

**SEAWATER AS A COAGULANT SUBSTITUTE AND THE EFFECTS ON
DISINFECTION IN CHEMICALLY ENHANCED PRIMARY
WASTEWATER TREATMENT:
A CASE STUDY OF PARATY, BRAZIL AND THE DEER ISLAND
WASTEWATER TREATMENT PLANT, BOSTON.**

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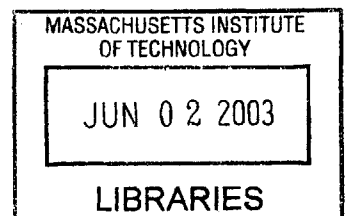
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Submitted to the Department of Civil and Environmental Engineering on May 9, 2003 in partial fulfillment of the requirements of the degree of Master of Engineering in Civil and Environmental Engineering

Abstract

Paraty is a small touristic city on the South Eastern coast of Brazil plagued with public health problems inherently linked to its poor water and sanitation practices. This thesis centers on the design of an appropriate wastewater treatment facility for the City. The new and interesting use of seawater as a wastewater treatment tool and its effect on the disinfection potential of the treated effluent is also reviewed and tested in a series of jar tests and laboratory experiments. Chemically Enhanced Primary Treatment (CEPT) is a wastewater treatment method that serves as an attractive alternative to conventional primary treatment and can also be used as an efficient preliminary step to biological secondary treatment processes. CEPT adopts coagulation and flocculation and accomplishes remarkable increases in the removals of common pollutants and contaminants from the influent. CEPT was chosen as the most favorable treatment alternative for Paraty as it is an expandable, economic and highly efficient system. The main advantage to CEPT is to generate an effluent that can be efficiently and economically disinfected at a low cost compared to secondary treatment. The most optimal dose of FeCl_3 , polymer and seawater to treat the Paraty sewage were estimated at 40mg/L, 0.1 mg/L and 5% seawater by volume respectively. Jar tests were also conducted at the Boston Deer Island WWTP to check the efficiency of using seawater as a coagulation enhancement mechanism on saline influents.

CEPT effluents treated with FeCl_3 were also tested for disinfection with both Paracetic acid (PAA) and Chlorine. Although PAA is an effective disinfectant, it was not included due to its high cost. The optimal chlorine dose for Paraty was estimated at 3 mg/L.

Additional tests were conducted to test for the disinfectability of the Deer Island effluent treated with CEPT. These tests helped show that the sensitivity of disinfection in the presence of seawater is limited by low seawater concentrations.

This thesis concludes with a detailed design of the treatment plant sedimentation tanks, chlorination/dechlorination chambers, and grit removal facility dimensions and draws general encouraging conclusions on the suggested use of seawater as a coagulation catalyst in chemically enhanced primary wastewater treatment.

Thesis Supervisor: Dr. Donald Harleman

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CHAPTER ONE: PROJECT INTRODUCTION; RESEARCH AND APPLICATIONS

The United Nations and World Health Organization (WHO) have repeatedly included Brazil on the list of nations required to immediately address sanitation infrastructure, appropriate treatment technologies and related public health issues (United Nations, WHO, 2003). The city of Paraty in Brazil is not an exception to these cases and is the setting for this work.

This thesis is part of a larger project aimed at providing a comprehensive solution to both potable water and sanitation infrastructure and treatment problems in the City of Paraty, Brazil.

The thesis centers on two main goals:

1. Design: To provide the conceptual and preliminary design of a chemically enhanced primary wastewater treatment plant. The raw sewage characteristics were qualified from an extensive set of jar tests performed on samples collected from representative collection points in the city. These results were also used to test for the disinfection potential of chemical treatment. Sludge treatment options, recommended chemical doses, and a suggested plant construction schedule are also presented as integral components of the design.
2. Research: The second goal of this thesis is to research the use of seawater as a coagulation enhancement mechanism in chemically enhanced primary treatment. A detailed literature review on seawater is presented and used as a preface to the jar tests performed in Paraty and in the Deer Island Wastewater Treatment Plant in Boston with the use of seawater. The effects of seawater use on the disinfection potential of chemical treatment are also a pivotal issue and are extensively explored both in Paraty and on sample influents from the Deer Island Wastewater Treatment Plant in Boston, MA.

This work comprises seven chapters:

1. Chapter 1 introduces Paraty and describes the constraints to designing a wastewater treatment plant for the city. A range of treatment alternatives is briefly presented and is analyzed for application in Paraty.
2. Chapter 2 introduces chemically enhanced primary treatment (CEPT) and explains why it is the treatment alternative of choice for Paraty. This chapter explains the kinetic theory behind using CEPT and includes an exhaustive list of sludge treatment and disposal alternatives.

3. Chapter 3 reviews chemically enhanced primary treatment in greater detail by providing a thorough literature review on previous CEPT applications, each of which highlight a distinct advantage to adding chemicals to influents. Seawater addition is introduced in detail here as a promising coagulation enhancement mechanism.
4. Chapter 4 presents the results from the January 2003 jar tests performed in Paraty to test for the efficiency of adding ferric chloride, polymers and seawater to wastewater influents. This chapter focuses primarily on making recommendations for the dosages of chemicals needed for a treatment plant in Paraty. It is also an important attempt at showing the advantages of using seawater in chemical treatment
5. Chapter 5 introduces the Deer Island Wastewater Treatment Plant in Boston, which is this project's second setting for additional jar tests. The results from these tests were compared to the results from chapter 4 to reiterate the potential for seawater use as a coagulation enhancement mechanism.
6. Chapter 6 consists of the project's disinfection component. The data presented in this chapter are the results from the numerous chlorination tests performed on wastewater effluent in Paraty and in Deer Island. The theory behind various methods of disinfection is included in thorough detail.
7. Chapter 7 is the design chapter and presents the preliminary conceptual design for the wastewater treatment plant in Paraty. Design parameters include estimated influent characteristics, chemical doses, and tank dimensions among others.

1.1 INTRODUCTION TO PARATY

Paraty is a historical, colonial city located in the state of Rio de Janeiro in Brazil, 252 kilometers south of the city of Rio de Janeiro. Paraty is located on the acclaimed Green Coast of Brazil (Figure 1.1) and due to its prime geographic location, is a key tourist attraction of the State of Rio de Janeiro (Figures 1.2,1.3 and 1.4). With a winter population of 3000, the small historical center in Paraty, alone, attracts close to 6000 visitors every summer. This three-fold population flux is not formally documented but was nonetheless assumed to be true based on communication with city representatives and residents. The population flux exerts a large demand on water consumption and wastewater production. However, Paraty does not currently treat its wastewater.

Paraty is currently actively pursuing a UNESCO World Heritage Site qualification but must treat its wastewater in order to be eligible for nomination. The issue of sanitation in Paraty has therefore become a pivotal and very critical issue not only from a public health perspective but from an economic standpoint as well. Both the infrastructure and treatment alternatives must be respectively revamped, retrofitted and designed to serve Paraty's fluctuating population both efficiently and economically.



Figure 1.1:
The State of Rio de Janeiro inclusive of Paraty.



Figure 1.2:
The Green Coast of Paraty
<http://www.bitourism.com>



Figure 1.3: Churches in Paraty

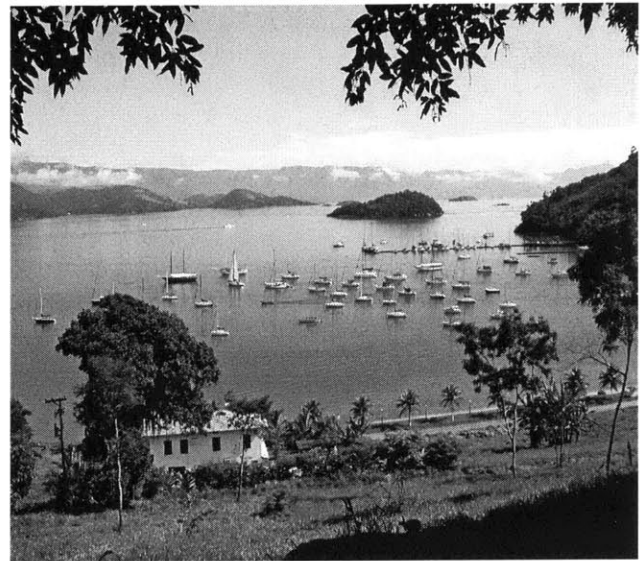


Figure 1.4: The Paraty Harbour

In January 2003, a team of four environmental engineering graduate students spent three weeks in Paraty to assess the extent of the drinking and wastewater problems in the city. The main tasks addressed by the design chapters of this particular thesis however are the collection and testing of the Paraty sewage to generate the most appropriate wastewater treatment plant design for the city. Figures 1.5 through 1.8 below are examples of the poor wastewater management currently plaguing the city of Paraty.



Figures 1.5: Sewage Discharge into stream



Figure 1.6: Sewage Discharge into street (1)



Figure 1.7: Sewage Discharge into Street (2)



Figure 1.8: Discharge into Paraty Bay

1.2 PROJECT CONSTRAINTS

There are a number of constraints that became very evident for the design of an appropriate wastewater treatment plant in Paraty. These are summarized below:

1.2.1 Old and New Infrastructure

Paraty is divided into three main sections: the historical downtown, the upper part of the city, and the “Ilha das Cobras”(Snake Island). Few of these areas are served by a functional sewage collection network. Sampling raw sewage for laboratory analysis therefore became a challenge and the samples were collected from open pipes, street ditches through which sewage was allowed to flow, or opened manholes. It is important to note however that the manholes did not yield fresh sewage since the pipes that the manholes serviced were not functional.

Examples of sampling points are shown in Figures 1.9 through 1.12. The main sampling point used for the jar testing (described in Chapter 4) was from an open ditch near a popular hotel that was assumed to represent domestic sewage during the summer months. The city of Paraty also only has one hospital that does not have wastewater treatment and discards directly to the adjacent rivers. This obviously causes serious contamination of the river waters by potentially very pathogenic organisms. The hospital wastewater however was not included in this project.



Figure 1.9: Dilute sewage in an open ditch



Figure 1.10: Claire collecting sewage from street



Figure 1.11: Collecting sewage from a tank



Figure 1.12: River of sewage

1.2.2 Space allotment for the treatment plant

One of the most important constraints in the design of a wastewater treatment plant for Paraty was the limited area of land allotted to the treatment facility. This area is approximately 6000 m² (Choi, 2003) and is located in a poor residential area. Extra efforts had to be taken into consideration therefore to ensure that the influent was treated quickly and efficiently and that the treated effluent was disinfected and properly discharged to the ocean, with maximum odor control.

1.2.3 Income gradients and Government funding

The “Ilha das Cobras” is home to many lower income people. This area has grown to be the largest portion of the city with a wintertime population of approximately 4500 people. Since the “Ilha Das Cobras” is a very poor and densely populated neighborhood, the risks to public health

are greater than in the wealthier, cleaner and more touristic area of the Historical Center of Paraty. Special attention therefore was paid to include the Ilha das Cobras in the early stages of the wastewater treatment plant for Paraty.

The Paraty region is one of the most famous vacation spots in Brazil and the historical center is a hub of entertainment and expenditure. This center was therefore chosen as this project's starting point in order to provide a design example and to incite the government of Paraty to expand its sanitary infrastructure and treatment capabilities.

The suggested design of a wastewater treatment plant (described in detail in Chapter 7) therefore suggests a phased treatment construction plan. Phase 1 addresses the Historical Center, Phase 2 addresses the Ilha das Cobras and Mangeira Island (another low income section of the city) and Phase 3 will address the remaining sections of the city (You, 2003)

1.2.4 Climate

The climate in Paraty is tropical with heavy rainstorms expected in the afternoons. As such, the samples that were collected (from open ditches for example) were often heavily diluted and considered unrepresentative of pure domestic sewage. These samples are identified and the treatment removals observed are discussed correspondingly in Chapter 4.

It is important to note that a significant limiting factor to the collection of information in Paraty was the general lack of information and engineering resources (engineering maps, past data, water quality monitoring reports, public health statistics etc.) in the city. Many assumptions were made and these are clearly identified in the project description and analysis

1.3 WASTEWATER TREATMENT METHODS FOR THE DEVELOPING WORLD

Several treatment alternatives are available to treat the wastewater in Paraty. These are briefly discussed in the following sections below. Conceptually effective, most of these treatment methods however can no longer stand independently due to increasing population demands, shrinking land availability, and increased pathogenic virulence. The advantages and disadvantages of each treatment alternative are discussed and facilitated choosing chemically enhanced primary treatment for the most optimal application in Paraty.

Natural and constructed wetlands, lagoons and septic tanks are few examples of the main treatment methods that have been traditionally adopted for the developing world and are briefly described in sections 1.3.1 and 1.3.3 below. The CEPT process will be presented and discussed in detail in chapter 2. It was however considered the best system for this project due to Paraty's heavily fluctuating population (and its correspondingly fluctuating flows), the limited amount of land allotted to the treatment facility and the demand for immediate low-cost solutions to the serious public health concern related to pathogens present in public water bodies.

1.3.1 Wetlands

Wetlands are inundated land areas with water depths typically less than 2 ft (0.6 m) that support the growth of emergent plants such as cattail, bulrush, reeds and sedges (Metcalf and Eddy, 1991). The vegetation provides surfaces for the attachment of bacterial films, aides in the filtration and adsorption of wastewater constituents, transfers oxygen into the water column and controls the growth of algae by restricting the penetration of sunlight (Metcalf and Eddy, 1991).

Natural wetlands in the developed world are typically considered receiving water bodies and are therefore designed to hold water that has already undergone secondary or advanced treatment. In the developing world, however, natural wetlands are often perceived as ideal holding tanks for the raw sewage. Consequently, these often become overloaded, breed pathogenic bacteriological agents, seep into and contaminate surrounding groundwater, and jeopardize neighboring crop quality. Constructed wetlands in the developed world, on the other hand, offer all the treatment capabilities of natural wetlands without the constraints associated with discharging to a natural ecosystem (Metcalf and Eddy, 1991). In the developing world, it is often very hard to distinguish between constructed and natural wetlands.

Wetlands were considered inappropriate for use in Paraty due to the limited land space allotted to the future treatment facility. The proximity of the allotted treatment facility area to the ocean also discouraged the use of wetlands in Paraty.

1.3.2 Lagoons

Lagoons are the world's oldest wastewater treatment technology and consist of in-ground earthen basins in which the waste is kept for a specified detention time and then discharged. An aerated lagoon is a basin in which wastewater is treated either on a flow-through basis or with solids recycle; the essential function of this treatment process being waste conversion. Oxygen is usually supplied by means of surface aerators or diffused air units. As with other suspended growth systems, the turbulence created by the aeration devices is used to maintain the contents of the basin in suspension. (Metcalf and Eddy, 1991). Lagoons can be classified under one of five categories: Anaerobic, facultative, aerobic, partial mix or completely mixed (Metcalf and Eddy, 1991). Lagoons were considered inappropriate however for Paraty principally due to the necessary land requirements and the high energy inputs.

1.3.3 Septic Tanks

Septic tanks were finally considered for Paraty and are defined as large, concrete or plastic tanks buried underground to receive household sewage. These however, were considered inappropriate for Paraty since the city was not interested in satellite treatment alternatives and due to the high maintenance costs. The old infrastructure in the city would also not be amenable to the installation of septic tanks.

Chemically enhanced primary treatment was chosen as the most appropriate treatment alternative for Paraty for several reasons:

1. The large number of successful past projects implemented in cities similar to Paraty for which CEPT had been the most efficient and cost effective treatment alternative (Harleman and Murcott, 2001).
2. The space constraints, high maintenance and capital costs etc of other treatment alternatives were considered limiting factors which made CEPT the best wastewater treatment process for Paraty.

CHAPTER 2: CHEMICALLY ENHANCED PRIMARY TREATMENT

The following chapter addresses CEPT in extensive detail and will certify that it is the most efficient and cost reducing first wastewater treatment step to address Paraty's immediate sanitation needs.

2.1 INTRODUCTION

Chemically Enhanced Primary Treatment (CEPT) is a wastewater treatment method that serves as an attractive alternative to conventional primary treatment and can also be used as an efficient preliminary step to biological secondary treatment (such as activated sludge and trickling filters). CEPT adopts the coagulation and flocculation processes and accomplishes a remarkable increase in the removals of common pollutants and contaminants such as BOD (biochemical oxygen demand), COD (chemical oxygen demand), TSS (total suspended solids), and TP (total phosphorous) present in the influent. The main advantage to CEPT therefore is to generate an effluent that can be efficiently and economically disinfected at a low cost compared to secondary treatment.

The CEPT process is principally derived from conventional primary treatment since the influent in both processes passes through a bar screen (to remove large objects from the flow), grit removal chamber and clarifier both designed to remove suspended solids. CEPT however enhances this process by injecting small doses of metal salts and/or cationic polymers prior to the grit removal process. An optional anionic polymer can also be added as a flocculent prior to clarification. Figure 2.1 below describes the processes involved in both conventional and chemically enhanced primary treatment. The red processes are the conventional primary treatment and the blue additions explain the role that CEPT plays in treating the influent.

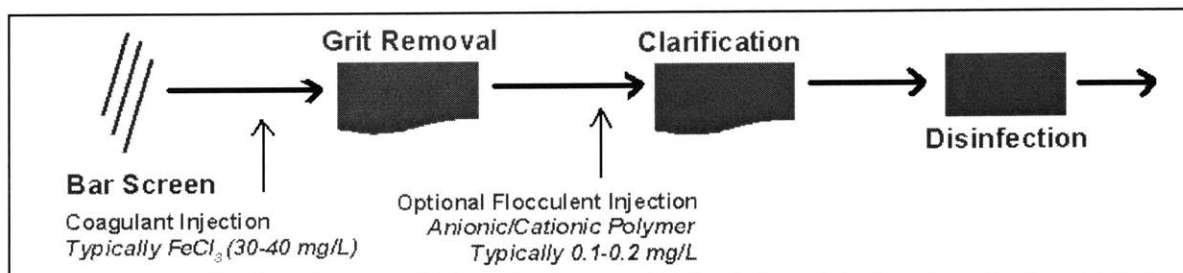


Figure 2.1: CEPT vs. Conventional Primary Treatment

The injected chemicals act as coagulants/flocculants forming large heavy flocs that settle to the bottom of the clarifier and form a sludge layer than can be appropriately collected and removed. Particulate and colloidal settling are the processes responsible for the formation and settling of floc. Consequently, the BOD, TSS, and phosphorus removal efficiencies in CEPT have repeatedly been observed to be higher than those in conventional primary treatment and appreciably close to biological secondary treatment (Harleman, 2003).

2.2 THEORY OF CEPT

Colloidal particles found in wastewater typically have a net negative surface charge. The size of colloids (about 0.01 to 1 μm) is such that the attractive body forces between particles are considerably less than the repelling forces of the electric charge. Under these stable conditions, Brownian motion keeps the particles in suspension. Coagulation is the process of destabilizing colloidal particles so that particle growth can occur as a result of particle collisions. (Metcalf and Eddy, 1991)

2.2.1 *Coagulation*

Coagulation encompasses all the reactions and mechanisms involved in the chemical destabilization of particles and in the formation of larger flocs by the aggregation of particulates in the size range from 0.01 to 0.1 μmeters otherwise known as perikinetic flocculation. In general, metal salts or cationic polymers are the chemicals added to destabilize the colloidal particles in wastewater so that floc formation can result. Figure 2.2 of the following page shows what typical floc in chemical treatment look like.

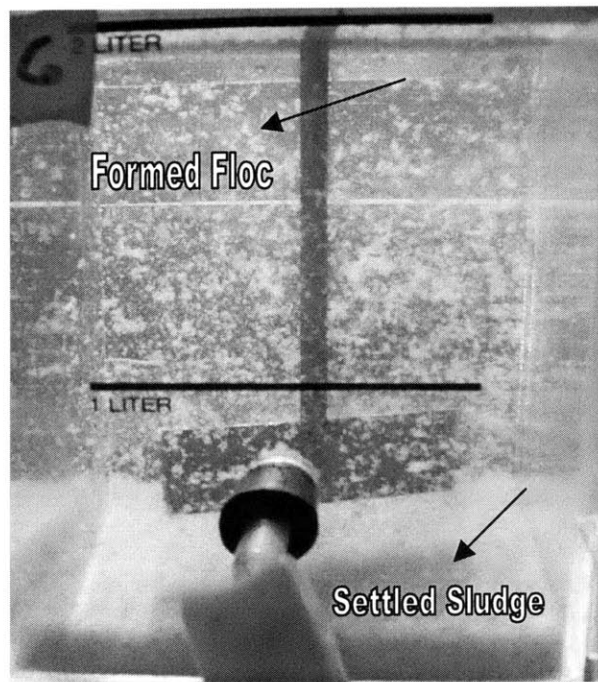


Figure 2.2: Floc formation due to chemical addition

Typical coagulants and flocculants include natural and synthetic organic polymers, metal salts such as alum, ferric sulfate or ferric chloride, and prehydrolyzed metal salts such as polyaluminum chloride (PACl). (Metcalf and Eddy, 1991)

Ferric chloride (FeCl_3) is an example of a common coagulant used in the chemical treatment of wastewaters. When added to the influent, FeCl_3 reacts with the alkalinity and with phosphates to form insoluble iron salts. The colloidal particle size of insoluble FePO_4 is small, requiring larger dosages of FeCl_3 to produce a well-flocculated iron hydroxide precipitate that carries the phosphate precipitate (Metcalf and Eddy, 1991). The exact dosages of ferric chloride are usually best determined by jar tests and full-scale evaluations. Typical average concentrations vary between 10 and 50 mg/L (Harleman, 2003). These concentrations can be kept at a minimum with the added use of polymers in the wastewater treatment.

Polymers or polyelectrolytes are high molecular weight compounds, usually synthetic, which, when added to wastewater, can also be used as coagulants, coagulant aids, filter aids or sludge conditioners. In solution, polymers may carry either a positive, negative or neutral charge and, as such, are characterized as cationic, anionic or nonionic. As a coagulant or coagulant aid, cationic polymers act as bridges, reducing charge repulsion between colloidal and dispersed floc particles and thereby increasing the settling velocities (Metcalf and Eddy, 1991).

The use of anionic polymers as flocculants for chemically enhanced primary treatment is a proven and acceptable technique (Harleman, 2003). Typical concentrations of anionic polymers in CEPT treatment average between 0.05 and 0.2 mg/L. Significant mixing (in the order of 100 rpm) is needed however for the cationic additive to bind to the suspended solids in the wastewater and form flocs appropriate to the coagulation and flocculation process. Therefore the coagulant is usually added as far upstream in the process as possible, or dosed in a contact chamber equipped with mechanical mixers.

2.2.2 Flocculation

Flocculation is the process in which the size of particles increases as a result of particle collisions. The two types of flocculation are: (1) *microflocculation* (or perikinetic flocculation), in which particle aggregation is brought about by the random thermal motion of fluid molecules known as Brownian motion and (2) *macroflocculation* (or orthokinetic flocculation) in which particle aggregation is brought about by inducing velocity gradients and mixing in the fluid containing the particles to be flocculated (Metcalf and Eddy, 1991).

Figure 2.3 below shows the typical difference in treated effluent quality compared to the raw wastewater influent. Beaker 1 on the left of Figure 2.3 represents conventional primary treatment (no chemical addition, rapid mix and 5 minutes settling), and beaker 6 to the right, contains the treated wastewater, after injection with 40mg/L of FeCl_3 , rapid mixing and 5 minutes of settling time. The advantage of adding chemicals to the influent is therefore obvious.



Figure 2.3: Difference in effluent quality

2.3 JAR TESTING

Jar tests are commonly conducted to test for the quality of raw sewage in an area preparing for a treatment plant design. These experiments are typically undertaken as a first step to establishing the efficiency of coagulants and flocculants at removing suspended solids (SS), chemical oxygen demand (COD) and turbidity.

The standard jar testing apparatus shown in Figure 2.4 below consists of six 2-liter beakers, each equipped with a stainless steel 1"x 3" mechanical mixer with a maximum mixing speed of 300 rotations per minute.

Figures 2.4 and 2.5 below show a complete and typical jar testing setup before and after a typical jar test run. Beakers 1 through 6 in Figure 2.5 (from left to right respectively) represent conventional primary treatment, 10, 20, 30, 40 and 50 mg/L FeCl_3 with rapid mixing and 5 minutes settling

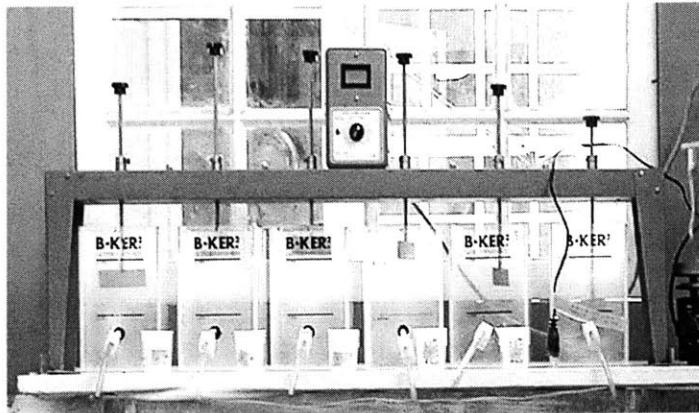


Figure 2.4: Typical jar tests apparatus

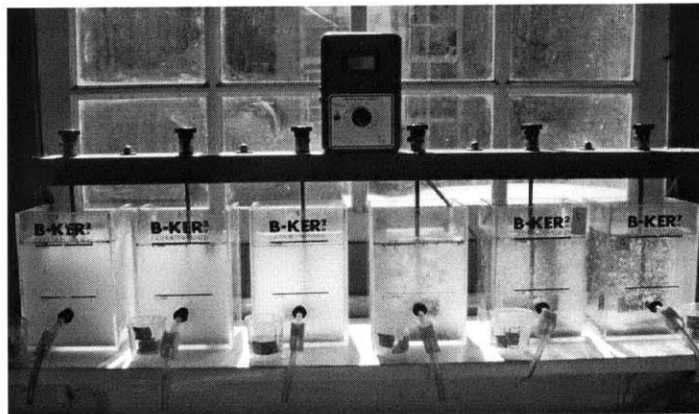


Figure 2.5: Difference in effluent quality due to chemical addition

Batch jar testing results are representative of a continuous flow treatment system because the overflow rates for both systems are equal. The efficiencies of the coagulation and flocculation processes are proportional to the time that the chemicals are in contact with the water. It is therefore possible to extrapolate data from the 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:

$$T = L.W.H / Q \qquad \text{Eq. (2-1)}$$

Where T is the residence time in days, L is the length of the tank in meters, W is the width of the tank in meters, H is the height of the tank in meters and Q is the flowrate in m³/sec.

2.3.1 Surface Overflow 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:

$$SOR = Q / L.W = H/T \qquad \text{Eq. (2-2)}$$

Where H is the height of the tank in meters and T is the residence time in days.

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

$$SOR = h/t \qquad \text{Eq. (2-3)}$$

Where h is the height of the outlet in the beaker below the water surface and equals 8 cm and t is the residence time in the beaker.

For a residence time t = 5 minutes, the SOR was then determined to be:

$$SOR = (8 \text{ cm}) / (5 \text{ minutes}) = 23 \text{ m/day approximately.}$$

For settling times of 1 or 2 minutes (instead of the 5 minutes used in this project's jar test) however, jar tests (with the beaker outlet located at 8 cm) typically display CEPT overflow rates in the range of 60 m/d (Harleman, 2003). Since jar tests are designed to model the wastewater

treatment process, the 60 m/d value is consistent with typical overflow rates for full-scale CEPT settling tanks (Metcalf and Eddy, 1991). The lower surface overflow rate of 23 m/d seen in these experiments however can therefore be attributed to the fact that the settling time allowed (5 minutes) was higher than average jar testing settling times of 1 to 2 minutes and therefore yielded overflow rates lower than 60 m/d. Also, it is important to note here that for the jar tests exhibiting high pollutant removal rates, the effluent was clear at settling times of approximately 2 minutes. If the clear effluent sample had been collected after the 2 minutes therefore rather than waiting the longer 5 minutes, the corresponding overflow rate would have therefore been equal to the expected 60 m/d.

2.3.2 Measured Parameters

Standard jar test experiments are performed to test the efficiency of chemically enhanced primary treatment in removing certain specific pollutants of concern in the wastewater influent. These pollutants are typically total suspended solids, chemical oxygen demand, and turbidity and can also encompass phosphorous and nitrogen removal tests, dissolved oxygen and pathogen levels. For the experiments described in Chapter 4, the prime emphasis was on tracking the removal rates of suspended solids, chemical oxygen demand and turbidity.

2.3.2.1 Total Suspended Solids

Total suspended solids (TSS) is defined for an influent sample to be the fraction of total solids retained on a filter of specified pore size, measured after being dried at 105 degrees Celsius. The filter most commonly used for the determination of total suspended solids is the Whatman glass fiber filter, which has a nominal pore size of 1.58 μm (Metcalf and Eddy, 1991)

Spectrophotometers are common pieces of equipment used to measure TSS quickly and efficiently. Suspended solids are another way of referring to total suspended solids (Metcalf and Eddy, 1991). For the experiments in this project therefore, the suspended solids were measured by the Hach Spectrophotometer (www.hach.com).

Suspended solids test results are routinely used to assess the performance of conventional treatment processes and the need for effluent filtration in reuse applications. These are also used as universal effluent standards (along with BOD) by which the performance of treatment plants is judged for regulatory control purposes. In chemically enhanced treatment therefore, which

achieves high-suspended solids removal rates (See Figures 2.8 and 2.9 below), measuring suspended solids is then of utmost importance to gauge the removal efficiency.

2.3.2.2 Turbidity

Turbidity is a measure of the light-transmitting properties of water and is another important test used to indicate the quality of waste discharges and natural waters with respect to colloidal and residual suspended matter. The measurement of turbidity is based on comparison of the intensity of light scattered by a sample to the light scattered by a reference suspension under the same conditions (Standard Methods, 1998). Formazin suspensions are used as the primary reference standard. The results of the turbidity measurements are read from a turbidimeter and are reported as nephelometric turbidity units (NTU) (Metcalf and Eddy, 1991). Figure 2.6 below is the Hach Pocket™ Turbidimeter Analysis System set that was used for the experiments in this project.



Figure 2.6: Hach Pocket™ Turbidimeter Analysis System

In general, there is no relationship between turbidity and the concentration of total suspended solids in untreated wastewater (Metcalf and Eddy, 1991). There is however, a reasonable relationship between turbidity and total suspended solids for the settled and filtered secondary effluent from the activated sludge process. Since the TSS removals for secondary treatment are very similar to those achieved by CEPT (See Table 2-6 on p.24), the following equation can be adopted to relate TSS and turbidity values in chemically enhanced jar tests as well:

$$\text{TSS, mg/L} \approx (\text{TSS}_f)(\text{T}) \quad \text{Eq. (2-4)}$$

Where TSS = total suspended solids, mg/L

TSS_f = factor to convert turbidity readings to total suspended solids, (mg/L TSS)/NTU

T = Turbidity in NTU

The specific value of the conversion factor depends on the wastewater treatment plant characteristics. For settled secondary effluent and for secondary effluent filtered with a granular medium-depth filter, the conversion factors will typically vary from 2.3 to 2.4 and 1.3 to 1.6 respectively (Metcalf and Eddy, 1991).

2.3.2.3 Chemical Oxygen Demand

The chemical oxygen demand (COD) is used to measure the oxygen equivalent of the organic material in wastewater that can be oxidized chemically using dichromate in an acid solution.

Biochemical oxygen demand (BOD) is also a common wastewater parameter used to qualify the characteristics of the wastewater and measures the dissolved oxygen used by microorganisms in the biochemical oxidation of organic matter (Metcalf and Eddy, 1991).

Although it would be expected that BOD and COD readings are similar, this is seldom the case. Some of the reasons for observed differences are as follows:

- 1) Many organic substances that are difficult to oxidize biologically (lignin for example) can be oxidized chemically.
- 2) Inorganic substances that are oxidized by the dichromate increase the apparent organic content of the sample.
- 3) High COD values may occur because of the presence of inorganic substances with which the dichromate can react (Metcalf and Eddy, 1991)

Interrelationships between BOD and COD have been researched however. Typical values for the ratio of BOD/COD are described in Table 2-1:

Type of Wastewater	BOD/COD
Untreated	0.3-0.8
After Primary Settling	0.4-0.6
Final Secondary Effluent	0.1-0.3

Table 2-1: BOD/COD Ratios

In chemically enhanced treatment plants, the ratios of BOD removal have been observed to be very close to the ratios of COD removals (Harleman, 2003). Since the BOD test is a 5-day test and the COD test is a 2-hour test, COD removals are commonly measured to represent the wastewater characteristics pre and post treatment instead of BOD in time-constrained laboratory

settings. Figure 2.7 below shows the Hach COD Reactor (www.hach.com) used in this project to incubate the COD vials containing effluent samples.



Figure 2.7: Hach COD Reactor

2.3.3. Effects of chlorides on chemical oxygen demand readings

In this project, seawater was added to the influent wastewater to test its efficiency as a coagulation enhancement mechanism. Seawater is naturally very abundant in chlorides that constitute slightly more than half of the percent by weight of dissolved ions (Table 2-2 below)

Dissolved Ion	Chemical Formula and Charge	% by weight of dissolved ions	% by weight of seawater
Chloride	(Cl ⁻)	55.04	1.898
Sodium	(Na ⁺)	30.61	1.0556
Sulfate	(SO ₄ ²⁻)	7.68	0.2649
Magnesium	(Mg ⁺)	3.69	0.1272
Calcium	(Ca ²⁺)	1.16	0.04
Potassium	(K ⁺)	1.1	0.038
Bicarbonate	(HCO ₃ ⁻)	0.41	0.014
Bromide	(Br ⁻)	0.19	0.0065
Boric Acid	(H ₃ BO ₃)	0.07	0.0026
Strontium	(Sr ²⁺)	0.04	0.0013
Fluoride	(F ⁻)	0.002	0.0001
Total		99.992	3.4482

Table 2-2: Dissolved Ions in Seawater

(<http://www.guilford.edu/original/academic/geology/Seawater.html>)

Chloride (Cl⁻) is however the prime interference when determining COD concentrations. Each COD vial contains mercuric sulfate that will eliminate chloride interference up to the level specified by Hach (Hach Water Analysis Handbook, 2003), in Table 2-3 below. Samples with higher chloride concentrations should be diluted enough to reduce the chloride concentrations to the level given in column three of Table 2-3.

For these experiments, the high-range Hach COD vials were used which are limited by the 2000 mg/L Chloride concentration.

Vial Type Used	Max. Cl ⁻ Conc. in sample (mg/l)	Suggested Cl ⁻ Conc. Diluted sample (mg/L)	Max. Cl ⁻ Conc. in sample when 0.5 HgSO ₄ was added
Ultra Low Range (0.7 - 40 mg/L)	2000	1000	N/A
Low Range (3 - 150 mg/L)	2000	1000	8000
High Range (20 - 1500 mg/L)	2000	1000	4000
High Range Plus (200 - 15,000 mg/L)	20,000	10,000	40,000

Table 2-3: Recommended Chloride Concentrations for accurate COD testing.

If sample dilution will cause the COD concentration to be too low for accurate determination, then 0.5 g of mercuric sulfate (HgSO₄) can be added to each COD vial before the sample is added. The additional mercuric sulfate will raise the maximum chloride concentration allowable to the level given in Column four of Table 2-3 (Hach Water Analysis Handbook, 2003)

The chloride concentrations added to the wastewater in a jar test must therefore be closely monitored to ensure that chloride interference is not yielding misleading COD results when using Hach Equipment. Two approaches were adopted to determine the concentration of chloride (Cl⁻) concentrations in different seawater volumes used for the 2-liter jar tests described in section 2.3. Both methods are based on the values from Table 2-3 above and are important at showing the sensitivity of COD removal readings in the Hach vials to the presence of chlorides.

2.3.3.1 % Weight of Chlorides

The percent of chlorides in seawater by weight is 1.898 % (Table 2-2). This means that 1 gram of seawater contains 0.018 grams of chlorides. The mass of seawater can therefore be calculated knowing the density of 1.0250 g/cm³ for seawater at a temperature of 16 degrees Celsius and a salinity of 35 parts per thousand (<http://duedall.fit.edu>). The following sample calculation was performed to monitor the addition of 10 ml of seawater to the 2-liter jar-testing beaker:

1 gram seawater = 0.018 grams Cl⁻
10 ml seawater = (1.025 g/10⁻³ L)(10 *10⁻³ L) = 10.25 grams
10.25 grams of seawater therefore contains 0.1845 grams Cl⁻

So the concentration of chlorides in the 10ml = 18450 mg/L

When the seawater is added to the 2-liter beaker, the total chloride concentration is:

Concentration = (18450 mg/L)(10 ml)(1L/1000ml)/(2 L)
C = 92 mg/L

The masses of chlorides for various volumes of seawater were therefore calculated following the method described above to check that the maximum concentration of chlorides had not been reached in the Hach COD vials. These are presented in Table 2-4 below:

Seawater by Vol. %	Vol. Seawater Added ml	Mass seawater g	Cl- mass g	Cl- conc. mg/L
0.5	10	10.25	0.1845	92
1	20	20.5	0.369	185
2	40	41	0.738	369
5	100	102.5	1.845	923
10	200	205	3.69	1845
15	300	307.5	5.535	2768

Table 2-4: Chloride concentrations Method 1

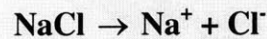
The results show therefore that the addition of 15% of seawater by volume to the 2-liter beaker contributes 2768 mg/L of chlorides to the solution. This is significantly larger than the 2000

mg/L chloride limit for the Hach vials described in Table 2-3. The COD readings for 15 % seawater additions are therefore incorrect.

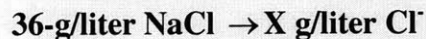
2.3.3.2 % Sodium Chloride in the seawater

To roughly estimate the amount of chlorides in seawater, NaCl can be used as an indicator. This is the basis for method 2 described in detail below to calculate the concentration of chlorides added to the beaker with the addition of various seawater volumes. The example shown below is for a 10ml seawater addition to the 2-liter beaker:

Assuming a seawater salinity of 36 ppt (parts per thousand) = 36 g NaCl / liter of seawater:



Atomic weight of sodium Na^+ is 23 g/mol and the atomic weight of Cl^- is 35 g/mol.



$$X = 22 \text{ g Cl}^-/\text{liter} = 2200 \text{ mg Cl}^-/\text{liter}$$

Therefore, when 0.5% by volume of seawater is added to the 2-liter beakers used in the jar testing apparatus, this volume equals 10 ml of seawater:

$$2200 \text{ mg Cl}^- \rightarrow 1000 \text{ ml}$$

$$Y \rightarrow 10 \text{ ml}$$

$$Y = 22 \text{ mg Cl}^- / \text{liter}$$

The values of chloride concentrations added to the wastewater in the jar tests for various concentrations of seawater by volume were therefore calculated and summarized in Table 2-5 of the following page:

% Seawater Added (by volume)	Volume of Seawater Added	Chlorides (mg/L)
0.5%	10 ml	22 mg/L
1%	20 ml	44 mg/L
2%	40 ml	88 mg/L
5%	100 ml	200 mg/L
10%	200 ml	400 mg/L
15%	300 ml	600 mg/L

Table 2-5: Chloride Concentrations in seawater volumes Method 2

Method 2 shows that the volumes of seawater added to the two-liter beakers did not cause the chloride concentrations to approach the 2000 mg/L maximum chloride concentration and therefore did not interfere with the COD readings using Hach equipment.

Method 2 can be considered less reliable than method 1 however because method 1 encompasses all the possible sources of chlorides in the seawater and does not limit the analysis to the salt concentrations in the water. Therefore, the results from method 1 are considered correct and the addition of 15% seawater does not yield correct COD readings using the Hach vials.

2.4 EXPECTED PERFORMANCE: CEPT VS SECONDARY AND PRIMARY WWT

Preliminary wastewater treatment is used to screen out, grind up, or separate debris and is the first step in wastewater treatment. Sticks, rags, large food particles, sand, gravel, toys, etc., are removed at this stage to protect the pumping and other equipment in the treatment plant. Treatment equipment such as bar screens, comminutors (a large version of a garbage disposal), and grit chambers are used as the wastewater first enters the treatment sequence. The collected debris is usually disposed of in a landfill.

Conventional primary treatment is the second step in treatment and separates suspended solids and grease from wastewater. Wastewater is held in a quiet tank for several hours allowing the particles to settle to the bottom and the greases to float to the top. The solids drawn off the bottom and skimmed off the top receive further treatment as sludge. The clarified wastewater flows on to the next stage of wastewater treatment. Clarifiers and septic tanks are usually used to provide primary treatment. (Ohioline, 2003)

As discussed in Section 2.1, chemically enhanced primary treatment therefore enhances conventional primary treatment and achieves significantly higher removal rates at lower costs compared to secondary treatment.

2.4.1 Removal Efficiencies

Table 2-6 below is a summary of expected removal rates in conventional primary, chemically enhanced and secondary wastewater treatment (NRC, 1996). These removal rates, graphed in Figures 2.8 and 2.9 below, and coupled with the financial estimates for the three treatment alternatives in Table 2-7, are critical to determining CEPT as the most efficient and economical wastewater treatment option.

	TSS (%)	BOD (%)	TP (%)	TN (%)	FOG (%)
Conv. Primary	55	35	20	15	51
Conv. Primary + Secondary	91	85	30	31	98
CEPT	85	57	85	37	71

Table 2-6: Relative Removal Efficiencies

Table 2-6 above shows that CEPT achieves pollutant removal rates significantly higher than those achieved in conventional primary treatment. When secondary treatment is used to complement the conventional primary treatment measures, the removal rate of TSS is only 7% more efficient than CEPT. BOD removals also increase by approximately 33% when secondary treatment is used. However, since the main goal of chemically enhanced primary treatment is to produce an effluent that can be disinfected (Harleman, 2003) and since suspended solids are a limiting factor to disinfection as opposed to BOD (Harrington, 2003) then the higher BOD removals in secondary treatment are not a limiting factor to using CEPT. It is also important to note that since the CEPT effluent is usually discharged into the ocean or other tolerant water body after disinfection, the BOD removals become less of a limiting factor compared to phosphorous or suspended solids for example and the 57% removal rate achieved is therefore considered acceptable for specific discharge locations (Harleman, 2003).

Phosphorous removals in CEPT are almost three-fold those in secondary treatment and nitrogen removals are very comparable for both secondary and CEPT treatment alternatives.

Figures 2.8 and 2.9 below therefore show that chemically enhanced primary treatment achieves significantly higher removal rates compared to conventional treatment and is comparable to secondary treatment especially with regards to suspended solids removals.

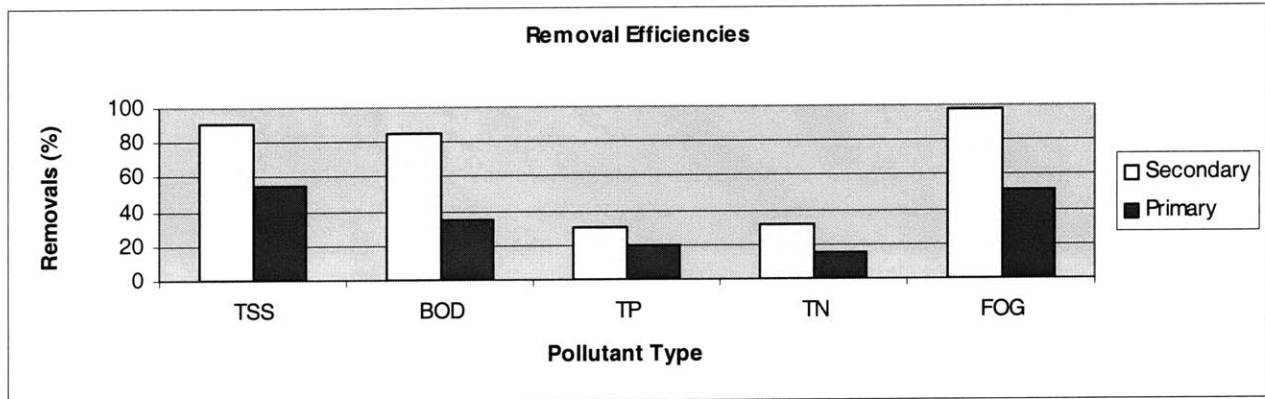


Figure 2.8: Secondary vs. Primary removal efficiencies

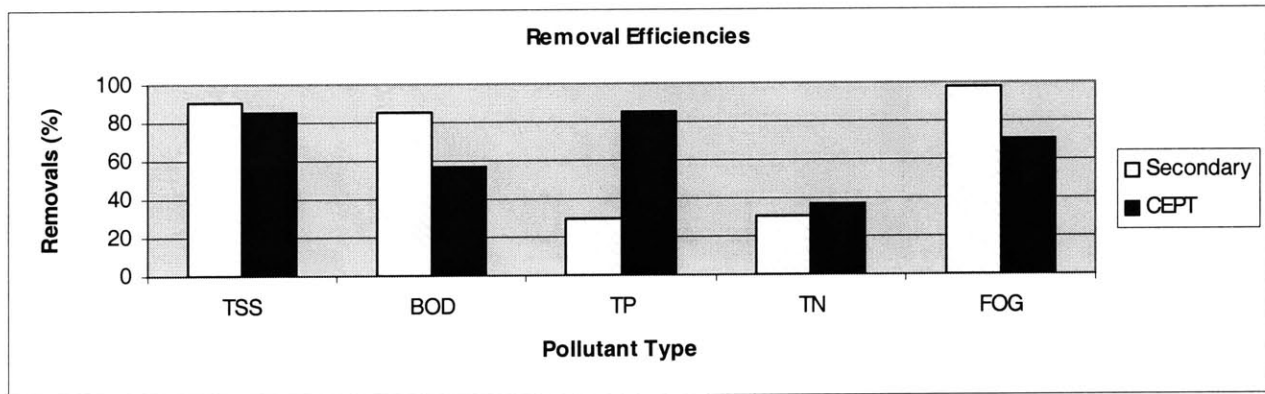


Figure 2.9: Secondary vs. CEPT removal efficiencies

2.4.2 Relative Costs

Table 2-7 below compares and contrasts CEPT, conventional primary and secondary treatment processes on a cost scale and proves that chemically enhanced treatment is a cheaper and more efficient alternative to reducing BOD and suspended solids prior to secondary treatment, or to using secondary treatment alone.

	Capital Costs \$/10 ³ m ³ .d ⁻¹	O&M Costs \$/10 ⁶ m ³	Total Costs \$/10 ⁶ m ³
Conv. Primary	3.1--4.2	0.8--0.9	1.7--2.1
Conv. Primary + Secondary	9.1--9.8	1.2--1.6	3.5--4.3
CEPT	4.2--5.3	0.9--1.1	2.1--2.6

Table 2-7: Relative Treatment Costs (NRC, 1996)

2.5 DISINFECTION

After leaving the settling tank, the clarified effluent is then disinfected to eliminate bacterial pathogens. Chlorination is the most commonly used disinfection process and will be discussed in greater detail in Chapter 6. Disinfection involves the addition of elemental chlorine or hypochlorite to the wastewater. When chlorine is used, it combines with water to form hypochlorous (HOCl) and hypochloric (HCl) acids. Hydrolysis goes virtually to completion at pH values and concentrations normally experienced in municipal wastewater applications (Metcalf and Eddy, 2002) Hypochlorous acid will ionize to hypochlorite (OCl) ion with the amount greatly affected by pH. However, in wastewater treatment the primary disinfectant species is monochloramine. Therefore the tendency of hypochlorous acid to dissociate to hypochlorite ion should be discouraged by maintaining a pH below 7.5 (Metcalf and Eddy, 2001).

Chlorine demand is determined by the difference between the chlorine added and the measured residual concentration after a certain contact time of usually 15-30 minutes (AWWA, 2001) The chlorine or hypochlorite is rapidly mixed with the wastewater, after which it passes through a detention tank, which normally contains baffled zones to prevent short-circuiting of the wastewater. The main limitation to the use of chlorine as a disinfecting agent is the potential formation of chlorinated hydrocarbons, some of which are known to be carcinogenic compounds. Chlorine gas is also a hazardous material and requires sophisticated handling procedures. The typical concentrations of chlorine required vary between 5 and 25 mg/L (Delaney, 2003).

This thesis will examine and prove the efficiency of using chemically enhanced primary treatment to produce an effluent that can be easily and efficiently. An example is taken from the Halifax wastewater treatment plant in Canada that will be upgraded by the use of CEPT (Civil Engineering News, 2002)

Figures 2.10 below shows how the three wastewater treatment plants to be built in Halifax will lower current bacteria levels in the harbor and keep them low. The left figure shows fecal coliform levels projected for the harbor in 2041 without treatment, and the figure on the right shows coliform levels with treatment. The white shades represent low E.Coli presences verses the highly pathogenic black shades:

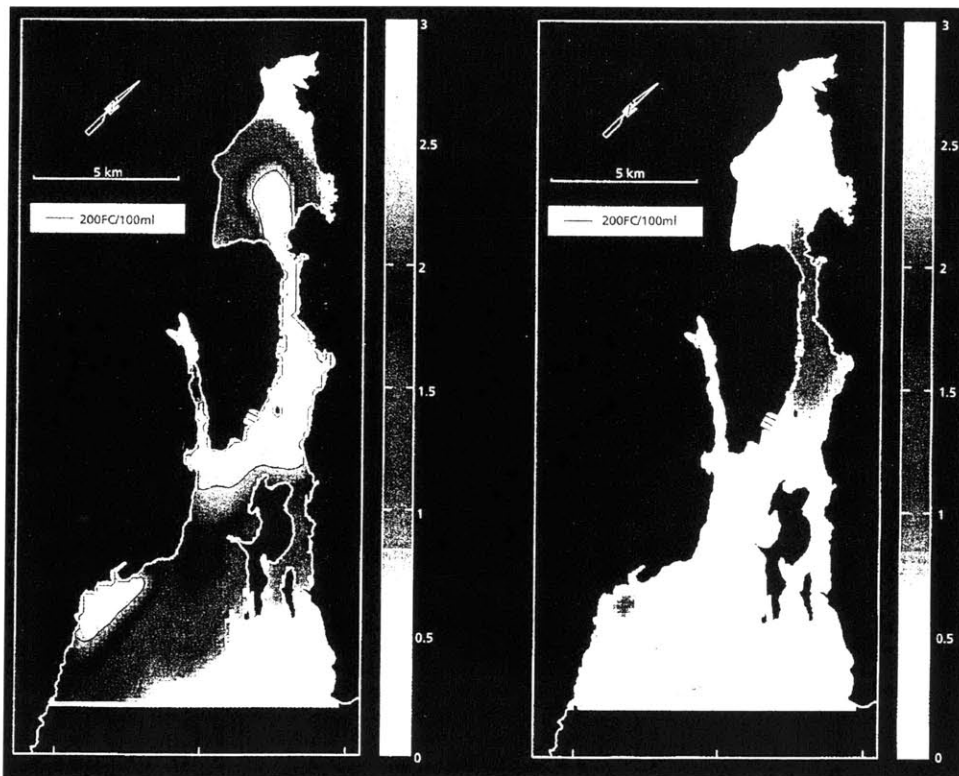


Figure 2.10: E.Coli levels in the Halifax Harbor with (right) /without (left) CEPT (Civil Engineering News, July 2002)

2.6 CEPT ADVANTAGES

The advantages to using chemically enhanced primary treatment revolve mainly around large reductions in the volume and concentrations of required chemicals, ecological effects downstream, maintenance and operation labor demands, all of which translate into substantial economic savings. CEPT also allows the sedimentation basins to operate at higher overflow rates, while still maintaining ideal removal rates of BOD and TSS at approximately 55 and 85% respectively (Refer to section 2.4.1 above). The footprint of the treatment plant's infrastructure can therefore be significantly smaller, reducing capital costs. Since CEPT can be easily used to retrofit already existing secondary treatment processes (such as activated sludge basins for example), and reduce the BOD and SS load entering the secondary treatment process, these latter units are therefore made smaller and more efficient. Also, the addition of metal salts and polymers only require the installation of injection valves from storage tanks.

2.7 SLUDGE PRODUCTION AND TREATMENT TECHNOLOGIES

Increased sludge production due to chemical addition has been one of the most common criticisms of the chemically enhanced wastewater treatment process. The focal goal of CEPT however is to remove more suspended solids and this inherently comes with an increased sludge volume. Sludge production is also not limited to the chemically enhanced process and plagues conventional primary and secondary treatment sequences as well. The sludge digestion processes used after secondary treatment are very expensive and contribute to significant capital, operation and maintenance costs and therefore pose another indirect disadvantage related to sludge production.

The dry weight per capita production of sewage sludge resulting from primary and secondary treatment is approximately 90 grams per day per person in most of the countries of the European Union where municipal communities are served by two stage physical, mechanical and biological processing plants (European Environment Agency, 1997). Sludge production therefore presents a large and impending problem at all levels of wastewater treatment including secondary treatment for example that contributes chemical precipitates and microorganisms to the sludge.

Many different techniques exist to handle and treat the sludge produced from wastewater treatment facilities. The agricultural use of raw sludge or other composting practices is encouraged by European national authorities as the best way to recycle, while incineration is considered the worst method of sludge treatment (European Environment Agency, 1997).

Sludge typically undergoes standard pre-treatment processes before it proceeds to advanced disposal and reuse processes. Common pre-treatment operations include dewatering, anaerobic stabilization, pasteurization and aerobic pretreatment. These processes are described in Figure 2.11:

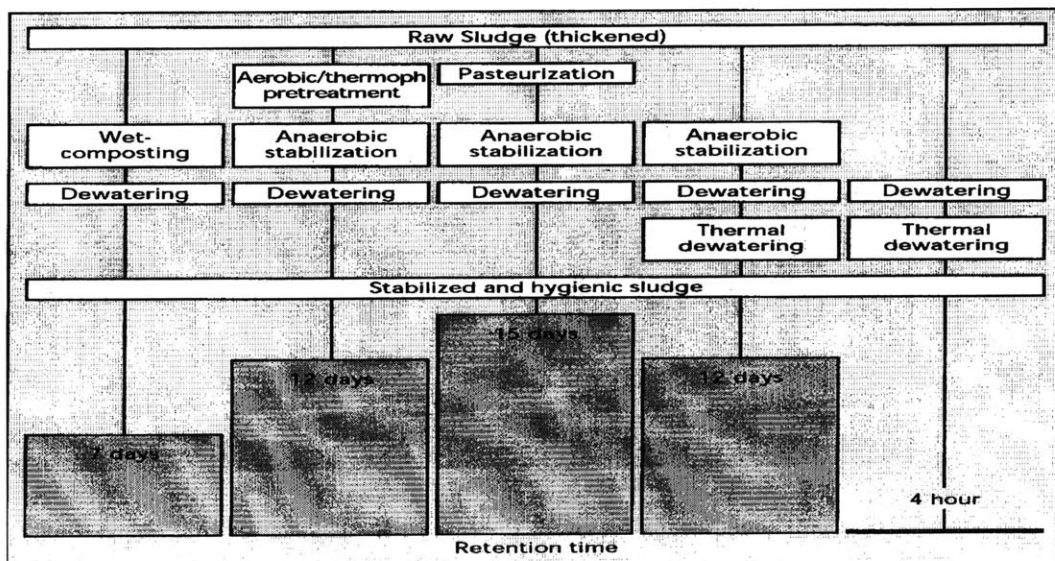


Figure 2.11: Sludge Pre-treatment options

Figure 2.11 is a summary of the several options for sludge treatment and disposal routes. Sections 2.7.1 through 2.7.6 will highlight the different fates that sludge has (as depicted in Figure 2.12) and will expand on the conditions, advantages and disadvantages of each process.

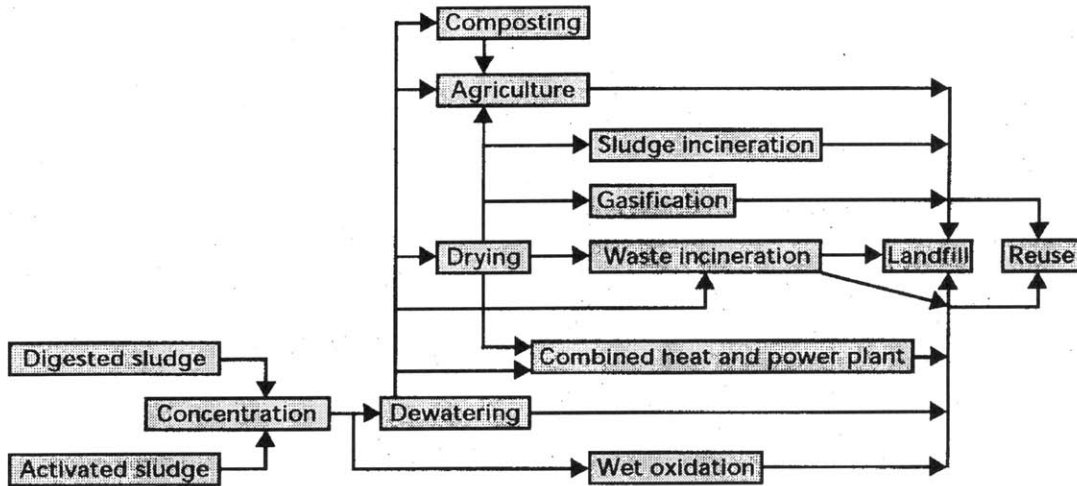


Figure 2.12: Sludge Treatment Options

2.7.1 Agricultural Use

The main reason for using sludge as an agricultural fertilizer is to make use of its essential nutrients (mainly phosphorous and nitrogen) and to utilize organic substances for soil improvement. As such, almost all sludges can be used as agricultural sources of nutrients and organic substances as long as they conform to the heavy metal and nutrient concentration, pH and crop type controls and limitations.

The sludge is normally spread on farmland once or twice a year in connection with ploughing and seeding. Hence the maximum uptake of nutrients by the plants is obtained, thus leading to a reduced washout of the nutrients to the ground and surface waters (European Environment Agency, 1997).

The advantages to spreading sludge on farmland are mainly:

1. Utilization of nutrients contained in the sludge (mainly phosphorous and nitrogen).
2. Utilization of organic substances contained in the sludge for the improvement of the humus layer of the soil
3. Often the cheapest disposal route

The disadvantages to using sludge as an agricultural resource however are the following:

1. Major investments in storage facilities since sludge can only be spread a few times a year.
2. Potential impact of micro-pollutants and pathogenic organisms on the food chain

It is important to note that by applying sludge from a wastewater treatment plant, one always runs the risk of introducing excess concentration of potentially toxic elements into the soil. These parameters were qualified by the Food and Agriculture Organization of the United Nations and are summarized in Table 2-8 below:

Potentially toxic element (PTE)	Maximum permissible concentration of PTE in soil (mg/kg dry solids)				Maximum permissible average annual rate of PTE addition over a 10 year period (kg/ha) ³
	PH 5.0 <5.5	pH 5.5<6.0	pH 6.0-7.0	PH > 7.0	
Zinc	200	250	300	450	15
Copper	80	100	135	200	7.5
Nickel	50	60	75	110	3
Cadmium	35				0.15
Lead	300				15
Mercury	1				0.1
Chromium	400				15 (provisional)
Molybdenum	4				0.2
Selenium	3				0.15
Arsenic	50				0.7
Fluoride	500				20

Table 2-8: Max Permissible Conc. of potentially toxic elements in soil after application of sewage sludge and max. annual rate of addition (www.fao.org)

2.7.2 Composting

Sludge composting aims at biologically stabilizing sludges in order to develop agricultural outlets that exploit the nutrient or organic value of sludges. Composting is also used to digest sludge and involves the aerobic degradation of organic matter as well as a potential decrease of the sludge water content, the efficiency of which depends on the composting efficiency (European Environment Agency, 1997).

Sludges can be composted if they have sufficient organic matter as well as relevant water content. As a general reference, the water content of a compostable mixture of organic wastes should be around 55% while the organic matter content should be greater than 70%, which facilitates effective bio-degradation. High moisture content above 60% reduces the temperature, porosity and thus the oxygen concentration while low moisture content, below 50%, could limit the rate of composting.

A balance of nitrogen and carbon content is necessary for the proper growth of microorganisms. Typical C/N ratios are between 25 and 30. (European Environment Agency, 1997)

Figure 2.13 below depicts a typical in-vessel sludge compostester:

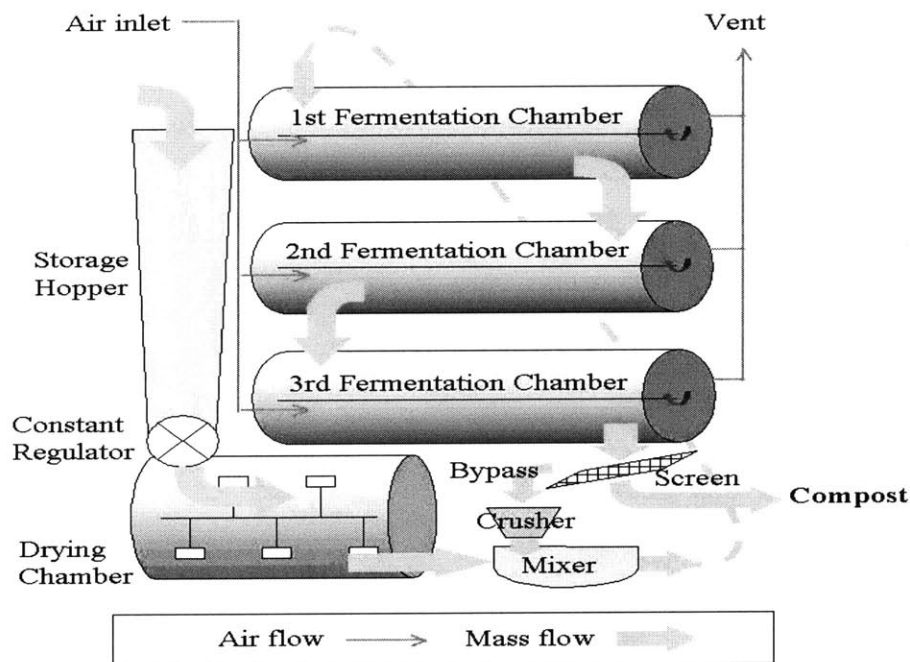


Figure 2.13: In-vessel Sludge Composter Process (<http://www.emc.or.kr/english/koetv>)

The advantages to composting sludge are:

1. Reductions in the volume of sludge to be transported to agricultural fields for example.
2. Easier storage and spreading capabilities.
3. Control of compost materials which leads to a more stable end-product

The disadvantages however are:

1. Higher treatment costs compared to direct sludge application to agricultural fields.
2. High energy costs of aeration
3. Need for an outlet market for the compost products.

2.7.3 Drying

The general flow sheet for a typical sludge drying process is shown in Figure 2.14 below:

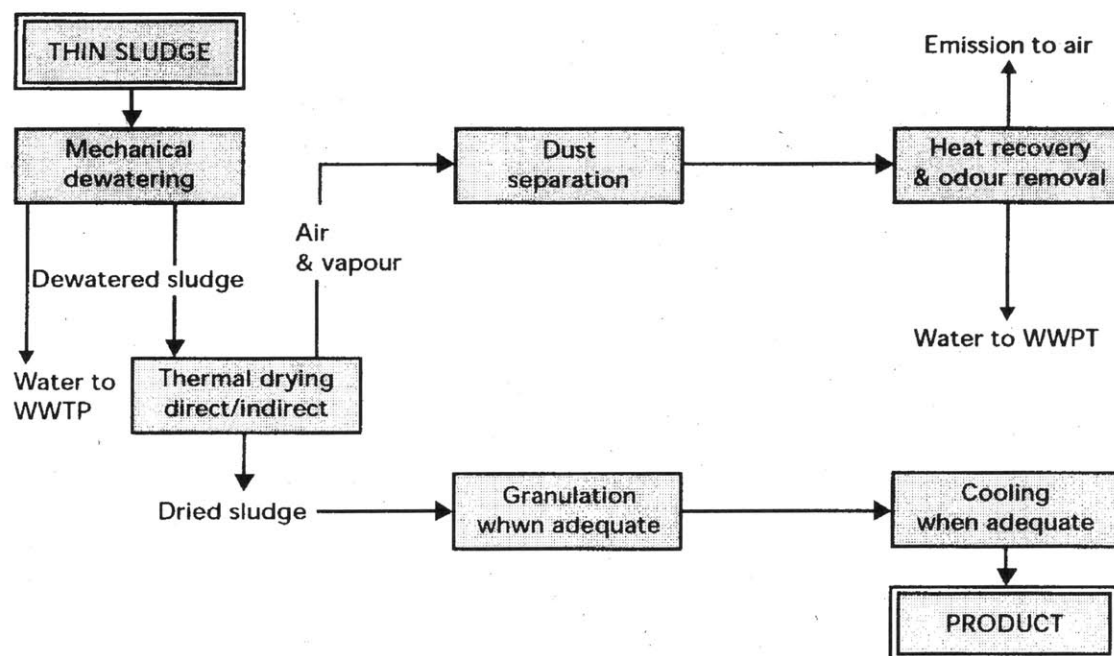


Figure 2.14: Typical Drying Process Diagram (European Environment Agency, 1997)

The two distinctly different drying methods are indirect and direct drying. In direct driers, there is a direct contact between the sludge and the heated gas supplying the required heat for evaporation and simultaneously carrying the water vapor formed out of the system. In indirect driers however, heat is transferred to the material to be dried indirectly by heat conduction through a heat transfer surface (European Environment Agency, 1997).

A drying plant that, in most cases, includes granulation is more expensive to install compared to mechanical methods such as pressing and centrifugation. Prior to drying, proper mechanical dewatering must therefore be installed. The greatest advantage to having sludge in a dry form as compared with various other methods, is the possibility of marketing the product for a number of applications including fertilizer/soil conditioners in agriculture and forestry, fuel in power plants and incinerators, as well as top soil, landscaping, landfilling and disposal (European Environment Agency, 1997).

2.7.4 Incineration

15% of Europe's sludge is currently incinerated (European Environment Agency, 1997). Since the agricultural uses of sludges, by direct application, as well as sludge landfilling are subject to increasingly stringent regulatory control, the incineration of sludges has been expected to gain some popularity even though it can be a capital intensive investment and is also subject to strict regulation pertaining to combustion criteria, management of the off-gas treatment residues and treatment of fly and bottom ashes.

Incineration of sludges is performed in designated incinerators or in municipal solid waste incinerators under specific constraints for each type, where the process results in the combustion of the sludge's organic matter. After pre-drying, sludges can also be incinerated in cement kilns because they have a high calorific value (European Environment Agency, 1997).

These methods of sludge treatment are only economical however for large volumes of sludge (2.5 tons of evaporated water per hour) and that are not appropriate for agricultural application.

It is also important to note that Japan has some experience with the vitrification of sludge. This process however remains very expensive and is therefore not considered, as of yet, a feasible sludge treatment solution.

The advantages to incinerating sludge are: (European Environment Agency, 1997)

1. A significant reduction in sludge volume, after incineration
2. Energetic valorization of sludges
3. Recycling of sludge treatment sub-products such as ashes and inert material that can be used in filler material for asphalt, concrete production, and in brick fabrication.
4. Low sensitivity to sludge composition

5. Reliable systems
6. Odor minimization due to closed systems and high temperatures

The disadvantages however are: (European Environment Agency, 1997)

1. Incinerators are capital intensive and usually justified only in larger volume situations.
2. With co-incineration, the treatment capacity and treatment efficiency depend on the saturation of the incinerator by other solid waste streams and/or the ratio of sludge mass to solid waste mass.

2.7.5 Landfilling

Since sludges are considered infectious materials and contain large concentrations of organic material (fat, proteins and carbohydrates) that are biodegradable, putrescible, and cause odor problems, it is of critical importance that sludges be stabilized.

Sludges are classified as stabilized when they have undergone either aerobic or anaerobic stabilization processes or have been chemically treated, which includes a liming step. The addition of lime to the sludge for stabilization theoretically results in a better disinfection efficiency (Metcalf and Eddy, 1991), compared to anaerobic digestion for example. The disinfection effect of aerobic stabilization is uncertain in that respect. Thermal aerobic stabilization processes are also used for pathogen removal and this system is considered to be much more efficient in that respect compared to other previous systems (European Environment Agency, 1997).

In smaller plants, sludge-drying beds are also popular, but mechanical dewatering is becoming more and more widespread (European Environment Agency, 1997). As a result of the mechanical dewatering, the original dry material content (2-3%) of the liquid sludge is increased to 20-30% that describes a sludge that can already be shoveled into a landfill. Dewatering machines require chemical preconditioning or treatment of the sludge, usually with lime. Stabilized, dewatered sludge always contains pathogenic microorganisms that have to be taken into account. Lime treatment can however increase the pH of the sludge up to values of pH = 12, but the inactivation effect on the pathogens is only temporary (European Environment Agency, 1997).

2.7.6 New Technologies: Gasification and Wet Oxidation

The processes of sludge gasification and wet oxidation are very new sludge-treatment technologies for which detailed information and data is not very readily available. They will nonetheless be briefly mentioned.

Gasification is a thermal process where a feedstock containing combustible material is converted with air (sometimes with oxygen or steam) to an inflammable gas. The most commonly used reactors for gasification are the fixed bed reactor, the fluid bed reactor, and the circulating bed reactor (European Environment Agency, 1997)

In wet oxidation, the organic content of sludge is oxidized in specific reactors at temperatures varying from 200 to 300 degrees Celsius and at pressures between 30 and 150 bar. The main output of the wet oxidation process is a sludge containing more than 95% of mineral components and less than 3% of low-molecular organic substances. The sludge is dewatered (typically using a belt filterpress) and then recycled or landfilled. (European Environment Agency, 1997).

2.7.7 Sludge Decision Making Tree

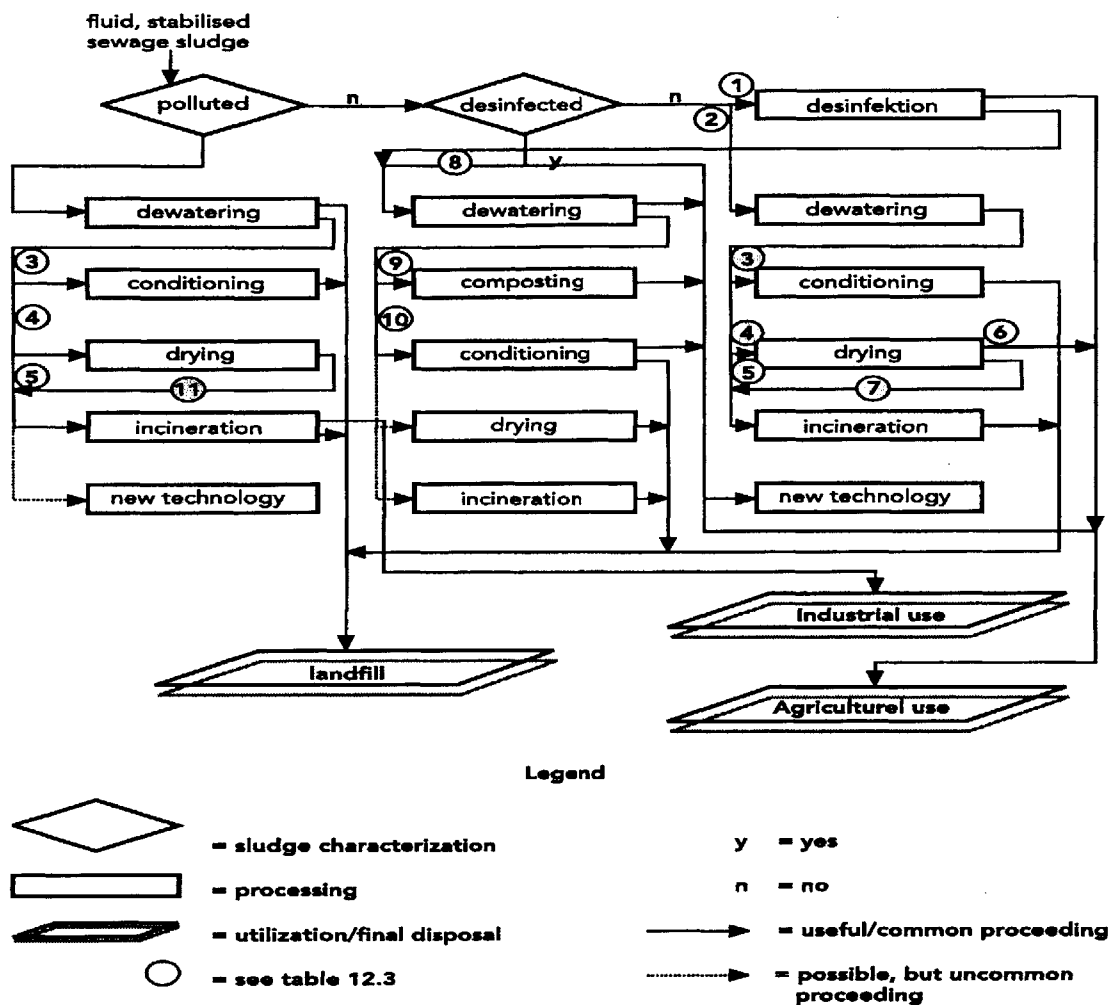


Figure 2.15: Sludge Decision Making Tree (European Environment Agency, 1997)

Figure 2.15 above is a suggested flow diagram to follow in the decision-making process concerning sludge management technologies. It classifies sludge management technologies according to the nature of the contaminants in the sludge

Table 2-9 below accompanies Figure 2.15 and is an explanation of the numbers in the decision tree:

Number	Conditions Influencing Sludge Decision
1	Utilization of thin and/or dewatered sludge in agriculture is possible
2	No use in agriculture or at the most as dry granular sludge
3	Mechanical properties have to be improved for intermediate storage, transportation or landfilling
4	Landfilling is not desired
5	Incineration in a waste incinerator or similar furnace allowing input of dewatered sludge. Limits and variations of dry substance content after dewatering have to be controlled.
6	Granular spreading in agriculture, due to seasonal storage of dried sludge or to non-acceptance of other types of sludges by farmers.
7	External valorization of dried sludge as a fuel or disposal of surplus stock of dried sludge or mixing of dried sludge with dewatered sludge in order to reach input limits of dry substance for the furnace.
8	Application of thin (non-dewatered) sludge in agriculture
9	Green waste from gardens or other compostable waste is available for mixing with sludge. Utilization of compost, e.g. as a soil conditioner, is possible.
10	Mechanical properties have to be improved for intermediate storage, transportation or application in agriculture. Landfilling of surplus sludge.
11	External valorization of dried sludge as a fuel or mixing of dried sludge with dewatered sludge in order to reach input limits of dry substance for the furnace

Table 2-9: Conditions Influencing Sludge Decisions

CHAPTER THREE: CASE STUDIES ON CEPT

Chemically enhanced primary treatment (CEPT) of wastewater is not a new technology. The use of chemicals to increase the level of coagulation and flocculation in municipal wastewater was widespread in England as early as the late nineteenth century (Harleman, 2002). At the time however, large chemical doses were used and these consequently increased the volumes of sludge produced in the treatment process. Groundbreaking research has been performed however on increasing the efficiency of chemically enhanced primary treatment with the use of smaller volumes of coagulants and stronger synthetic polymer aids. The case studies presented in the sections below are a collection of the relatively recent studies performed on CEPT in addition to a summary of the removal efficiencies reported by functioning CEPT plants.

The following are examples of current CEPT plants around the world and highlight the different aspects and advantages of using chemicals in the preliminary treatment of wastewater.

3.1 Point Loma Wastewater Treatment Plant in San Diego: The Point Loma Outfall

3.1.1 Introduction

The Point Loma Wastewater treatment plant in San Diego is typically considered the landmark case used to prove the effectiveness of CEPT (Harleman, 2002). When operators were faced with strict effluent requirements in 1985, they retrofitted the treatment plant by using potable water treatment schemes and adding various doses of metal salts and polymers to the influent in the primary sedimentation basin. Increased removal efficiencies at three times the design overflow rate with minimal amounts of additional chemical sludge were consequently reported, certifying the efficiency of CEPT. (Hansson et al, 1994). The mean annual percent removal of total suspended solids remains at 80% and the 5-day biochemical oxygen demand removal is 58% on a mean annual basis. (City of San Diego, 1995). These removal rates are significantly higher than the 60% TSS and 30% BOD removals typically expected in conventional primary treatment.

The key parameters to the Point Loma design are summarized in Table 3-1 below:

Flow	190 MGD
Peak Flow	240 MGD
Dry Base FeCl₃	40 ppm
40% Liquid FeCl₃	100 ppm
Anionic Polymer	0.2 ppm
Surface Load	2 m/h
Peak Surface Load	3 m/h
Total Retention	2 hours
Avg. Sludge Density	3% dry solids
Digester Retention	15 days

Table 3-1: Design Specifications at Point Loma WWTP

Jar tests were also conducted as part of a preliminary study designed to test the efficiency of using ferric chloride as a coagulant, with the added efficiency of an anionic polymer (Hansson and Langworthy, 1994) The results from the jar tests are presented in Table 3-2 below and prove the efficiency of upgrading the conventional primary treatment plant to a chemically enhanced treatment facility. Close inspection of the economic analyses of the data shows that adding chemicals is a cost-efficient method since the cost to turbidity removal ratio increases steadily with the addition of small doses of chemicals.

Product	Dose Ml/m³	Turbidity NTU	Removed Turbidity	Cost \$/m³	Cost/Rem. Turb.
Blank	0	145	0	0	0
Ferric Chloride	32	62	83	0.0036	8.59
	64	26	119	0.0059	13.11
	96	23	122	0.0107	22.46
	128	17	128	0.0143	28.92
	160	14	131	0.0178	36.37
PAC poly-aluminum chloride	24	38	107	0.006	56.07
	48	9.4	135	0.012	88.52
	72	6.4	139	0.018	129.87
	96	5.2	140	0.024	171.67
	120	4.2	141	0.03	212.92

Table 3-2: Point Loma Jar Test Results

3.1.2 Key Issue: The Point Loma Outfall

The Point Loma outfall is an essential component of the treatment and discharge system. Treated wastewater is discharged to ocean waters at a depth of approximately 310 feet through the 23,472-foot-long Point Loma ocean outfall. The outfall was extended in 1993 by approximately 12,246 feet and discharges through a 4992-foot long diffuser with 416 ports (City of San Diego, 1995).

Discharging treated effluent through a carefully engineered outfall must meet the following constraints: (City of San Diego, 1995).

1. Not adversely affect recreation or other beneficial uses of the ocean waters
2. Not adversely stimulate phytoplankton growth to the point of creating an aesthetic or nuisance condition.
3. Not alter the balance or diversity of benthic species to a biologically significant degree outside the zone of dilution
4. Not introduce concentrations of toxic compounds into the ocean which could be toxic to humans, mammals, fish, or other marine species and
5. Not significantly reduce water clarity outside the zone of initial dilution or significantly reduce dissolved oxygen in the water column

Many tests are constantly relied upon at Point Loma to monitor the discharges from the outfall and its compliance with Ocean Plan Standards and Federal Criteria. As such, common tests used to prove the inoffensive effects of outfall discharges on the surrounding ocean environments are tests on dilution, physical oceanography, dissolved oxygen, and marine biology.

3.2 Sao Paulo –Tests at the Ipiranga Facility: The Retrofitting Capabilities of CEPT

3.2.1 Introduction

The full scale and jar tests that were performed at the Ipiranga Facility serving the Greater Sao Paulo area were primarily centered on proving that the existing and deteriorating WWTP facility could be retrofitted using CEPT. The results from the full-scale test are presented in section 3.2.2 below.

Greater Sao Paulo, the largest city in South America, has a population of approximately 17 million distributed in 339 municipalities (www.brazil.com). Data from the period of 1993 to 1996 show that Ipiranga's primary sedimentation basins were achieving removal efficiencies of only 20% of influent TSS in contrast to the 60% expected from a well-operated plant. BOD and COD removals were similarly low, at 20 and 30% respectively (Harleman, 2002).

3.2.2 Full Scale Tests Results

The results from the Ipiranga full-scale tests are presented in Table 3-3 below. These are consistent with the removal efficiencies described in San Diego's Point Loma:

Ferric Chloride Dose mg/L	Polymer Dose mg/L	Flow Rate L/sec	COD Rem. %	BOD Rem. %	TSS Rem. %
0	0	25	34	37	52
0	0	50	27	28	36
25	0.5	50	45	44	50
50	0.5	50	52	52	64
25	0.25	50	58	60	52
50	0.25	50	63	62	69
50	0.5	50	62	58	80

Table 3-3: Full Scale Results from Ipiranga Tests (Sao Paulo Brazil)

Table 3-3 above shows the positive effects of adding chemicals to the influent. When 0 mg/L Ferric chloride was used to treat the wastewater, the COD and TSS removals were 34 and 52% respectively. When 50 mg/L Ferric chloride was added with 0.5 mg/L polymer, the removals increased to 52 and 64% respectively.

3.3 Rio de Janeiro: Phosphorous Removal in CEPT

Phosphorous and nitrogen are commonly present in WWTP discharges and are the limiting nutrients in algal growth. Guanabara Bay in Rio de Janeiro suffers from extensive environmental contamination and algal growth due to high nutrient inputs (Harleman, 2002). Discharge of untreated industrial and residential wastes has resulted in high coliform levels, eutrophication problems, and low dissolved oxygen in the surrounding waters (Harleman, 2002). Therefore the WWT facility near the Bay was chosen as a test site to explore using CEPT in future new plants.

The full-plant test performed in Rio de Janeiro was divided into two streams; one that was fed to a conventional settling tank and the other was routed through a parallel sedimentation tank with the addition of metal salts and polymers (Harleman, 2002). As the testing period was too short to report significant amounts of data, the general conclusions from the full-scale tests were that using ferric chloride in doses between 35 and 59 mg/L almost doubled the removal efficiencies of TSS and BOD; In contrast to unaided primary settling which achieved maximum removal rates of 43% of the influent TSS, 44% of the BOD and 29% of the COD, the chemically dosed

streams reported levels as high as 76% TSS removals, 75% BOD and 65% COD (Harleman, 2002).

Figure 3-1 below depicts the locations of the treatment plants in Rio de Janeiro that were considered for CEPT's retrofitting capabilities.

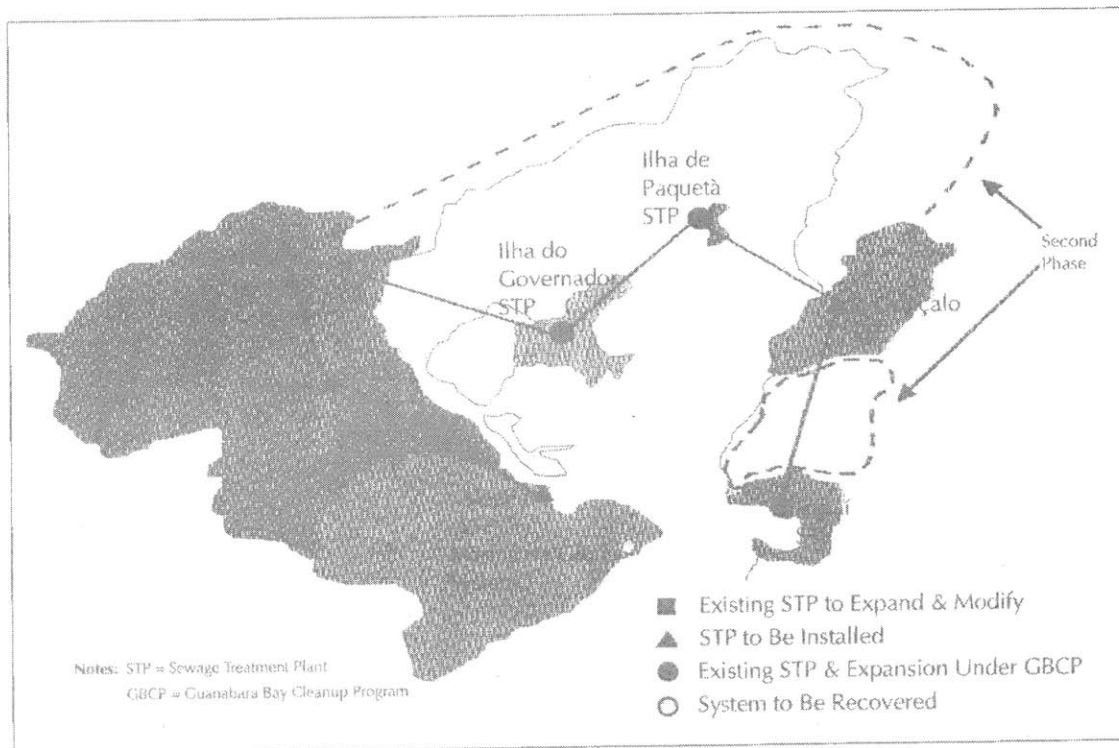


Figure 3-1: Rio de Janeiro Plants

3.4 Seawater and CEPT

3.4.1 Seawater in Hong Kong

The Stone Cutter's Island plant in Hong Kong is the world's largest and most efficient chemically enhanced primary treatment plant. It was placed in operation in July 1997 and has a maximum capacity of 40 m³/sec.

The plant was originally designed for conventional primary treatment with expected removals of 55% SS and 30% BOD (Harleman, 2003). Following recommendations of an International Review Panel in 1995, the design was changed to a CEPT plant (Harleman, 2003). The number of settling tanks was reduced from the original 58 to 38; the smaller plant was made possible by the more rapid settling of particles after coagulation by ferric chloride.

The average performance data for the years between 1997 and 2000 are tabulated in Table 3-4 below:

Flow rate	3.7 m ³ /sec
Surface Overflow Rate	66 m/d
Influent BOD	156 mg/L
Effluent BOD	39 mg/L
BOD Removal Efficiency	75%
Influent SS	200 mg/L
Effluent SS	32 mg/L
SS Removal Efficiency	84%
Ferric Chloride	10 mg/L
Anionic Polymer	0.1 mg/L
Seawater by volume	20%

Table 3-4: Hong Kong Stonecutter's Island Performance Data

The use of seawater in chemically enhanced primary treatment is a new phenomenon that has not been intentionally included in any CEPT plants as of yet. The use of seawater in Hong Kong's Stonecutter's Island was not by choice since the sanitary systems are flushed with seawater so as to avoid buying fresh water from Mainland China. The 20% seawater (by volume) was therefore already present in the wastewater influent and had to be accounted for in the design. The presence of the 20% seawater in the influent is believed to have caused the reduction in ferric chloride demand from an average 40 mg/L in other CEPT plants to the 10 mg/L used at Stonecutter's Island.

Sections 3.4.2 to 3.4.5 below are a collection of the available literature review on the use of seawater as a coagulation enhancement mechanism in wastewater treatment. Many of the studies described below focus strongly on the use of lime in wastewater treatment and then look at the combined effect of seawater. Although this research-focus is not recommended due to the large volumes of chemical sludge produced with the addition of lime (Harleman, 2003), the use of seawater certainly enhances treatment performances and these studies can therefore be considered catalysts for future research. It is also important to note that the use of seawater was very carefully studied and considered for use in Paraty. Detailed data analysis is described in Chapters 4 and 5.

3.4.2 Bagot (1990): Chemical Treatment of Sewage: Experiences in San Francisco

In early March of 1970, the California Regional Water Quality Control Board called for immediate improvements in the sewage collection and treatment facilities belonging to the City and County of San Francisco. An immediate upgrading of the primary-type treatment by the use of one or more of the synthetic organic chemical polymers was favored by both the city and the state. (Bagot, 1990) The North Point Plant was chosen to test the idea of chemical addition and Figure 3-2 is the flow diagram of the North Point Plant process.

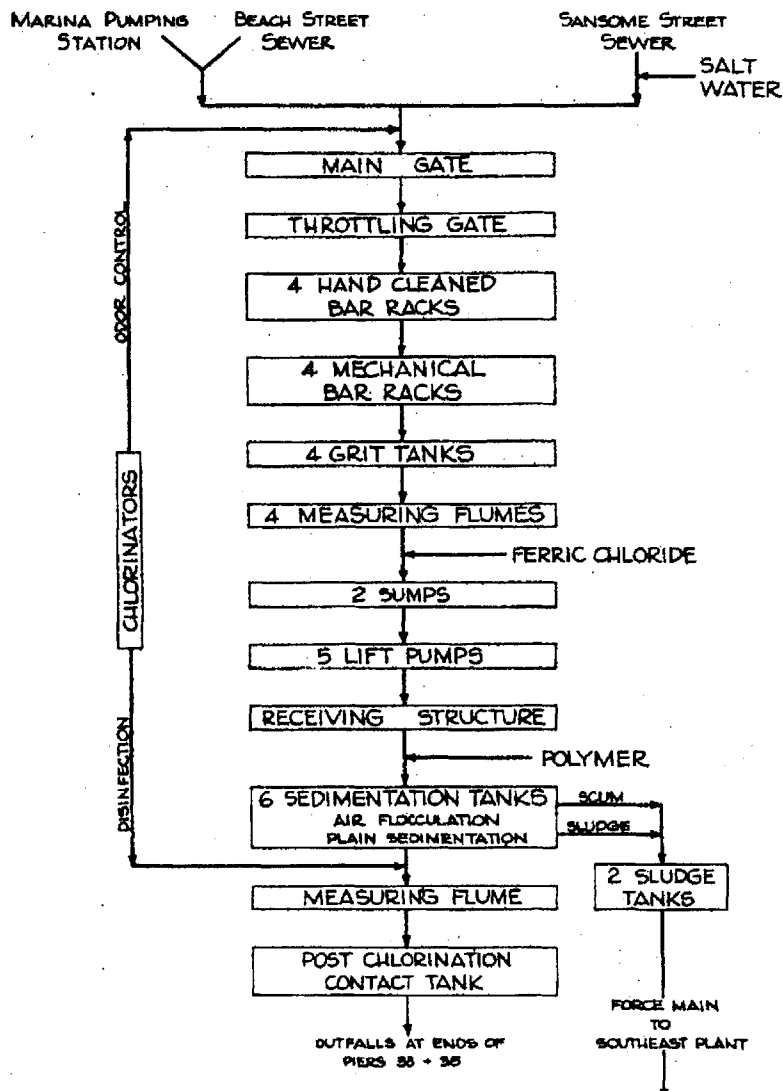


Figure 3-2: North Point Treatment Process

Several doses and different types of polymers were used with varying doses of ferric chloride in the North Point initial tests. In April 1971, the following feed program had been proposed:

Hours	Ferric Chloride	Anionic Polymer
12:00 am – 7:00 am	15 mg/L	
7:00 am – 5:00 pm	25 mg/L	0.4 mg/L
5:00 pm – 12:00 am	40 mg/L	0.4 mg/L
10% Bay Water added on a continuous basis.		

Table 3-5: North Point CEPT Feed Program

The qualities of effluents derived from different types of treatment at North Point are summarized in Table 3-6 below. It is very important to notice that the behavior of ferric chloride was influenced by the salt content of the raw sewage. Variations of the salt concentration are caused by infiltration of tidal water into the sewer system (Bagot, 1990)

Type of Treatment	SS (mg/L)	BOD (mg/L)	COD(mg/L)	Grease (mg/L)
Sedimentation	78	164	X	33.1
Sedimentation and Polymers	72	164	X	28.7
15 mg/L FeCl₃ + 10% salt water plus 0.4 mg/L cationic polymer	45	120	166	22.2

Table 3-6: Effluent Qualities in North Point for varying treatment schemes (Bagot, 1990)

The use of salt water yielded better results and as such, further tests were conducted on the use of saltwater to enhance the chemical treatment of wastewater in San Francisco. Bay water was tested in percentages varying from 0-10% by volume (Chloride ion concentrations ranged from 140 mg/L to 1880 mg/L) and were all tested with 10 mg/L of ferric chloride (Bagot, 1990). The removal efficiencies from these tests are tabulated in Table 3-7 below:

% Bay water by vol.	FeCl ₃ mg/L	Chlorides mg/L	SS mg/L
Scheme A: With Ferric Chloride			
0	10	140	50
1	10	316	43
2	10	476	33
4	10	761	33
6	10	1100	28
8	10	1430	34
10	10	1880	28
Scheme B: Without Ferric Chloride			
0	0	125	41
1	0	295	51
2	0	436	41
4	0	750	54
6	0	1060	50
8	0	1330	51
10	0	1840	77

Table 3-7: Effluents in Salt Water treatment with/out FeCl₃ (Bagot, 1990)

The pH values used in these tests were consistent at approximately 7.2. It is obvious that large volumes of seawater did not necessarily increase the removal efficiencies of suspended solids since 6% seawater performed as well as 10% seawater for example (Refer to Table 3-7 above). Finally, Table 3-8 of the following page provides a comparison of various chemical treatment schemes on North Point Raw sewage and is efficient at proving the effectiveness of using saltwater in small concentrations to enhance chemical treatment of influent.

Table 3: Comparison of various chemical treatment schemes on North Point raw sewage				
Ferric Dose (mg/l)	Suspended Solids (mg/l)	Light Absorption (700 mμ)	% Light Transmission (700 mμ)	pH
Scheme A: Ferric Chloride + 10% Water				
0	61	0.135	73.0	7.30
5	57	0.105	78.0	7.20
10	37	0.095	80.5	7.15
20	33	0.080	83.0	7.10
25	32	0.077	83.5	7.00
30	35	0.075	84.5	6.90
40	37	0.063	86.5	6.80
Ferric Dose (mg/l)	Suspended Solids (mg/l)	Light Absorption (700 mμ)	% Light Transmission (700 mμ)	pH
Scheme B: Ferric Chloride + 10% Salt Water				
5	40	0.070	85.0	7.20
10	22	0.055	88.3	7.10
20	21	0.043	90.5	7.00
25	22	0.040	91.5	6.95
30	22	0.037	92.0	6.90
40	19	0.030	93.5	6.80
Ferric Dose (mg/l)	Suspended Solids (mg/l)	Light Absorption (700 mμ)	% Light Transmission (700 mμ)	pH
Scheme C: Ferric Chloride + 10% Water + 0.5 mg/l Polymer				
5	22	0.097	80.0	7.15
10	18	0.095	81.0	7.15
20	21	0.072	85.0	7.10
25	17	0.071	85.0	7.10
30	22	0.065	86.0	7.00
40	23	0.052	89.0	6.85
Ferric Dose (mg/l)	Suspended Solids (mg/l)	Light Absorption (700 mμ)	% Light Transmission (700 mμ)	pH
Scheme D: Ferric Chloride + 10% Salt Water + 0.5 mg/l polymer				
5	40	0.062	86.5	7.10
10	28	0.050	89.0	7.10
20	24	0.040	91.5	7.00
25	15	0.035	92.5	7.00
30	20	0.030	93.0	6.95
40	16	0.025	95.0	6.80

Table 3-8: Various chemical treatment schemes on North Point

3.4.3 Ferguson and Vrale (1984): Seawater in wastewater treatment with lime

Ferguson and Vrale (1984) looked closely at the use of lime and seawater in wastewater treatment. The use of lime is not often recommended because of the large quantities of chemical sludge that incur (Harleman, 2003). However, a brief description of the findings is useful (Ferguson and Vrale, 1984):

1. "Seawater adds enough magnesium to precipitate at least 0.6 millimoles/L of magnesium hydroxide. The amount of seawater needed depends on the solubility of magnesium hydroxide that is a function of pH. As little as 1 to 2% of seawater is needed if pH values above 11 are used; if pH values are below 10.5, 10% or more seawater is required."

2. "Phosphate solubility is seldom limiting in the process, but can be described as a function of pH and calcium concentration. At pH values below 10.5, orthophosphate is a significant portion of the total phosphate"
3. Sludge recycle improves the steady state performance of the process.
4. "The lime seawater process is capable of producing an effluent suitable for marine discharge if no more than 75% soluble BOD removal is required. Removal of suspended solids and phosphorous is characteristically greater than 90%. The effluent is very clear."

The Ferguson and Vrale study can be considered somewhat misleading because the large expected quantities of sludge produced from the addition of significant concentration of lime (varying from 165 to 240 mg/L) are not very clearly addressed. However, the efficiency of seawater at enhancing the process provides other researchers with encouraging data.

3.4.4 Ayoub and Koopman (1986) and Ayoub et al (1986): Seawater and Algae

The Ayoub and Koopman (1986) study focused on determining the effectiveness of the lime-seawater process in the removal of algae from oxidation pond effluents. The second study was identical except that lime was substituted by sodium hydroxide to evaluate the effectiveness of the process at reduced Ca^{2+} ions. The main conclusion of interest concerning seawater is as follows:

"The effectiveness of seawater is largely a function of the extent of the $\text{Mg}(\text{OH})_2$ reaction. In the presence of ample Mg^{+2} ions the optimal lime dosage required will supply enough OH^- ions to react with the Mg^{+2} ions"

As the lime process is not typically encouraged, it is important to note that in chemically enhanced primary treatment with the use of seawater, the naturally abundant Mg^{+2} ions will react with the wastewater particles that are negatively charged therefore replacing the negatively charged OH^- ion.

3.4.5 Ayoub et al (1991): Seawater as a demulsification agent.

Laboratory investigations were conducted to explore additional applications of seawater flocculation and to gain a better understanding of the conditions under which effective flocculation is achieved. The pollutants tested included emulsified oil, high alkaline industrial wastewaters and a pilot scale oxidation pond. The conclusions reached were as follows (Ayoub et al, 1991):

1. Seawater is effective for demulsification of oily wastewater. Flocculation of alkaline wastewaters may be accomplished merely by the addition of seawater
2. Seawater addition is also highly efficient for suspended solids and phosphorous removals as well as particulate forms of COD and nitrogen, and is also effective against textile dyes.
3. Seawater serves as a source of magnesium ions, which precipitate as $Mg(OH)_2$ at high pH. The ability to destabilize oil emulsion and particulates that are negatively charged, suggests that the $Mg(OH)_2$ floc carry a positive charge in the seawater-wastewater mixture. A minimum precipitate quantity of 2 to 3.5 g eq/m³ was required to achieve good flocculation.

These conclusions are encouraging despite the fact that the tests were applied to sources of wastewater that were not domestic. It can be implied however that further research on the use of seawater as a coagulation enhancement mechanism in the chemical treatment of domestic wastewater will yield comparable results. Please refer to Chapter 4 for data analysis on jar tests performed in Paraty, Brazil.

CHAPTER FOUR: PARATY ANALYSIS: FeCl₃, SEAWATER AND POLYMERS

4.1 INTRODUCTION

In January 2003, several jar test experiments were conducted to assess the sewage quality in the city of Paraty. The measured suspended solids (SS), turbidity and chemical oxygen demand (COD) removal rates were then used to estimate appropriate FeCl₃ and polymer doses and then to design a chemically enhanced primary treatment plant for Paraty. Seawater was also considered for use as a coagulant enhancement tool.

This chapter will then introduce the results from three sets of experiments that were conducted in the laboratory in Paraty. Each experiment is a collection of several comparable jar tests conducted on one raw sewage sample or on a sample of similar raw wastewater characteristics and to which ferric chloride, FeCl₃, was added either alone or with a combination of seawater and/or anionic polymer in assigned percent volumes. The use of seawater as a coagulation enhancement tool (Chapter 3 for literature review) was a critical examination point for the jar tests results. These experiments are summarized in Table 4-1 below and are very effective at comparing and contrasting the effect of FeCl₃ and seawater on the SS, turbidity and COD removal efficiencies and are therefore critical at determining the optimal coagulant, seawater and polymer dose required for the proposed CEPT plant in Paraty.

Experiment Number	Jar Test Number	Description	Page
1			54
1A	2 and 6	FeCl ₃	54
1B	4, 5 and 8	FeCl ₃	57
2			
2A	8,9 and 10	0.5% seawater and FeCl ₃	61
2B	8,11 and 12	1,2% seawater and FeCl ₃	64
2C	8, 25 and 26	5, 10% seawater and FeCl ₃	68
3			
3A		FeCl ₃ and Polymer	73
3B		FeCl ₃ and Seawater	76
3C		FeCl ₃ , seawater and polymer	78

Table 4-1 Summary of Experiments in Paraty, Brazil

4.1.1 Constraints

It is critical to note that the lack of a sewage collection system in Paraty made finding representative raw wastewater samples a challenging task. Also, once a sampling point in an open sewer was finally located, the continuous rain falls diluted the samples significantly thereby reducing the suspended solids and turbidity contents in the collected raw wastewater samples.

Please refer to figures 1.5 through 1.8 in Chapter 1 for examples of the state of wastewater discharge in Paraty.

Figures 4-1 and 4-2 below show the effect of dilution on SS and COD removals. As the sample is diluted, the influent SS and COD concentrations steadily decrease as do the SS and COD removals. The response of SS and COD removals to increasing SS and COD influent concentrations are compared to the South Essex treatment plant (Harleman, 2003):

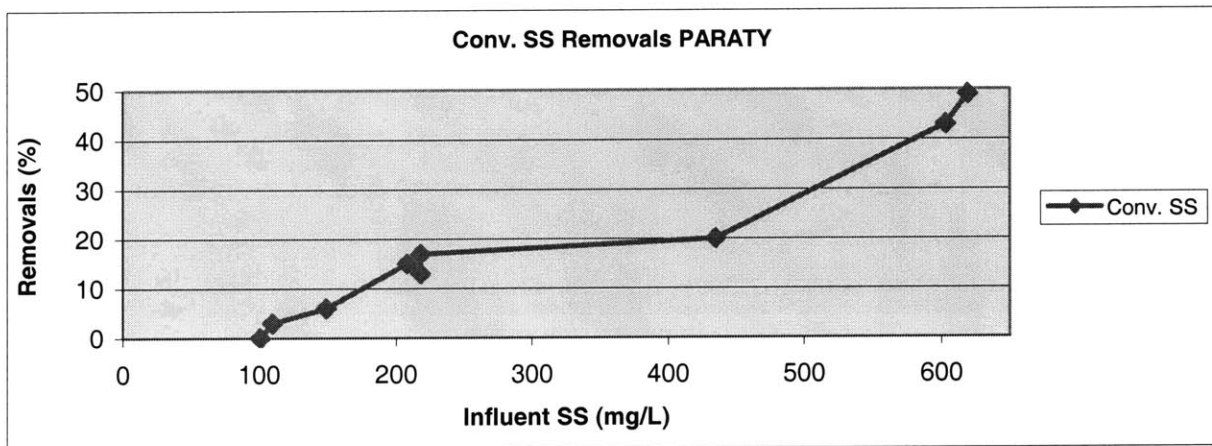


Figure 4-1: SS removals with increasing influent concentration

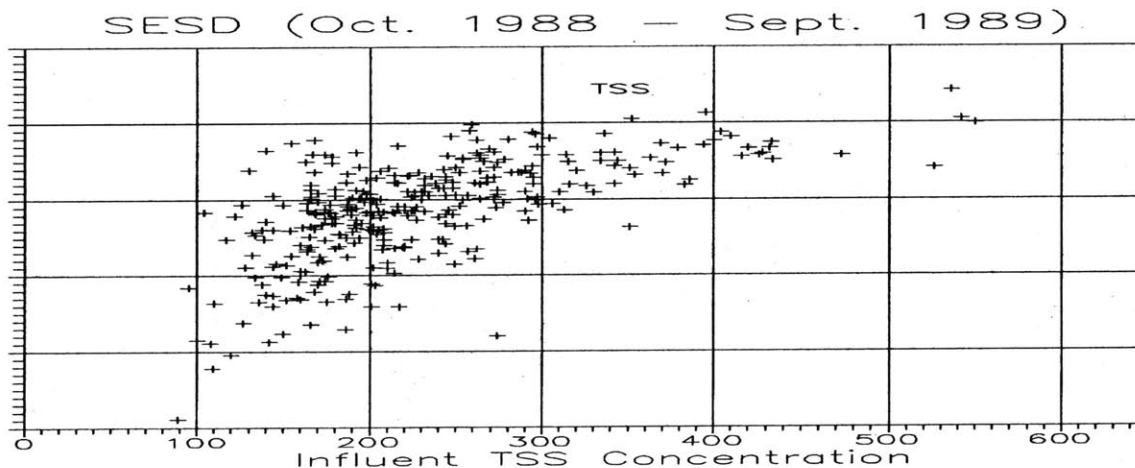


Figure 4-2: TSS removals with increasing TSS concentration

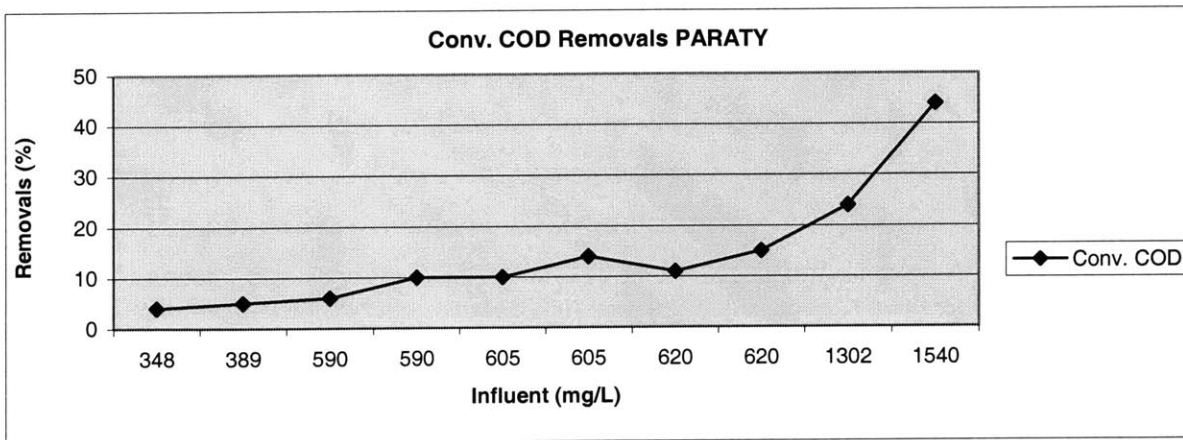


Figure 4-3: COD removals with increasing influent concentration

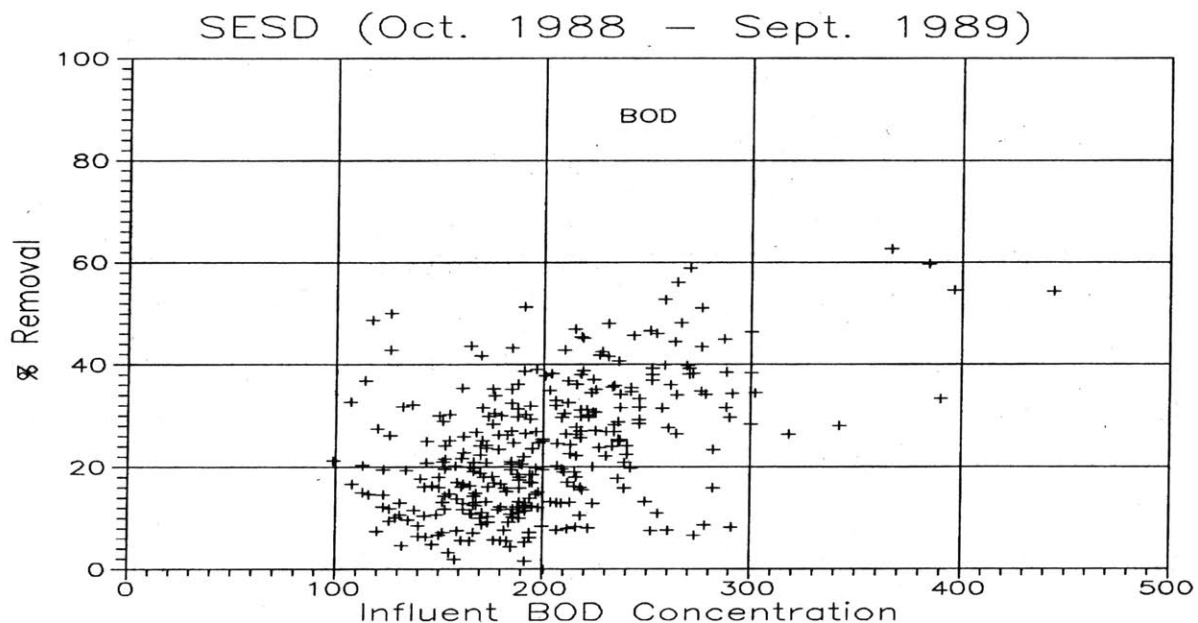


Figure 4-4: BOD removals with increasing BOD concentration

It is also important to note that BOD removal rates were approximated by measuring the COD removals of the raw and treated wastewater. This has been shown to be an acceptable technique for the estimation of BOD (Harleman, 2003). Since the BOD test requires 5 days to yield final results whereas the COD tests only requires 2 hours, measuring the COD was therefore more practical for our time-constrained experiments in Paraty.

4.2 EXPERIMENT ONE

4.2.1 Introduction

For this experiment, raw sewage was injected with ferric chloride, FeCl_3 , at different concentrations to test for the optimal dose that would yield the most economical level of SS, turbidity and COD removal. The optimal FeCl_3 dose was chosen to most closely match the expected and published removal rates for a typical chemically enhanced primary treatment plant: 85% for SS and 57% for BOD (NRC, 1996). As described in the introduction above, and for the jar tests described below, COD removals were measured instead of BOD removals since the 2 hour COD test was more practical than the 5-day BOD test.

Experiment one consists of two sets of jar tests: In set one, Jar tests 2 and 6 were tested on different raw wastewater samples to test the effectiveness of a 40 mg/L FeCl_3 dose for the removal of suspended solids, turbidity and COD.

Similarly in the second set of jar tests under experiment one, a single sample of raw wastewater was used for jar tests 4 and 5. This raw wastewater had characteristics very similar to the raw sample used for jar test 8 and was therefore expected to perform similar to jar tests 4 and 5 under the same FeCl_3 conditions.

4.2.2 EXPERIMENT 1A: Jar tests 2 and 6

These jar tests were performed on two distinct samples of sewage having very similar raw wastewater characteristics and collected from the same sampling spot. They were therefore considered comparable in quality and, at identical FeCl_3 doses, expected to yield similar SS, turbidity, and COD removal rates. The raw wastewater characteristics and removal rates are shown in Table 4-2. It is important to note that the samples settled for 5 minutes after mixing thus representing an overflow rate of approximately 23 m/day (Chapter 2). The blank sample was not injected with any ferric chloride and therefore represents conventional primary treatment. The turbidimeter was not functional at the time that Jar Test 2 was conducted.

			Removal Rates (%)					
Jar Test	Raw	FeCl ₃ (mg/L)	0	20	40	60	80	100
2	1650	COD	5	34	47	45	5	57
	435	SS	20	54	81	82	94	97
	440	Turb	N/A	N/A	N/A	N/A	N/A	N/A
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
6	1540	COD	44	49	49	48	58	59
	603	SS	43	40	53	53	63	90
	350	Turb	36	50	60	62	63	62

Table 4-2: Experiment 1A Percent Removals Summary

4.2.2.1 Suspended Solids Removal

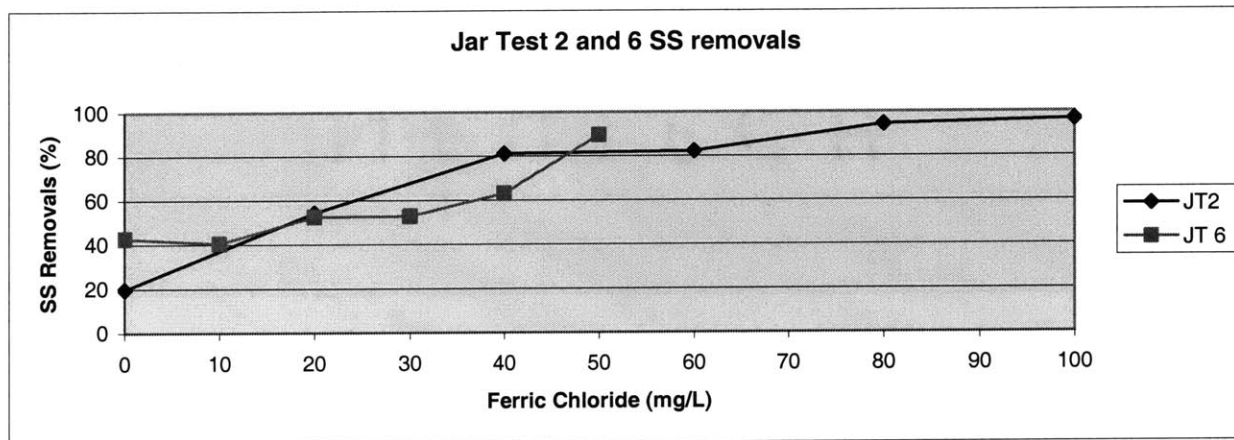


Figure 4-5: Experiment 1A Suspended Solids Removals

The suspended solids removal rates were higher in Jar Test 2 peaking at 97% for 100mg/L of FeCl₃. The results from Jar Test 6 were also considered within acceptable range and the observed discrepancy in removal rates can be attributed to the fact that the initial SS reading in Jar Test 6 was 1.5 times larger than the initial SS in Jar Test 2. Lower removal rates would be therefore expected for more dilute samples. The removals after conventional treatment (mixing with 0mg/L FeCl₃) were also lower than the removals in jar test 6 because of the dilution effect (Refer to Figure 4-1 above)

The most economical dose for Jar Test 2 was between 40 and 50 mg/L. The economic dose was determined by finding the point at which increased doses of FeCl₃ did not result in similar increases in removal rates. Similarly, suspended solids removals in Jar Test 6 reached a somewhat constant removal rate of 80% for FeCl₃ doses between 40 and 65 mg/L. The optimal dose of FeCl₃ was therefore determined to be 40 mg/L.

4.2.2.2 Turbidity Removals

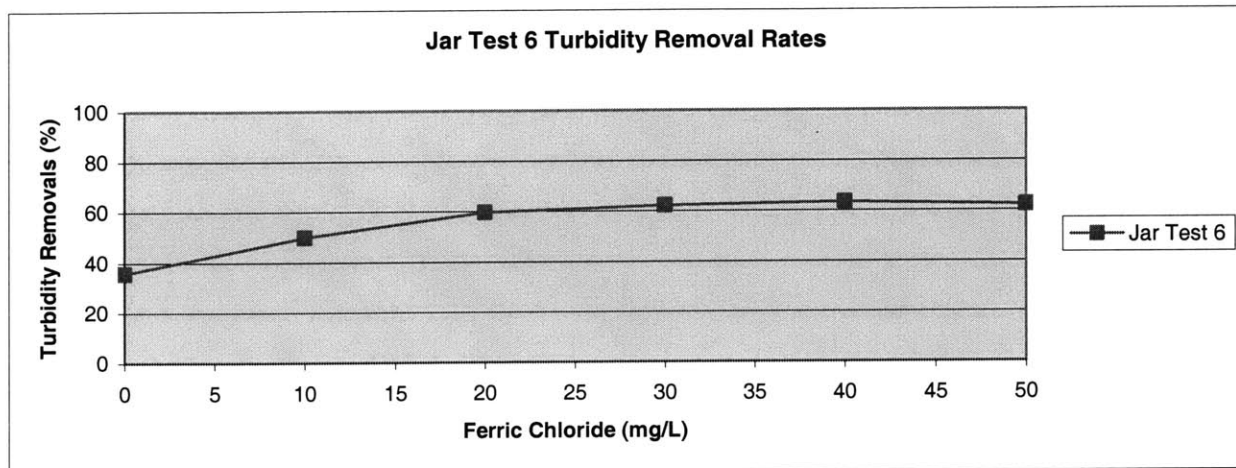


Figure 4-6: Experiment 1A Turbidity Removals

Turbidity measurements were not made for Jar Test 2 since the turbidimeter was not functional at the time of the test. For Jar Test 6, however, turbidity removal rates peaked at 60% for FeCl_3 doses ranging from 20 to 60 mg/L. The optimal coagulant dosage for the turbidity alone was therefore chosen to be the 20 mg/L. The most optimal FeCl_3 dose however which takes suspended solids into account is 40 mg/L.

4.2.2.3 COD Removals

Sewage in Jar Tests 2 and 6 reached 60% COD removal rates at 50 and 80 mg/L of FeCl_3 respectively. These values are comparable to the published and expected CEPT COD removal rate of 57% (NRC, 1996).

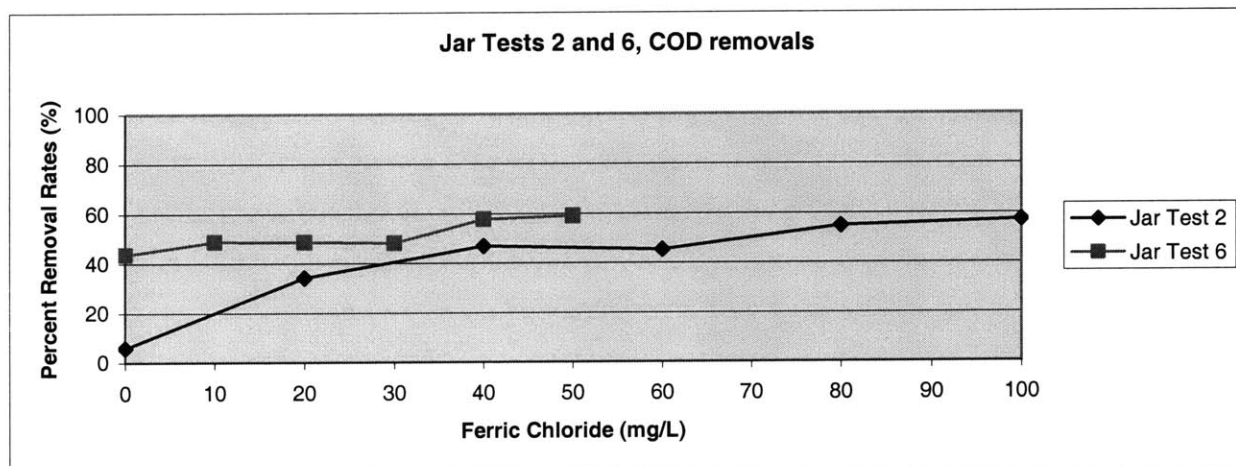


Figure 4-7: Experiment 1A COD removals

There was not a large difference however in the removals at 40 mg/L and therefore the Ferric Chloride concentration at a dose of 40 mg/L was therefore considered the optimal dose that achieved 60% removal rates of suspended solids, turbidity and chemical oxygen demand for jar tests 2 and 6.

4.2.3 **EXPERIMENT 1B: Jar Tests 4, 5 and 8**

Similar to jar tests 2 and 6 above, experiments using raw wastewater and varying ferric chloride doses were used to determine optimal coagulant doses in jar tests 4,5 and 8. These experiments are identical in procedure and methodology to jar tests 2 and 6 above and were performed to check the efficiency of the chosen 40mg/L FeCl₃ dose.

The raw wastewater sample from which Jar tests 4 and 5 were taken had raw characteristics very similar to the sample from which Jar test 8 was taken. The three jar tests were therefore grouped together and assumed to be similar in wastewater quality and therefore expected to achieve similar removal rates. It is also important to note that the samples from which jar tests 4,5 and 8 were taken were significantly more dilute than those for jar tests 2 and 6 in section 4.2.2 above. Removal rates can therefore be expected to be lower.

The summary of raw waste characteristics and removal rates are shown in Table 4-3 below
The highlighted jar tests indicate samples taken from the same raw wastewater source:

			Removal Rates (%)					
Jar Test	Raw	FeCl ₃ (mg/L)	0	20	40	60	80	100
4	605	COD	10	11	19	23	26	34
	149	SS	6	6	19	21	30	48
	200	Turb	24	35	40	46	47	58
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
5	605	COD	14	10	18	18	21	19
	149	SS	6	6	7	9	14	15
	200	Turb	40	35	43	42	46	54
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
8	590	COD	6	6	8	15	16	20
	208	SS	36	37	46	50	58	64
	166	Turb	34	29	42	50	54	56

Table 4-3: Experiment 1B Percent Removals Summary

4.2.3.1 Suspended solids removal

The results for the three different jar tests showed a clear sensitivity to initial suspended solids concentrations: At 40 mg/L of FeCl₃, Jar Tests 4 and 5, at an initially low SS concentration of 149 mg/L, achieved an SS removal rate of less than 15%, which is very low compared to the expected removal of 57% (Harleman, 2002). Jar Test 8 however, at an initial concentration of 208 mg/L, achieved removal rates of approximately 60% at the prescribed FeCl₃ 40 mg/L concentration.

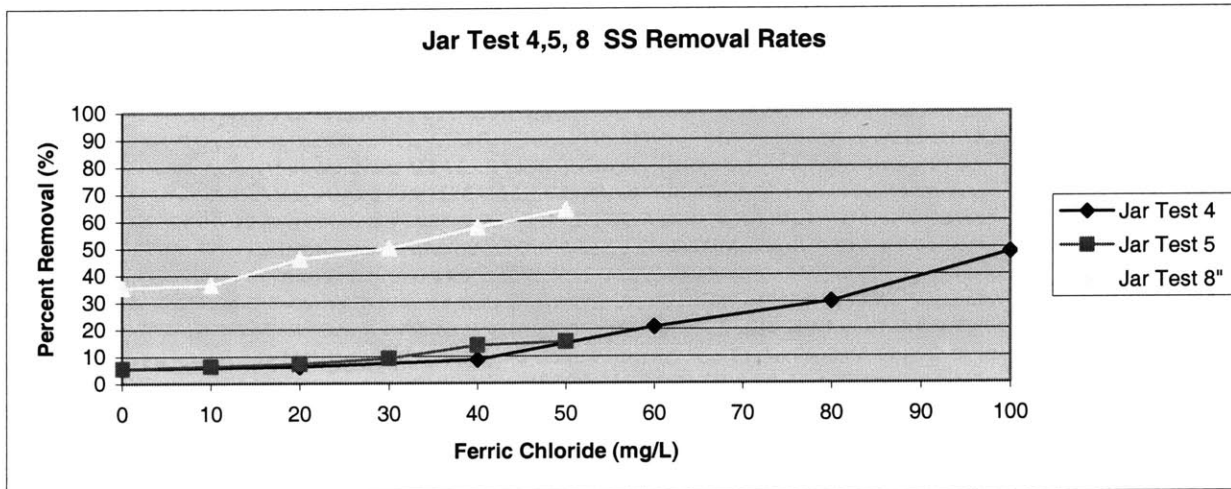


Figure 4-8: Experiment 1B Suspended Solids Removal Rates

Dilution caused by continuous heavy rains in Paraty and infiltrating into the sampling location was therefore considered a limiting factor to the suspended solids results in these jar tests.

It is important to note that dilution significantly affected the conventional primary treatment of jar tests 4 and 5 where 0mg/L of FeCl₃ achieved SS removals much lower than the expected 30%. Avoiding sewage dilution with precipitation or storm water is therefore critical since the coagulation process is impeded when the initial SS concentrations are low.

(Refer to the laboratory study constraints section in section 4.1.1 above.)

4.2.3.2 Turbidity Removal

The turbidity removals were more consistent between jar tests 4,5 and 8. Close examination of Figure 4-9 of the following page shows that at a FeCl₃ dose of 40 mg/L, turbidity removal rates for jar tests 5 and 8 were 50% and 40% in jar test 4.

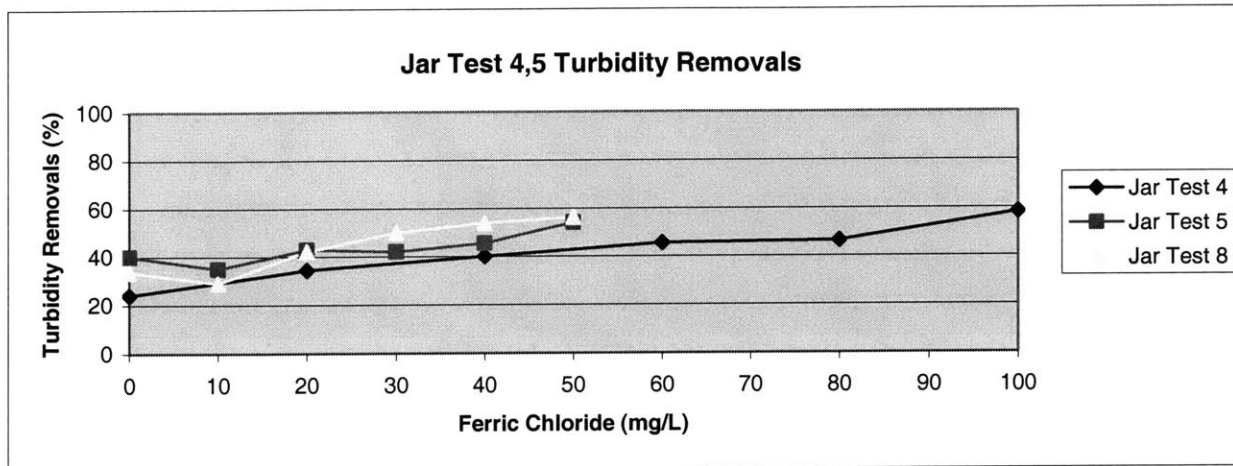


Figure 4-9: Experiment 1B Turbidity Removals

4.2.3.3 COD removal

COD removals for jar tests 4,5 and 8 reached a 20% removal rate at FeCl_3 doses of 40 mg/L. This value is lower than the expected and representative 57% BOD removal for chemically enhanced primary treatment but the low removals can be attributed, again, to the diluted sample and to the low initial COD readings of the raw wastewater.

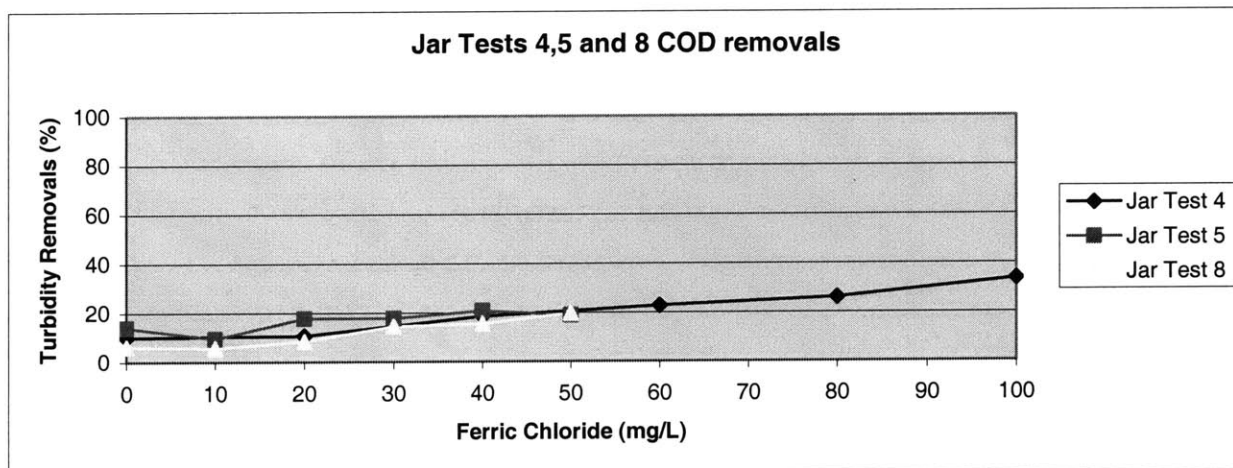


Figure 4-10 Experiment 1B: COD removals

A summary table of the removals achieved with injection of 50 mg/L FeCl₃ alone is provided below. The average removals expected from FeCl₃ additions to the sewage in Paraty are 53, 80, and 62% COD, SS and Turbidity for undiluted sewage (Jar Tests 2 and 6) and 19, 32 and 52 % COD, SS and Turbidity for diluted sewage (Jar Tests 4,5 and 8)

Jar Test	Raw	40 mg/L FeCl ₃ (mg/L)	% Removal
2	1650	COD	47
	435	SS	81
	440	Turb	N/A
Jar Test	Raw	40 mg/L FeCl ₃ (mg/L)	% Removal
6	1540	COD	58
	603	SS	63
	350	Turb	63
Jar Test	Raw	40 mg/L FeCl ₃ (mg/L)	% Removal
4	605	COD	20
	149	SS	14
	200	Turb	43
Jar Test	Raw	40 mg/L FeCl ₃ (mg/L)	% Removal
5	605	COD	19
	149	SS	16
	200	Turb	54
Jar Test	Raw	40 mg/L FeCl ₃ (mg/L)	% Removal
8	590	COD	16
	208	SS	58
	166	Turb	54

Table 4-4: Summary Removal Rates: Experiment One

4.3 EXPERIMENT TWO

4.3.1 Introduction

For this second experiment, fresh domestic wastewater was collected and treated with 0 to 50 mg/L doses of ferric chloride. Different doses of seawater ranging from 0.5 to 15% of seawater were also added to the samples to test the efficiency and feasibility of using seawater as a coagulant enhancement. The seawater was collected from a beach nearby at a measured salinity of 36 ppt (parts per thousand).

4.3.2 EXPERIMENT 2A: Seawater at 0.5% by volume.

The raw wastewater used for Jar test 8 in section 4.2 above was also used in Jar Test 9 for Experiment 2 here to test the effect of adding 0.5 % seawater by volume as a coagulation enhancement mechanism. Jar Tests 8 and 9 described were therefore supplied by the same raw wastewater sample. Jar Test 10 is an independent test of importance here because the raw sample from which it was taken was significantly less dilute than the sample from which jar tests 8 and 9 were taken. Jar test 10 is therefore important to test the doses of ferric chloride and volumes of seawater needed to achieve appropriate SS, Turbidity and COD removals at all dilution levels.

Since the beakers in which the jar tests were conducted contain 2 liters of wastewater, then adding 0.5% seawater by volume equals the addition of 10ml of seawater.

Table 4-5 below summarizes the raw wastewater characteristics and achieved SS, turbidity and COD removal rates in the three jar tests. It is important to note the difference and compare removal rates in samples with and without seawater. Again, the highlighted jar tests indicate same raw wastewater sources:

			Removal Rates (%)					
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
8	590	COD	6	6	8	15	16	20
	8	SS	36	37	46	50	58	64
	166	Turb	34	29	42	50	54	56
9	590	COD	10	13	17	28	31	30
	8	SS	37	43	55	70	60	75
	166	Turb	42	46	57	64	69	68
10	1302	COD	24	25	30	35	41	37
	619	SS	49	59	62	65	68	73
	395	Turb	31	48	51	59	71	76

Table 4-5: Experiment 2A Summary of Removals

4.3.2.1 Suspended solids removal

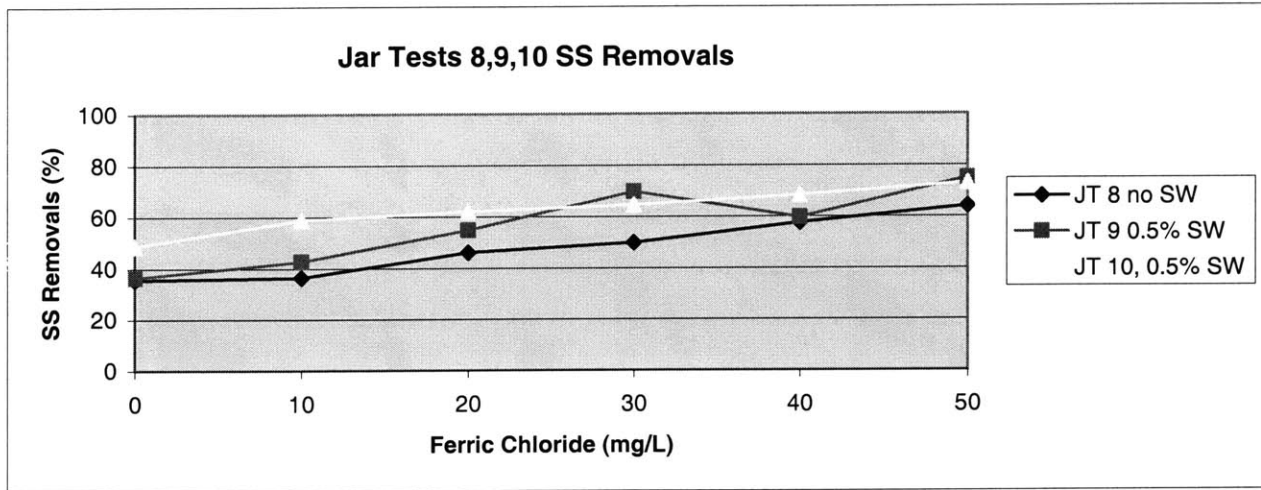


Figure 4-11: Experiment 2A Suspended solids removals

The suspended solids removals for Jar Test 8 to which no seawater was added, was 64% at a FeCl_3 dose of 50 mg/L. When the same wastewater was well mixed and injected with 50 mg/L FeCl_3 and 10 ml of seawater (Jar Test 9, pink line on Figure 4-11), the suspended solids removals increased to 75% marking a 17% increase in suspended solids where:

$$\% \text{ Increase in solids} = \frac{[\text{final (mg/L)} - \text{initial (mg/L)}]}{[\text{initial (mg/L)}]}$$

Similarly for Jar Test 10, the SS removal rate at 50 mg/L FeCl_3 was 73%, marking a 14% increase from the 64% removals when no seawater was added.

For suspended solids removal, the addition of small volumes of seawater was therefore effective at achieving the following essentially identical goals:

1. Reduce the amount of ferric chloride needed to achieve a specified SS removal rate
2. For the same concentration of ferric chloride, increase the suspended solids removal rate.

Therefore, as a first conclusion, seawater enhances the coagulation process and leads to significant reductions in suspended solids removals

4.3.2.2 Turbidity Removals

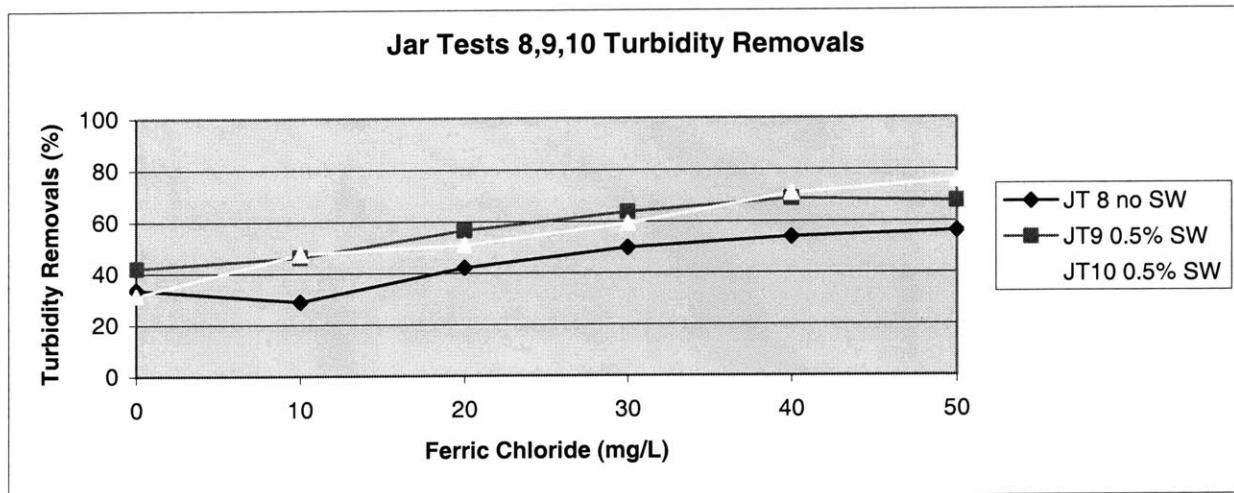


Figure 4-12: Experiment 2A Turbidity Removals

The turbidity removal for jar test 8 to which no seawater was added was 56% at 50 mg/L of FeCl_3 . However, in jar test 9, and for the same concentration of ferric chloride, a 69% turbidity removal was achieved marking a 23% increase in removal efficiency. In jar test 10, the removal efficiency increased from the original 56% to 75% at 50 mg/L FeCl_3 and with 0.5% seawater, thus marking a 34% increase in removal efficiency.

It is interesting to note that, unlike in the suspended solids removal above, the difference between removal efficiencies with and without seawater, in jar tests 8 versus 9, remained consistent at approximately 55% on average for all values of ferric chloride tested. Therefore, if the wastewater treatment objective for example, is 55% turbidity removal, then a concentration of 50mg/L of ferric chloride could be used (Blue line). Alternatively, a ferric chloride concentration of approximately 23 mg/L could be used with 0.5% seawater by volume. The second alternate suggests a 54% decrease in required ferric chloride concentration that translates into significant economic savings

4.3.2.3 COD Removals

COD removals for jar test 8, at FeCl_3 concentration of 50 mg/L reached a 20% value. When the 0.5% seawater was added to jar test 9, however, and at the same FeCl_3 concentration of 50 mg/L, the COD removal rate increased to 30%, marking a 50% increase. When the results from jar test 10 were compared to those from jar test 8 for COD removals, an 85% increase was noted, bringing the COD removals from 20% to 37%.

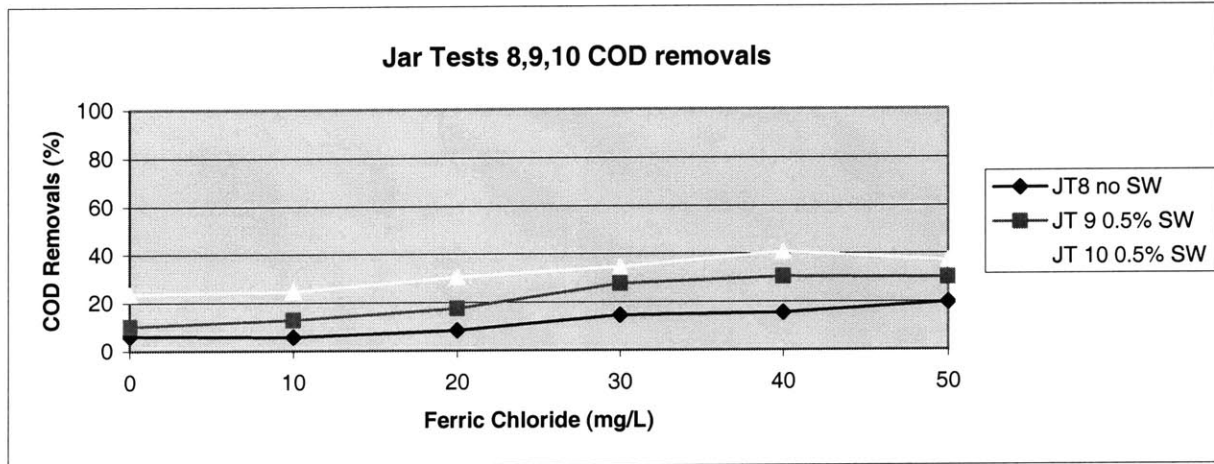


Figure 4-13: Experiment 2A COD removals

It is important here to note again, that because of the initially diluted raw wastewater samples (especially for that used in jar tests 8 and 9), the COD removals for conventional primary treatment (at 0mg/L FeCl_3) were significantly lower than expected. The importance of avoiding diluted sewage is therefore of primary importance. It is also important to note that increases in the concentration of seawater contributed significantly to increases in SS, turbidity and COD removals. Seawater therefore might prove to be an in-plant solution to treating influents with low suspended solids, turbidity and COD readings.

4.3.3 EXPERIMENT 2B: Seawater at 1 and 2% by volume

Since the results from the addition of 0.5% on jar tests 9 and 10 were very positive, additional jar tests were conducted with the addition of 1 and 2% seawater. This was done to test for the increased efficiency of using seawater as a coagulation enhancement in the removal of suspended solids, turbidity and COD. In Jar test 11, ferric chloride doses up to 50 mg/L were used with 1% by volume of seawater (20 ml of seawater in the 2 liter jar testing beaker). In Jar test 12, 40ml of seawater (2% seawater by volume) was used with the same doses of ferric chloride and on the same raw wastewater sample from which jar test 11 was used. The raw wastewater from which jar tests 11 and 12 were taken was very similar in characteristics to jar test 8 raw wastewater. Therefore the removal results from jar tests 11 and 12 were compared to those from jar test 8 to which no seawater was added. Table 4-6 below summarizes the raw water characteristics and the observed removal efficiencies.

Jar Test	Raw	FeCl ₃ (mg/L)	Removal Rates (%)					
			0	10	20	30	40	50
8 <i>no SW</i>	590	COD	6	6	8	15	16	20
	208	SS	36	37	46	50	58	64
	166	Turb	34	29	42	50	54	56
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
11 <i>1% SW</i>	620	COD	11	25	34	36	39	44
	218	SS	13	27	41	50	64	69
	131	Turb	8	16	48	51	59	59
Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
12 <i>2% SW</i>	620	COD	15	28	32	38	46	50
	218	SS	17	33	47	61	70	65
	131	Turb	10	33	45	60	68	68

Table 4-6 Experiment 2B Summary of Removals

4.3.3.1 Suspended Solids Removals

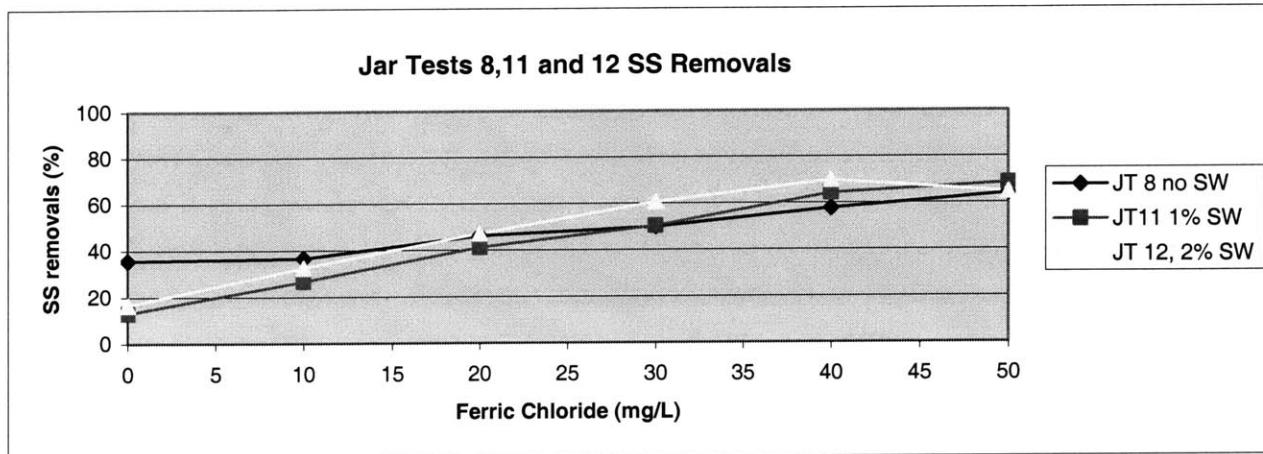


Figure 4-14: Experiment 2B Suspended Solids Removals

Increases in suspended solids removal efficiencies caused by the addition of seawater occurred after the addition of 30 mg/L FeCl₃ to jar tests 8, 11 and 12. These results vary slightly therefore from the SS removal rates at lower seawater concentrations where differences in SS removals as large as 50% occurred at FeCl₃ concentrations of 10 and 20 mg/L (Figure 4-14 above). The increase in SS removal efficiency was 20% with a 2% seawater addition at 30 mg/L FeCl₃ and 21.5% at 40 mg/L FeCl₃. The increases in SS removals due to seawater addition seemed to stabilize in excess of 50 mg/L FeCl₃ indicating a potential limit to the level at which seawater and FeCl₃ can be effectively mixed and used as coagulants.

4.3.3.2 Turbidity Removals

Turbidity removals followed the same trends as suspended solids with the addition of 1 and 2% of seawater. Increases of turbidity removal efficiencies caused by seawater addition were not noted until after 20mg/L of FeCl₃ was added to the influent. At 30 mg/L, the addition of 1% of seawater also did not have any differentiating effect on the turbidity removals and the 2% seawater addition instigated a 20% increase in removal efficiency. The highest removal efficiencies observed were at 40 mg/L FeCl₃ whereby a 1% seawater addition caused a 15% turbidity removal increase and 28% increase with the addition of 2% seawater.

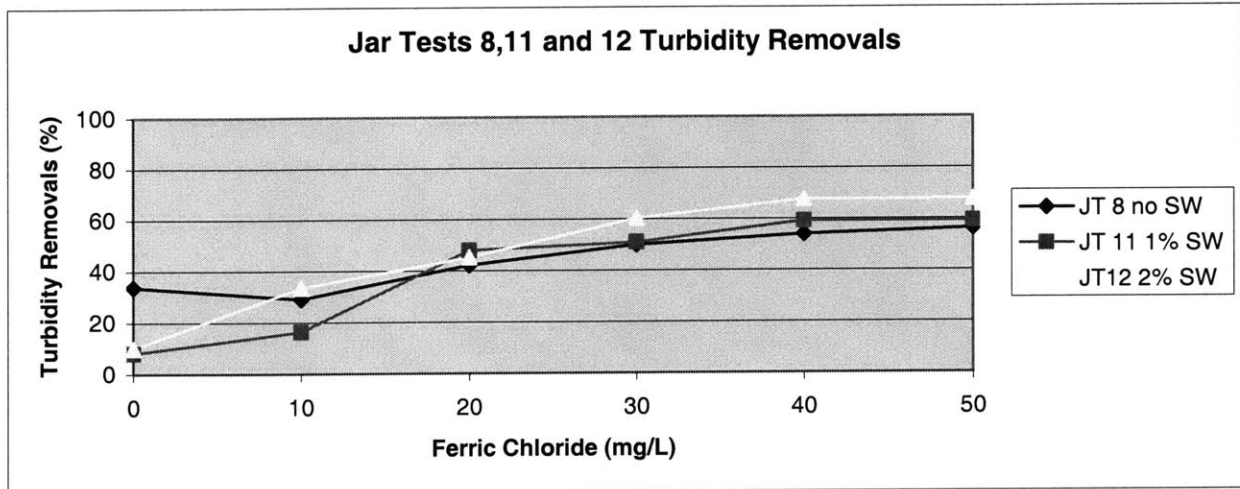


Figure 4-15: Experiment 2B Turbidity Removals

4.3.3.3 COD Removals

COD removals followed significantly different trends compared to their suspended solids and turbidity counterparts. At FeCl_3 concentrations as small as 10mg/L, a 1% seawater addition caused a 300% increase in removal efficiency, from 5.5% to 26%. In addition, at the estimated most economic FeCl_3 dose of 50mg/L, increases in COD removals reached a 170% difference. COD was therefore very strongly affected by the increased seawater presence in the influent and greatly increased coagulation. It is important to note here that seawater enhanced the FeCl_3 coagulation process and yet appears to be more effective in removing colloidal COD compared to removing colloidal suspended solids (or turbidity) for small concentrations of seawater less than 2%. This theory will be checked for confirmation after analyzing the jar tests with larger concentrations of seawater additions.

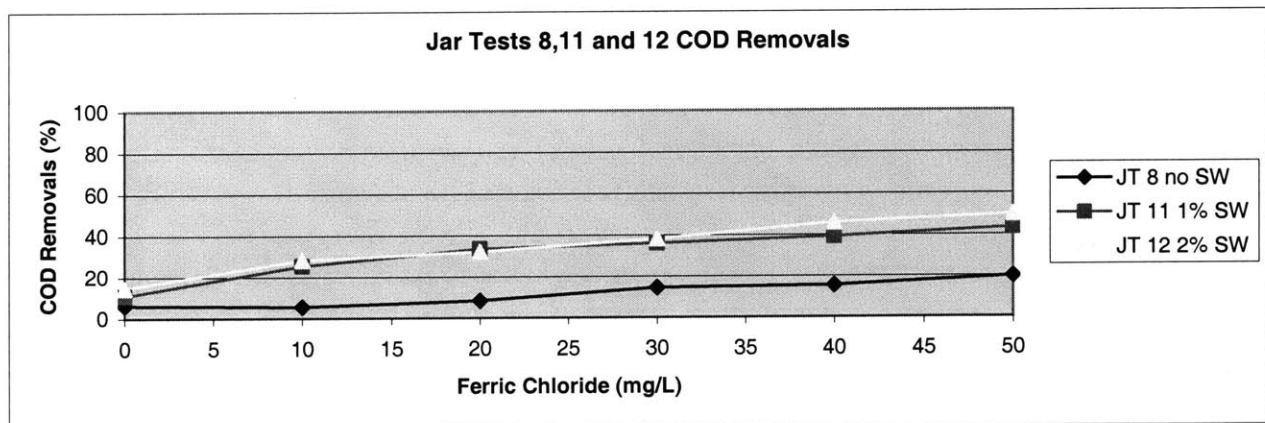


Figure 4-16: Experiment 2B COD Removals

4.3.4 EXPERIMENT 2C: Seawater at 5 and 10% by volume

Similar to the experiments above, additional jar tests were performed to test for the added efficiency of injecting 5 and 10% seawater by volume into the influent. The drive for performing these additional tests was the positive results observed in the SS, turbidity and COD removal efficiencies for the injection of 0.5%, 1% and 2% seawater into the influent. The results from jar tests 25 and 26 will be presented in this section. Both jar tests were injected with FeCl₃ concentrations ranging from 0 to 40mg/L and seawater concentrations of 5 and 10% by volume. Although jar tests 25 and 26 were not taken from an identical raw wastewater source, their respective raw sources are very similar in SS, turbidity and COD values, as shown in Table 4-7 below:

Jar Test	Raw	FeCl ₃ (mg/L)	0	10	20	30	40	50
8 <i>No SW</i>	590	COD	6	6	8	15	16	20
	208	SS	36	37	46	50	58	64
	166	Turb	34	29	42	50	54	56
Jar Test	Raw	FeCl ₃ (mg/L)	0	20(5)	30(10)	40(0)	40(5)	40(10)
25 <i>5 and 10%</i>	389	COD	5	0	0	2	1	3
	101	SS	0	25	42	11	66	68
	84	Turb	5	19	38	4	63	68
Jar Test	Raw	FeCl ₃ (mg/L)	0	30	30(5)	30(10)	40(5)	40(10)
26 <i>5 and 10%</i>	348	COD	4	20	14	25	14	22
	110	SS	3	9	58	60	65	72
	143	Turb	8	7	53	58	62	70

Table 4-7: Experiment 2C Summary of Removals

4.3.4.1 Suspended Solids Removal

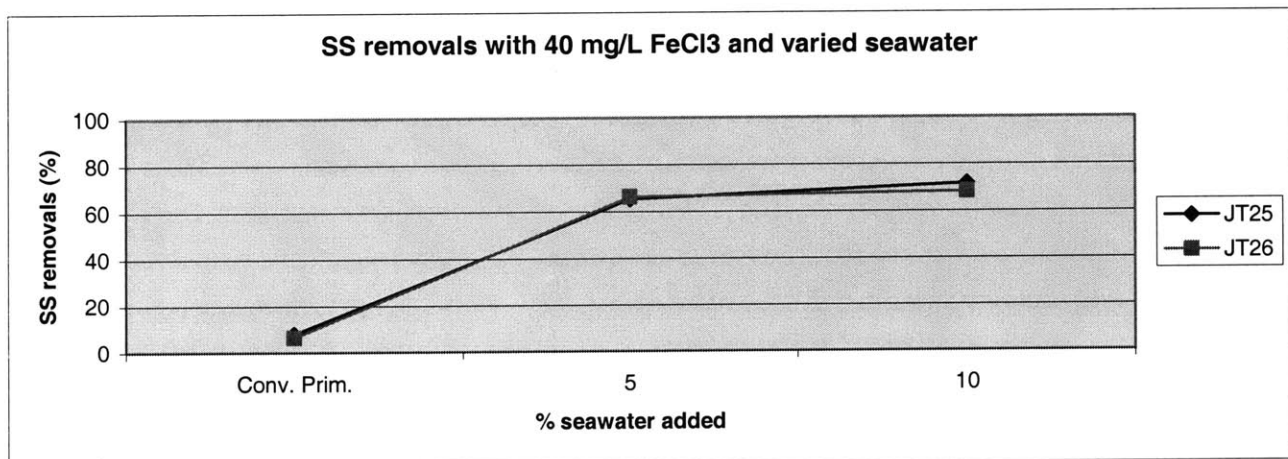


Figure 4-17: Experiment 2C Suspended Solids Removals

In this experiment, the ferric chloride concentration was held constant at 40 mg/L and the jar tests were enhanced with seawater concentrations varying between 0% (to represent conventional primary treatment) and 10%. The SS removals increased significantly between the 0% and 5% marks (at about 660% difference in removal efficiencies) and yet remained approximately equal for the 10% addition of seawater, varying from 68 to 70% removals.

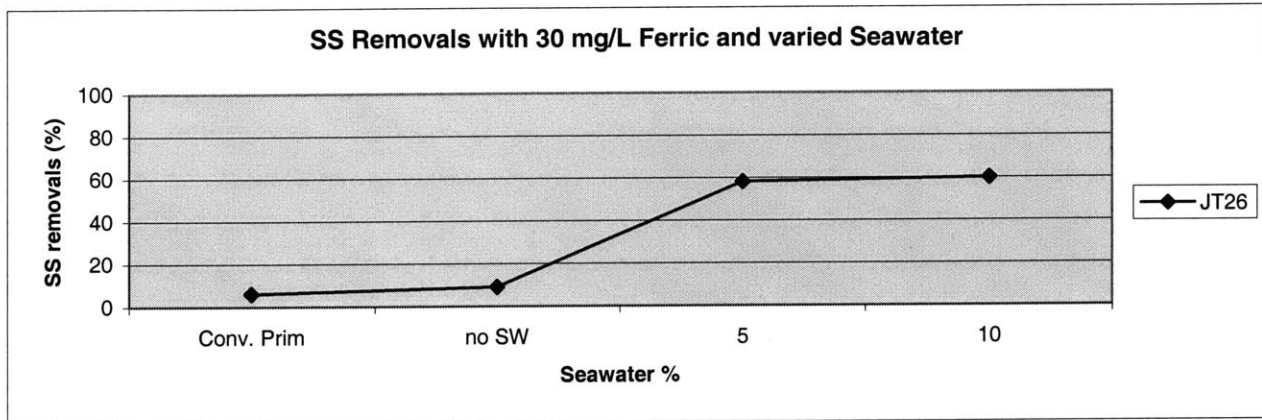


Figure 4-18: Experiment 2C Suspended Solids removals (2)

Since finding the optimal ferric chloride concentration is the end goal of these experiments, the jar test described above was repeated to test for the efficiency of SS, turbidity and COD removals for the addition of 30 mg/L of ferric chloride and various seawater concentrations.

Figure 4-18 above shows the clear improvement in SS removals for the sample to which no seawater was injected and which achieved a removal rate of 10%. After the addition of 5% seawater, with the same 30 mg/L FeCl₃ concentration, the SS removal rate achieved was 60% thus marking a 500% increase in removal efficiencies. It is interesting to note here that the removal efficiencies remained constant for 5 and 10% seawater injections.

Comparing the results of 30 mg/L of FeCl₃ to the previous sample where 40 mg/L FeCl₃ was used, it is obvious that the removal efficiencies did not differ appreciably and that 30 mg/L is enough coagulant to achieve very high removal rates and that using 40 mg/L FeCl₃ with 5% seawater is not the optimally economic coagulant dose.

4.3.4.2 Turbidity Removals

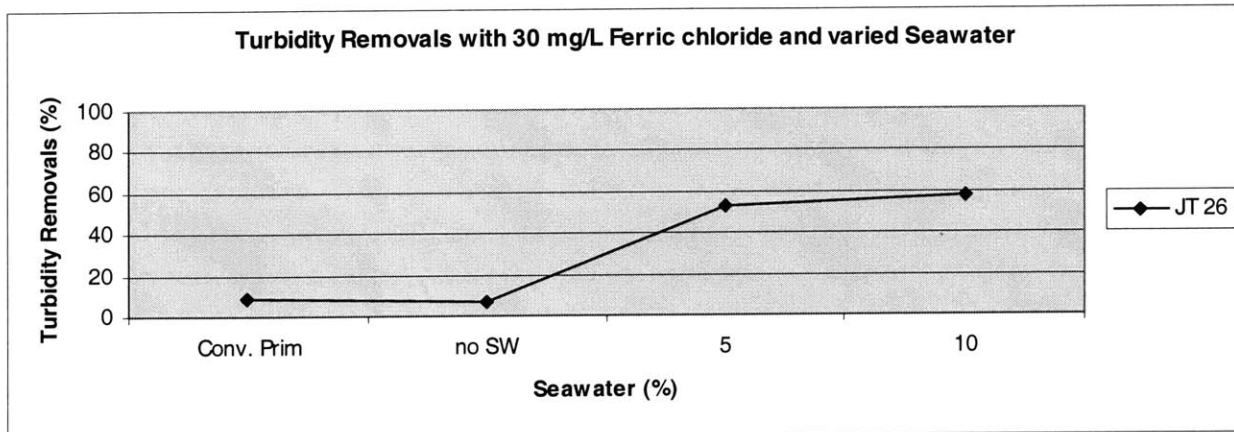


Figure 4-19: Experiment 2C Turbidity Removals

Similar to most of the previous jar tests, turbidity removals followed the same trend as that seen in suspended solids. Therefore, at a constant FeCl_3 concentration of 30 mg/L and with varied seawater volume additions, the turbidity removals increased by 400% between using no seawater to injecting the 5% seawater by volume (from 9 to 52% at 5% seawater).

It is important to note that turbidity removals were slightly lower when compared to the removals in suspended solids for the same FeCl_3 and seawater concentrations; at a 5% seawater concentration, SS removals were 60% compared to 51% for turbidity for example. It is also important to note that, unlike suspended solids, the turbidity removals continued to increase (although not as dramatically) with increased seawater concentrations; removals at 5% were 51% and 59% at 10%. It is this type of situation where the cost of pumping the extra seawater would have to be compared to the extra assumed benefit of reducing the turbidity by an extra 8 percentage points.

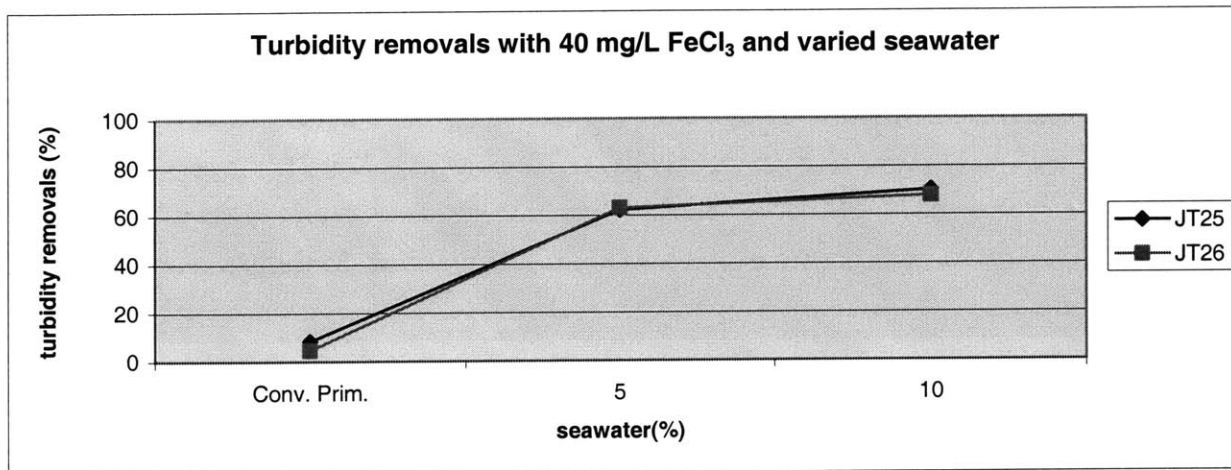


Figure 4-20 Experiment 2C Turbidity Removals (2)

Figure 4-20 above shows the results from an identical experiment, in which a FeCl₃ concentration of 40 mg/L was used instead of 30 mg/L. Again, the turbidity removals followed very similar trends to those seen in suspended solids. The increase in ferric chloride concentration to 40 mg/L also did not yield significant increases in turbidity removals since at a seawater injection of 5%, the turbidity removals were 62% compared to 55% removal when 30 mg/L was used with 5% seawater.

4.3.4.3 COD Removals

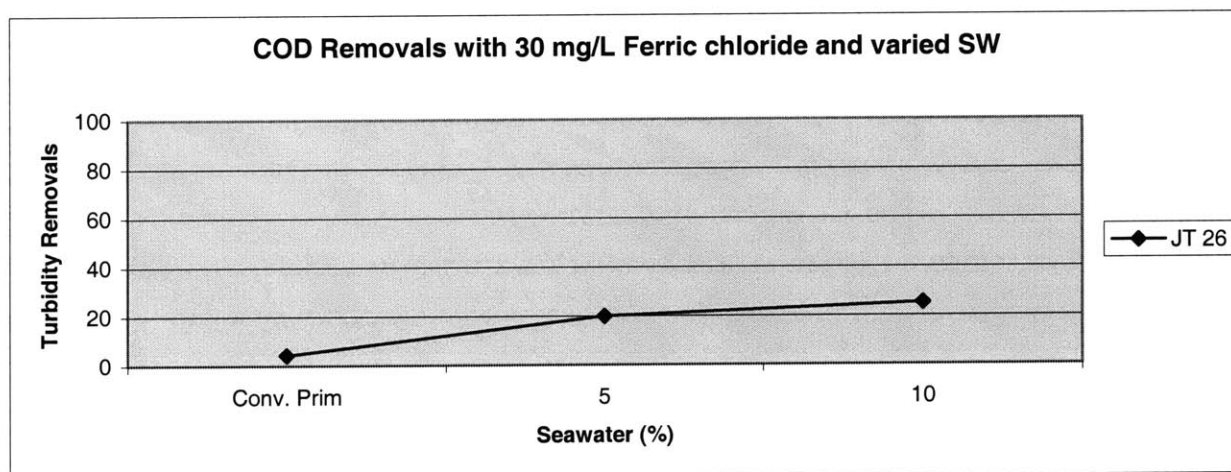


Figure 4-21: Experiment 2C COD Removals

COD removals were, as expected, lower than the suspended solids and turbidity removals for the same FeCl₃ and seawater concentrations. However, the COD removal rates still increased with the addition of seawater, although not to the same extent as the SS or turbidity measurements. A 5% addition of seawater yielded a 30% increase in COD removal efficiency. Also at 10%

seawater and with 30 mg/L FeCl₃, the removal efficiency increased to 25% from the initial 6% in conventional primary treatment marking a 316% in removal efficiency.

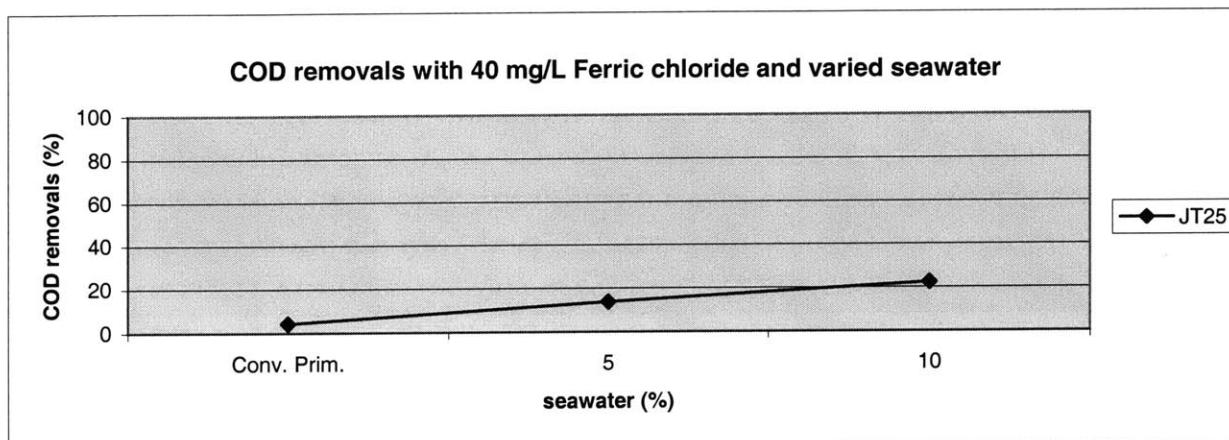


Figure 4-22: Experiment 2C COD Removals (2)

At a ferric chloride concentration of 40 mg/L, the COD removals increased linearly with the increased addition of seawater. The observed efficiencies in COD removal were not significantly different from those observed in graph 4-22 above where only 30 mg/L FeCl₃ was used. At 5% seawater injection, the COD removal was approximately 17% at 40 mg/L and 20% at 30 mg/L. Similarly, at 10% seawater, the COD removals for 40 mg/L FeCl₃ were 21% and approximately 24% for 30 mg/L.

4.4 GENERAL FeCl₃ CONCLUSIONS & DOSAGE RECOMMENDATIONS IN PARATY

The general conclusions on the observed trends that the jar tests described above yielded are listed below:

1. Seawater has a positive effect on the removal efficiencies of diluted wastewater samples.
2. At FeCl₃ doses higher than 50 mg/L, the effect of seawater decreases significantly.
3. Minimum FeCl₃ doses for seawater to take effect are approximately 20-25 mg/L.
4. Large seawater additions do not necessarily yield large increases in removal efficiencies.
5. COD removals are mostly affected with small seawater volume additions.
6. Relatively negligible increases in removal efficiencies of SS and COD for seawater additions larger than 5% by volume.

Therefore, based on these preliminary tests, the recommended chemical doses for chemically enhanced primary treatment in Paraty are:

40 mg/L FeCl₃ or 30 mg/L FeCl₃ with 5% seawater

4.5 EXPERIMENT 3: POLYMER ANALYSIS

Polymers are frequently used in chemically enhanced primary treatment to aid in the removals of suspended solids and chemical oxygen demand. Typical doses vary between 0.05 and 0.25 mg/L depending on the characteristics of the raw wastewater (Harleman, 2003).

The anionic polymer, OPTIFLOC, was tested in Paraty and the SS and COD removals were closely monitored for increases in removals caused by the presence of small doses of polymers.

The jar tests involving polymers did not all use identical raw wastewater samples and, as such, focused primarily on identifying the trends in SS and COD removals caused by the presence of polymers in varying wastewater samples. The first set of jar tests therefore tested FeCl₃ doses combined with polymer doses. FeCl₃ was then tested with seawater alone. Finally, a combination of FeCl₃, polymer and seawater at varying concentrations were combined to test for the most efficient and economically optimal dosage to treat the wastewater in Paraty

4.5.1 EXPERIMENT 3A: Ferric Chloride and Polymers

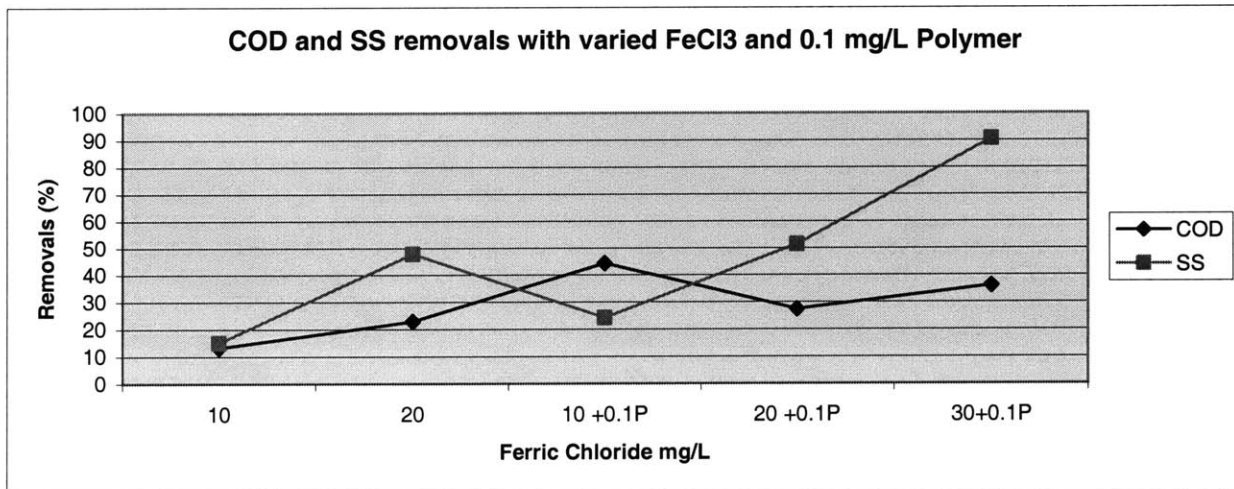


Figure 4-23: Experiment 3A: COD and SS Removals with FeCl₃ and 0.1 mg/L Polymer

In figure 4-23 above, the FeCl₃ doses were varied while the polymer dose was kept constant at 0.1 mg/L. Suspended solids removals increased significantly with the increase of ferric chloride doses. The most optimal ferric chloride/polymer dose was therefore chosen off the graph to be at 30 mg/L FeCl₃ and 0.1 mg/L Polymer which yielded approximately 38% COD removal and 90% SS removal.

It is also important to note that at relatively low FeCl₃ doses of 20 mg/L, adding 0.1mg/L of polymer did not have the desired effect of an increase in COD or SS removal efficiencies. It

wasn't until at least 30 mg/L of FeCl₃ was used that the polymer displayed an effect in enhancing removal efficiencies.

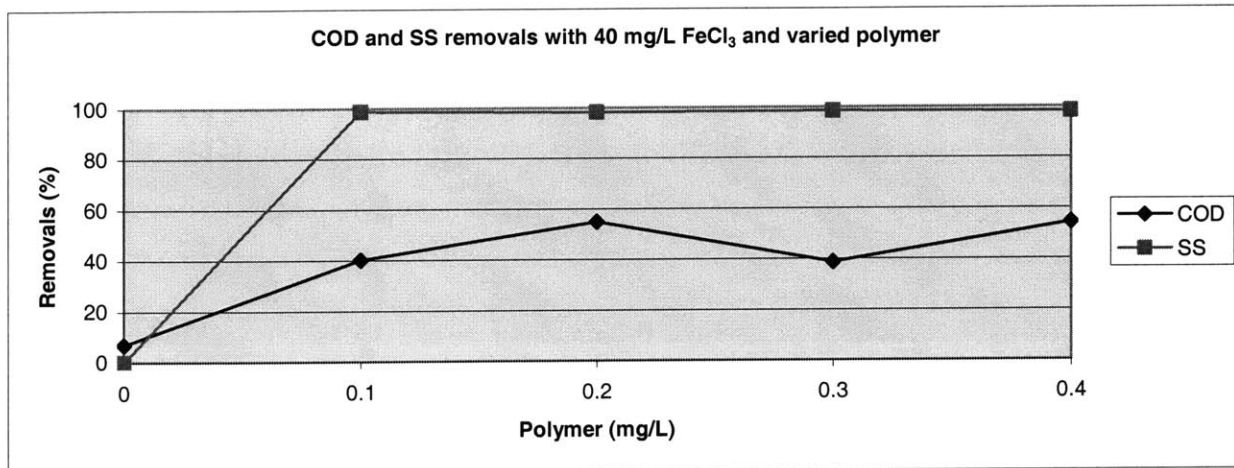


Figure 4-24: Experiment 3A COD and SS removals with 40 mg/L FeCl₃ and polymer

In figure 4-24 above, the same jar test was reiterated (with different raw wastewater) to check for the accuracy of using ferric chloride with varying polymer doses. The FeCl₃ concentration was held constant here at 40 mg/L while the polymer concentration varied between 0.1 and 0.4 mg/L. Again, using more than 0.1 mg/L of the polymer did not yield any increases in SS removal efficiency and only caused a slight increase in COD removals. Bearing in mind that 0.4 mg/L would be a relatively expensive investment, the 13% increase in COD removal does not appear significant. The optimal dose was therefore selected here to be 40 mg/L FeCl₃ with 0.1 mg/L of polymer.

It is very important to note there that the observed removal efficiencies for conventional treatment are significantly lower than those published and expected for treatment without chemical addition.

The two jar tests above therefore indicate that 0.1 mg/L of polymer is very sufficient to treat the Paraty wastewater. Although doses of recommended FeCl₃ varied between 30 and 40 mg/L, both figures remain within acceptable range.

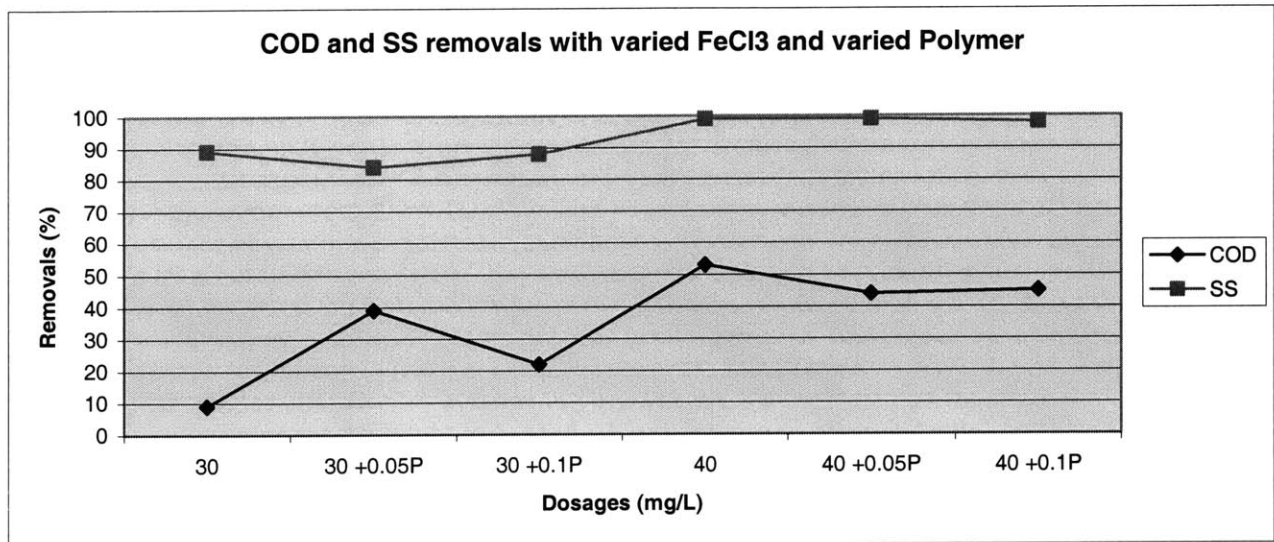


Figure 4-25: Experiment 3A: COD and SS removals with varied FeCl₃ and polymer

The jar test above was used to try to confirm the FeCl₃ and polymer doses shown in Figure 4-24 before. Both FeCl₃ and polymer concentrations here were varied, in particular, to look for trends in the addition of polymers to the raw wastewater and to note the observed removals of COD and SS.

Since the SS removals were very high (greater than 85% on average), COD became the limiting factor in analyzing the results from this jar test. 0.05 mg/L of polymer had a larger effect on COD removals than did 0.1 mg/L, both being used with 30 mg/L FeCl₃. The COD removals at 40 g/L FeCl₃ however were significantly high (50%) and only decreased with the addition of polymers. This test therefore pointed to using 40 mg/L FeCl₃ alone without the use of polymers

Based on the results from the three jar tests above, it is hard to determine what polymer dose is most suitable for use in conjunction with FeCl₃. General conclusions can be made however regarding the general performance and effect of polymers on jar tests with FeCl₃ as the only coagulant:

1. As FeCl₃ doses increase, suspended solids removals increase. This effect is not as noticeable if SS removals are already very high.
2. The use of small polymer doses seems to display better COD removals with larger doses of FeCl₃ (i.e. polymers used with FeCl₃ concentrations smaller than 20 mg/L did not show large increases in removal efficiencies)
3. The use of polymers (even in relatively small doses of 0.1 mg/L) caused very large jumps in SS removals (typically varying from 30 to 90%). Using additional polymer doses did not lead to further removals. This is simply because SS removals were already 90% with polymer doses of 0.1mg/L
4. Increasing polymer concentrations does not necessarily increase removals.

4.5.2 EXPERIMENT 3B: Ferric Chloride and Seawater

These tests were designed to observe the reaction of FeCl_3 to using seawater as a coagulation enhancement mechanism. Jar tests were therefore performed with varying FeCl_3 and seawater concentrations to test for the most optimal seawater dose to use with FeCl_3 .

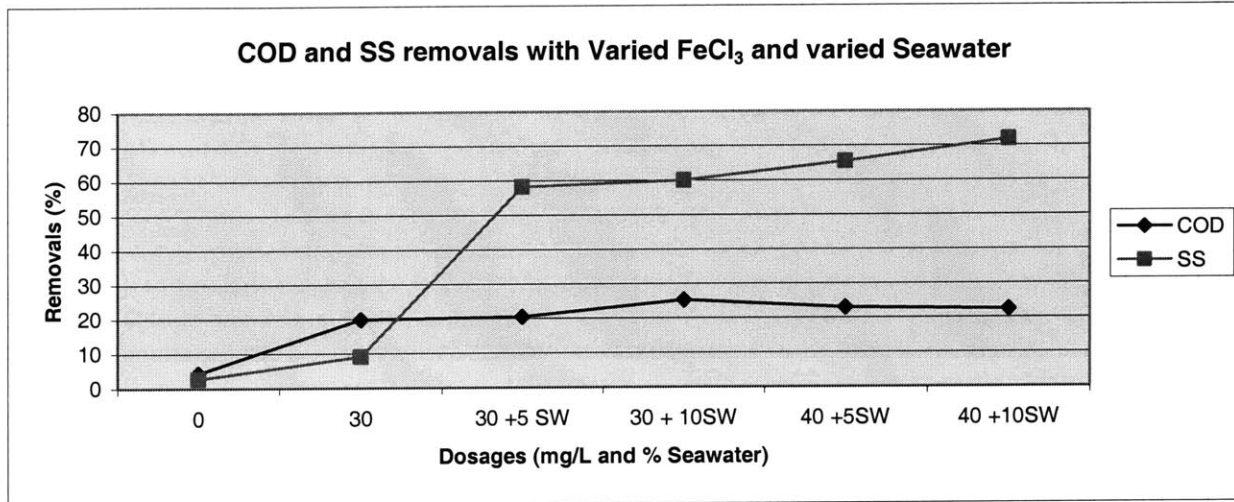


Figure 4-26: Experiment 3B COD and SS removals with varied FeCl_3 and seawater

In this test, both FeCl_3 and seawater concentrations were varied to test for the most optimal combination to yield the highest COD and SS removals.

Adding 5% seawater to the 30 mg/L FeCl_3 , increased SS removals from 20 to 60% marking a 200% increase. COD removals remained constant at 20% removal for both tests. The use of additional seawater (10%) with 30 mg/L FeCl_3 did not induce increases in SS removals that remained constant at 60%.

Using 40 mg/L FeCl_3 did not yield very significant increases in SS removals and seemed to cause COD removals to begin to decrease. These COD readings should not have been altered by the presence of chlorides in the samples as was explained in Section 2.3.3 above of Chapter 2.

The 30 mg/L FeCl_3 and 5 % seawater was therefore considered the most optimal dose for this jar test.

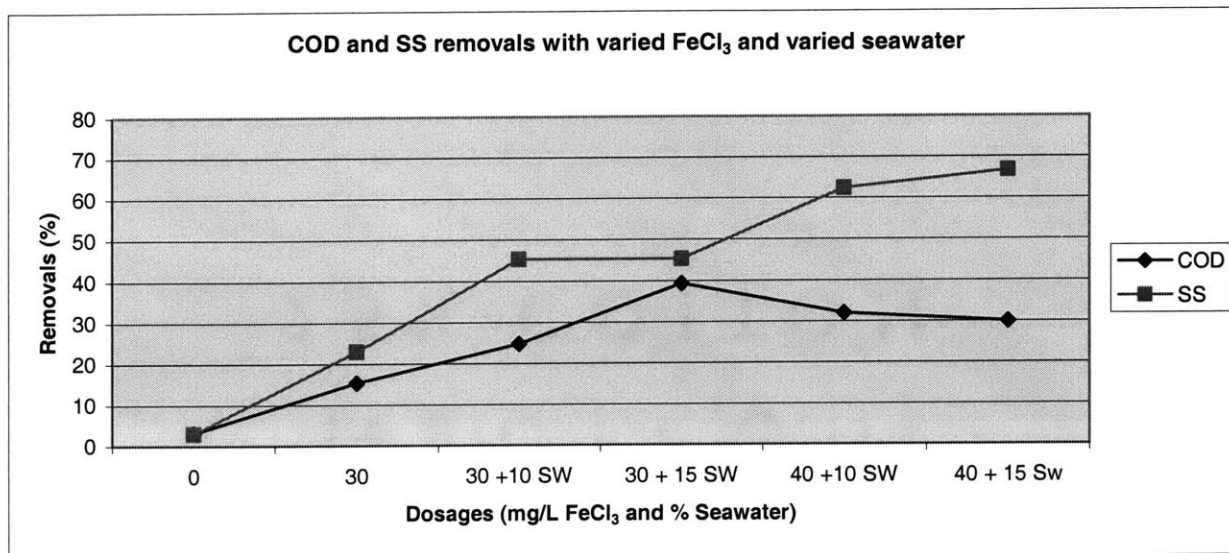


Figure 4-27: Experiment 3B COD and SS removals with varied FeCl₃ and seawater

In this test, various FeCl₃ concentrations were again tested with different seawater volumes and the SS and COD removals were consequently observed.

When FeCl₃ was tested again with the use of seawater, the observed removal efficiencies were lower than in previous tests (See Figure 4-26 above). At a FeCl₃ dose of 30 mg/L and 10% seawater the SS removal was 46% and the COD 25%, compared to the 20% COD and 60% SS removals from using 30mg/L FeCl₃ and 5% seawater in Figure 4-26 above. The addition of seawater only enhanced COD removals at relatively low FeCl₃ doses (i.e. less than 30 mg/L). When larger doses of FeCl₃ were used, the COD removal rate steadily declined whereas the suspended solids continued to increase. It is also critical to note that chloride interference is an important aspect of adding 15% of seawater and that the COD readings were therefore incorrect (Please refer to Section 2.3.3. on chloride interference with the COD Hach vial readings)

The general conclusions on the use of seawater and FeCl₃ in conjunction therefore are:

1. Higher volumes of seawater seem to cause larger increases in SS and COD removals with smaller concentrations of FeCl₃ (i.e. 30 mg/L FeCl₃ and 15% Seawater yielded higher results than 40 mg/L FeCl₃ and 15% seawater)
2. COD removals seem to decrease as the percentage of seawater increases. This is probably due to the chloride interference with Hach COD vial readings (Section 2.3.3)
3. Small seawater additions caused large increases in suspended solids removals (removals with seawater increasing from 20 to 90% marking a 350% increase in removal efficiencies.)

4.5.3 EXPERIMENT 3C: Ferric Chloride, Seawater and Polymer

Finally, FeCl_3 , seawater and polymers were tested simultaneously to gauge the effect of the multiple presences on the SS and COD removals.

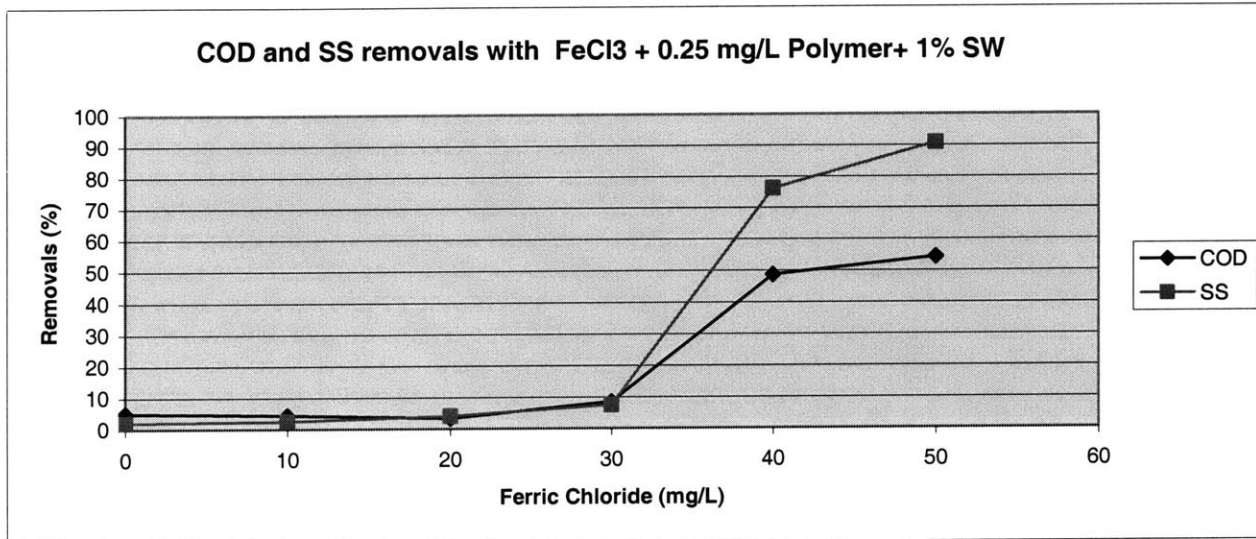


Figure 4-28: Experiment 3C: COD and SS removals with varied FeCl_3 , 0.25 mg/L polymer and 1% Seawater

In this jar test, varying FeCl_3 concentrations (ranging from 10 to 60 mg/L) were tested in conjunction with 0.25-mg/L of polymer concentration and 1% seawater.

The polymer and seawater did not have an effect on either SS or COD removals until a FeCl_3 of 30 mg/L was used. This is consistent with the preliminary conclusions made in section 4.5.2 above concerning threshold limits for polymers and seawater to take effect on contaminant removals. The COD and SS removals (10% for both) at 30 mg/L FeCl_3 , 0.25 mg/L (which can be considered a strong dose of polymer) and 1% seawater are significantly lower than expected removal rates at these dosages of FeCl_3 and polymer especially. The results from this jar test cannot be therefore completely relied upon. This is particularly noticeable since the removals from conventional treatment are so low. Overdosing on polymers might be a potential caused for the low removals in these jar tests.

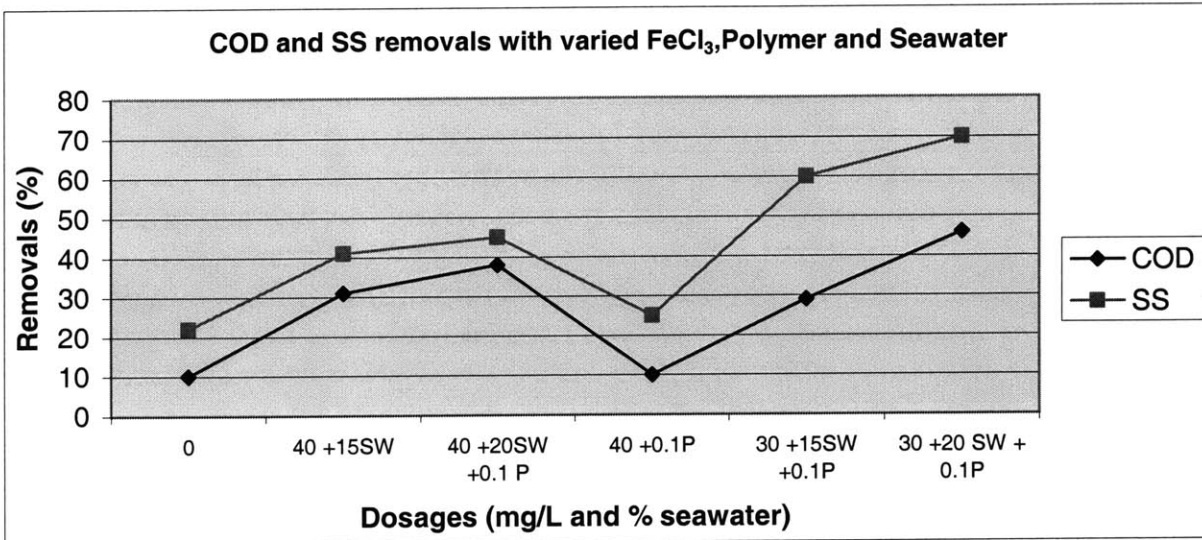


Figure 4-29 Experiment 3B COD and SS removals with varied seawater, polymer and FeCl₃

In this jar test, all of FeCl₃, seawater and polymer concentrations were varied to observe for trends in COD and SS removals. The COD results from the addition of 15% of seawater are discarded due to chloride interference.

When 0.1 mg/L polymer alone was added to the 40 mg/L FeCl₃, SS and COD removals were low compared to when 15% seawater was used with the same FeCl₃ dose. The use of seawater seems to be more effective in this jar test, therefore, at increasing SS and COD removals.

Using 40mg/L FeCl₃ with 20% Seawater and 0.1mg/L polymer instead of 40 mg/L FeCl₃ with 15% Seawater also did not yield significantly higher SS and COD removals. In fact, using 30mg/L FeCl₃, 0.1 mg/L polymer and 20% seawater yielded much higher SS and COD removal rates and this dosage was therefore chosen as the most optimal for this jar test.

The conclusions on the combined use of FeCl₃, seawater and polymer are as follows:

1. Polymer alone with FeCl₃ is not as efficient as seawater acting with FeCl₃.
2. When FeCl₃, seawater and polymer were used together, the use of smaller FeCl₃ concentrations performed as well as larger FeCl₃ concentration dosages.
3. It is also important to note that the results from using 40 mg/L FeCl₃ and 0.1 mg/L polymer in this jar test are not consistent with other jar test results and indicate that experimental error might have occurred.

4.5.4 POLYMER RECOMMENDATIONS FOR PARATY

Based on the results from the jar tests described above, the most optimal polymer dose recommended for the Paraty CEPT plant is 0.1 mg/L

This polymer dose seems to work most efficiently with FeCl₃ doses ranging from 30mg/L to 40 mg/L and with small seawater concentrations by volume ranging from 1 to 5%.

4.5.5 DESIGN PARAMETERS FOR PARATY

Based on the jar tests results displayed in sections 4.2 through 4.5.4 above, and taking from the conclusions on the general trends that ferric chloride, seawater and polymers, the following Table 4-7 was generated to summarize the raw wastewater characteristics in Paraty and the required dosages of chemicals for the design of the CEPT plant:

Raw Wastewater Characteristics	
Influent SS	200 mg/L
Influent COD	350 mg/L
Chemical Doses	
Ferric Chloride mg/L	40 mg/L
Seawater Volume	5%
Polymer mg/L	0.1 mg/L
Expected Removals	
SS removals	85%
COD removals	55%

Table 4-7: Design parameters for Paraty

CHAPTER 5 :WASTEWATER TREATMENT AT DEER ISLAND WWTP, BOSTON

5.1 Introduction

The Boston Deer Island wastewater treatment plant (Figure 5-1 below) is the second largest wastewater treatment plant in the United States and serves a total population of 2 million people producing 390 million average gallons of influent per day, with a maximum capacity of 1.27 billion gallons per day (MWRA, 2003). Although the plant is a secondary treatment plant and only uses conventional primary treatment for preliminary suspended solids and grit removal, jar tests were performed for this project to check the results that led to the conclusions on FeCl_3 , seawater and polymer in Paraty.



Figure 5-1: Boston Deer Island Wastewater treatment plant (<http://www.bryant-engrs.com/projects/deer.htm>)

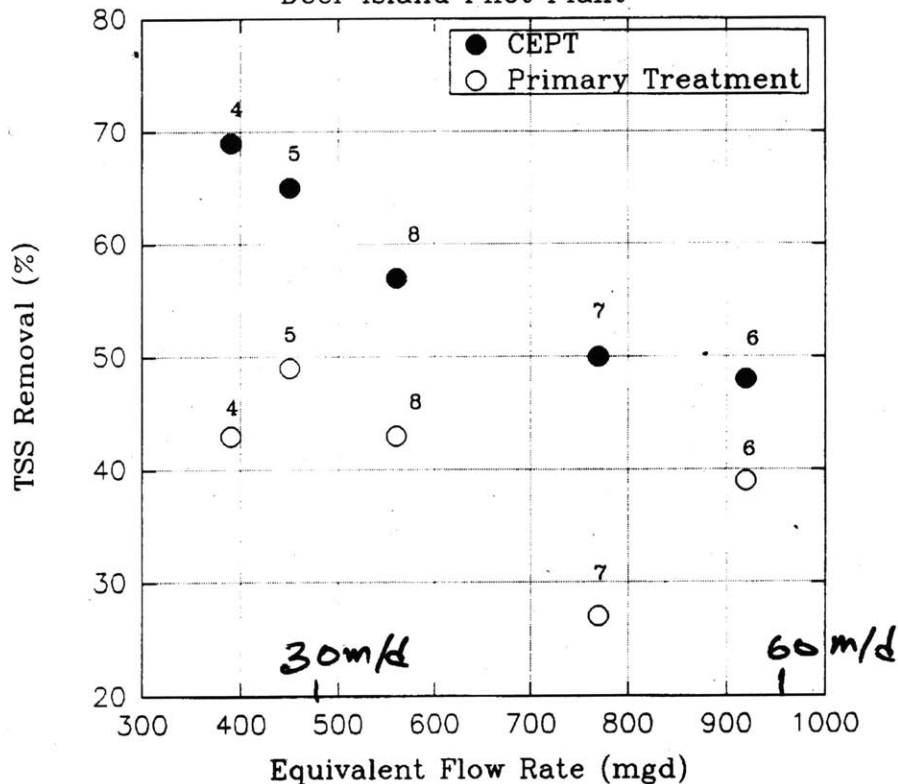
5.2 CEPT Pilot Plant Test at Deer Island Wastewater Treatment Plant

A series of pilot scale tests were performed at the Deer Island Wastewater Treatment Plant to test the efficiency of using CEPT to treat the influent of the Boston area served by the plant. The results from these tests are shown in Figures 5-2 through 5-6 below (Harleman, 2003). The results from the Deer Island Jar tests performed as part of this project were then compared and contrasted to the pilot scale results:

	TSS	COD	BOD ₅	Total P
No Chemicals	40	29	33	11
15 mg/L FeCl ₃	61	53	58	45
30 mg/L FeCl ₃	61	54	57	50

Table 5-1: Pilot Plant Summary of Removals

Comparison of Chemical Treatment with Conventional Primary
TSS % Removal vs. Equivalent Flow Rate
Deer Island Pilot Plant

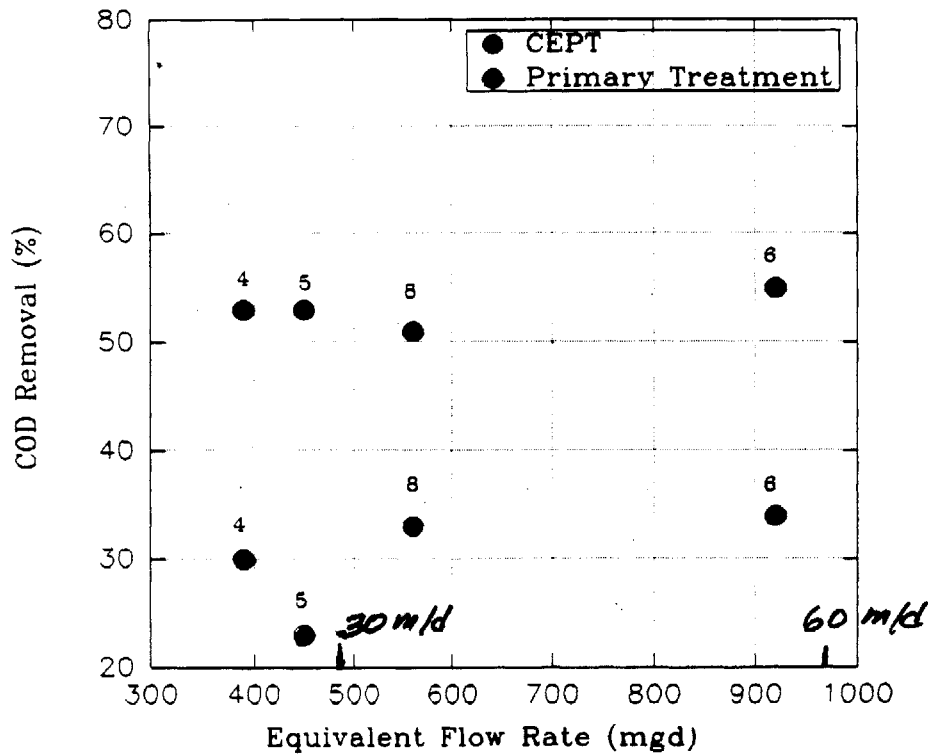


Tests 4, 7 & 8 = 15 mg/l FeCl₃ + 0.2 mg/l anion; Tests 5 & 6 = 30 mg/l FeCl₃ + 0.2 mg/l anion

Figure 5-2: TSS removals with conventional and CEPT treatment

Figure 5-2 above shows the increase in TSS removal efficiencies when CEPT was used as opposed to conventional primary treatment in which chemicals were not added. The TSS removals with conventional primary treatment did not exceed 50%, whereas CEPT removals reached a high of 70%, marking a large increase in removal efficiency.

Comparison of Chemical Treatment with Conventional Primary
 COD % Removal vs. Equivalent Flow Rate
 Deer Island Pilot Plant Study



Tests 4, 7 & 8 = 15 mg/l FeCl₃ = 0.2 mg/l anion; Tests 5 & 6 = 30 mg/l FeCl₃ + 0.2 mg/l anion

Figure 5-3: COD removals at the Deer Island Pilot Scale Test

The COD removals for the primary treatment did not exceed 40% and those of CEPT were consistently higher at approximately 55% (Figure 5-3 above). This is an obvious increase in removal efficiencies and warrants the use of CEPT as an ideal treatment alternative.

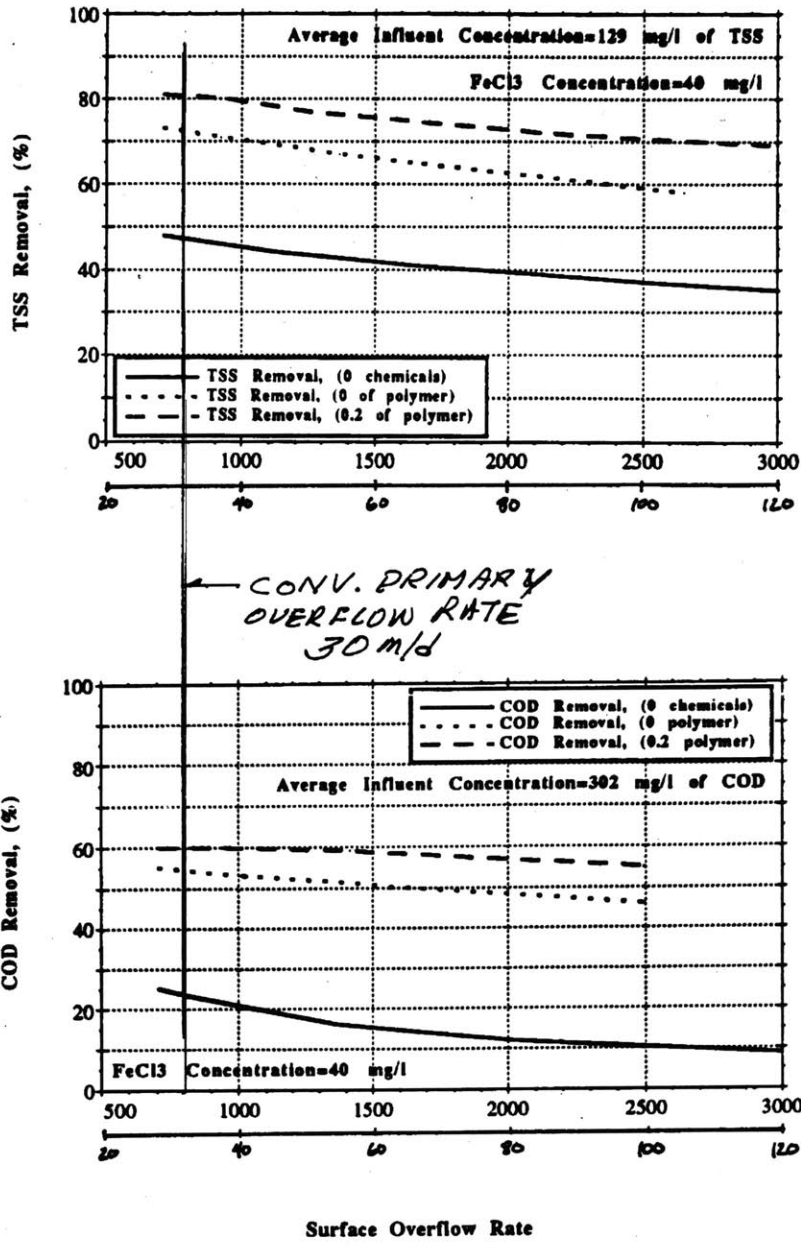


Figure 5-4: Pilot Scale Removal Rates

Since the Deer Island Pilot-scale test yielded such good results, the Deer Island influent was seen as an ideal sampling location to test the reliability of the Paraty results.

5.3 Salinity in the Boston Influent

The influent to the Deer Island Wastewater Treatment Plant was estimated to contain an initial volume of seawater that would therefore affect the removal rates of suspended solids and COD in the jar tests for this project.

The salinity of the Boston effluent was estimated by two methods. These are described in the sections below:

5.3.1 *Conductivity*

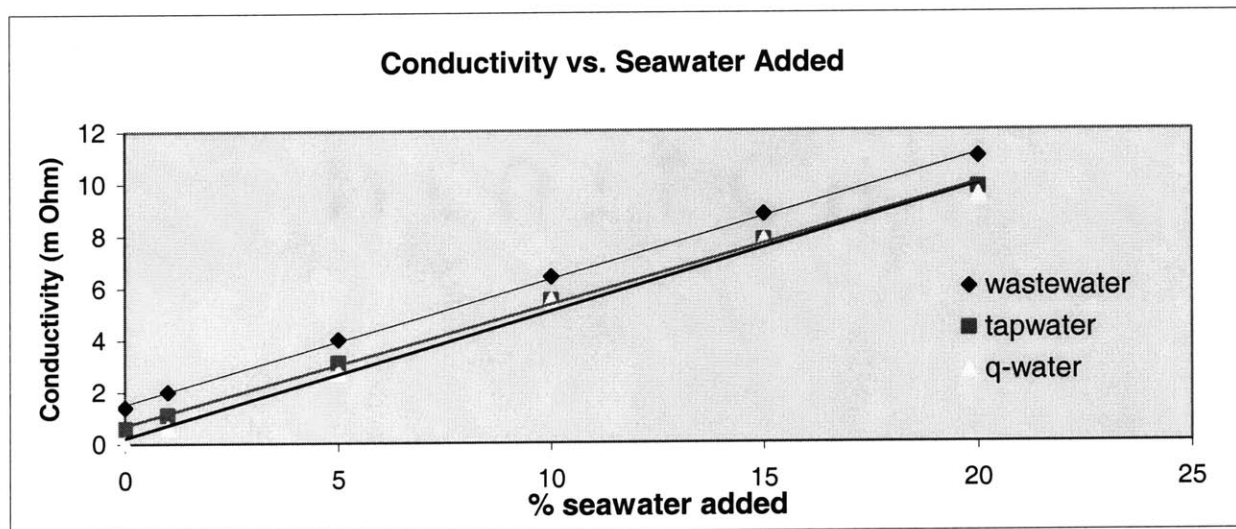


Figure 5-4: Conductivity verses Seawater Added

The conductivity of the wastewater sample was compared and contrasted to the conductivity of q-water (or distilled water) and to that of tapwater for varying concentrations of seawater added. Figure 5-4 therefore shows that the wastewater contains a maximum concentration of 2% seawater already present in the influent. This is important to further data analyses of the laboratory experiments that were performed on the Deer Island influent with the addition of seawater. Adding 5% of seawater by volume would therefore have the net effect of looking at the reaction of the influent to a seawater addition of 7% since the sample already contained an assumed maximum seawater concentration of 2%.

5.3.2 Salinity Equation

The standard methods manual (Standard Methods, 2003) encourages the use of conductivity as a measure of salinity since a seawater with a conductivity at 15 degrees Celsius equal to that of a KCL solution containing a mass of 32.4356 g in a mass of 1 Kg solution is defined as having a salinity of 35 parts per thousand” (Standard Methods, 2003).

The salinity dependence on resistivity (the inverse of conductivity), R_t , as a function of temperature of a given sample to a standard $S = 35$ seawater is used to determine the salinity:

$$S = 0.008 + (-0.1692)R_t^{1/2} + (25.3851)R_t + (14.0941)R_t^{3/2} + (-7.0261)R_t^2 + (2.7081)R_t^{5/2} + \Delta S$$

Where:

$$\Delta S = [(t-15)/(1+0.0162(t-15))](0.0005 - 0.0056)R_t^{1/2} - 0.0066)R_t - 0.0375)R_t^{3/2} + (0.0636)R_t^2 - (0.0144)R_t^{5/2}$$

Solving this equation also yields a salinity of approximately 2‰.

5.4 DEER ISLAND Data Analysis

5.4.1 Experiment One

Experiment one was performed on raw wastewater collected from Deer Island consisted of two jar tests. Jar Test 1 used ferric chloride in varying concentrations ranging from 1 to 30 mg/L and Jar Test 2 used the same $FeCl_3$ concentrations but also used 5% seawater (by volume) in the influent. Results from COD, SS, and turbidity removals are presented in Figure 5-6, 5-7 and 5-8

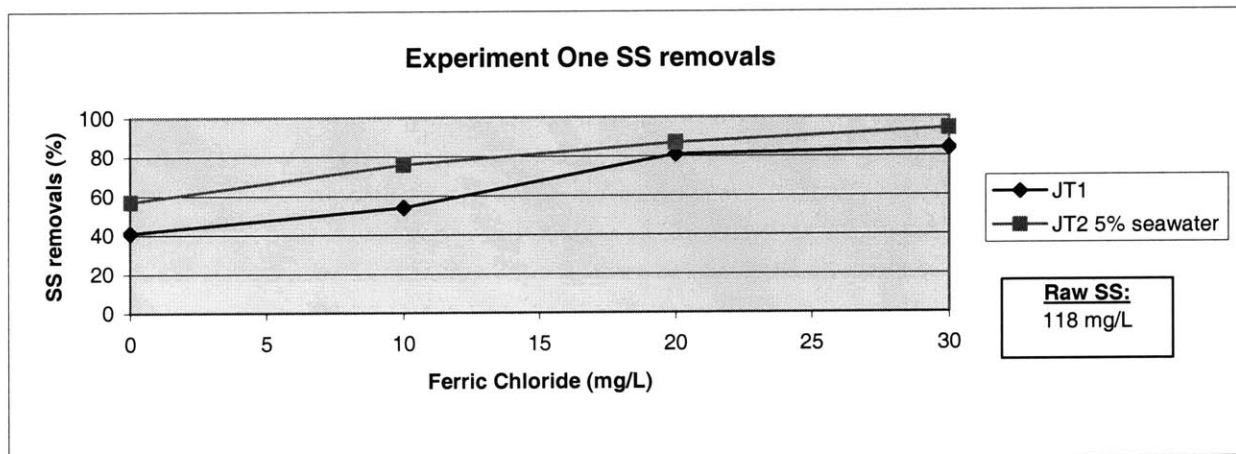


Figure 5-6: Experiment One: SS removals

Jar Test 2, (for which 5% seawater was added to the influent), yielded higher SS removal rates compared to jar test 1 to which no seawater was added. The minimum difference in removals however occurred at a FeCl_3 dose of 20 mg/L and at the maximum difference at a FeCl_3 dose of 10 mg/L. It is important to note that the SS removals without seawater at 20mg/L FeCl_3 were already high at 80% and that additional removals would not be expected. The most optimal doses of FeCl_3 therefore would be 10 mg/L with 5% seawater addition and 20 mg/L FeCl_3 without seawater.

It is of critical importance to note that the Deer Island influent is assumed to already contain a certain concentration of seawater as was explained in Section 5.3 above. Adding 5% seawater to the Deer Island wastewater is therefore expected to yield removals identical to adding 7% to a corresponding wastewater that does not contain initial seawater content.

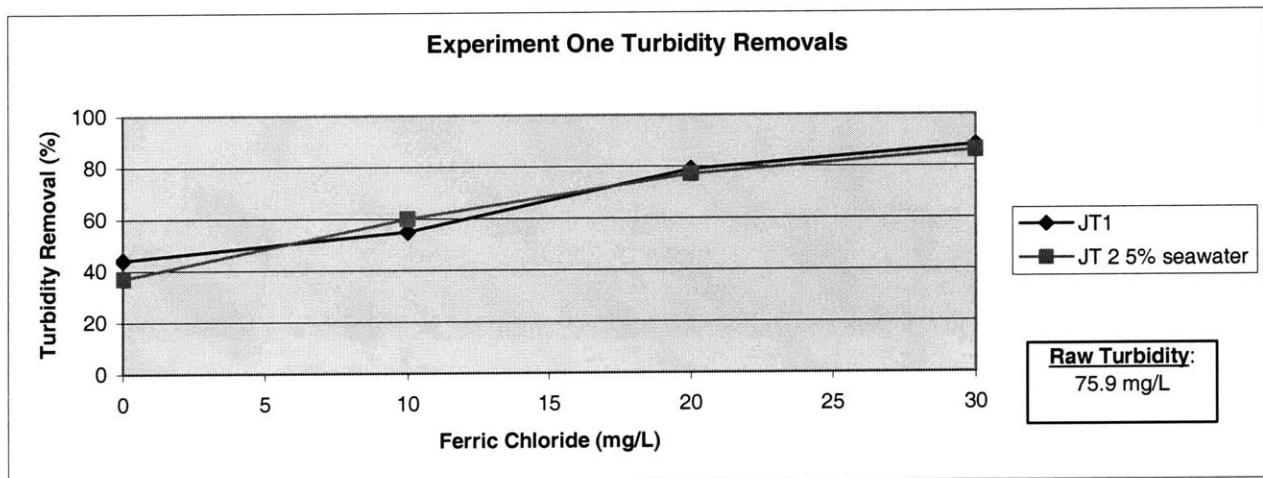


Figure 5-7: Experiment One Turbidity Removals

Turbidity removals were unchanged with the addition of seawater to jar test 2 compared to jar test 1. This points to concluding that the addition of seawater does not affect turbidity.

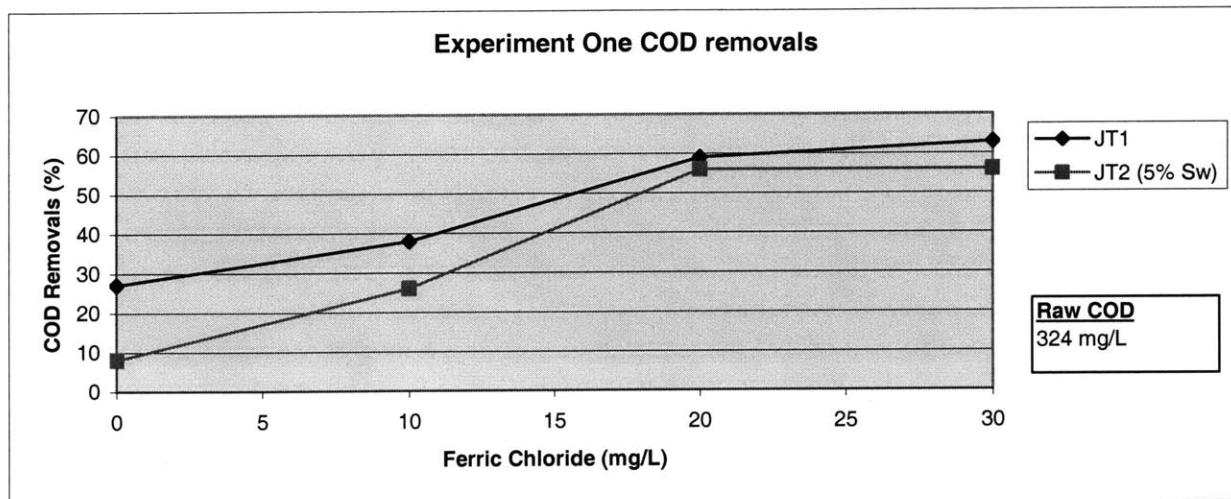


Figure 5-8: Experiment One COD Removals

Jar test 2 shows that COD removals decreased with the addition of seawater to the influent compared to jar test 1. The removals were similar in the two jar tests when 20 mg/L FeCl₃ was used and the difference in removals remained relatively approximate with doses of FeCl₃ higher than 20 mg/L. This points to a potential sensitivity of the effect of seawater for flows treated with low doses of FeCl₃. This is inconsistent however with the results from the Hong Kong Stone Cutter's Island Plant which achieves 58% COD removals using 10 mg/L FeCl₃ and 20% seawater. It may be that higher concentrations of seawater are more compatible with lower FeCl₃ doses and that lower seawater concentrations are therefore more reactive with higher FeCl₃ concentrations.

5.3.2 Experiment Two:

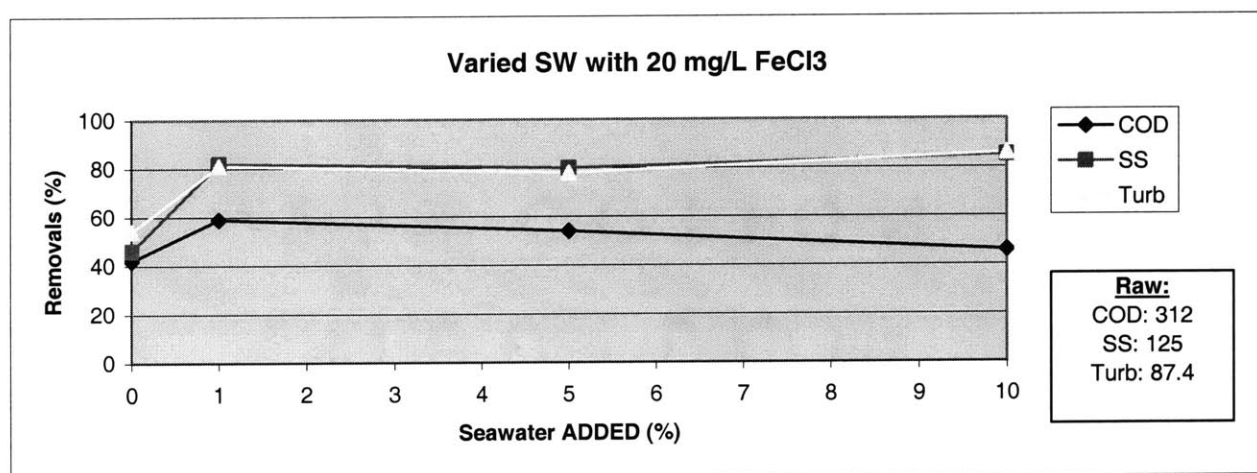


Figure 5-9: Experiment Two Removals

In Experiment two, the FeCl_3 dose was held constant at 20 mg/L while the concentrations of seawater were varied between 0 and 10% by volume. Again, it is of critical importance to realize that the abscissa of Figure 5.9 above represents the volume of seawater manually injected into the influent and does not represent the total volume of seawater in the beaker at any time since the Deer Island influent has seawater present initially.

The suspended solids and turbidity removals followed identical trends were only affected by the addition of small volumes of seawater ranging from 1 to 2%. With the ongoing addition of seawater, the suspended solids and turbidity removals remained constant at 80%. This is considered a very good SS removal for 20 mg/L FeCl_3 and the small seawater injection of 1%. COD removals also increased with the addition of 1% but then began to steadily decrease with the addition of more seawater. The optimal seawater dose from this jar test and with 20 mg/L FeCl_3 can therefore be identified at 1%.

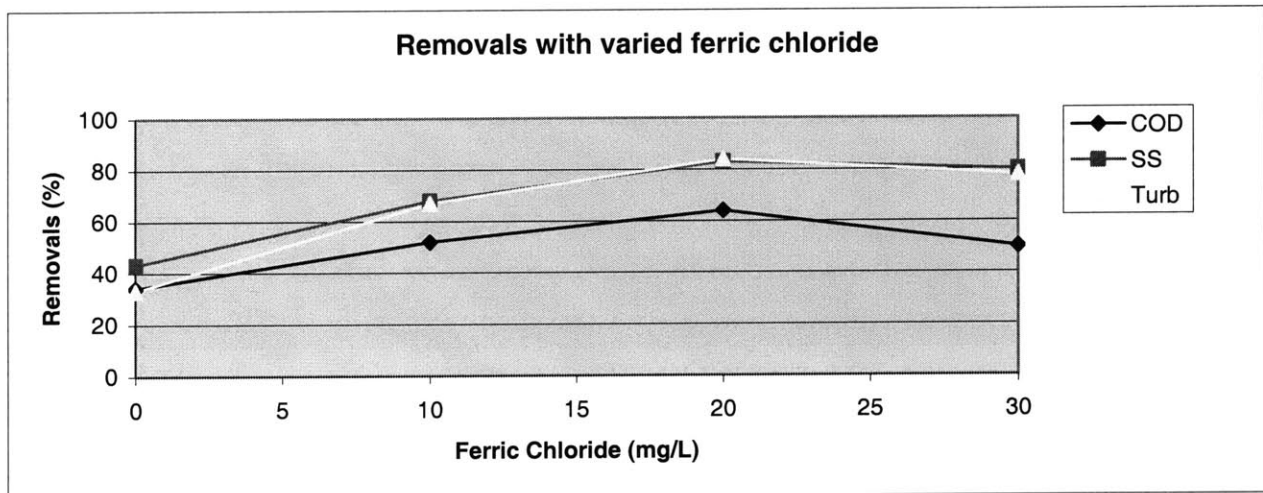


Figure 5-10: Removals with varied Ferric Chloride

In this jar test, ferric chloride concentrations were varied between 0 and 30 mg/L and no seawater was added to the beakers. Again, it is important to expect variations in the effects of FeCl_3 on SS, COD and turbidity removals since the influent in Deer Island is assumed to naturally contain 2% of seawater (See section 5.3)

The SS and turbidity removals increased steadily with the added FeCl_3 concentration and did not fluctuate very much higher than 80% past 20mg/L FeCl_3 .

COD removals followed the same trend and increased from the initial 34% to 60% at 20 mg/L FeCl₃. However, at 30mg/L FeCl₃, the COD decreased from 60% to 50%. This might be attributed to the natural presence of seawater in the Deer Island influent and the sensitivity of COD removals to the presence of seawater with the use of FeCl₃ as a coagulant.

It is also important to compare the results from this test to those from Section 5.4.1 above which help to show that the presence of seawater is responsible for higher SS and COD removals at relatively lower FeCl₃ concentrations.

5.3.3 EXPERIMENT THREE:

Experiment three was used again to test for the efficiency of adding seawater to ferric chloride and gauging the respective effects on SS and COD removals. It is essential to note that since the addition of seawater to a wastewater influent is a relatively new technique, a large number of jar tests and significant amount of research are required. Therefore the jar tests were repeatedly tried on the Deer Island influent to test the conclusions made in experiments one and two concerning seawater addition.

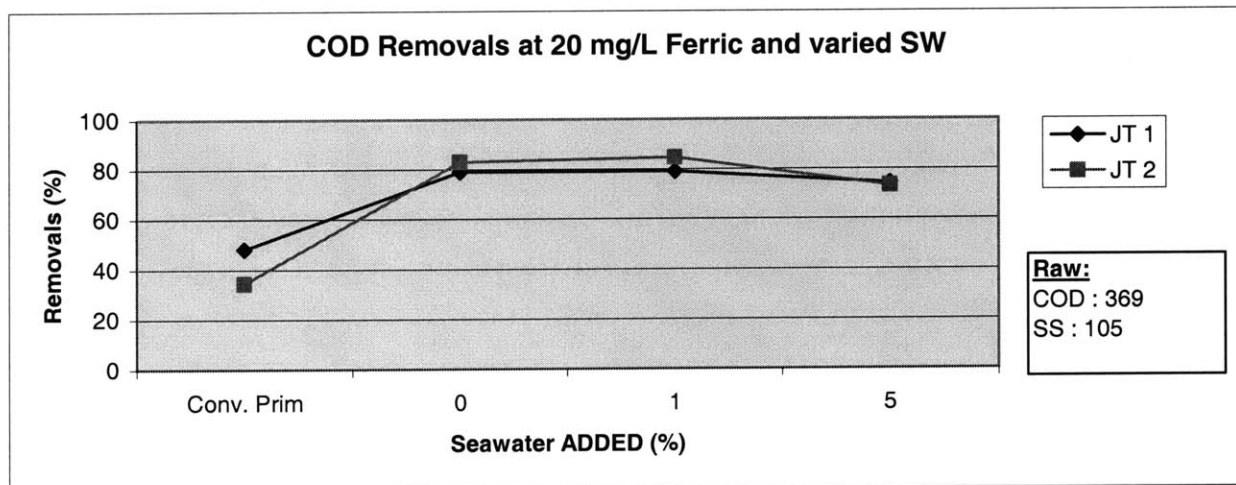


Figure 5-11: COD removals with varied Ferric Chloride and seawater

In this jar test, 20 mg/L FeCl₃ was added to every beaker in the jar test (except for the beaker representing conventional primary treatment). Seawater was also injected a % volumes varying between 0 and 5%. The Deer Island influent already contains seawater and therefore the abscissa of Figure 5-11 only represents the percentage of seawater added. It is does not represent the total % of seawater in the beaker.

The COD removals did not vary very much with the addition of seawater since with no seawater and at 20 mg/L FeCl₃, removals were 80% and remained constant at 80% with the addition of 1% seawater. This fact points to the same prior conclusion regarding seawater addition and its efficiency with lower FeCl₃ doses.

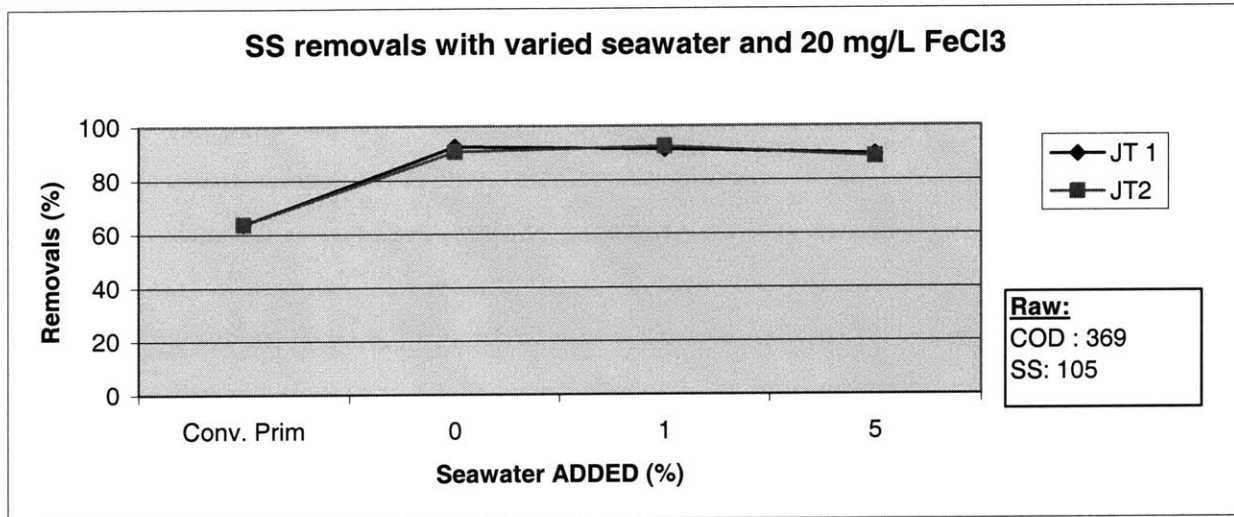


Figure 5-12: SS removals with varied seawater and 20 mg/L FeCl₃

For the same jar test described above, the suspended solids removals also followed trends identical to COD removals. The addition of seawater did not increase the SS removal that remained constant at 93%.

EXPERIMENT FOUR

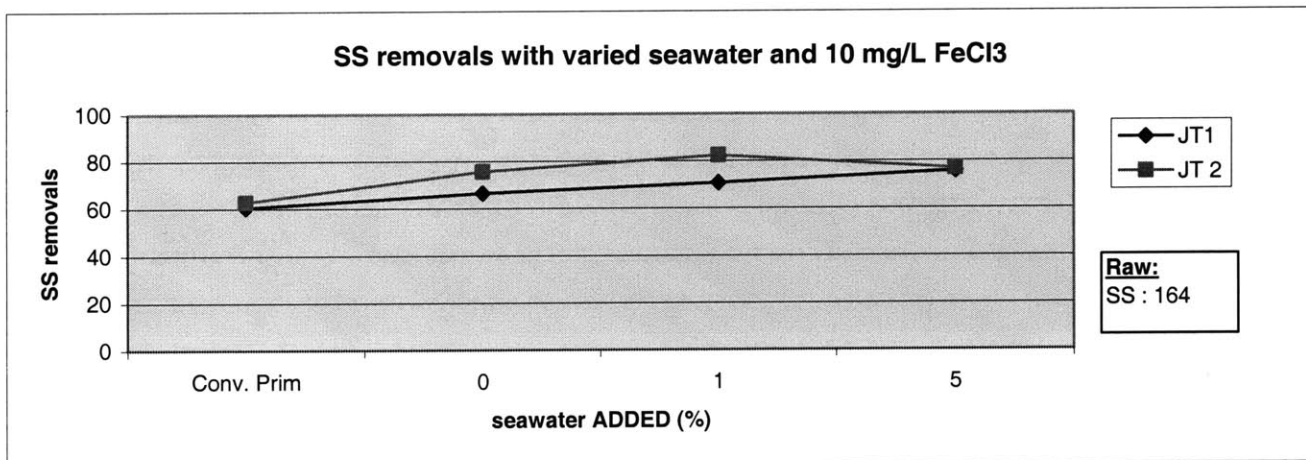


Figure 5-13: SS removals with varied seawater and 10 mg/L FeCl₃

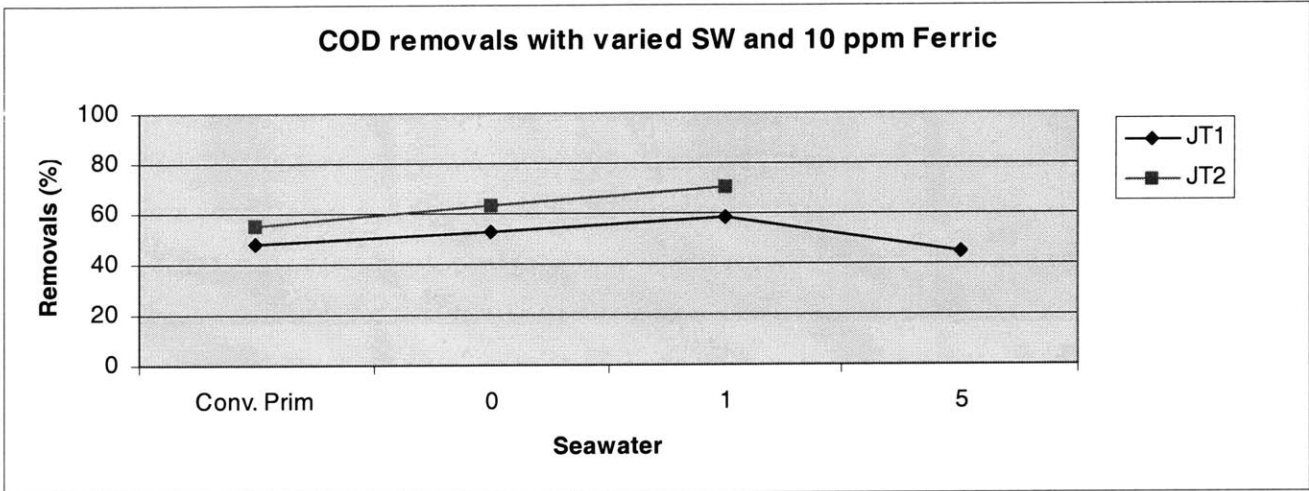


Figure 5-14: COD removals with varied SW and 10mg/L Ferric Chloride

CHAPTER SIX: DISINFECTION IN PARATY AND IN DEER ISLAND

6.1 Introduction

6.1.1 Constraints

Treated wastewater effluent is commonly discharged to a natural surface water body, such as a river or the ocean. Since surface water generally sustains human life and is an ecological habitat for large numbers of species, disease-causing organisms must be removed from treated wastewater effluent before being discharged to nature. Disinfection is the process used for the reduction of pathogenic microorganisms responsible for various diseases such as diarrhea or infectious hepatitis. Although pathogens can be removed with suspended solids during the sedimentation process, the settling processes alone do not meet the regulations for the treated wastewater effluent. Therefore the disinfection process is required in wastewater treatment. Before choosing a proper disinfection method for a wastewater treatment plant, the following criteria designated by the EPA should be considered (EPA, 1999).

1. Ability to destroy infectious organisms under normal operation conditions.
2. Safe and easy handling, storage, and shipping
3. Absence of toxic residuals and harmful byproducts
4. Affordable capital, operation and maintenance (O&M) costs

Indicator organisms are generally used to monitor the concentration of pathogens in water. Indicator organisms are microorganisms that originate from the same sources as the pathogens of interest and are often found in high numbers. Thus, it is assumed that pathogens exist in water when the indicator organisms are detected. Characteristics for an ideal indicator organism are described in the following.

Indicator organisms must:

1. Be present when fecal contamination is present
2. Be equal to or greater number than pathogenic organisms
3. Have same or greater survival characteristics in the environment as the target pathogens
4. Not reproduce during the culturing procedure
5. Be cheap and easy to cultivate compared to the target pathogen
6. Be a member of the intestinal microflora of warm-blooded animals (Metcalf & Eddy, 2002)

6.1.2 Regulations in the U.S and Brazil

Although no ideal indicator organism has been found, coliform is commonly used as an indicator organism. Humans discharge approximately one hundred billion coliforms per day per capita on average. Thus the water is considered free from disease-producing organisms when there are no detectable coliform bacteria in the water. The regulations for secondary treatment examine and control the levels of biochemical oxygen demand (BOD), total suspended solids (TSS), pH, and fecal coliform bacteria. In the United States, the fecal coliform bacteria standards vary from less than 2.2 to 5000 MPN/100 ml depending on the quality of receiving water and the reuse application. For the receiving water, ≤ 200 MPN FC/100 ml is the most common standard. According to state ocean water quality standards in California, which have some of the strictest standards in the United States, the minimum protective fecal coliform bacteria standards for waters adjacent to public beaches and public water-contact sports areas is 200 MPN/100ml based on the results of at least five weekly samples during any 30-day sampling period (Blumenthal. U. J. et al, 2000).

In Paraty, Brazil, there are no regulations concerning the acceptable level of coliform concentration in discharged treated wastewater. According to Brazilian regulation issued by the Environmental Policy Commission, however, the maximum level of fecal coliforms in treated wastewater effluent discharged into the natural water is 1000MPN/100ml. Considering the proposed CEPT effluent discharging points, which are near the beach, and regulation of the United States and Brazil, however, 200 FC MPN/100ml can be adopted for the effluent standard because the beaches are main popular attractions in Paraty.

6.2 Characteristics of an Ideal Disinfection Agent

Disinfection can be performed with the use of chemical agents, physical agents, mechanical means, and ultraviolet (UV) radiation. To safely achieve the desired concentration of coliform safely, disinfectants would have to cover the wide range of wastewater quality. The characteristics for an ideal disinfection agent are shown in Table 6.1, and are critical to choosing an appropriate disinfection agent.

Characteristic	Properties/response
Availability	Should be available in large quantities and reasonably priced
Deodorizing ability	Should deodorize while disinfecting
Homogeneity	Solution must be uniform in composition
Interaction with extraneous material	Should not be absorbed by organic matter other than bacterial cells
Noncorrosive and nonstaining	Should not disfigure metals or stain clothing
Nontoxic to higher forms of life	Should be toxic to microorganisms and nontoxic to humans and other animals
Penetration	Should have the capacity to penetrate through surfaces
Safety	Should be safe to transport, store, handle, and use
Solubility	Must be soluble in water or cell tissue
Stability	Should have low loss of germicidal action with time on standing
Toxicity to microorganisms	Should be effective at high dilutions
Toxicity at ambient temperatures	Should be effective in ambient temperature range

Table 6.1 Characteristics of an ideal disinfectant (Metcalf & Eddy, 2002)

In addition, several factors that affect the efficiency of disinfection agents should be considered before application. These are contact time, concentration of the disinfectant, intensity and nature of physical agent or means, temperature, types of target organisms, and nature of suspending liquid.

6.3 Disinfection with chlorine

Chlorine is one of the most commonly used disinfection agents throughout the world. Chlorination technology is therefore well established. Since chlorination is cheap relative to UV radiation and ozone disinfection, it can significantly reduce the cost of wastewater treatment. This can be an important factor of consideration in the developing areas such as Paraty. The forms of chlorine used for wastewater treatment process are compressed gas (Cl_2), solutions of sodium hypochlorite (NaOCl), or solid calcium hypochlorite (Ca(OCl)_2) which are chemically equivalent. Chlorine dioxide (ClO_2) is also another form of chlorine. Safety precautions must be

taken in the storage, shipping, and handling because of the corrosion and toxicity of all forms of chlorine. The characteristics of various forms of chlorine are presented in Table 6.2 below.

Compound	Molecular weight	Chlorine equivalent	Actual Chlorine, %	Available Chlorine, %
Cl ₂	71	1	100	100
ClO ₂	67.5	5	53	260
Ca(OCl) ₂	143	2	50	99
HOCl	52.5	2	68	135
NaOCl	74.5	2	48	95
NHCl ₂	86	2	83	165
NH ₂ Cl	51.5	2	69	138

Table 6.2 Actual and available chlorine in compounds containing chlorine (Metcalf & Eddy, 2002)
Actual Chlorine = % of Cl₂ in compounds, w/w
*Available Chlorine = Actual Chlorine * Chlorine Equivalent*

The disinfection efficiency of chlorine is dependent on the wastewater characteristics. Table 6.3 shows the impact of wastewater characteristics on chlorine. Other factors that affect the disinfection efficiency include contact time, temperature, alkalinity, and nitrogen content (EPA, 1999).

Wastewater Characteristic	Effects on Chlorine Disinfection
Ammonia	Forms chloramines when combined with chlorine
Biochemical Oxygen Demand (BOD)	The degree of interference depends on their functional groups and chemical structures
Hardness, Iron, Nitrate	Minor effect, if any
Nitrite	Reduces effectiveness of chlorine and results in THMs
pH	Affects distribution between hypochlorous acid and hypochlorite ions and among the various chloramines species
Total Suspended Solids (TSS)	Shielding of embedded bacteria and chlorine demand

Table 6.3 Wastewater characteristics affecting chlorination performance (EPA, 1999)

As mentioned in table 6-3, the level of suspended solids in treated wastewater affects the chlorination performance. According to the research of Robert Armon et al., 1995, suspended solids and soluble organic compounds are important in disinfection efficiency. Since suspended solids surround and shield microorganisms, disinfection agents cannot go through suspended solids and cannot inactivate the target microorganisms. Moreover, low suspended solids removal efficiencies can indicate that the coliform levels in the treated wastewater effluent is not quite different from the concentration in the effluent, since coliforms are included in the suspended solids (Metcalf & Eddy, 2002; Water Quality and Treatment, 2000). As shown Chapter 3, SS removal efficiency in CEPT reaches approximately 85 % that is appropriate level for the disinfecting the treated wastewater effluent. Moreover, seawater addition increases the efficiency of SS removal. Thus, chlorine may be more effective when seawater is added to the raw wastewater in CEPT because seawater help raise the SS removal efficiency. Typical chlorine dosages are showed in Table 6.4.

Type of wastewater	Initial Coliform, MPN/100mL	Chlorine dose, mg/L			
		Effluent standard, MPN/100mL			
		1000	200	23	2.2
Raw wastewater	10^7 - 10^9	15~40			
Primary effluent	10^7 - 10^9	10~30	20~40		
Trickling filter effluent	10^5 - 10^6	3~10	5~20	10~40	
Activated-sludge effluent	10^5 - 10^6	2~10	5~15	10~30	8~30
Filtered activated-sludge effluent	10^4 - 10^6	4~8	5~15	6~20	8~20
Nitrified effluent	10^4 - 10^6	4~12	6~16	8~18	8~16
Filtered nitrified effluent	10^4 - 10^6	4~10	6~12	8~14	4~10
Microfiltration effluent	10^1 - 10^3	1~3	2~4	2~6	0~2
Reverse osmosis	0	0	0	0	0
Septic tank effluent	10^7 - 10^9	20~40	40~60		
Intermittent sand filter effluent	10^2 - 10^4	1~5	2~8	5~10	8~18

Table 6.4 Typical chlorine dosages, based on combined chlorine unless otherwise indicated, required to achieve different effluent total coliform disinfection standards for various wastewaters based on a 30-min contact time (Metcalf & Eddy, 2002)

According to the table, the concentration of fecal coliform is rarely reduced by primary treatment. Due to the high levels of fecal coliform, it is impossible to disinfect primary effluent

efficiently by chlorine. The fecal coliform concentration in the activated sludge effluent is less as three-order magnitude as raw wastewater. Due to the reduction of fecal coliform, activated sludge effluent is disinfectable with chlorine. Since the removal efficiency of suspended solids of CEPT plant is as good as activated sludge treatment, the same order magnitude of reduction of fecal coliform by CEPT is expected. Therefore, CEPT effluent can be disinfected with less amount of chlorine saving the cost of chemical.

6.3.1 Types of Chlorine

6.3.1.1 Molecular Chlorine (Cl₂)

Molecular chlorine is a dense gas that, when subjected to pressures in excess of its vapor pressure, condenses into a liquid with the release of heat and with a 450-fold reduction in specific volume. Hence, chlorine is provided as a form of liquid under high pressure to reduce shipment volume (Metcalf & Eddy, 2002; Water Quality and Treatment, 2000).

6.3.1.2 Sodium Hypochlorite (NaOCl)

Sodium hypochlorite can be supplied in liquid form and available chlorine at the time of manufacturing is usually 12.5 to 17 percent. The decomposition rate of the solution depends on concentrations, exposure to light and heat. Therefore it must be stored in a cool location in a corrosion-resistant tank (Metcalf & Eddy, 2002). One of the disadvantages of sodium hypochlorite is cost. The cost of different types of chlorine will be discussed in section 6.3.5. It is available to generate sodium hypochlorite from sodium chloride (NaCl) or seawater. However, the use of onsite generation systems is limited due to high electric power cost.

The hydrolysis reaction of sodium hypochlorite is as follows:

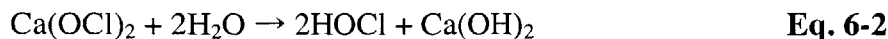


6.3.1.3 Calcium Hypochlorite (Ca(OCl)₂)

Calcium hypochlorite is available in a dry or a wet form, and is commonly used to treat the wastewater effluent from textile and paper mills under controlled conditions (PPG Industries, Inc. 1999). High quality calcium hypochlorite contains more than 70% available chlorine. Its oxidizing potential is high, so it should be stored in a cool, dry location separated from other chemicals in corrosion-resistant storage containers. Calcium hypochlorite is more expensive than

molecular chlorine, and its available strength is reduced on storage. Handling of calcium hypochlorite can be difficult, since metering pumps, piping and valves can be clogged because calcium hypochlorite is likely to crystallize.

The hydrolysis reaction of calcium hypochlorite is as follows:



6.3.2 Reactions of Chlorine

6.3.2.1 Hydrolysis of Chlorine

When molecular chlorine is added to water, it equilibrates with aqueous chlorine, and then aqueous chlorine is hydrolyzed to form hypochlorous acid, a chloride ion, and a proton as described in equation 1 below.



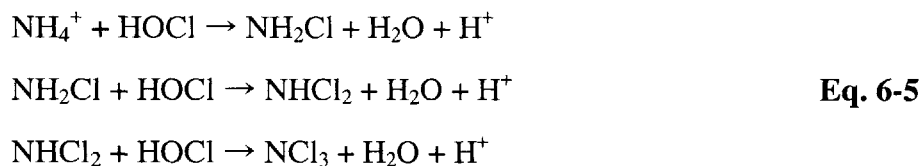
Hypochlorous acid is a weak acid and dissociates to the hypochlorite ion and to a proton.



The concentration of hypochlorous acid and hypochlorite ion is determined by the dissociation constant ($\text{pK}_a \approx 7.6$ at 25°C) depending on the pH and the total concentration of chlorine. The total amount of HOCl and OCl⁻ in water is the “free available chlorine.” Because the disinfection efficiency of HOCl is about 40 to 80 times that of OCl⁻, the actual disinfection efficiency of chlorine varies according to pH (Water Quality and Treatment, 2000).

6.3.2.2 Reaction of Chlorine with Ammonia

Chlorine may react with ammonia and amino nitrogen compounds to transform into a less biocidal form. In the presence of ammonium ion, free chlorine reacts with it to form chloramines.



The ratio of concentrations of each compound depends on the pH, temperature, contact time, and the ratio of chlorine to ammonia (White, 1999). Each of the chloramines (monochloramine (NH₂Cl), dichloramine (NHCl₂), and trichloramine (NCl₃)) contributes to the total or combined chlorine residual in water. Total chlorine includes free chlorine compounds and reactive

chloramines. The combined chlorine forms are considerably less effective for viruses and cyst, and the reaction rate is slower than that of free chlorine (Water Quality and Treatment, 2000). Chlorine readily oxidizes inorganic, and organic substances as it is added in water. When these reactions are finished, the additional chlorine reacts with ammonia to form chloramines between points A and B (Fig. 6.1).

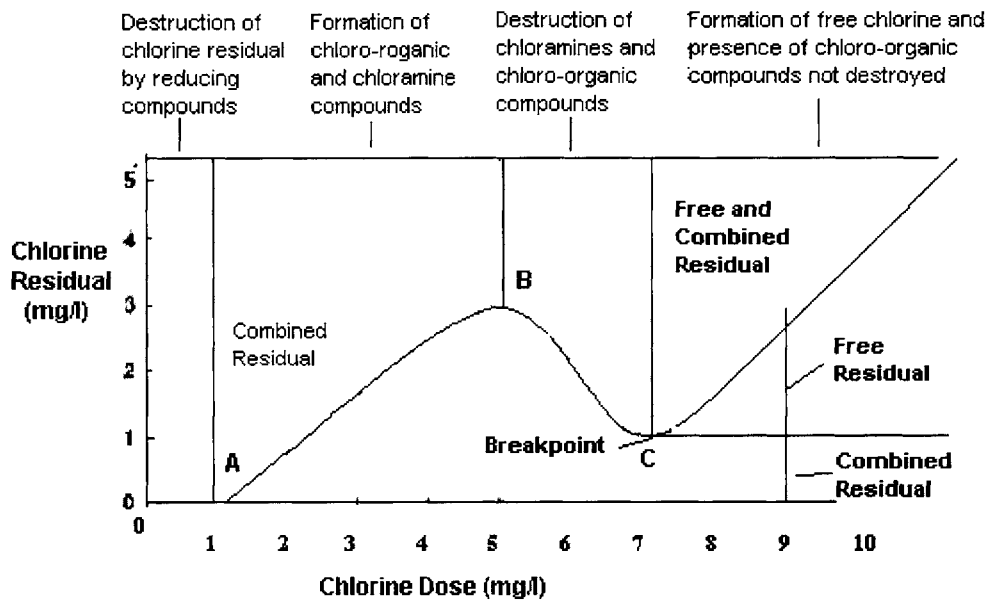


Fig 6.1 Chlorine breakpoint (Metcalf & Eddy, 2002)

Between point B and point C, the breakpoint, chloramines are oxidized to nitrous oxide (N_2O) and nitrogen (N_2), and ammonia nitrogen can be removed by this oxidation reaction. The residual chlorine increases linearly with additional dosage after the breakpoint. Theoretically, the weight ratio of chlorine to ammonia nitrogen at the breakpoint is 7.6 to 1, and the weight ratio at point B is about 5.0 to 1. When free residual chlorine is obtained, the effective disinfection then can be assured. Therefore, the amount of chlorine over the breakpoint should be added to water. The amount of chlorine required to achieve a desired level of residual is called the “chlorine demand” (Metcalf & Eddy, 2002).

6.3.3 Chlorine Dioxide (ClO_2)

Chlorine Dioxide is another form of chlorine. The disinfection capability of chlorine dioxide is equal to or greater than chlorine. The half reaction for ClO_2 is as follows:



Free dissolved chlorine dioxide has an extremely high oxidation potential. The equivalent available chlorine content based on the reaction is equal to 263 % as compared to molecular chlorine. This means that 1g/L of ClO_2 is equivalent to 2.63g/L of chlorine (Water Quality and Treatment, 2000). Based on this information and Table 1 from the section 2 above, the required chlorine dioxide dosages for disinfection can be calculated. Because the data on the appropriate dosages of chlorine dioxide are limited, however, site-specific testing is recommended to determine proper dosage ranges.

The advantage of using chlorine dioxide as a disinfection agent is that the chlorine dioxide residuals and end products are degraded more quickly than chlorine residuals. This means that chlorine dioxide may not endanger aquatic life as chlorine does. In addition, chlorine dioxide does not produce the potentially toxic chlorinated organic compounds (Metcalf & Eddy, 2002; Water Quality and Treatment, 2000).

The DBPs of using chlorine dioxide are chlorite (ClO_2^-) and chlorate (Cl_2O_2), both of which are toxic. Chlorite can be produced during the generation of the chlorine dioxide and reduction of chlorine dioxide. The chlorate ion is produced by the oxidation of chlorine dioxide, the impurities in the sodium chlorite that is the source of chlorine dioxide generation, and the photolysis of chlorine dioxide.

6.3.4 Dechlorination

Chlorine is one of the common disinfectants for pathogenic organisms that endanger human health. At the same time, however, chlorine affects the natural environment. It may harm natural organisms directly, and may react with organic matter to form toxic compounds that can adversely affect the environment including water resource into which effluent is discharged. According to the EPA's Quality Criteria for water (1986), 0.019 mg/l of chlorine is acutely toxic to freshwater organisms, and 0.011 mg/l of chlorine is chronically toxic. In seawater, the acute and chronic concentration is 0.013 mg/l and 0.0075 mg/l. Since chlorine disinfection normally produces a total residual chlorine concentration of 1.0 to 5.0 mg/l in the effluent, it is required to dechlorinate the treated wastewater before discharging to the surface water.

The most common dechlorination agent is sulfur dioxide (SO_2). Sodium sulfite (Na_2SO_3) and sodium metabisulfite ($\text{Na}_2\text{S}_2\text{O}_5$) and activated carbon have also been used as dechlorination agents. Following Table shows dechlorination reaction related to each agent and theoretical ratio of residual chlorine to agent.

Chemical	Reaction	Chemical Use Ratio
Sulfur Dioxide	$\text{SO}_3 + \text{Cl}_2 + 2\text{H}_2\text{O} \rightarrow \text{H}_2\text{SO}_4 + 2\text{HCl}$	1.1
Sodium Sulfite	$\text{Na}_2\text{SO}_3 + \text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{Na}_2\text{SO}_4 + 2\text{HCl}$	1.8
Sodium Metabisulfite	$\text{Na}_2\text{S}_2\text{O}_5 + 2\text{Cl}_2 + 3\text{H}_2\text{O} \rightarrow 2\text{NaHSO}_4 + 4\text{HCl}$	1.5
Sodium Bisulfite	$\text{Na}_2\text{HSO}_3 + \text{Cl}_2 + \text{H}_2\text{O} \rightarrow \text{NaHSO}_4 + 2\text{HCl}$	1.5
Hydrogen Peroxide	$\text{H}_2\text{O}_2 + \text{Cl}_2 \rightarrow 2\text{HCl} + \text{O}_2 (\text{g})$	0.5

Table 6.5 Dechlorination agents and reactions (The Dow Chemical Company, 2000)

6.3.5 Cost

The cost of chlorine disinfection depends on the chemical and equipment manufacturer, the site, the capacity of the plant, and the characteristics of the wastewater. In general, chlorine gas is the cheapest one among the forms of chlorine. Sodium hypochlorite and calcium hypochlorite is more expensive than chlorine gas. On the basis of available chlorine, sodium hypochlorite costs three times more than chlorine gas, and calcium hypochlorite costs four times more than chlorine gas. Accord to the Fact Sheet reported by EPA, however, the total cost of disinfection will be increased by approximately 30 to 50% if dechlorination is added although chlorination is the most inexpensive way of disinfection.

6.3.6 Environmental impacts of using chlorination

After dechlorinating the disinfected wastewater, microorganisms can regrow in receiving water bodies and in long transmission pipelines. It is assumed that the regrowth of pathogenic microorganisms on the pipe surfaces exposed to treated wastewater results because the organic matter present in treated wastewater effluent maintains a high number of microbes even after treatment of wastewater. Regrowth also occurs because of the lack of predators such as protozoa. Due to this problem, it is important to maintain the proper concentration of residual chlorine in effluent discharging into the nature. Typical residual is from 0.1 to 0.5 ppm for free available chlorine, and 2 ppm for combined chlorine because of the less effectiveness.

Another environmental impact is DBPs of chlorination. It has been reported that very small amount of DBPs can negatively affect the human health as well as aquatic lives. Residual chlorine in dechlorinated wastewater produces chlorinated organic byproducts by reacting with organic compounds. Among organic compounds, phenols, amines, aldehydes, ketones, and pyrrole groups are very susceptible to chlorination. The most common disinfection byproducts

are trihalomethanes (THMs) and haloacetic acids (HAAs) (Water Quality and Treatment, 2000). The DBPs formation rate is dependent on the presence of organic substances, free chlorine concentration, bromide concentration, the pH, and temperature. The principal means of controlling the formation of DBPs in wastewater is not to add free chlorine directly because the reactivity to produce byproducts is higher for the free chlorine than chloramine. Although the use of chloramine can prevent from forming high levels of DBPs, however, alternative disinfection means such as UV radiation should be concerned if specific precursors such as humic materials are present in water.

6.4 Alternative Disinfection Agents

Although chlorine is a highly effective disinfectant, alternative disinfection methods have been considered because of some serious concerns of its use. Followings are important concerns.

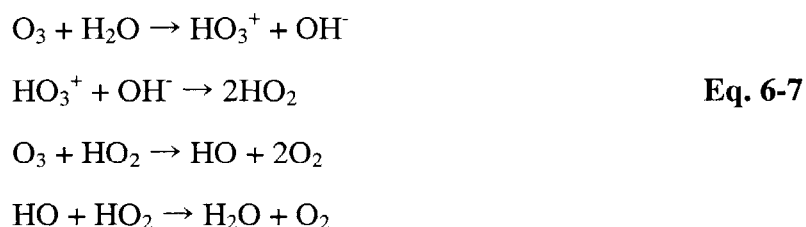
1. The high risk of transportation of chlorine
2. Potential health risks to treatment plant operators because of the high toxicity of chlorine
3. Formation of odorous compounds by reaction with the organic compounds in wastewater
4. Formation of disinfection byproducts (DBP) which are carcinogenic by reaction with the organic substances
5. Toxicity of residual chlorine to aquatic lives in treated wastewater effluent

For the alternative disinfection agents, ozone, ultraviolet (UV) radiation, and peracetic acid will be discussed in following section.

6.4.1 Ozone

Effectiveness

Ozone is an unstable and highly reactive form of oxygen, and therefore must be produced on-site. Ozone has fewer safety problems related with shipping and handling especially compared to chlorine. On the other hand, ozone is highly reactive and corrosive, and therefore corrosion-resistant material is required. The reactions of ozone in water are as follows:



The free radicals, HO₂ and HO, are very good oxidation agents and are very active in the disinfection process. These radicals also oxidize other impurities in water.

The typical values for the ozone demand are shown in Table 6-6.

Type of wastewater	Initial Coliform, MPN/100mL	Ozone dose, mg/L			
		Effluent standard, MPN/100mL			
		1000	200	23	2.2
Raw wastewater	10 ⁷ -10 ⁹	15~40			
Primary effluent	10 ⁷ -10 ⁹	10~40			
Trickling filter effluent	10 ⁵ -10 ⁶	4~10			
Activated-sludge effluent	10 ⁵ -10 ⁶	4~10	4~8	16~30	30~40
Filtered activated-sludge effluent	10 ⁴ -10 ⁶	6~10	4~8	16~25	30~40
Nitrified effluent	10 ⁴ -10 ⁶	3~6	4~6	8~20	18~24
Filtered nitrified effluent	10 ⁴ -10 ⁶	3~6	3~5	4~15	15~20
Microfiltration effluent	10 ¹ -10 ³	2~6	2~6	3~8	4~8
Reverse osmosis	0				1~2
Septic tank effluent	10 ⁷ -10 ⁹	15~40			
Intermittent sand filter effluent	10 ² -10 ⁴	4~8	10~15	12~20	16~25

Table 6.6 Typical ozone dosages required to achieve different effluent coliform disinfection standards for various wastewaters based on a 15-min contact time (Metcalf & Eddy, 2002)

This table shows the same results as Table 6.4. Although ozone is more effective on viruses and bacteria than chlorine, primary effluent cannot be disinfected efficiently. Since, ozone can destroy chlorine-resistant organisms with relatively short contact time, approximately 10 to 30 minutes (EPA, 1999), basins for disinfection of ozone could be smaller than chlorination basins.

Advantages and Disadvantages

During ozonation, taste, odor, and color of water can be controlled. Moreover, ozonation can elevate the dissolved oxygen concentration of the effluent because ozone readily decomposes to water and oxygen. In contrast to chlorine compounds, ozone does not produce halogenated organic matter. However, ozone can produce bromate which is harmful to human health when water contains raw bromide concentrations at high pH. Ozone may also produce oxygenated byproducts and assimilable organic carbon which bacteria may use to grow. The toxic byproducts of ozonation are usually unstable, so they exist in water only for minutes.

Cost

Ozonation is more expensive than chlorination in terms of capital and O&M expenses. Especially, ozonation is not appropriate to the effluent which levels of SS, BOD, or COD are high. Therefore, ozonation is not appropriate for where want to treat wastewater more economically. Table 6.7 in section 6.4.3 compares the cost of various disinfection methods.

6.4.2 Ultraviolet (UV) Radiation

Effectiveness

The range of an ultraviolet (UV) wave is between 40nm and 400nm, and the UV germicidal range is between 250nm to 270nm. The disinfection efficiency of UV radiation depends on the characteristics of the wastewater, the intensity of UV radiation, and the contact time. Disinfection efficiency is also directly related to the level of turbidity and SS.

Advantages and Disadvantages

The biggest advantage of UV as a disinfectant is that UV does not form disinfection by-products and does not have toxic residuals in contrast to chlorine compounds. UV is effective against protozoan pathogens as well as bacteria and viruses with relatively short contact time of approximately 20 to 30 seconds with low intensity UV lamps. However, UV radiation is not effective to disinfect wastewater with high turbidity and TSS level. UV disinfection with low intensity lamps is not as effective for the treated wastewater effluent with TSS levels above 30 mg/l (EPA, 1999). In addition, microorganisms can regrow after UV radiation through a repair mechanism.

Cost

UV radiation is more expensive than chlorination technology although the costs of UV radiation have recently decreased due to improved technology and competition between suppliers. In particular, facilities and O&M that includes electric power represent most of the high costs for UV radiation. When dechlorination and fire codes are added to the chlorination, however, costs of UV radiation are comparable (EPA, 1999).

6.4.3 Peracetic Acid

Effectiveness

Peracetic acid (PAA, CH₃COOOH) is a very strong oxidizing solution containing peracetic acid, glacial acetic acid, hydrogen peroxide, and water at the equilibrium.



According to the research of M.G.C. Baldry et al. in 1995, PAA performs better than sodium hypochlorite against vibrio cholera species. Its efficiency was better at 30 °C than at 20 °C. PAA is effective for treatment of sewage especially for cholera control in warm climates. In addition, according to C. Sanchez-Ruiz et al. in 1995, increased concentration of PAA and contact time do not substantially improve PAA efficiency against total coliform bacteria. It is easy to achieve the corresponding guideline (1000 CFU/100ml of fecal coliform) of the WHO with concentration of PAA 10 ppm for 30min. However, a much higher dosage, 400 ppm for 20min. was required to achieve the stringent guideline for agricultural reuse (2 CFU/100ml of total coliform).

Advantages and Disadvantages

PAA has been used as a disinfectant for years in various industries, and research into the use of PAA as a wastewater disinfectant began in the late 1980s. PAA was included among 5 disinfectants by the EPA 1999 report despite the lack of quantitative information about the activity of PAA against the microorganisms in water. The desirable attributes of PAA listed in the report were the absence of persistent residuals and by-products (DBPs), independence of pH, short contact time, and high effectiveness as a bactericide and virucide. The biggest advantage in using PAA for disinfection is that PAA is hydrolyzed and produces acetic acid and hydrogen peroxide which are easily biodegradable in water.

The disadvantages of PAA are the increase of organic content in the treated wastewater effluents, the potential microbial re-growth due to the remaining acetic acid which is the product of PAA hydrolysis, the limited efficiency against viruses and parasites, and the strong dependence on wastewater quality.

Cost

PAA disinfection is more expensive than chlorination. For example, according to Industrial Water Treatment Bulletin by Houghton Chemical Corporation, 1 lb of 5% peracetic acid solution costs 44 dollars which is 10 times more expensive than sodium hypochloride solution.

According to the pilot investigation performed by L. Liberti and M. Notarnicola in 1999, the total cost of disinfection using peracetic acid is the most expensive, including operation and maintenance costs, among UV, peracetic acid, and ozone. The following table shows the results of the cost estimation in 1999.

Disinfectant	Dose	Flowrate (m ³ /h)	Total Coliform Target achieved (CFU/100ml)	O&M Costs (US\$/1000m ³)		
				Electric Power	Replacement Chemicals	Total
UV	100mWs/cm ²	30	1	6.7	10.6	17.3
NaOCl	5ppm, 30min	30	1		10.5	10.5
NaOCl + Dechlorination	5ppm, 30min	30	1		10.5 + 5.3	15.8
PAA	10ppm, 30min	30	240		64.5	64.5
Ozone	15ppm. 10min	30	97	34.2	3.1	37.3

Table 6.7 Cost estimation for UV, chlorination, chlorination/dechlorination, PAA, and O₃ disinfection of clarified filtered effluent (Source: Liberti and Notarnicola, 1999, Currency exchange for EURO to US\$ is 1; REGAL-Chlorinators Inc.)

Since it is essential to provide an economical wastewater treatment system to the city of Paraty, peracetic acid, which is much more expensive than chlorine, cannot be used as a disinfectant in Paraty.

Handling

To use PAA for disinfection, some safety precautions are required because of its corrosive properties and oxidizing power. PAA should be stored in a cool, dry, well-ventilated area in original shipping containers with hazard labels. PAA should be separated from acids, alkalines, organic materials, and heavy metals. Because of its explosive potential, PAA should be kept away from sources of ignition and heat. Operators should wear protective equipment because PAA can cause severe health problems such as eye irritation, skin burns, and gastrointestinal tract problems.

6.5 Data Collection and Analysis

6.5.1 *Procedure*

CEPT effluents were used in the disinfection studies by peracetic acid and chlorine. To measure the level of total coliforms and fecal coliforms, ColiPlate™ which has 96-micro-well was used. Samples were injected in microplate, and incubated at 35°C for 24 hours. Total coliform positive test results in a blue color, and fecal coliform positive test results in a fluorescence color.

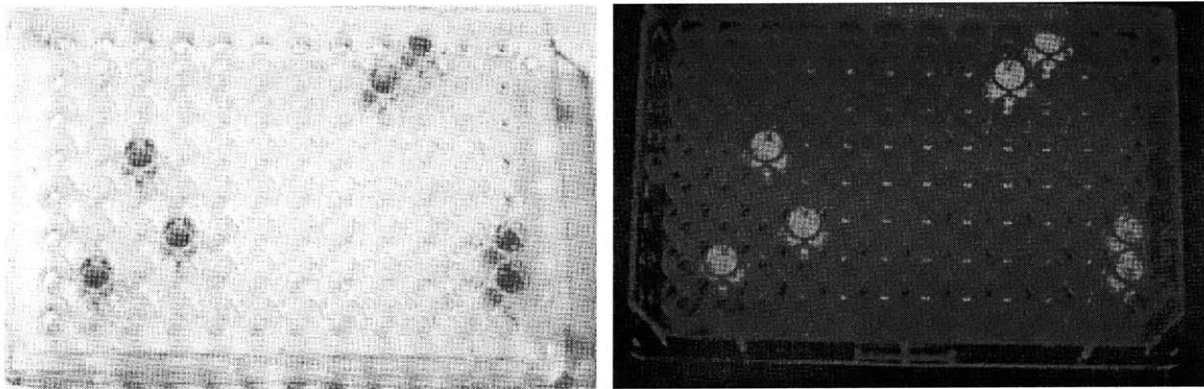


Fig 6.2. Blue colour indicative of coliforms (left); Fluorescence indicative of fecal coliform (right)
(<http://www.ebpi-kits.com/>)

6.5.2 *Data Analysis*

6.5.2.1 **Paraty Data**

For disinfection experiment, CEPT effluent treated by 30 mg/l of ferric chloride was used. The levels of COD and SS of this effluent were 19.8 mg/l and 9.1 mg/l. To compare the coliform removal efficiencies of peracetic acid (PAA) and chlorine, 5, 10, 15, 20 mg/l of PAA and 20 mg/l of chlorine were added to the effluent, and contact time was 30 minutes. Following figure shows the coliform reductions by various concentrations of PAA and chlorine.

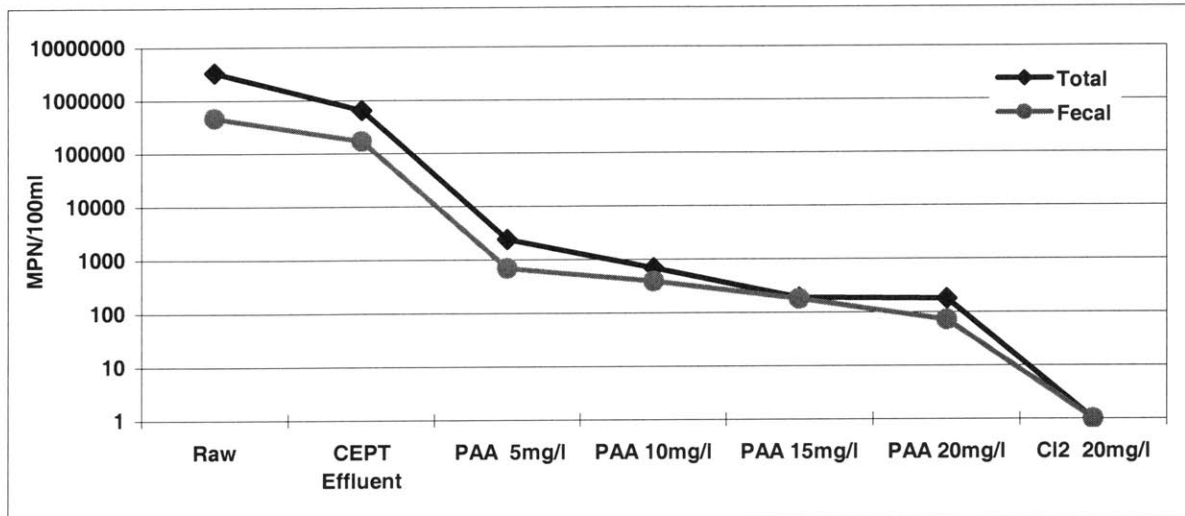


Fig 6.3 Coliform reduction of CEPT effluent by PAA and chlorine

As shown in the figure 2, coliforms in treated wastewater reduced with increased dosage of PAA although PAA did not eliminate all the coliforms while chlorine at a 20mg/l concentration did. Disinfection with PAA 15 mg/l achieved a FC concentration below 200 MPN/100ml which level is recommended in section 6.12.

It is important to note that there is only one set of disinfection test in Paraty due to unrepresentative jar-test effluents. Considering the high cost of PAA, however, it is obvious that PAA is not a good disinfection agent for Paraty.

6.5.2.2 Deer Island Data

For the disinfection experiments, wastewater from the Deer Island wastewater treatment plant was used. The raw sewage was treated by 20 mg/l and 10 mg/l of ferric chloride, and the effluent was used for the disinfection tests. To examine the effect of additional seawater on the coliform reduction, 1% and 5% of seawater was used. The effluent of conventional and CEPT was disinfected by 5 and 10 mg/l of Cl₂ with 30-min contact time. The fecal coliform levels in the raw sewage were 800000 and 500000 MPN/100ml and these are acceptable concentrations based on the average fecal coliform level, 918000 MPN/100ml, in the influent of the Deer Island wastewater treatment plant.

6.5.2.2.1 Conventional Effluent vs. CEPT Effluent

According to the collected data, there is a significant difference of fecal coliform reduction between conventional effluent and CEPT effluent. Fecal coliform level was reduced below 10⁵ MPN/100ml by CEPT without additional seawater while fecal coliform level was approximately

10⁶ MPN/100ml in the conventional effluent. The differences of fecal coliform reduction between conventional and CEPT effluent can be explained by the relationship between SS removal efficiency and coliform removal efficiency. Following graph shows the levels of COD, SS, fecal coliforms in the effluent of conventional and CEPT using 20mg/l of ferric chloride.

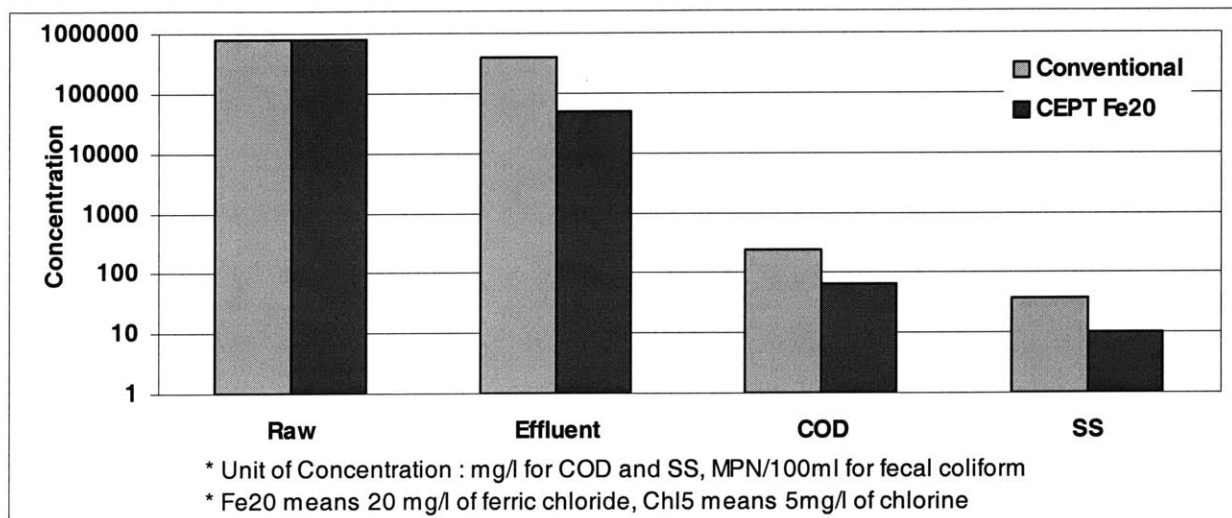


Fig 6.4 COD, SS, and fecal coliforms in the effluent treated by 20mg/l of FeCl₃ (1)

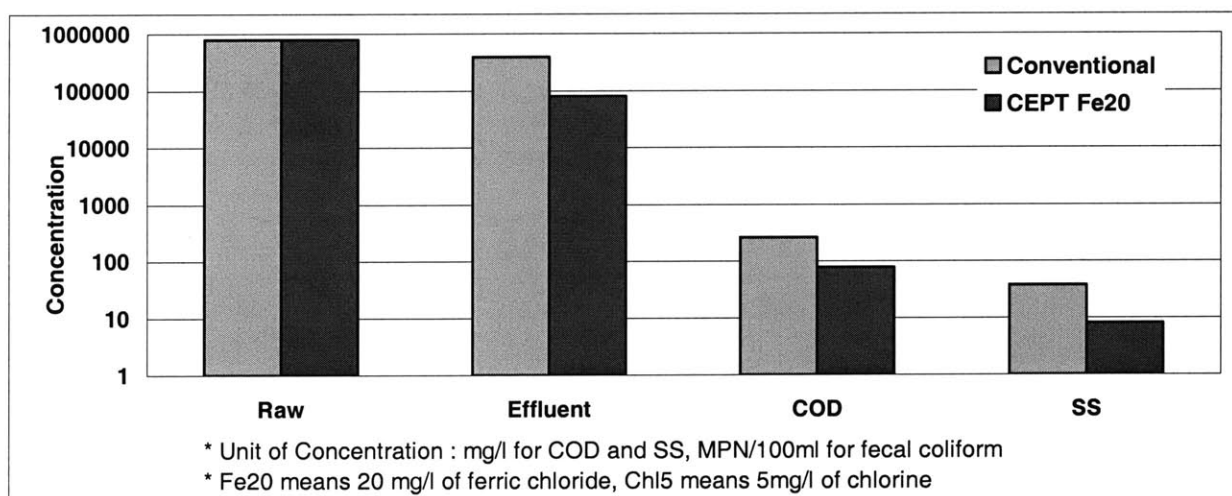


Fig 6.5. COD, SS, and fecal coliforms in the effluent treated by 20mg/l of FeCl₃ (2)

As shown in the graphs, the quality of CEPT effluent treated by 20 mg/l of ferric chloride was significantly better than that of conventional effluent.

In contrast, the chemical treatment with 10mg/l of ferric chloride did not make a significant difference in SS and fecal coliform removals.

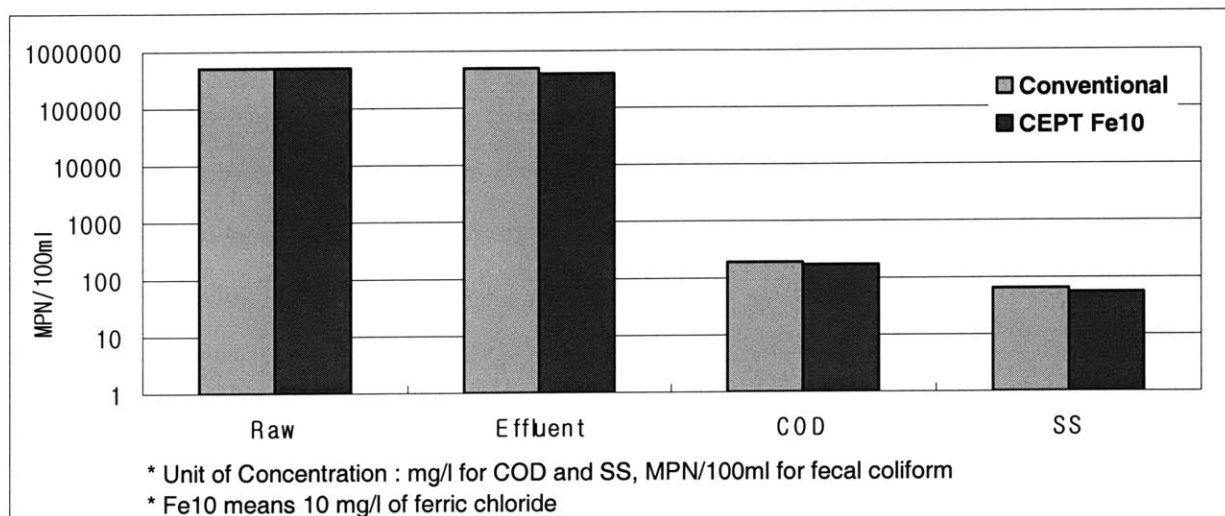


Fig 6.6 COD, SS, and fecal coliforms in the effluent treated by 10mg/l of FeCl₃ (1)

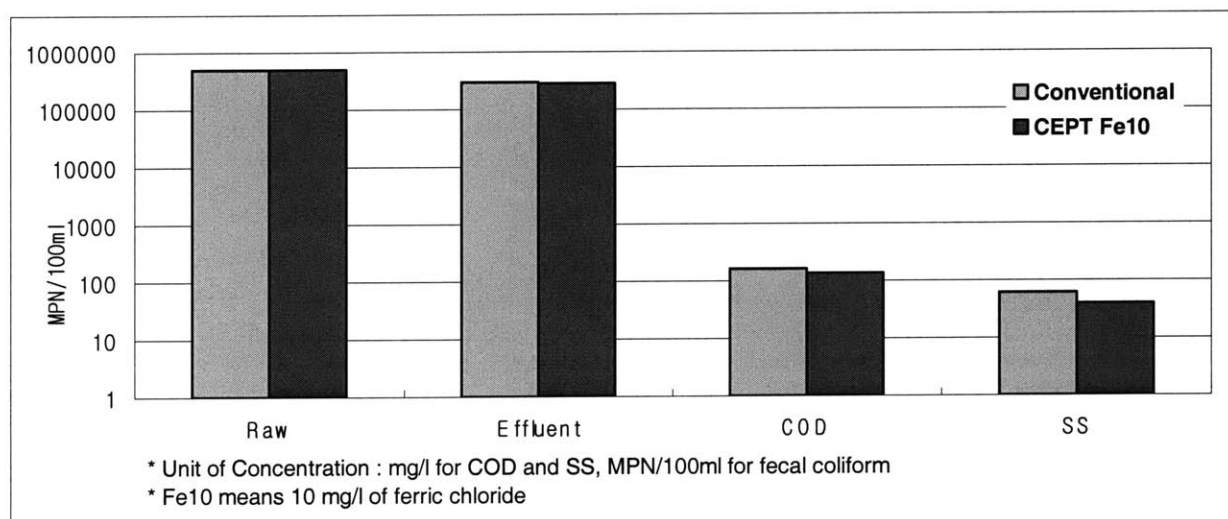


Fig 6.7 COD, SS, and fecal coliforms in the effluent treated by 10mg/l of FeCl₃ (2)

According to the graphs, it is obvious that 20mg/l of ferric chloride is more effective than 10mg/l of ferric chloride for wastewater treatment in Boston.

It is noticed that the trends of reduction of SS and COD levels in the effluent are equal to the trend of reduction of fecal coliform concentrations. As discussed above (section 6.2.1.), microorganisms are partly removed with the removal of suspended solids. Since SS removal efficiency of CEPT is much higher than that of conventional treatment, the concentration of fecal coliforms in the CEPT effluent is much lower than that in the conventional effluent. In the same way, the level of coliforms in the effluent is lower when seawater is added due to the higher SS removal efficiency. The following graph shows the relationship between concentration of SS and fecal coliforms.

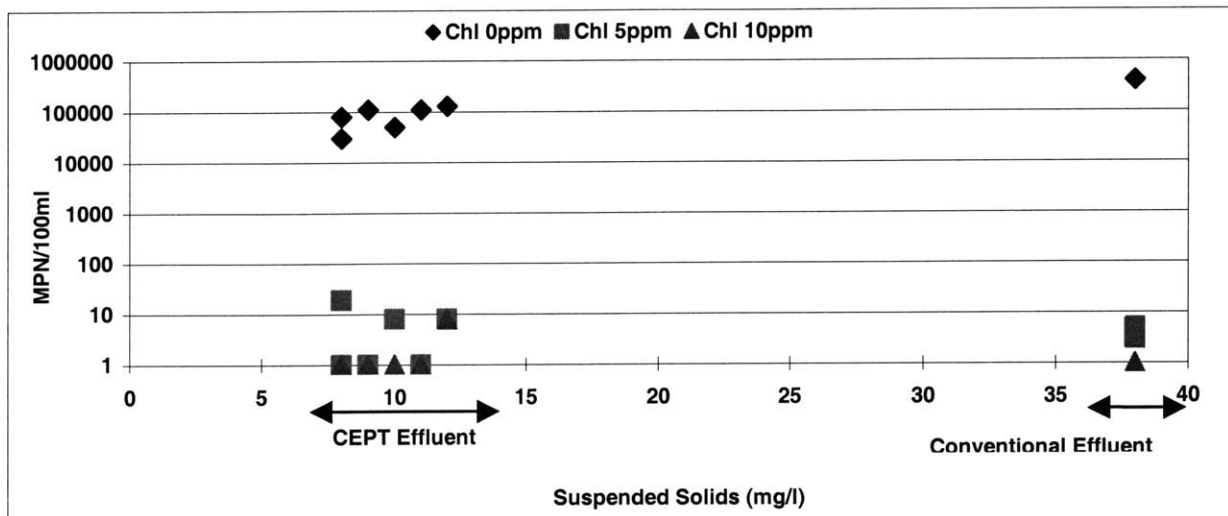


Fig 6.8 SS versus Fecal coliform concentration in the effluent treated with 20mg/l of FeCl₃

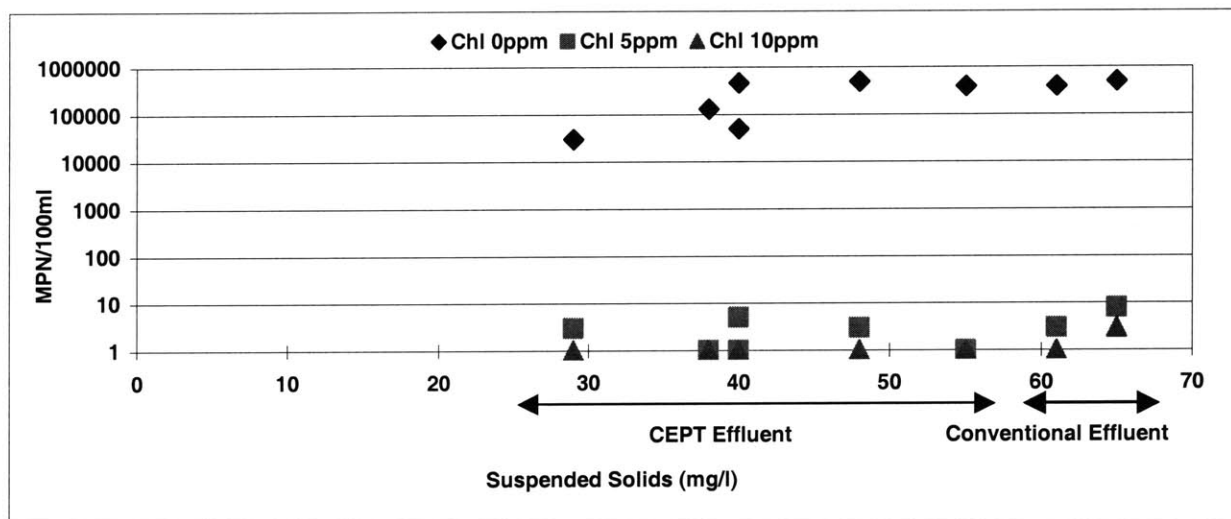


Fig 6.9 SS versus Fecal coliform concentration in the effluent treated with 10mg/l of FeCl₃

6.5.2.2.2 Effect of Additional Seawater on the Fecal Coliform Reduction

For the CEPT effluent treated with 20 mg/l of ferric chloride, the effect of additional seawater on the suspended solids removal efficiency is not constant. 5% of additional seawater negatively affected the SS removal efficiency, and 1% of additional seawater made a little difference from no additional seawater. The qualities of conventional and CEPT effluent are shown in the following graphs.

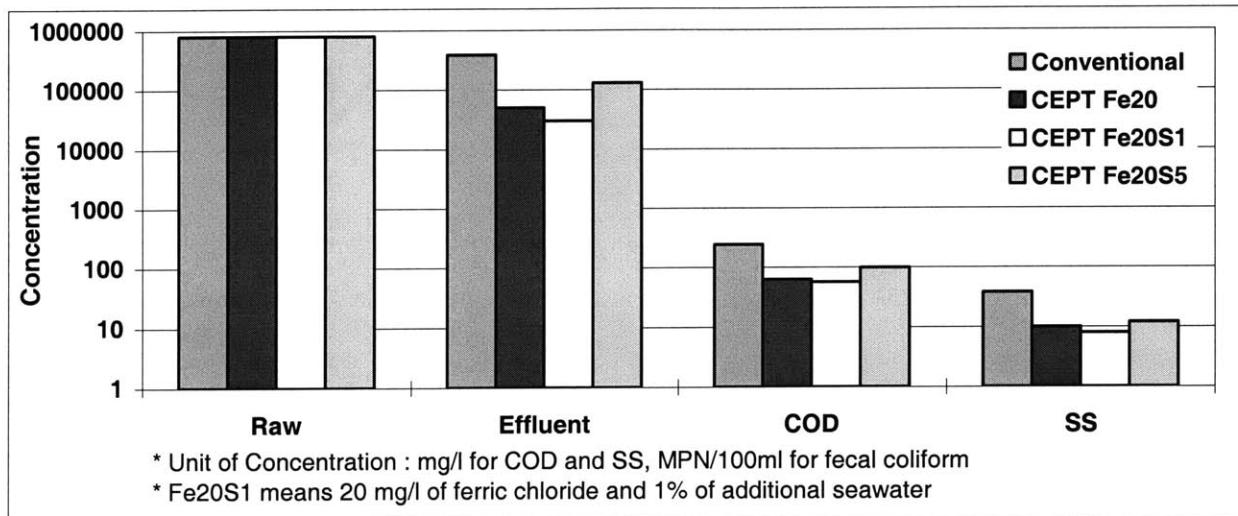


Fig 6.10 The concentration of COD, SS, and fecal coliform (1)

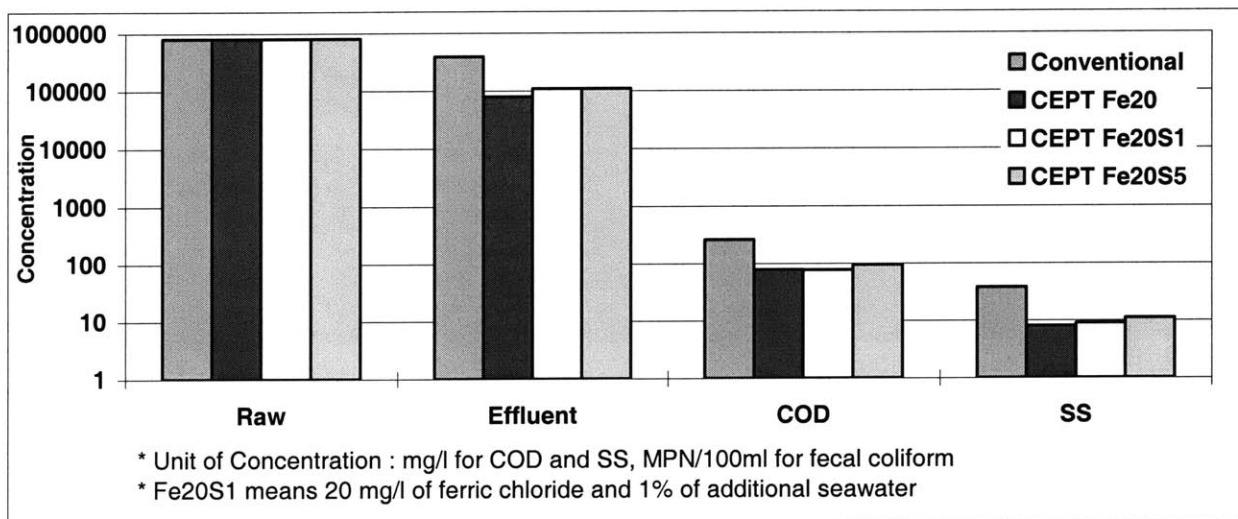


Fig 6.11 The concentration of COD, SS, and fecal coliform (2)

Since raw sewage in Boston already includes approximate 2% of seawater (see section 5.3), SS removal efficiency of conventional treatment for the Boston sewage is higher than the typical removal efficiency. Therefore, it leads no significant difference of fecal coliform reduction with additional seawater. However, it is obvious that 1% of additional seawater is more effective to reduce the coliform levels than 5% of additional seawater for the Boston sewage.

For the CEPT effluent treated by 10 mg/l of ferric chloride, neither 1% nor 5% of additional seawater made significant differences for the fecal coliform reduction. It is because the effect of additional seawater on the SS removal efficiency. As shown in Fig. 6.12, SS removal efficiency did not increase with additional seawater. Following graph shows the effects of additional seawater on pollutant reduction in the effluent treated by 10mg/l of ferric chloride.

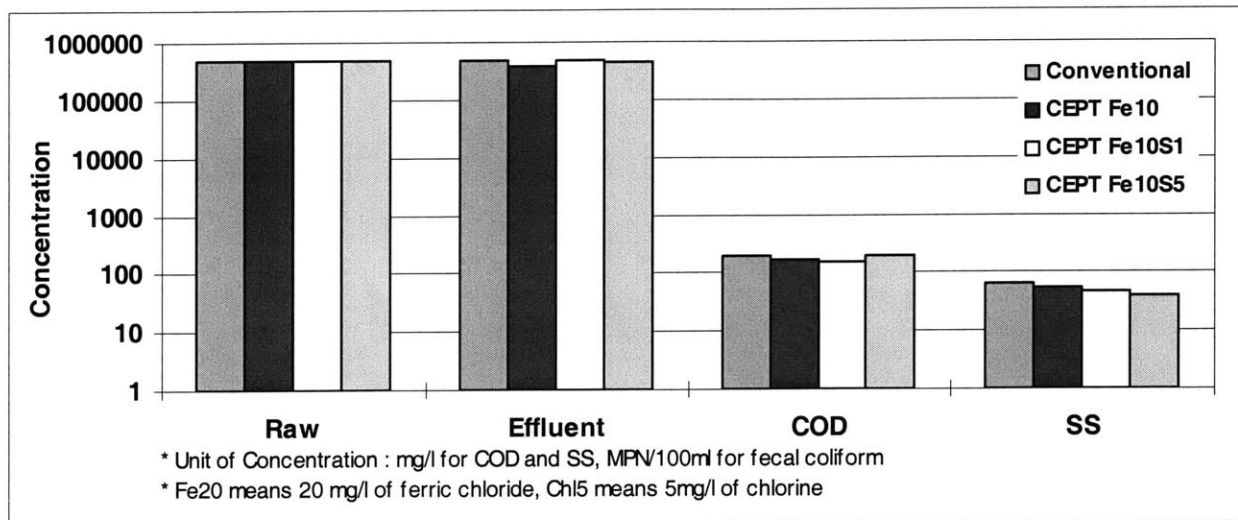


Fig 6.12 Concentration of COD, SS, and fecal coliform in the effluent according to the additional seawater

6.5.2.2.3 Disinfectability of the Effluent

Typically, the concentration of fecal coliforms in the conventional effluent is $10^7 \sim 10^9$ MPN/100ml based on $10^7 \sim 10^9$ MPN/100ml in raw sewage. It means that conventional treatment barely removes fecal coliforms, and it is impossible to reduce the coliform level below 200MPN/100ml with high dosage of chlorine (see section 6.3 and Table 6.4). In contrast, the concentration of coliforms reduces to 10% of raw sewage with CEPT, and this value is the same as the secondary effluent (WPCF, 1986). This represents that the disinfectability of CEPT effluent is higher than that of conventional effluent.

In the experiments in Boston, however, not only the CEPT effluent but also the conventional effluent were disinfected with 5 mg/l of chlorine. The reason even the conventional effluent was disinfected with low concentration of chlorine is that the level of fecal coliforms, 390000 MPN/100ml, in the effluent is much lower than the typical value. The level of fecal coliforms in the conventional effluent of Boston sewage is similar to the typical value of the activated sludge effluent. This low level of fecal coliforms in the conventional effluent is due to the lower level of fecal coliforms, 10^6 MPN/100ml, in the raw sewage in Boston than the typical value, $10^7 \sim 10^9$ MPN/100ml, and relatively high SS removal efficiency of conventional treatment for Deer Island wastewater. The reason of high SS removal efficiency of conventional treatment for Deer Island wastewater was discussed in the section 6.5.2.2.2.

6.5.2.3 Chlorine Demand for CEPT Effluent of Deer Island Wastewater

Following graphs shows the coliform reduction in the effluent with 5 mg/l and 10 mg/l of chlorine.

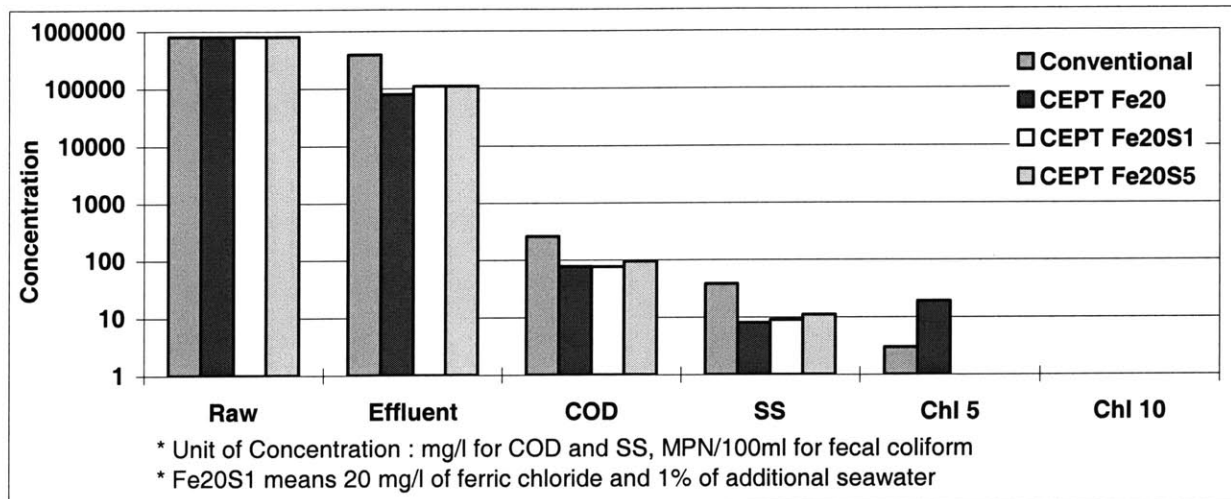


Fig 6.13 Fecal coliforms reduction in the effluent with 5, 10 mg/l of chlorine (1)

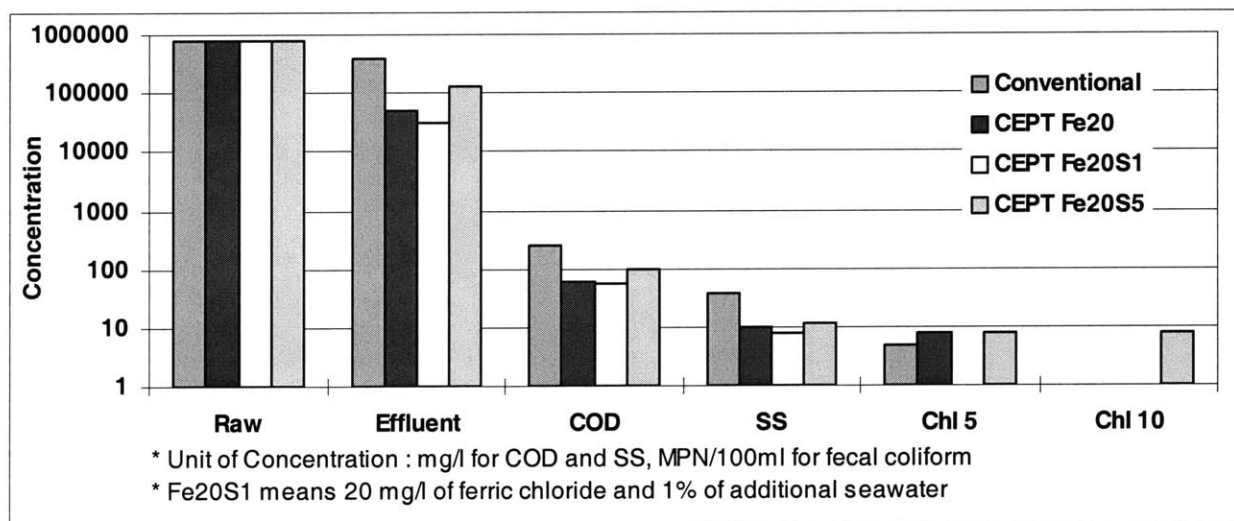


Fig 6.14 Fecal coliforms reduction in the effluent with 5, 10 mg/l of chlorine (2)

As shown in the Fig. 6.13 and 6.14, 5 mg/l of chlorine achieved the same coliform removal efficiency as a dose of 10 mg/l. It probably means that 5 mg/l of chlorine is much higher than the actual chlorine demand. According to the disinfection/dechlorination performance report of Deer Island wastewater treatment plant, the average chlorine dose is 2.2 mg/l and the range is from 1 to 4 mg/l with 45-min average contact time. The average fecal coliform in the effluent is 10 MPN/100ml that is similar to results of experiments. In the experiments, 5 mg/l of chlorine with 30-min contact time achieved similar results to the Deer Island plant test. As mentioned in

section 6.3, the disinfection efficiency of chlorine depends on the concentration of chlorine and the contact time. Since the similar results in the experiments to the Deer Island wastewater treatment plant were achieved with shorter contact time, it can be represented that 5 mg/l of chlorine is much higher than the chlorine demand.

6.6 Conclusions and Recommendation

In Paraty, 15 mg/l of PAA achieved about 200 MPN/100ml that meets the Brazilian Regulations. although the removal efficiency of PAA disinfection in Paraty was lower than chlorine. Considering the cost if PAA is 10 times higher than chlorine, however, chlorine is a more proper disinfectant than PAA in Paraty.

Coliform removal efficiency of wastewater treatment depends on the SS removal efficiency. It is because coliforms are removed with SS during the settling process, and SS protect coliforms from the disinfection agents. Since the SS removal efficiency of CEPT is higher than the conventional treatment, the disinfectability of CEPT effluent is better than the conventional effluent. However, both of CEPT and conventional effluent of Deer Island wastewater are disinfectable. The reasons are:

1. Raw wastewater in Boston is relatively weaker than the typical value.
2. Coliform reduction in the conventional effluent is relatively higher than the typical because approximate 2 % of seawater, already included in the Boston wastewater, enhances the SS efficiency.

Additional seawater does not enhance the coliform reduction in the CEPT effluent in Boston. The reasons are;

1. Initial SS and coliform removal efficiencies by CEPT with no seawater are already high.
2. Additional seawater did not make significant differences of SS removal.

Since raw sewage in Paraty does not include seawater, however, the level of fecal coliforms in the effluent would be significantly reduced with small amount of additional seawater in Paraty as if COD and SS removal efficiency were increased with additional seawater.

According to the data of Deer Island wastewater treatment plant where use secondary treatment, the average dose of chlorine is 2.2 mg/l with an average contact time of 45 min. Since the SS removal efficiency of CEPT is similar to the secondary treatment, it is expected that the amount

of chlorine in Paraty would be also similar to that of in Deer Island wastewater treatment plant. Considering cases that higher amount of chlorine demand and relatively short contact time during the summer season, 3 mg/l of chlorine is recommended (see Table 6.3). If chlorine dosage is well-controlled, the residual chlorine will not be varied, and therefore the amount of dechlorination agent will not be varied. Based on the data of Deer Island wastewater treatment plant, 0.5 mg/l of dechlorination agent is recommended. This dosage is for the case sulfur bisulfate is used as a dechlorination agent. Following table shows the average chlorine dose in Deer Island wastewater treatment and recommended chlorine dose in Paraty.

	Efficiency (%)		Dosage of Chemical (mg/l)	
	SS	COD/BOD	Chlorine	SBS
CEPT	85	55	3	0.5
Secondary Treatment	91	85	2.2	0.5

Table 6.8 Recommended dosage of Chlorine and sulfur bisulfate (SBS) in Paraty based on SS Removal Efficiency of CEPT and Secondary Treatment (Source: Chemical Dosage in Secondary Treatment: Deer Island WWTP)

CHAPTER SEVEN: DESIGN OF THE PLANT IN PARATY

The CEPT tanks, chlorination and dechlorination basins are the main components to a CEPT plant design and will be described in detail in the following sections:

7.1 CEPT Tanks

The plan to build a CEPT plant is divided into three stages. The following table 7-1 shows the populations, and average and peak wastewater flow rates for each stage. The Paraty CEPT plant will serve the Historical Center during the first stage, the area of Mangueira and the Ilha das Cobras will be added during the second stage. The old city will be finally added during the third stage.

	Stage 1 Hist. Cntr	Stage 2 + Man+Ilha	Stage 3 + Old City
Population (capita)	3000	7500	3000
Water Consumption (L/day-capita)	180	180	180
Seasonal Factor	3	1	3
Peak Factor	1.8	1.8	1.8
	m ³ /day	m ³ /day	m ³ /day
Qavg	540	1890	2430
Qp	972	3402	4374
Qsp	1620	2970	4590
Qspp	2916	5346	8262

*Table 7-1. The population, average and peak wastewater flow rate for each stage; Qavg = average flow rate, Qp = peak flow rate, Qsp = average flow rate during the summer = Qavg * seasonal factor, Qspp = peak flow rate during the summer season.*

To ensure the plug flow of wastewater in the tanks, the ratio of width to height and length to height should be at least 1 to 5 (W:L = 1:5, H:L = 1:5). The CEPT plant should also be able to serve the summer flow rate at each design stage. When the typical overflow rate (SOR), 60 m/day, is used to estimate the CEPT tanks dimension, the tank length was too short for the first stage of the design:

$$Length = \frac{1620 \frac{m^3}{day}}{60 \frac{m}{day} \times 3 m} = 9 m$$

The calculation above is based on the summer flow rate of the first stage and a 3 m width. In this case, the ratio of width to height ($W/H=1.5$) does not meet the 1:5 ratio, so a shorter width could be designed for. Taking the future phases into consideration, however, it would be better to decrease the overflow rate than to shorten the width of tanks. Therefore, the CEPT tanks in Paraty are designed based on a 30 m/day of overflow rate (SOR). In addition, CEPT also must serve the peak flow during the summer season (Q_{spp}). Since the removal efficiency of CEPT is constant up to 90 m/day, then a 90-m/day overflow rate (SOR) is used for Q_{spp} to ensure the appropriate removal efficiencies in CEPT. The following table shows the estimated flow rate of CEPT with 30, 60, 90 m/day of overflow rate in the tanks with a dimension of $15 * 3 * 3 \text{ m}^3$.

Width (m)		3		
Height (m)		3		
Length (m)		15		
OFR (m/day)		30	60	90
Foot Print for 1 Tank (m^2)		45	45	45
Flow Rate (m^3/day)	Q for 1 Tanks	1350	2700	4050
	Q for 2 Tanks	2700	5400	8100

*Table7-2. Estimated flow rates of CEPT tanks with the dimension of $15 * 3 * 3$*

According to the table, one tank has the capacity to meet the flow rate of summer season of the first stage. Two CEPT tanks meet the flow rate of summer season of the second and the third stage. Therefore, two CEPT tanks with dimension of $15 * 3 * 3 \text{ m}^3$ are required for Paraty at the end of the stage. For the maintenance of CEPT, one more tank is need. Therefore, total number of CEPT tanks is three, and footprint is 135 m^2 ($45 \text{ m}^2 * 3$).

7. 2. Chlorination Basins

To maintain the proper disinfection efficiency, the ratio of width to length should be 1 to 20 ($W:L = 1:20$, Metcalf & Eddy, 2002). Also, the detention time of the average flow is 30 to 120 min, and 25 to 90 min for the peak flow. Due to the difficulty of obtaining the proper length of contact basins for the whole stages, trial and error method is used to find proper dimension of basins. The following table shows the proper dimension of chlorination basins and detention time according to the various flow rates.

Width (m)	1					
Height (m)	2					
Length (m)	20					
# of basins	1			2		
Volume (m ³)	40			80		
Foot Print (m ²)	20			40		
Stage	1	2	3	1	2	3
Qavg (m ³ /d)	540	1890	2430	540	1890	2430
Detention time (min)	107	30	24	213	61	47
Qp (m ³ /d)	972	3402	4374	972	3402	4374
Detention time (min)	59	17	13	119	34	26
Qsp (m ³ /d)	1620	2970	4590	1620	2970	4590
Detention time (min)	36	19	13	71	39	25
Qspp (m ³ /d)	2916	5346	8262	2916	5346	8262
Detention time (min)	20	11	7	40	22	14

Table 7-3. The detention time in the chlorine basins with various flow rates

As shown above, the detention time of the flow during the summer season (Qsp) of the third stage is in the acceptable range, 25, 50, 90 minutes with two basins (Metcalf & Eddy, 2002). For the maintenance of chlorination basins, one more basin is required. Therefore, total number of basins is three, and footprint is 60 m² (20 m² * 3).

7. 3. Chemical Storage Tanks

For the treatment of wastewater, ferric chloride (40 mg/l) will be used, and small amount of polymer could be used. If the concentration of ferric chloride is 40%,

$$\frac{40 \frac{mg}{l} \times 4590 \frac{m^3}{day} \times 10 \text{ day}}{1.4 \frac{g}{ml} \times 1000 \frac{ml}{l} \times \frac{40}{100}} \times \frac{m^3}{1000 l} \cong 3.3 m^3$$

For the case of higher doses, 20% more ferric chloride is stored and therefore:

$$3.3 m^3 \times 1.2 \cong 4.0 m^3$$

To ensure the proper storage capacity, the volume of tank is therefore chosen to be 4.5 m³. When the height of the tank is 1.5 m, the footprint is 3 m². Since the amount of polymer is much smaller than that of ferric chloride, the footprint of the polymer tank is negligible.

Liquid sodium hypochlorite (3 mg/l) will be used for disinfection, and sulfur bisulfate (0.5 mg/l) can be used for dechlorination. Other chemicals, such as sulfur dioxide, can be used instead of sulfur bisulfate. If the concentration of sodium hypochlorite is 13 %,

$$\frac{3 \frac{mg}{l} \times 4590 \frac{m^3}{day} \times 10 \text{ day}}{1.2 \frac{g}{ml} \times 1000 \frac{ml}{l} \times \frac{13}{100}} \times \frac{m^3}{1000 \text{ l}} \cong 0.9 m^3$$

For the case for the higher chlorine demand, 10% more can be stored and therefore:

$$0.9 m^3 \times 1.1 \cong 1.0 m^3$$

To ensure proper storage capacity, the volume of the tank will be 1.2 m³. When the height of the tank is 1.2 m, the footprint becomes 1 m².

If the concentration of sulfur bisulfate is 25 %,

$$\frac{0.5 \frac{mg}{l} \times 4590 \frac{m^3}{day} \times 10 \text{ day}}{1.23 \frac{g}{ml} \times 1000 \frac{ml}{l} \times \frac{25}{100}} \times \frac{m^3}{1000 \text{ l}} \cong 0.07 m^3$$

It is expected that the amount of sulfur bisulfite (SBS) will not change a lot considering the proper dosage of chlorine. To ensure the proper storage capacity, the volume of the tank will be 0.1 m³, and the footprint will be 0.1m³ and 1 m height.

The volume, footprint of the CEPT tanks, chlorine basins, and chemical storage tank are summarized in the following table. Bar screens, grit chambers, parshall flumes, and pumps will also added.

	CEPT Tank	Chlorine Basins	FeCl ₃ Tank	NaOCl Tank	SBS Tank
Volume (m ³)	135/each	40/each	4.5	1.2	0.1
Footprint (m ²)	135	40	3	1	0.1

Table 94. Volume and footprint of CEPT facilities

7.4 Plant Plan

The rough plan of the treatment plant is shown in Figure 7-1 below:

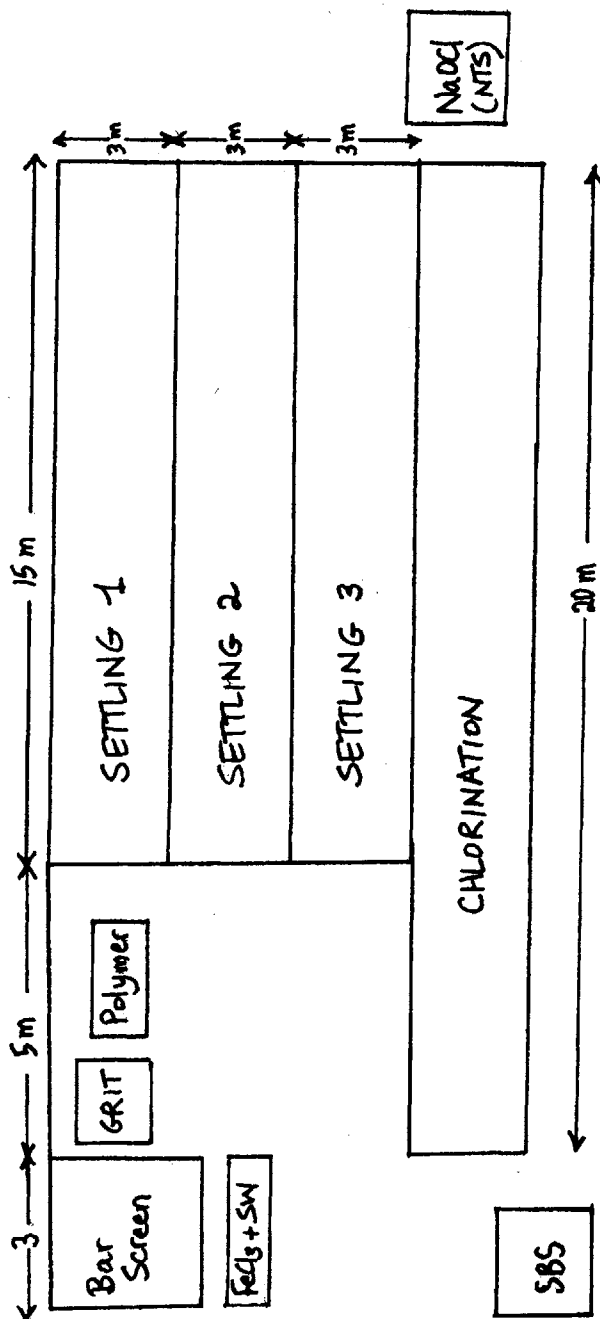


Figure 7-1: CEPT treatment unit locations (not to scale)

REFERENCES

- Deer Island Wastewater Treatment Plant (March, 2003). Disinfection/Dechlorination Performance Overview Report
- EPA (September, 1999), Wastewater Technology Fact Sheet Chlorination Disinfection. EPA. 832-F-99-062
- EPA (September, 1999), Combined Sewer Overflow Technology Fact Sheet; Alternative Disinfection Methods. EPA. 832-F-99-033
- EPA (September, 1999), Wastewater Technology Fact Sheet Ozone Disinfection. EPA. 832-F-99-063
- EPA (September, 1999), Wastewater Technology Fact Sheet Ultraviolet Disinfection. EPA. 832-F-99-064
- EPA (February 2002), Onsite Wastewater Treatment Systems Manual. EPA. pp. TFS17-TFS23
- Colgan. S. and Gelhr, R (November, 2001). Disinfection. *Water Environment & Technology*, vol. 5, pp. 29-33
- Blumenthal. U. J., Mara. D., Peasey. A., Ruiz-Palacios. G. and Scott. R (2000). Guidelines for the microbiological quality of treated wastewater used in agriculture: Recommendations for revising WHO guidelines. *Bulletin of the WHO*. vol. 78(9). pp. 1104-1116.
- Houghton Chemical Corporation. Water Front: Industrial Water Treatment Bulletin
- Solvay Interlox (2002). Material Safety Data Sheet: Proxitane® EQ Liquid Sanitizer. pp. 1-10.
- U.S.Army. Impact of Water Quality Based Permitting and the Risk Management Program Rule on Wastewater Disinfection (Winter 1998). *U.S.Army Center for Health Promotion and Preventive Medicine*
- Baldry. M. G. C., Cavadore. A., French. M. S., Massa. G., Rodrigues. L. M., Schirch. P. F. T. and Threadgold. T. L. (1995). Effluent Disinfection in Warm Climates with Peracetic Acid. *Water Science and Technology*. vol 31(5-6). pp. 161-164
- Sánchez-Ruiz. C., Martínez-Royano. S. and Tejero-Monzón. I. (1996). An Evaluation of the Efficiency and Impact of Raw Wastewater Disinfection with Peracetic Acid prior to Ocean Discharge. *Water Science and Technology*. vol 32(7). pp. 195-166
- Narkis. N., Armon. R., Offer. R., Orshansky. F. and Friedland. E. (1995). Effect of Suspended Solids on Wastewater Disinfection Efficiency by Chlorine Dioxide. *Water Resource Journal*. vol. 29(1). Pp. 227-236
- Jiménez. B., Chávez. A., Maya. C. and Jardines. L. (2001) Removal of Microorganisms in Different Stages of Wastewater Treatment for Mexico City. *Water Science and Technology*. vol. 43(10). pp. 155-162
- Liberti. L. and Notarnicola. M. (1999) Advanced Treatment and Disinfection for Municipal Wastewater Reuse in Agriculture. *Water Science and Technology*. vol 40(4-5). pp. 235-245

Metcalf & Eddy. (2002) *Wastewater Engineering Treatment and Reuse*. 4th ed. McGraw Hill. pp. 1219-1332

Haas. C. N. (2000) *Water Quality and Treatment - A Handbook of Community Water Supplies*. 5th ed. McGraw Hill. pp. 14.1-14.8

Palmer. T. (1999) *Compound Costs of Instrument Inaccuracies*. Instrumentation Testing Association

The Dow Chemical Company. (2000) *Filmtec Membranes: Biological Fouling Prevention*. pp.1-6

PPG Industries, Inc. (December 1999) *Bulletin: Calcium Hypochlorite*

REGAL-Chlorinators Inc. http://www.regalchlorinators.com/html/poultry_details.html

Department of Health Services. State Ocean Water Quality Standards. Section 100275, *Health and Safety Code. Los Angeles County*

United Nations , www.un.org, May 13, 2003

World Health Organization, www.who.org, May 14, 2003

Choi Nancy, *Design of Wastewater Infrastructure for the Historical Center of Paraty*, M.Eng Thesis, MIT, 2003.

You Evelyn, *Policy for development of water and sanitation infrastructure in Paraty, Brazil*, M.Eng Thesis, MIT, 2003.

Harleman and Murcott, *An innovative approach to urban wastewater treatment in the developing world*, *Water 21*, June 2001.

www.hach.com, and *Hach Water Analysis Handbook (2003)*.

Harleman, by personal communication, 2003

Landers Jay, *Civil Engineering News*, Orange County and San Diego Contend With Waiver Woes, July 2002, p.29

Ayoub and Koopman (1986), *Seawater induced algal flocculation*, *Water Resources Journal* Vol. 20, No.10, pp.1265-1271

Ayoub et al, *Coagulation of alkalinized municipal wastewater using seawater bittern*, *Water Environment Research*, Volume 71, Number 4, p443

Ayoub and Koopman (1992), *Seawater Flocculation of Emulsified Oil and Alkaline Wastewaters*, *Water Resources Journal*, Volume 26, number 6, pp. 817-823

Harleman et al, *International Panel reviews plans for treatment upgrade*, *Water Environment and Technology*, pg 46-50 March 1997.

National Research Council (NRC), editor, *Wastewater Management in Urban Coastal Areas*, *National Academy Press*, Washington, D.C., 1992.

Harleman, Innovative Wastewater Treatment in the Developing World, *Civil Engineering Practice*, p.25, 2002.

Bagot, Chemical Treatment of sewage: Experiences in San Francisco, *Water and Sewage Works*, September 1990, p.295

City of San Diego, *Annual Monitoring Report of the Point Loma Outfall at the Point Loma Wastewater Treatment Plant*, 2000.

Massachusetts Water Resources Authority, www.mwra.com

Ferguson and Vrale, Chemical aspects of the lime-seawater process, *Journal of Water Pollution Control Federation*, 56,355-363