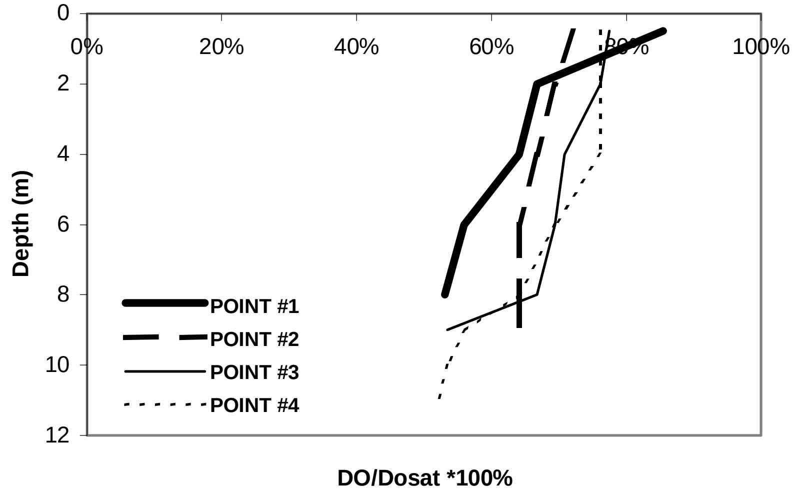


Figure E-4: Concentration Profiles for % Saturation of DO Concentrations



January 21, 2002 Field Measurements

Figure E-5: Temperature and DO Concentration Profiles

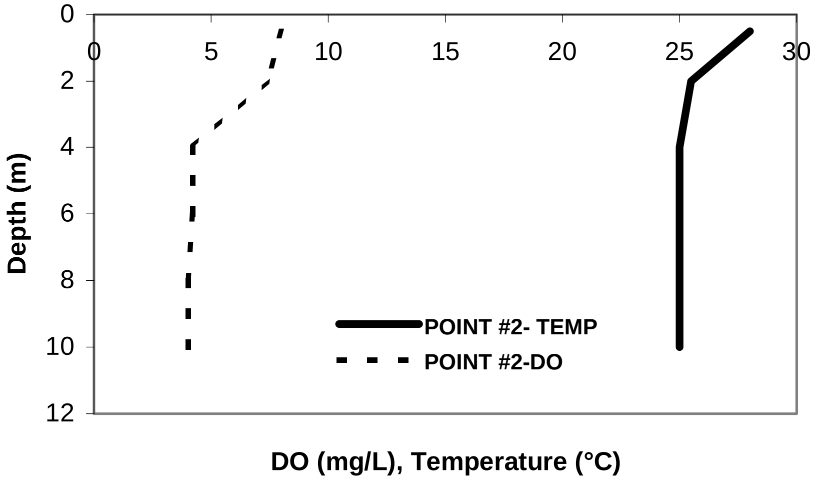
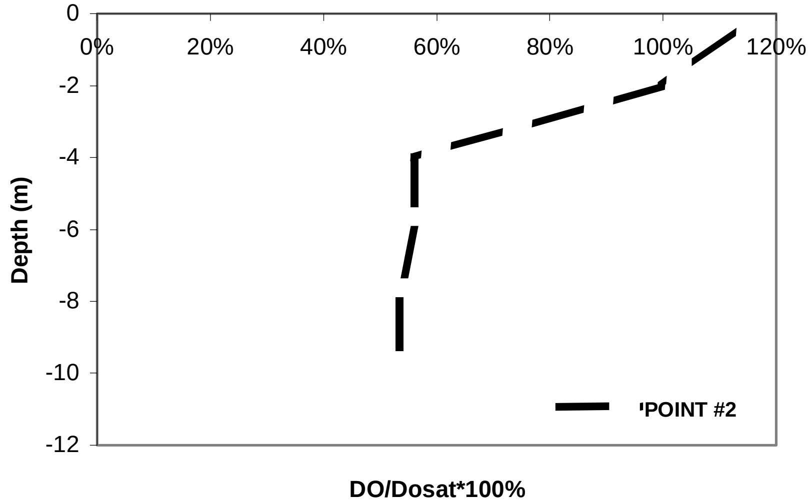


Figure E-6: Concentration Profiles for % Saturation of DO Concentrations



January 24, 2002 Field Measurements

Figure E-7: Temperature and DO Concentration Profiles

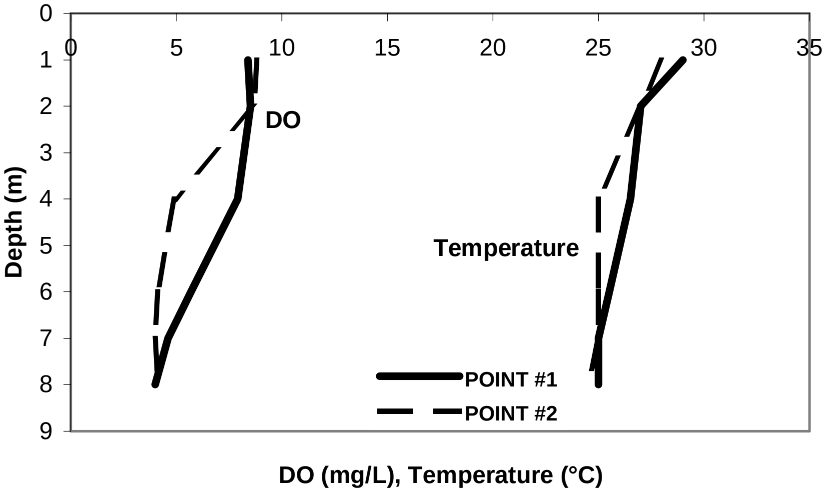


Figure E-8: pH Profiles versus Depth

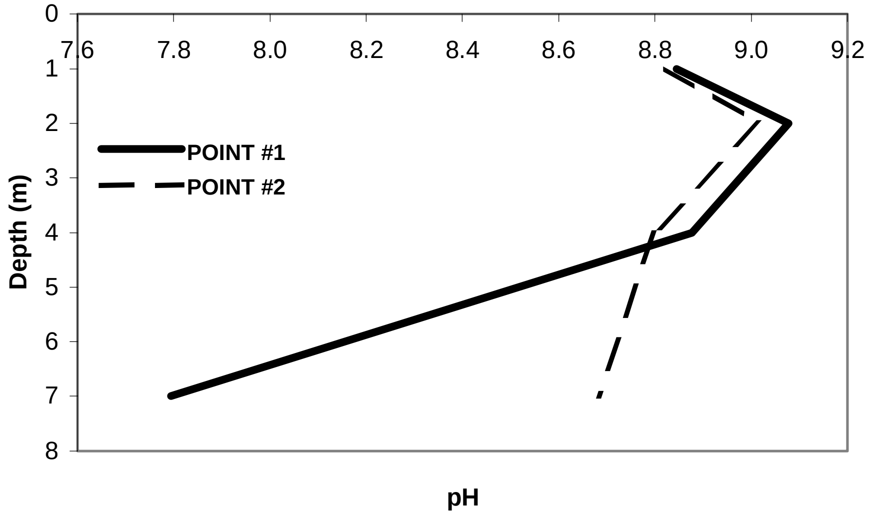
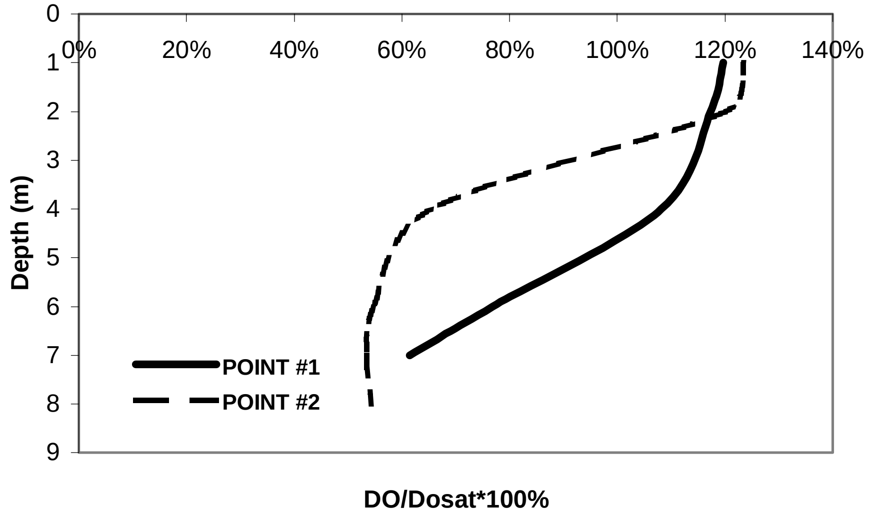


Figure E-9: Concentration Profiles for % Saturation of DO Concentrations



Appendix F – Furnas Reservoir Field Measurements

January 17, 2002 Field Measurements

Sampling Point Coordinates for January 17, 2002

CODE	Sample #	Depth (m)	Elevation	Latitude, S	Longitude, W	Sampling Time
1-0117	1	7	756	21°23.213'	46°00.937'	3:00 PM
2-0117	2	7	767	21°22.819'	46°00.532'	3:15 PM
3-0117	3	8	767	21°21.730'	46°00.184'	3:30 PM
4-0117	4	9	767	21°20.786'	45°59.940'	4:00 PM
P3-0117	Stream	Pedra Branca	781	21°21.531'	45°57.472'	9:30 AM
P4-0117	Stream	JBE	768	21°23.924'	45°58.900'	10:30 AM

Nutrient and pH Measurements for January 17, 2002

CODE	Sample #	Sample Location	Rep #	Ammonia N (mg/L Nitrogen)	Ortho-phosphate (mg/L Phosphate)	Ortho-phosphate (mg/L Phosphorus)	Nitrate (mg/L Nitrate)	pH
1-0117-1	1	Reservoir	1	<0.1	0.02	0.007	ND	7.47
1-0117-2	1	Reservoir	2	--	0.02	0.007	--	--
2-0117-1	2	Reservoir	1	<0.1	0.02	0.007	ND	8.16
2-0117-2	2	Reservoir	2	--	0.02	0.007	--	--
3-0117-1	3	Reservoir	1	<0.1	0.03	0.010	ND	7.64
3-0117-2	3	Reservoir	2	--	0.02	0.007	--	--
4-0117-1	4	Reservoir	1	<0.1	0.02	0.007	ND	7.7
4-0117-2	4	Reservoir	2	--	0.03	0.010	--	--
P3-0117-1	Stream	Pedra Branca	1	1.2	0.16	0.053	0.001	--
P3-0117-2	Stream	Pedra Branca	2	1.4	0.22	0.073	--	--
P4-0117-1	Stream	JBE	1	2.45	0.2	0.067	0.001	7.68
P4-0117-2	Stream	JBE	2	2.5	0.18	0.060	--	--

Total and Fecal Coliforms and COD for January 17, 2002

CODE	Sample #	Sample Location	Repetition #	MPN Total Coliform/100ml	MPN Fecal Coliform/100ml	COD (mg/L)
1-0117-1	1	Reservoir	1	--	--	23.3
1-0117-2	1	Reservoir	2	--	--	--
2-0117-1	2	Reservoir	1	--	--	15.5
2-0117-2	2	Reservoir	2	--	--	--
3-0117-1	3	Reservoir	1	--	--	25.8
3-0117-2	3	Reservoir	2	--	--	--
4-0117-1	4	Reservoir	1	790	170	21.6
4-0117-2	4	Reservoir	2	--	--	--
P3-0117-1	Waste stream	Pedra Branca	1	920000	17000	12.2
P3-0117-2	Waste stream	Pedra Branca	2	--	--	--
P4-0117-1	Waste stream	JBE	1	35000000	2200000	37.8
P4-0117-2	Waste stream	JBE	2	--	--	--

DO and Temperature Measurements in the Reservoir

	Sample 1		Sample 2		Sample 3		Sample 4	
	7 meters deep		7 meters deep		8 meters deep		9 meters deep	
Depth (m)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)
0.5	7.4	29	7.8	28	7.6	26.5	7.8	29
2	7	26	7.8	25.5	6.8	25.9	7	26.2
4	6.1	25.5	5	25	6.4	25	5.6	25.5
5	4.6	25.2	4.8	25	6	25	4.8	25
6	4.6	25.1	4.8	25	5.2	25	4.6	25
7	4.4	25	4.4	25	5	25	4	25
8	--	--	--	--	4.8	25	2	25
9	--	--	--	--	--	--	0.8	24.8

January 18, 2002 Field Measurements

Sampling Point Coordinates for January 18, 2002

CODE	Sample #	Depth (m)	Elevation	Latitude, S	Longitude, W	Sampling Time
1-0117	1	8	764	21°22.912'	46°00.937'	9:00 AM
2-0117	2	9	765	21°21.816'	46°00.532'	9:15 AM
3-0117	3	10	768	21°21.506'	46°00.184'	9:30 AM
4-0117	4	11	764	21°18.593'	45°59.940'	9:45 AM
P3-0117	Stream	Pedra Branca	781	21°21.531'	45°57.472'	2:00 PM
P4-0117	Stream	JBE	768	21°23.924'	45°58.900'	3:45 PM

Nutrient and pH Measurements for January 18, 2002

CODE	Sample #	Sampling Location	Repetition #	Ammonia N (mg/L Nitrogen)	Ortho-phosphate (mg/L Phosphate)	Ortho-phosphate (mg/L Phosphorus)	Nitrate (mg/L Nitrate)	pH
1-0118-1	1	Reservoir	1	<0.1	0.27	0.090	ND	7.39
1-0118-2	1	Reservoir	2	--	0.10	0.033	--	--
2-0118-1	2	Reservoir	1	<0.1	0.03	0.010	ND	7.57
2-0118-2	2	Reservoir	2	--	0.02	0.007	--	--
3-0118-1	3	Reservoir	1	<0.1	0.02	0.007	ND	7.51
3-0118-2	3	Reservoir	2	--	0.03	0.010	--	--
4-0118-1	4	Reservoir	1	<0.1	0.04	0.013	ND	7.79
4-0118-2	4	Reservoir	2	--	0.02	0.007	--	--
P3-0118-1	Stream	Pedra Branca	1	1.2	0.08	0.027	0.001	7.54
P3-0118-2	Stream	Pedra Branca	2	1.4	0.04	0.013	0.001	--
P4-0118-1	Stream	JBE	1	2.45	0.17	0.057	0.002	7.54
P4-0118-2	Stream	JBE	2	2.5	0.50	0.167	0.002	--

Total and Fecal Coliforms and COD for January 18, 2002

CODE	Sample #	Repetition #	MPN Total Coliform/100 ml	MPN Fecal Coliform/100 ml	COD (mg/L)
1-0118-1	1	1	--	--	23.10
1-0118-2	1	2	--	--	--
2-0118-1	2	1	--	--	7.80
2-0118-2	2	2	--	--	--
3-0118-1	3	1	--	--	22.70
3-0118-2	3	2	--	--	--
4-0118-1	4	1	2000	1200	23.70
4-0118-2	4	2	--	--	--
P3-0118-1	Pedra Branca	1	1.10E+06	2.20E+05	54.40
P3-0118-2	Pedra Branca	2	--	--	--
P4-0118-1	JBE	1	2.40E+08	1.60E+08	148.40
P4-0118-2	JBE	2	--	--	--

DO and Temperature Measurements in the Reservoir for January 18, 2002

	Sample 1		Sample 2		Sample 3		Sample 4	
	8 meters deep		9 meters deep		10 meters deep		11 meters deep	
Depth (m)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)	DO (mg/L)	Temperature (°C)
0.5	6.4	25	5.4	25	5.8	25	5.7	25
2	5	25	5.2	25	5.7	25	5.7	25
4	4.8	25	5	25	5.3	25	5.7	25
6	4.2	24.8	4.8	25	5.2	25	5.2	25
8	4	24.6	4.8	25	5	25	4.8	25
9	--	--	4.8	25	4	25	4.2	25
10	--	--	--	--	--	--	4	25
11	--	--	--	--	--	--	3.9	25

January 21, 2002 Field Measurements

Sampling Point Coordinates for January 21, 2002

CODE	Sample #	Depth (m)	Elevation	Latitude, S	Longitude, W	Sampling Time
1-0121	1	8	767	21°23.051'	46°00.163'	2:00 PM
2-0121	2	9	765	21°21.716'	46°00.651'	2:15 PM
3-0121	3	10	768	21°19.506'	45°59.508'	2:30 PM
4-0121	4	11	770	21°18.115'	45°58.946'	2:45 PM
5-0121	5	farm	775	21°22.687'	46°00.103'	3:00 PM
6-0121	6	farm	767	21°23.589'	46°00.892'	3:15 PM
7-0121	7	farm	764	21°23.729'	46°01.133'	3:30 PM
WB1	at 8 m	11	765	21°21.716'	46°00.651'	3:45 PM
WB2	at 7 m	10	768	21°19.506'	45°59.508'	4:00 PM
WB3	at 7 m	12	770	21°18.115'	45°58.946'	4:15 PM
P3-0117	Pedra Branca	Waste stream	781	21°21.531'	45°57.472'	9:30 AM
P4-0117	JBE	Waste stream	768	21°23.924'	45°58.900'	10:00 AM

Total and Fecal Coliforms and COD for January 21, 2002

CODE	Sample #	MPN Total Coliform/100 ml	MPN Fecal Coliform/100 ml	COD (mg/L)
1-0121-1	1	--	--	36.50
2-0121-1	2	--	--	21.90
3-0121-1	3	--	--	125.30
4-0121-1	4	1800	93	101.10
5-0121-1	5	3500	700	39.50
6-0121-1	6	--	--	88.10
7-0121-1	7	16000	460	33.40
WB1-0121-1	8/11 m	--	--	78.30
WB2-0121-1	7/10 m	--	--	78.40
WB3-0121-1	7/12 m	700	45	30.90
P3-0121-1	Pedra Branca	700000	2000	11.80
P4-0121-1	JBE	7900000	450000	36.50

Nutrient Concentrations and pH Measurements for January 21, 2002

CODE	Sample #	Repetition #	Ammonia N (mg/L Nitrogen)	Ortho-phosphate (mg/L Phosphate)	Ortho-phosphate (mg/L Phosphorus)	Nitrate (mg/L Nitrate)	pH
1-0121-1	1	1	<0.1	0.08	0.027	ND	9.19
1-0121-2	1	2	--	0.05	0.017	--	--
2-0121-1	2	1	<0.1	0.06	0.020	ND	9.29
2-0121-2	2	2	--	0.05	0.017	--	--
3-0121-1	3	1	<0.1	0.04	0.013	ND	9.31
3-0121-2	3	2	--	0.05	0.017	--	--
4-0121-1	4	1	<0.1	0.08	0.027	ND	9.23
4-0121-2	4	2	--	0.06	0.020	--	--
5-0121-1	5	1	<0.1	0.04	0.013	ND	9.3
5-0121-2	5	2	--	0.03	0.010	--	--
6-0121-1	6	1	<0.1	0.04	0.013	ND	9.23
6-0121-2	6	2		0.04	0.013	--	--
7-0121-1	7	1	<0.1	0.06	0.020	ND	8.83
7-0121-2	7	2	--	0.04	0.013	--	--
WB1-0121-1	8/11 m	1	<0.1	0.04	0.013	ND	8.96
WB1-0121-2	8/11 m	2	--	--	--	--	--
WB2-0121-1	7/10 m	1	<0.1	0.02	0.007	ND	9.11
WB2-0121-2	7/10 m	2	--	0.04	0.013	--	--
WB3-0121-1	7/12 m	1	<0.1	0.04	0.013	ND	9.17
WB3-0121-2	7/12 m	2	--	0.03	0.010	--	--
P3-0121-1	Pedra Banca	1	2.2	0.24	0.080	ND	6.96
P3-0121-2	Pedra Banca	2	2.5	0.22	0.073	--	--
P4-0121-1	JBE	1	2.5	0.46	0.153	ND	7.05
P4-0121-2	JBE	2	2.5	0.42	0.140	--	--

DO and Temperature at Point 2 in the Reservoir for January 21, 2002

	Sample 2	
	11 meters deep	
Depth (m)	DO (mg/L)	Temperature (°C)
0.5	8	28
2	7.4	25.5
4	4.2	25
6	4.2	25
8	4	25
10	4	25

January 24, 2002 Field Measurements

Sampling Point Coordinates for January 24, 2002

CODE	Elevation	Latitude, S	Longitude, W
P1-0124	757	21°22.715'	46°00.409'
P2-0124	751	21°22.021'	46°00.320'

DO, Temperature and pH Measurements for January 24, 2002

	Point 1			Point 2		
Depth (m)	DO (mg/L)	Temperature (°C)	Average pH	DO (mg/L)	Temperature (°C)	Average pH
1	8.4	29	8.84	8.8	28	8.82
2	8.5	27	9.08	8.7	27	9.02
4	7.9	26.5	8.88	4.9	25	8.80
6	5.7	25.5	--	4.1	25	--
7	4.6	25	7.79	4	25	8.68
8	4	25	--	4.1	24.5	--

Appendix G– DO Saturation Concentrations

January 17, 2002

Dissolved Oxygen at Saturation, Sample Points 1 & 2, January 17, 2002

	Sample Point 1				Sample Point 2			
	7 meters deep				7 meters deep			
Depth (m)	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}
0.5	7.4	7.0	29.0	105.5%	7.8	7.1	28.0	109.4%
2	7.0	7.4	26.0	95.1%	7.8	7.4	25.5	105.1%
4	6.1	7.4	25.5	82.2%	5.0	7.5	25.0	66.8%
5	4.6	7.5	25.2	61.7%	4.8	7.5	25.0	64.1%
6	4.6	7.5	25.1	61.5%	4.8	7.5	25.0	64.1%
7	4.4	7.5	25.0	58.8%	4.4	7.5	25.0	58.8%
8	--	--	--	--	--	--	--	--
9	--	--	--	--	--	--	--	--

Dissolved Oxygen at Saturation, Sample Points 3 & 4, January 17, 2002

	Sample Point 3				Sample Point 4			
	8 meters deep				9 meters deep			
Depth (m)	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}
0.5	7.6	7.3	26.5	104.1%	7.8	7.0	29.0	111.2%
2	6.8	7.4	25.9	92.2%	7.0	7.3	26.2	95.4%
4	6.4	7.5	25.0	85.5%	5.6	7.4	25.5	75.4%
5	6.0	7.5	25.0	80.1%	4.8	7.5	25.0	64.1%
6	5.2	7.5	25.0	69.5%	4.6	7.5	25.0	61.4%
7	5.0	7.5	25.0	66.8%	4.0	7.5	25.0	53.4%
8	4.8	7.5	25.0	64.1%	2.0	7.5	25.0	26.7%
9	--	--	--	--	0.8	7.5	24.8	10.6%

January 18, 2002

Dissolved Oxygen at Saturation, Sample Points 1 & 2, January 18, 2002

	Sample Point 1				Sample Point 2			
	8 meters deep				9 meters deep			
Depth (m)	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}
0.5	6.4	7.5	25.0	85.5%	5.4	7.5	25.0	72.1%
2	5.0	7.5	25.0	66.8%	5.2	7.5	25.0	69.5%
4	4.8	7.5	25.0	64.1%	5.0	7.5	25.0	66.8%
6	4.2	7.5	24.8	55.9%	4.8	7.5	25.0	64.1%
8	4.0	7.5	24.6	53.1%	4.8	7.5	25.0	64.1%
9	--	--	--	--	4.8	7.5	25.0	64.1%
10	--	--	--	--	--	--	--	--
11	--	--	--	--	--	--	--	--

Dissolved Oxygen at Saturation, Sample Points 3 & 4, January 18, 2002

	Sample 3				Sample 4			
	10 meters deep				11 meters deep			
Depth (m)	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}	DO (mg/L)	DO_{sat} (mg/L)	Temperature (°C)	DO/DO_{sat}
0.5	5.8	7.5	25.0	77.5%	5.7	7.5	25.0	76.1%
2	5.7	7.5	25.0	76.1%	5.7	7.5	25.0	76.1%
4	5.3	7.5	25.0	70.8%	5.7	7.5	25.0	76.1%
6	5.2	7.5	25.0	69.5%	5.2	7.5	25.0	69.5%
8	5.0	7.5	25.0	66.8%	4.8	7.5	25.0	64.1%
9	4.0	7.5	25.0	53.4%	4.2	7.5	25.0	56.1%
10	--	--	--	--	4.0	7.5	25.0	53.4%
11	--	--	--	--	3.9	7.5	25.0	52.1%

January 21, 2002

Dissolved Oxygen at Saturation, Sample Point 2, January 21, 2002

	Sample 2			
	8 meters deep			
Depth (m)	**DO (mg/L)**	**DO_{sat} (mg/L)**	**Temperature (°C)**	**DO/DO_{sat}**
0.5	8	7.1	28	112.3%
2	7.4	7.4	25.5	99.7%
4	4.2	7.5	25	56.1%
6	4.2	7.5	25	56.1%
8	4	7.5	25	53.4%
10	4	7.5	25	53.4%

January 24, 2002

Dissolved Oxygen at Saturation, Sample Points 1& 2, January 24, 2002

	Sample 1				Sample 2			
Depth (m)	**DO (mg/L)**	**DO_{sat} (mg/L)**	**Temperature (°C)**	**DO/DO_{sat}**	**DO (mg/L)**	**DO_{sat} (mg/L)**	**Temperature (°C)**	**DO/DO_{sat}**
1	8.4	7.0	29.0	119.7%	8.8	7.1	28.0	123.5%
2	8.5	7.2	27.0	117.4%	8.7	7.2	27.0	120.1%
4	7.9	7.3	26.5	108.2%	4.9	7.5	25.0	65.4%
6	5.7	7.4	25.5	76.8%	4.1	7.5	25.0	54.8%
7	4.6	7.5	25.0	61.4%	4.0	7.5	25.0	53.4%
8	4.0	7.5	25.0	53.4%	4.1	7.6	24.5	54.3%