Potable Water and Wastewater Infrastructure Planning for the City of Paraty, Brazil

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EXECUTIVE SUMMARY

Paraty is a beautiful coastal city in the State of Rio de Janeiro that is thriving as a tourist city with its abundant natural beauty and cultural charm. With its esteemed Historical Center, which has well-preserved colonial architecture of considerable historical and cultural significance, the City is currently making efforts to qualify for a UNESCO World Heritage Site, which encompasses improving its existing sanitation system. In congruence to the City's objectives, this project is undertaken for a general purpose of assessing the condition of the City's existing water and sanitation infrastructure and associated public health problems, and of providing a preliminary design of wastewater collection system and treatment plant for the City.

The inadequacy of the City's existing potable water quality and supply system, well-known to and fully felt by the local population, include: (i) shortage of water supply in the summer; (ii) ineffective disinfection; (iii) inadequate protection of water sources; and (iv) substandard water quality (Chapter 2). In order to address these problems, the existing disinfection system for the potable water is to be upgraded immediately, with a flow meter and an automated chlorinator, and the potable water intake points are to be fenced around the perimeter, in order to protect the source waters. A new drinking water treatment plant, with the capacity for the entire City of Paraty including Jabaquara, is to be constructed in one stage, immediately following the completion of the development of wastewater collection system and treatment plant (Chapter 3).

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The City's sanitation system, which is almost completely lacking, has greater impact on the humans and the environment, and the need for its development is therefore more imminent. Public health problems related to water and sanitation, reflected in the high incidence of diarrhea in the City, are representative of not only the poor potable water quality, but also the direct consequence of discharging untreated wastewater into nearby surface water bodies, with which people easily come into contact (Chapter 2). It is therefore evident that the City must construct a wastewater collection system and infrastructure and a wastewater treatment plant, in order to reduce the environmental pollution and associated public health risks.

It is proposed that a gravity sewer system be used for the collection of wastewater, and that a chemically enhanced primary treatment (CEPT) plant be used for the treatment of wastewater. The wastewater collection system and infrastructure and the treatment plant are to be constructed concurrently in three stages for the City, in order to fully utilize the easily upgradeable characteristics of the CEPT plant and thereby reduce the cost of developments. The Historical Center is to be developed in the first stage; Mangueira and Ilha das Cobras in the second stage; and the Old City and rest of the City in the third stage. Each stage is expected to last approximately two years, and the wastewater treatment plant is to serve the developed areas immediately after the completion of each development stage (Chapter 3).

The conceptual design of the wastewater collection system is limited to the Historical Center, but applicable to the whole city. A gravity sewer system is proposed as the system of wastewater collection, based on four selection criteria: economics, expandability, adaptability, and simplicity. The feasibility analysis of conventional gravity sewers, pressure sewers, vacuum sewers, and small diameter gravity sewers reveals that the vacuum system is the least expensive alternative. However, the vacuum system is also a relatively new technology, and requires high operation and maintenance skills. Therefore, the study suggests that the gravity sewer system, which is also relatively inexpensive, may be most appropriate for the City since is a well-established, simple technology.

A chemically enhanced primary treatment (CEPT) plant is proposed for the treatment of the City's wastewater. The addition of 40 mg/l of ferric chloride and 5 % seawater are recommended for CEPT, according to the results of jar test experiments. Furthermore, 0.1 mg/l of polymer can be added to enhance the SS and COD removal efficiencies. For chlorination and dechlorination, 3 mg/l of chlorine and 0.5 mg/l of sulfur bisulfate are recommended. Expected removal efficiencies are 85 % for SS, 55 % for COD, and 100% for fecal coliform. Two CEPT tanks with dimensions of 15m x 3m x 3 m, and 2 chlorination basins with dimensions of 20m x 1m x 2 m are required, and one additional tank of each is recommended for maintenance. The wastewater plant requires a total footprint of approximately 180 m^2, excluding the area required for the treatment of sludge.

The total cost of the wastewater treatment collection system and treatment plant is approximately R$ 4 million for capital, and R$ 0.5 million per year for operation and maintenance (O&M). The total cost of a new drinking water treatment is approximately R$ 1 million for capital, and R$ 0.4 million per year for O&M. In order to fully recover these costs, an annual revenue of R$1.2 million must be collected from water and sewage tariffs. The following water and sewage tariffs, which are based on willingness to pay (WTP), are to be billed for each income group: R$1.40/$m^3$ for Mangueira and Ilha das Cobras population; R$2.40/$m^3$ for Historical Center and Old City population; and R$1.20/$m^3$ for Jabaquara population. Since these tariffs can be seen as a substantial increase from the existing tariffs, appropriate interim tariffs are to be designed and implemented in one or more steps to phase in the final design tariff.

The construction of wastewater collection infrastructure and treatment plant, and drinking water treatment plant is expected to bring numerous and substantial benefits to the City, which include: improvements in public health, environmental quality, and aesthetics in the city, as well as increases in productivity and economic value of the environment. It is also expected that these water and sanitation improvements will encourage tourism and promote general economic growth in the City, providing large economic returns.

EXTENDED ABSTRACT

The United Nations and World Health Organization (WHO) have repeatedly included Brazil on the list of nations required to immediately address water and sanitation infrastructure, appropriate treatment technologies and related public health issues. The city of Paraty, which is not an exception to these cases, is the setting for this work. In January 2003 a team of four environmental engineering graduate students together with staff and faculty supervisors spent three weeks in Paraty to assess the extent of the potable and wastewater problems in the city. The overall goal of this project is to provide a master plan that addresses the water and wastewater situation in Paraty. The main tasks addressed by this project center around the following goals:

- Identify existing problems with the city's water and sanitation system and quantify the drinking and surface water quality.
- Design a sewage collection system for the Historical Center of Paraty that can be used as a modular example for expansion to the entire urban area.
- Select and design an appropriate wastewater treatment technology that results in an effluent that can be effectively and economically disinfected.
- Provide a construction schedule and approximate costs for the implementation of the suggested water and wastewater infrastructures.

Background on Paraty

Paraty is located on the acclaimed Green Coast of Brazil in the state of Rio de Janeiro, 250 kilometers south of the city of Rio de Janeiro. Due to its prime geographic location and historical architecture, Paraty is a key tourist attraction. The estimated winter population in the historical center of Paraty is 3,000 inhabitants. This is estimated to reach 6,000 inhabitants in the summer. The seasonal population change causes a large increase in water consumption and wastewater production. Paraty does not currently have a wastewater collection or treatment system. The treatment of its potable water is inadequate and future demands for potable water will not be met. Paraty is currently and actively pursuing a UNESCO World Heritage Site qualification but must treat its wastewater in order to be eligible for nomination. The issue of water and sanitation in Paraty has therefore become a 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 designed or revamped to serve Paraty's fluctuating population both efficiently and economically.

Figure 1: The state of Rio de Janeiro

Policy for Water and Wastewater Infrastructure

Introduction

Paraty's current state of water supply and sanitation, the extent of environmental degradation, and appropriate response measures were studied in the first part of this project. Paraty currently suffers from poor public health, polluted surface waters and degraded aesthetics, all of which are directly related to poor water and sanitation systems. In addition, the City's goal of becoming a

UNESCO World Heritage Site has been deferred due to the lack of functioning sanitation system in the Historical Center.

In order to clearly identify the problems associated with water and sanitation, a number of water quality tests were performed for Paraty's drinking water supply, and for a number of ambient surface water bodies in and near the City. In addition, the diarrhea incidence in the City was also studied in order to better understand the health consequences of poor water and sanitation. The results of these studies are summarized below:

Problems with Potable Water Supply

The potable water supply system for the City of Paraty has a number of problems that should be addressed. The most important of these are: (i) shortage of water supply in the summer; (ii) ineffective disinfection; (iii) inadequate protection of water sources; and (iv) substandard water quality.

Numerous water quality analyses revealed that the quality of Paraty's potable water is heavily influenced by the quality of surface waters from which it is derived. It often fails to comply with international drinking water standards due to high turbidity after rainstorms, and bacterial contamination. These analyses show that the present method of disinfection is insufficient.

The City of Paraty should adopt various measures to improve the quality of its drinking water. In addition to procuring a sufficient supply of drinking water to meet demand at all times, the City must protect its drinking water at the sources. Coagulation and filtration are necessary steps before disinfection in order to reduce turbidity in water, which frequently rises to unacceptable levels after rainstorms. In addition, a more effective method of chlorination must be adopted.

Problems with Wastewater Disposal

Due to the lack of collection and treatment of wastewater, the City of Paraty suffers from serious environmental degradation and associated health consequences. The environmental degradation in the City results from direct discharge of untreated sewage into surrounding water bodies, and from tidal inflows that flood the streets with a mixture of sewage and seawater.

Four surface water bodies (Jabaquara Beach, Matheus River, Pereque River, and an open ditch, "sewer stream," were tested for water quality and found to have fecal coliform concentrations that suggest contamination from untreated sewage. Consequently, Jabaquara Beach was found to be unsafe for swimming, and Matheus River and Pereque River unsafe for all aquatic activities. In addition, the “sewer stream” was found to have the water quality of diluted raw sewage.

The uncontrolled disposal of wastewater damages the aesthetics of the rivers, and reduces the commercial value of the environment. The sources of pollution must be controlled in order to preserve the environment from further degradation and an appropriate treatment and disposal of the City’s wastewater is critical.

Problems with Public Health

Poor public health is a direct consequence of poor potable water quality and polluted environment. Diarrhea, a widely studied indicator of water and sanitation-related diseases, was found to be prevalent in both the urban and the rural areas of Paraty, especially in Mangueira and Ilha das Cobras, the more densely populated, low-income areas within the City of Paraty.

It is assumed that a significant proportion of diarrhea cases are caused by waterborne pathogens, although it is difficult to estimate the exact proportion caused by the consumption of poorly disinfected drinking water, or by the contact with polluted surface waters. For the City of Paraty, it is speculated that both the ineffectively disinfected drinking water, and the highly polluted surface waters are the causes of diarrhea and other water and sanitation related diseases.

Improvements

In order to mitigate the problems identified above, improve the quality of life, and foster economic growth, Paraty’s water and wastewater infrastructure must be improved.

Areas of improvement in the potable water supply are: (i) treatment of drinking water, (ii) protection of drinking water sources, and (iii) procurement of sufficient drinking water supply. It is evident that the City’s potable water must be filtered and better disinfected in order to make it safe for drinking, and that the drinking water sources must be isolated in order to prevent accidental contamination of the source waters. Furthermore, to improve the quality of life for the local population, as well as the tourists, potable water shortages must be eliminated.

Areas of improvement in wastewater supply are: (i) collection of wastewater, and (ii) treatment of wastewater. New wastewater infrastructure must be put in place to collect sewage, and a new wastewater treatment plant must be constructed in order to treat the wastewater before it can be safely discharged into the surrounding waters.

Development Stages

The wastewater collection infrastructure and treatment plant are to be constructed in three stages for the City of Paraty, excluding the Jabaquara area. The Historical Center should be developed in the first stage; Mangueira and Ilha das Cobras in the second stage; and the Old City and the remainder of the City in the third stage. Each development stage is expected to last 2 years. The completion of each stage should initiate an immediate start of the subsequent stage.

The potable water disinfection system should be upgraded as soon as possible by providing flow meters and automated chlorination. Intake points should be fenced around the perimeter, in order to protect the source waters. A conventional potable water treatment plant sufficient for the capacity for the entire City, including Jabaquara, should be constructed in one stage, immediately following the third stage of wastewater infrastructure development.

Wastewater Collection System

Introduction

The city is in need of a plan to collect and control its wastewater. The second part of this project presents a conceptual design for a wastewater collection system in the historical center of Paraty. The design of this collection system involved investigating wastewater flow requirements, alternatives for wastewater collection, possible locations for a treatment plant, a feasibility study, and cost estimates for the system.

Design Criteria

Investigation of the area in demand of sewerage is important for design and construction. Paraty's sewer design was based on field visits of the proposed sewerage area, a review of the city's mapping, and a preliminary analysis of different sewage collection alternatives.

The alternatives of collection considered for the historical center were conventional gravity sewers, pressure sewers, vacuum sewers, and small diameter gravity sewers. Four criteria were chosen as critical in the analysis and search for the most appropriate collection system. These are economics, adaptability, expandability, and simplicity.

A conventional gravity design was chosen for the historical center of Paraty based on an analysis of various collection systems. Conventional sewers were seen as the optimal collection infrastructure for the following reasons:

- Paraty requires a system that is easy to maintain and does not require extensive technical support. The overall plan for a treatment and collection system therefore needs to be expandable, adaptable and centralized and a conventional gravity sewer system is more easily expandable than the alternative systems.
- Paraty needs an overall collection and treatment system that is adaptable and robust to the changes in seasonal population and rainfall. A conventional gravity system coupled with chemically enhanced primary treatment (CEPT) is an ideal technology for coping efficiently with seasonal variations. Finally, a simple system that is adaptable and expandable is ideal for Paraty because it would minimize personnel needed to handle operation and maintenance.

Wastewater Collection Design Process

The design of the gravity flow collection system respects local restrictions. The system was designed for peak hourly flow of the base winter population. A detailed profile and model of the sewer network was created. A spreadsheet was prepared in Microsoft Excel to record the data and steps in the computations for each section of sewer between manholes. Using Haestad Method's SewerCAD, the sewer invert elevations, pipe diameters, pipe slopes and velocities were determined by trial and error to find the best fit design given the design factors and constraints on the depth of excavation.

Wastewater Collection System Design

The gravity sewer system consists of 2,500 meters of gravity sewer, 22 manholes and 1 pump station. Figure 2 displays the network of the gravity system for the historical center.

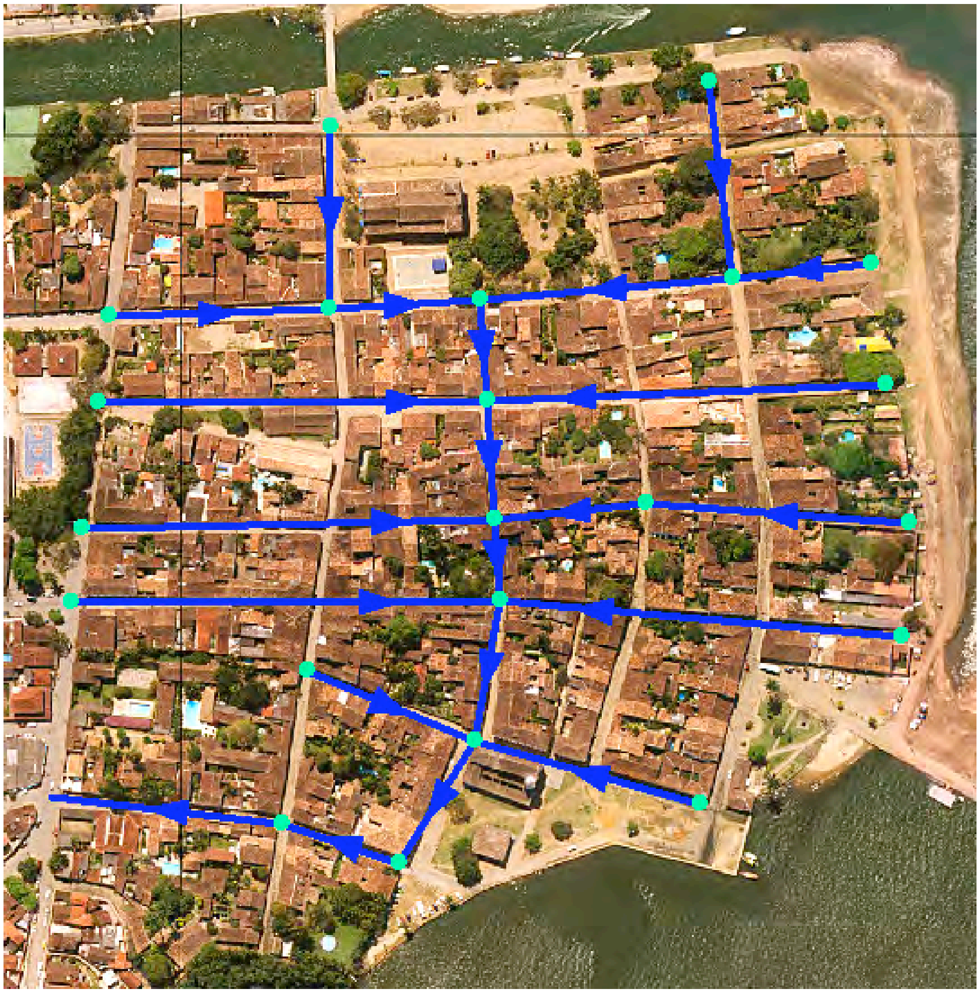

Figure 2: Sewer Network Design

Wastewater Treatment Design

Introduction

Chemically enhanced primary treatment (CEPT) is a simple and cost effective wastewater treatment technology that is an attractive alternative to biological treatment. CEPT adopts the coagulation and flocculation processes typically used for potable water treatment (with the addition of typical coagulants and flocculants such as $FeCl_3$ and anionic polymers) 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 phosphorus) 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. CEPT is robust in that it can effectively handle seasonal flows two to three times the winter rate without a significant reduction in removal efficiency.

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

- 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).
- The space constraints, high maintenance and capital costs etc of other treatment alternatives were considered limiting factors which made CEPT the most appropriate wastewater treatment process for Paraty.

CEPT Design Process

A series of jar tests were conducted in Paraty to determine the most cost-effective dose of ferric chloride ($FeCl_3$) and polymer concentrations that would achieve high suspended-solids, turbidity, and chemical oxygen demand removal rates. The addition of small quantities of seawater was also tested as an innovative and inexpensive method of enhancing coagulation and reducing ferric chloride demand. A 2% volume of seawater added to the influent reduced FeCl3 demand by 50%. Similarly, a 5% seawater volume reduced FeCl3 by 85%. The addition of seawater makes use of coagulant, such as magnesium salts, naturally present in ocean water. In Hong Kong, the world's largest and most efficient CEPT plant, a high degree of pollutant removal occurs because of seawater used for toilet flushing. The recommended chemical and seawater doses, along with expected influent concentrations and effluent removals are summarized in Table 1.

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 removal	85%
COD removal	55%

Table 1: Design Parameters for Paraty

CEPT Plant Design

The following dimensions were determined for the CEPT plant based on the three phases planned (Table 4) and for various surface overflow rates (SOR). The plant in its completed three phases will consist of two functioning CEPT settling tanks and one extra tank for maintenance.

Dimension of 1 CEPT Tank	Width (m)	3		
	Height (m)	3		
	Length (m)	15		
Stage		1	2	3
Number of CEPT Tanks		1	1	2
Footprint (m^2)		45	45	90
Flow Capacity (m^3/day)	Minimum	1350	1350	2700
	Median	2700	2700	5400
	Maximum	4050	4050	8100
Expected Daily Flow (m^3/day)	Non-Summer Season	540	1890	2430
	Summer Season	1620	2970	4590

Table 2: Dimension of the CEPT tanks and capacities for each stage

Minimum, median, maximum capacities are calculated with overflow rates of 30, 60, 90 m/day. Expected wastewater flow is based on the water consumption for each stage.

Finally, the volumes of the chemical storage tanks were also included in the design and are summarized in Table 3 below:

	CEPT Tanks	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 3: Various Tank Dimensions

Disinfection

In Paraty, there are no regulations concerning the acceptable level of coliform concentrations in the treated wastewater. According to Brazilian regulations 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. The 200 FC MPN/100ml standard adopted in the United States was considered appropriate for this design since the discharge point of the CEPT plant is near touristic beaches.

Peracetic acid (PAA) was considered for its use as a disinfection agent instead of chlorine because of the absence of disinfection by-products (DBPs). PAA was nonetheless considered an inappropriate disinfection agent in this project because PAA costs approximately 10 times more than chlorine. The ideal chlorine concentration appropriate for the Paraty wastewater was therefore chosen to be 3 mg/L of liquid sodium hypochlorite and 0.5 mg/L of sulfur bisulfate for dechlorination.

Due to the limited disinfection data in Paraty, and to gauge the effect of seawater addition on the disinfectability of a treated effluent, more research was done at the Boston MWRA's Deer Island Wastewater Treatment plant. It is important to note that fecal coliform reductions depend on the suspended solids removals and that these increase incrementally.

The raw influent to Deer Island already contains about 2% seawater, probably due to leakage into the collection system. Further addition of seawater to the Boston influent did not yield significant increases in suspended solids removals, compared to those seen in Party. Table 4 is a summary of the detention times for the chlorine basins for the various flows considered in Paraty:

Dimension of 1 Chlorine Contact Basin	Width (m)	1		
	Height (m)	2		
	Length (m)	20		
Stage		1	2	3
Number of Chlorine Contact Basins		1	1	2
Footprint (m^2)		20	20	40
Expected Daily Flow (m^3/d)	Non-Summer Season	540	1890	2430
	Summer Season	1620	2970	4590
Detention time (min)	Non-Summer Season	107	30	47
	Summer Season	36	19	25

Table 4: Dimension of the Chlorination basin and detention time for each stage

Costs of Improvements

The total capital costs and O&M costs associated with the above improvements are as follows:

Total Capital Costs and O&M Costs for Water and Sanitation Improvement Projects	
Total WW Collection Infrastructure and Treatment Plant CC	R$ 4 million
Total WW Collection and Treatment Annual O&M Cost	R$ 0.5 million/yr
Total DW Treatment Plant CC	R$ 1 million
Total DW Treatment Annual O&M Cost	R$ 0.4 million/yr

Table 5: Total Costs for Water and Sanitation Improvement Projects

The total annual cost is approximately R$1.2 million, with the capital cost amortized over a 30-year project life at 10% annual interest rate. In order to fully recover costs of water and sanitation improvements, annual revenue of R$1.2 million should be collected from water and sewage tariffs. The following water and sewage tariffs, which are based on willingness to pay (WTP), are suggested for each income group:

- R$1.40/$m^3$ for Mangueira and Ilha das Cobras population;
- R$2.40/$m^3$ for Historical Center and Old City population; and
- R$1.20/$m^3$ for Jabaquara population.

Since these tariffs can be seen as a substantial increase from the existing tariffs, appropriate interim tariffs should be designed and implemented in one or more steps to phase in the final design tariff.

Benefits

The benefits associated with water and sanitation improvements are numerous and substantial, although it is difficult to associate these benefits with monetary values for cost-benefit analysis. Some of the benefits include:

- Disease reduction and improved human productivity;
- Healthier environment, improved aesthetics, and associated increase in amenities, economic values, and intrinsic values of the environment;
- Encouraged tourism, poverty alleviation, and general economic growth; and
- UNESCO World Heritage Site candidacy, and associated distinction and merit.

Conclusion

The purpose of this project is to address and study the water and wastewater situation in Paraty, Brazil and then propose a recommended course of action. Based on the analysis done in this project, a strong recommendation is given to the city to plan for a wastewater collection system and a wastewater treatment system. The two components should not stand-alone. A gravity collection system in conjunction with Chemically Enhanced Primary Treatment would be the best-fit strategy for Paraty’s wastewater issue. Also, the drinking water disinfection system should be upgraded immediately, with flow meters and an automated chlorinator, and the drinking water intake points should be fenced around the perimeter, in order to protect the source waters. Ultimately a conventional potable water treatment plant with coagulation, filtration and chlorination should be provided.

Acknowledgements

This work would not have been possible without the initiative and cooperation of many concerned citizens of the city of Paraty. Specifically the team would like to recognize the tireless and extremely helpful efforts of Ricardo Tsukamoto and his family. The team also wished to express their gratitude to the administration of the city of Paraty for their hospitality and help during the field visit to Paraty.

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CHAPTER 1 – INTRODUCTION TO PARATY, BRAZIL

The area of focus for the water and sanitation studies in this report is the City of Paraty, located in the State of Rio de Janeiro, Brazil. Paraty is a historical city, with much natural and cultural charm, that has a potential to grow as a tourist city. However, doubtful drinking water quality and polluted rivers and beach water, which are associated with lack of wastewater treatment, could very well threaten the health of tourists and local population, and hinder the development of the tourism industry. Therefore, a careful study of the City's current state of water supply and sanitation, the extent of environmental degradation, and appropriate response measures are to be studied for the City of Paraty in this report.

1.1 Location, Area, Climate, and Population

The City of Paraty is located within the Municipality of Paraty, which is located in the south coast of the State of Rio de Janeiro, Brazil (See Figure 1.1). The Municipality of Paraty covers an area of 930 km^2, with the average elevation of 5 meters (Prefeitura, "Patrimony" 3). Embracing the Bay of Ilha Grande (Baia da Ilha Grande), Paraty has the mild climate that is hot in the afternoon most of the year, and receives more than 1.5 m of rainfall each year (Canaldotempo.com).

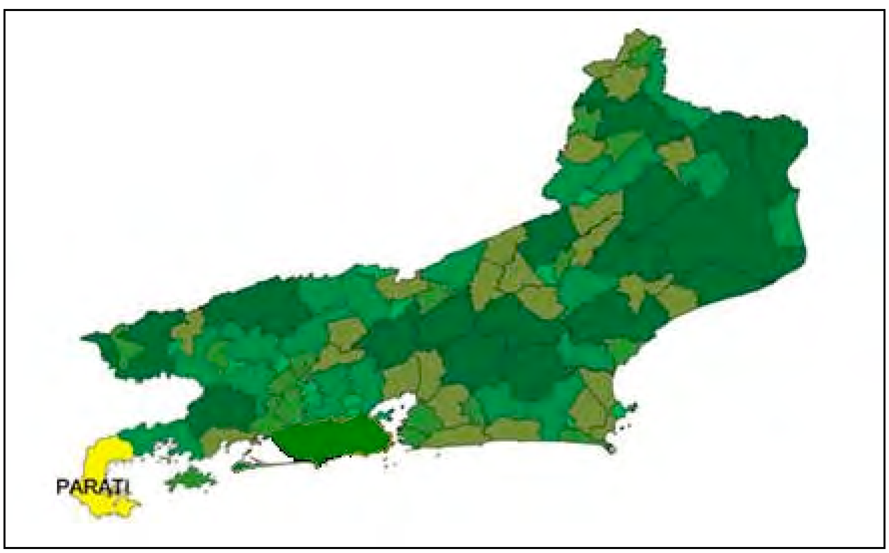

Figure 1.1. Location of Municipality of Paraty in the State of Rio de Janeiro, Brazil (not to scale)

The Municipality of Paraty has a population of 30,000 (Census 2000), approximately 15,000 of which are concentrated in the urban area, in and near the City of Paraty. The other 15,000 are dispersed in smaller rural communities around the Municipality (See Figure 1.2)

Figure 1.2. Municipality of Paraty (not to scale)

1.2. City of Paraty

The City of Paraty, which has the highest population density in the Municipality, has two rivers, Pereque River (Rio Pereque-Acu) and Matheus River (Rio Matheus-Nunez), running through it and discharging into the Paraty Bay (Baia Paraty) (See Figure 1.3) Matheus River, in the South, forms the southern boundary of the City, and the northern end of Jabaquara Beach (Praia Jabaquara) forms the northern boundary.

The City can be subdivided into five sections: (1) Historical Center; (2) the Old City; (3) Manguera; (4) Ilha das Cobras; and (5) Jabaquara (See Figure 1.3) Paraty's Historical Center, which preserves the authentic colonial architecture, from the 17th century when Paraty was a major staging post for Brazilian gold passing from Minas Gerais to Portugal, is a national monument, considered by UNESCO (United Nations Educational, Scientific and Cultural Organization) to be one of the most important surviving examples of colonial architecture in the world. The streets in the Historical Center are paved with irregular stones, which form a canal that drains off storm water and allows for the sea to enter and wash the streets at full moon and high tides. Manguera and Ilha das Cobras are the poorer areas of the City. The Old City and Jabaquara consists mainly of inns and other accommodations for tourists, and are generally wealthier areas.

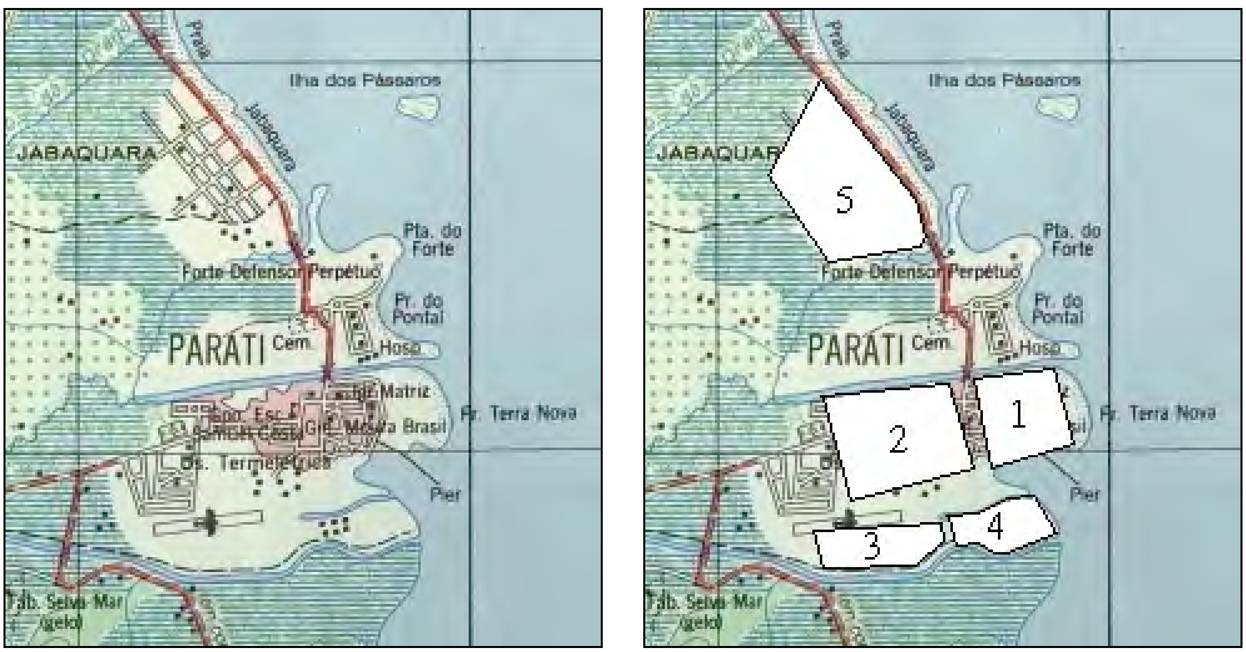

Figure 1.3. City of Paraty (not to scale)

The City has a total population of approximately 15,000, which increases manifold during summer due to tourism. The increases in population during summer is greatest for the Historical Center, a great tourist attraction, and for Old City and Jabaquara, which are mainly summer resort areas. In contrast, population increase is not expected for Mangueira and Ilha das Cobras areas, which are mainly residential areas for the local population.

1.3. Tourism Industry

The tourism industry in Paraty is active and strong, and is considered one of the largest contributors to the City and Municipality's economy, next to fishing, trade, and craft (Prefeitura, "Patrimony," 2003). Reflecting the City's thriving tourism industry, are many lodgings and hotels, pubs and restaurants, stores and boutiques, and travel agencies located in the City. Besides the Historical Center, there are many more tourist attractions, some of which include: islands; waterfalls; beaches; natural parks of preservation; museums; historical monuments; military forts; and folkloric parties (Prefeitura, "Patrimony," 2003). The City's location, situated advantageously between the two largest cities in Brazil, Rio de Janeiro and Sao Paulo, helps the tourism industry by allowing tourists to travel conveniently through either of the two cities. Sao Paulo and Rio de Janeiro have the two busiest airports in Brazil, and there were approximately 2.8 million international arrivals in Sao Paulo Airport, and 1 million in Rio de Janeiro Airport, in 2001, according to a poll taken by the Brazilian Tourist Office.

1.4. Candidacy for UNESCO World Heritage Site

The well-preserved 17th century colonial architecture in Paraty's Historical Center is the Brazilian national historic monument, and a candidate for UNESCO World Heritage Site. The World Heritage List, a direct result of the adoption of the Convention Concerning the Protection of the World Cultural and National Heritage by UNESCO in 1972, authenticates, in an area or monument, the existence of heritage that belongs to and is important to humanity. To be included in the World Heritage List, sites must satisfy severe selection criteria, following an extensive nominating procedure. A cultural criteria for the World Heritage Site follows: "works of man or the combined works of nature and of man, and areas including archaeological sites which are of outstanding universal value from the historical, aesthetic, ethnological or anthropological points of view" (UNESCO, 1997).

Paraty, which has initiated the nomination process for the UNESCO World Heritage Site, is in the stage of planning the improvements in water and sanitation, which are a few of the requirements specified by ICOMOS (the International Council on Monuments and Sites), one of

UNESCO’s two technical advisory bodies. The current, non-existing, wastewater treatment and disposal system in Paraty was identified as unsatisfactory, and a system that complies with domestic and international standards is required, in order for Paraty to qualify as a candidate for World Heritage Site.

CHAPTER 2 – PRESENT CONDITION OF WATER AND SANITATION

2.1. Institutional Framework in Paraty

The water and wastewater sector in the Municipality of Paraty is in a state of instability and faced with an uncertain future. Since the concession period, from the Municipality to CEDAE (the Rio de Janeiro State-owned water and sewage company), expired approximately 6 years ago, the Municipality has neither renewed its contract with CEDAE nor completed a full transfer of the control of its water and sanitation systems. While the Municipality remains undecided in its approach toward its repossessed water and sanitation systems, CEDAE continues to provide services without having established a new concession agreement with the Municipality. In the past, CEDAE has made apparent efforts to renew its concession with the Municipality, by making propositions such as: (i) spending R$0.2 million for fixing and making operational a partially constructed and abandoned drinking water treatment plant; and (ii) spending R$10 million for the operation and maintenance of potable water treatment and distribution (Lemos Padua, 2003). However no agreement has been reached.

The extent of Municipality's participation in its own water and sanitation sector depends largely on the interests of the individuals in political power. During the seat of previous mayor, Dede, the Municipality had constructed new water supply pipeline, begun the construction of a potable water treatment plant, and measured domestic water consumption using water meters. However, with the election of a new mayor in 2000, many of these projects were abandoned while new projects were devised and undertaken. For example, the Municipality had abandoned the construction of the treatment plant, discontinued the reading of water meters, set the tariffs for water and sanitation according to property size, and informally entrusted CEDAE with much of the water and sanitation services since 2001 (Reis, 2003).

2.2. Services Coverage

Paraty has a coverage of water supply and sanitation services that is lower than the national average, which is itself far below the desired universal coverage. According to a report prepared by the Municipality of Paraty in 2002, 60% of the total population in Paraty is supplied with public water that is disinfected with chlorine, and 12% is provided with sewage collection, that discharges, untreated, directly to surrounding surface water bodies (Prefeitura, "Laudo," 2002). This figure is lower than the nationwide average of 75% domestic water connection, and 48% connection to public sewer system.

The disparity is even greater when the coverage in Paraty is compared with the coverage in Rio de Janeiro, one of the nation's largest cities, that is also near Paraty. In Rio de Janeiro, where more than 99% of the population is in the urban area, 90% of the urban population has domestic water supply connection and 84% has connection to public sewer system (CEPIS, 2000). To compare more equitably, it is important to note that approximately 100% of the urban population in Paraty receives water that is disinfected with chlorine, and 0% of the wastewater collected is treated before discharge. In contrast, 77% of the total population in Rio de Janeiro receives effectively disinfected water through the distribution network; and 41% of the total wastewater produced in Rio de Janeiro is treated (CEPIS, 2000).

2.3. Existing Potable Water Supply System

Paraty, which receives more than 1.5 m of rainfall each year, is well endowed with an abundant supply of drinking water sources at the mountains. These drinking water sources, most of which are surface waters in the form of streams or rivers, have pristine water quality most of the time. Unfortunately, however, surface waters are easily contaminated with increased amounts of particulate matter in the water after rainstorms, due to erosion of sediments caused by rapid currents. In addition, surface waters are contaminated by the runoffs from upstream areas; so the presence of farms upstream or nearby can easily pollute the waters with fertilizers and animal feces. Therefore, the potable water, with surface waters as its source, has highly variable water quality, and requires filtration and disinfection treatment.

Despite these problems of frequent rainstorms and farms located near and upstream of the water intake points, the Municipality of Paraty disinfects only two of its many water sources, mainly those that serve the urban population in the City of Paraty. The disinfection is performed by the addition of chlorine and without filtration or any other form of pretreatment.

In the entire Municipality of Paraty, there are two other systems of potable water that receive treatment, and they are provided by private sectors for private developments. The first system, Condominio Laranjeiras, serves approximately 500 households in Laranjeiras and Vila Oratorio, and the other system, Vila Residencial da Eletronuclear, serves approximately 680 households in private developments in the Mambucaba area (Prefeitura, "Vigilancia," 2001). Both systems are described as conventional treatment with disinfection. The rest of the rural communities in Paraty consume water that is brought from various surface water sources in the mountains, and some groundwater sources.

Potable Water Infrastructure

The City of Paraty is supplied with disinfected water that is brought from two surface water sources, called Pedra Branca and Caboclo. The intake points of Pedra Branca and Caboclo are located in the mountains, approximately 7 km and 4 km west of the City, respectively. Pedra Branca withdraws water from Pereque River (Rio Pereque-Acu), which also flows through the City of Paraty further downstream, immediately before discharging into Paraty Bay (Baia Paraty).

Pedra Branca and Caboclo operate in a complementary system, supplying water to the City of Paraty (See Figure 2.1) The water from Pedra Branca is disinfected with chlorine gas at the source and transported to a reservoir located next to the City of Paraty, where it is combined with the water from Caboclo that has not been chlorinated. The water is disinfected with chlorine gas at the reservoir, before it is distributed to the City of Paraty. The water is not filtered before disinfection. The complementary water supply system, Pedra Branca and Caboclo combined, supplies water to approximately 3,850 households in the City of Paraty as well as the rural areas near the intake points. The average water consumption in the City is approximately 180 L of water per capita per day, according to a report prepared by the Municipal City Hall of Paraty (Prefeitura, "Vigilancia," 2001).

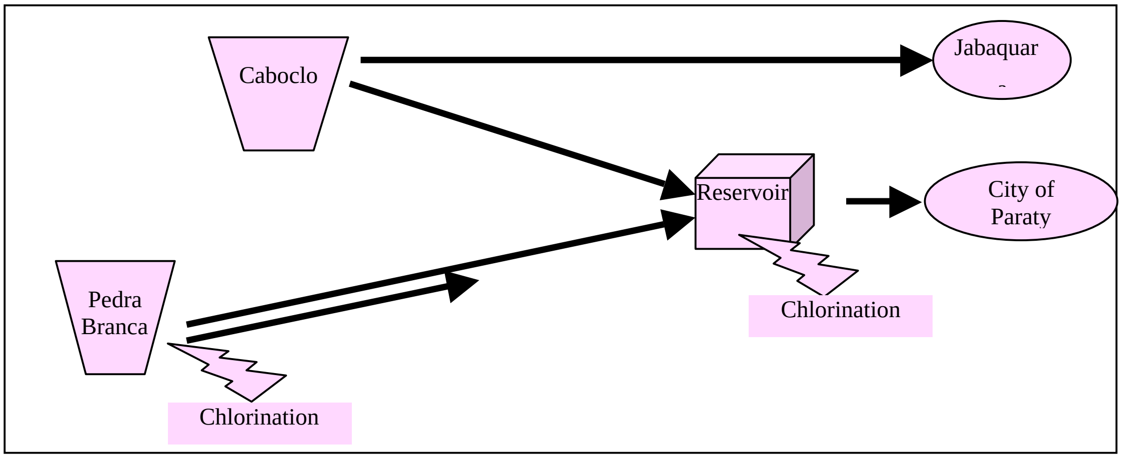

Figure 2.1. Schematic diagram of water supply system for the City of Paraty

Pedra Branca Intake

The water intake system at Pedra Branca, which withdraws water from Pereque River, consists of a concrete dam (W = 23 m, H = 2.1 m), a grit box (L = 6.4 m, W = 1.2 m, H = 2.9 m), also constructed in concrete, and 48 meters of 400 mm intake pipe that connects the two structures. The grit box, located below the dam, captures sand that is mixed with water, and the collected sand is removed from the grit box periodically. Water is disinfected with chlorine gas after it leaves the grit box, before it is taken to the city's reservoir by two 200 mm pipes. One of the two 200 mm pipes is iron pipe, constructed by CEDAE in 1975, and it stretches 6,000 meters from the grit chamber to the City's reservoir. However, the other 200 mm pipe, which is PVC pipe, extends only 3,000 meters and does not connect to the reservoir, although it was built by the City to serve as a duplicate of the iron pipe (Prefeitura, "Laudo," 2002).

Caboclo Intake

The water intake system at Caboclo consists of a concrete dam (W=5.3 m, H=1.1 m), a narrow concrete channel (L=17.9 m, W=1.2 m, H=0.8 m), and two stabilizing basins also in concrete, which act as grit boxes. A 150 mm iron pipe stretches 3,000 meters from Caboclo to Jabaquara, and a 150 mm PVC pipe transports water from Caboclo to the City's reservoir. The Caboclo intake system was constructed by the City of Paraty in 1999 (Prefeitura, "Laudo," 2002).

Reservoir

The City's reservoir, which receives chlorinated water from Pedra Branca and raw water from Caboclo, is located on a small hill, near the City of Paraty. The reservoir, built by the CEDAE in 1975, consists of two adjacent tanks, each with dimensions of L=16 m, W=11 m, and H=3.2 m. The total capacity of the reservoir is 10^6 liters, with a hydraulic residence time of approximately 9 hours. The hydraulic residence time is estimated by assuming that the flow into and out of the reservoir is equal to the daily consumption of 0.7 million gallons, by the City of Paraty.

System of Disinfection by Chlorination

The disinfection of water by chlorination, at Pedra Branca and at the City's Reservoir, is performed in a crude, trial-and-error method. The City has no water meter at the reservoir to measure the flow into and out of the reservoir, which varies daily, and thus, no reliable method to determine the required chlorine dosage. In general, an administrator of chlorine adds approximately 200 grams of chlorine gas to the reservoir water each day, after adding an unknown amount of chlorine at the Pedra Branca intake (de Sigueira Baffo, 2003). The administrator adds as much as 400 grams of chlorine gas at the reservoir each day if no chlorine is added at Pedra Branca. The administrator does not measure chlorine demand in the reservoir water, but measures residual chlorine concentration in the City's tap water using a swimming pool kit, to adjust the subsequent day's chlorine dosage using this measurement. For example, if the residual chlorine concentration in the City's tap water were below the target concentration of 0.5 mg/l today, the administrator would increase the chlorine dosage tomorrow. The time lag of 1 day between the measurement and adjustment makes correct chlorine dosage difficult.

The residual chlorine in the City's tap water, measured by the administrator using a swimming pool kit, is approximately 2.5 mg/l on average. However, the residual chlorine concentration varies widely when it is measured with a more precise method. The residual chlorine measured with Hach standard methods, ranges from 0.0 mg/l to 1.5 mg/l. The recommended concentration of residual chlorine in drinking water is 0.5 mg/l for effective disinfection. The residual chlorine concentration in water is discussed further in Appendix B.

2.4. Problems with Potable Water Supply

The potable water supply system for the City of Paraty has a number of problems that must be addressed. The most important problems are: (i) shortage of water supply in the summer; (ii) ineffective disinfection; (iii) inadequate protection of water sources; and (iv) substandard water quality.

Supply Shortage

The City experiences water shortage during summer time, when the City's population increases dramatically with tourists. The problem with water shortage has been prevalent in the past, although the situation has improved in the recent years. Despite the abundant amount of source water, which increases in the summer with frequent rainstorms, the supply often does not meet increased demand. It is estimated that the City's population increases manifold in the summer, as much as 3 to 10 times according to some local people. In the past, summer water shortage was very frequent and some events lasted as long as three days (Lemos Padua, 2003). In the more recent years, since the construction of duplicate water supply pipelines, from 1997 to 2000, the water shortage has become less frequent, but has not been eliminated.

Water shortages impose much inconvenience and distress to anyone who experiences it. Therefore, water shortages, especially those that last long, have the capacity to generate enormous public discontent, and can affect the local people and tourists alike.

Ineffective Disinfection

The disinfection of the City's potable water is as unreliable as the method of chlorine addition is imprecise. Due to inaccurate chlorine dosage, the drinking water is distributed with variable amounts of residual chlorine. The residual chlorine in the City's tap water is sometimes undetectable, according to laboratory measurements.

Ineffective disinfection is problematic, mainly because tests of fecal coliform bacteria show that the City's water source is contaminated with fecal matter. Pathogenic fecal coliform bacteria, E-Coli, which occurs naturally in the intestines and feces of most warm-blooded animals, including humans, is a direct result of fecal contamination when found in water, and a clear indication of unsafe water, whereas other types of coliform that are not fecal contamination related, including those commonly found in soil, on the surface of leaves, and in decaying matter, are not necessarily so. Some common health effects of bacterial ingestion include abdominal cramps and diarrhea. E-Coli is transmitted through fecal-oral ingestion of the bacteria (i.e. drinking), primary contact recreation (i.e. swimming), or secondary contact (i.e. fishing). Hemorrhagic colitis (HC), is an acute disease caused by E-Coli, which results in severe abdominal cramps, watery diarrhea, and lower intestinal bleeding with occasional vomiting and fever (US Dept of Interior, "Total Coliform" 2001).

Inadequate Protection of Water Sources

The City's water sources, Pedra Branca and Caboclo, are not completely isolated from sources of fecal contamination, although they are located at high elevations, and do not have sewage discharged into them. Due to lack of physical barriers around the potable water intake structures, domestic animals, such as chickens and dogs, wander dangerously close to the source waters. In fact, it is highly likely that the fecal contamination of the City's water originates from domestic animals wading around the intake. Therefore, it is important to take measures to protect the City's water sources, by placing fences around the intake structures and the upstream waters, for example.

Potable Water Quality

The quality of the City's potable water is heavily influenced by the quality of surface waters, from which it is derived, and thus is highly variable. As surface waters often do, the City's potable water quality often falls substandard due to high turbidity after rainstorms, and bacterial contamination. The description and analysis of the City's potable water quality is described in detail in Appendix B.

The water quality analysis of the City's potable water not only asserts that the City's present method of disinfection is ineffective, but also that filtration of drinking water before disinfection is necessary in order to remove suspended particulate matter, and the harmful pathogens adsorbed on those particles, from water. The turbidity in drinking water that rises as high as 68

NTU makes filtration obligatory. Chlorination, a method of disinfection that kills organic contaminants in water through the oxidizing ability of chlorine, is ineffective against hard-shelled cysts like those produced by Cryptosporitium, although it can effectively treat biological pathogens like coliform bacteria and lelegionella. Filtration, a method of disinfection, physically removes biological contaminants present in water. The benefits of drinking water filtration are extensive and include: (i) removal of suspended particulate matter; (ii) disinfection by the removal of harmful pathogens adsorbed on those particles; and (iii) reduction of disinfection by-products by the removal of natural organic matter, which are their precursors.

Numerous water quality analyses reveal that many rural communities in the Municipality of Paraty, as well as the City of Paraty, consume drinking water that fails to comply with international drinking water regulations. Two principal causes of substandard water quality are high turbidity and bacterial contamination. The rural communities, which currently do not treat their drinking water, must disinfect their drinking water at the least, with chlorine addition for example.

The City of Paraty must adopt various measures to improve the quality of its drinking water. In addition to procuring a sufficient supply of drinking water to meet demand at all times, the City must better protect its drinking water at the sources, and treat the water by filtration and disinfection. The drinking water must be filtered in order to reduce the turbidity in water, which frequently rises to unacceptable levels after rainstorms, and a more precise method of chlorination must be adopted in order to make disinfection of drinking water more effective.

2.5. Existing Wastewater Disposal System

Reflecting the City's preference of the drinking water system to wastewater systems, Paraty has a very low percentage (12%) of connection to public sewer system, which lacks sewage treatment. As a consequence, large quantities of untreated sewage is discharged into two rivers, Pereque-Acu and Matheus-Nunez, that pass through the City; Jabaquara beach, a popular spot for swimming that is situated North of the City within walking-distance; and Paraty Bay. It is estimated that approximately 2,600 m^3 of wastewater is discharged into these water bodies on average, and as much as 7,900 m^3 of is discharged in the highly populated summer season.

Wastewater Infrastructure

The City has short networks of sewerage pipe connections, which are mainly used to transport sewage from individual households into the nearest receiving water body. The sewerage network is incomplete and run-down, and its exact structure and location is unknown, due to the misplacement of the plans containing such information.

The incomplete, and often broken, sewerage pipes lead to an additional problem of polluting the streets with wastewater in the high tides. As the City sits at a low altitude, near sea level, with a high water table, large parts of the Historical Center is flooded with seawater periodically during high tidal periods. During these times, wastewater leaks out of broken sewerage pipes and floods the streets mixed with seawater, before it can discharge into the Bay with reversing tides.

Storm water Infrastructure

While the City has some wastewater collection infrastructure, it has no storm water infrastructure. The streets in the City are lined with cobblestones, in shapes of a canal, in V or U-shapes. In congruence with this design, the storm water drains into the Bay naturally by gravity.

2.6. Problems with Wastewater Disposal

The two major problems associated with Paraty's current mode of wastewater disposal are: (i) environmental degradation resulting from direct discharge of sewage into surrounding water bodies, and from tidal inflows that flood the streets with sewage and seawater mixture; and (ii) health consequences resulting from exposure to such environment. The latter will be discussed in Section 2.7

Environmental Degradation

The pollution of surface water bodies, such as rivers and beaches, due to untreated sewage, result in increased health risks, loss of aesthetics and other amenities, and violation of their intrinsic values. For those water bodies intended for recreational use, the health risks are very high when they are polluted with fecal matter. Many environmental regulatory agencies limit the amount of fecal contamination allowed in recreational water bodies for this reason. For example, the maximum concentration of fecal coliform bacteria in beach waters, where people swim, is 200 colonies/100ml, and those waters exceeding this limit are required to prohibit these recreational activities. Therefore, the environmental degradation results in limited recreational activities and diminished commercial value of the water body.

The loss of aesthetics, due to the discoloration of water and the odor, which becomes more unpleasant in the summer, also contribute to the diminishment of water body's commercial value. The damage to aesthetics also reduces the amenities value and intrinsic value of the water body.

The environmental degradation not only occurs in the water bodies, due to direct discharge of wastewater, but also in the streets due to the tidal flows that flood the streets with sea water and sewage mixture. Similar costs apply to this mode of environmental degradation.

Quality of Surrounding Water Bodies

Surface water bodies near the City of Paraty are heavily polluted from human activities. In order to characterize the quality of these surface water bodies, samples were collected from numerous locations and tested. The water quality analysis of the surrounding surface water bodies in the City is described in detail in Appendix B.

All four surface water bodies, Jabaquara Beach, Matheus River, Pereque River, and a “Sewer Stream” in an open ditch in Ilha das Cobras, show fecal coliform concentrations that suggest contamination from sewer discharge. Among the four, Jabaquara Beach shows the least amount of contamination, most likely benefited by tidal dilution. Matheus River and Pereque River are approximately equally contaminated, and Sewer Stream shows characteristics of diluted raw sewage.

Jabaquara Beach, a popular recreational water body where people swim, that is within walking distance from the City of Paraty, is inadequate for primary recreation, which includes swimming. Jabaquara Beach water has a slightly low pH, adequate levels of turbidity and suspended solids, and high COD.

Neither Matheus Rivers nor Pereque River is adequate for secondary recreation, due to high levels of fecal contamination. Matheus River showed acceptable pH, but especially high COD level that is most likely due to oil spills from small boats anchored at the riverbank. Pereque River had pH that is in the lower end of the acceptable range, and low COD that is within acceptable range most of the time. The turbidity and suspended solids for both Rivers suggest that they are often, but not always, in the safe range for aquatic life.

From the water quality analysis above, it is evident that the City’s current mode of wastewater disposal degrades its surface waters, rendering Jabaquara Beach unsafe for swimming, and Matheus River and Pereque River unsafe for all aquatic sports. The uncontrolled disposal of wastewater damages the aesthetics of the rivers, and reduces the commercial value of the environment. The source of pollution must be controlled in order to preserve the environment from further degradation, and therefore an appropriate treatment and discharge of the City’s wastewater is critical. The collection and treatment of wastewater is expected to limit pollution of the surface waters, as well as the streets, in the City of Paraty.

2.7. Problems with Public Health - Diarrhea

A direct consequence of poor potable water quality and polluted environment is the negative impact on public health. Of the many different diseases and illnesses, diarrhea is the most widely studied public health problem that is associated with poor water and sanitation.

Incidence

A total of 443 diarrhea cases were recorded at local hospitals and health clinics in the Municipality of Paraty, from September 1, 2002 to December 28, 2002, according to an epidemiological study conducted by Wilsa Mary S. Barreto (Barreto, 2003) (See Table 2.1). Of these 443 cases, 228 cases (51%) were of those individuals living in the City of Paraty, 204 cases (46%) of individuals living in the rural areas, and 11 cases (2%) of individuals from outside. Among the 228 people from the City of Paraty, 60% were from Mangueira and Ilha das Cobras, the poorer parts of the City.

Area	Population	Number of diarrhea cases			Probability of diarrhea incidence per person
			In 4 months	In 1 year	
Urban	1,5000	51%	228	680	4.6%
Mangueira and Ilha das Cobras	*7,500*	*60%*	*137*	*410*	*5.5%*
Other	*7,500*	*40%*	*91*	*270*	*3.6%*
Rural	1,5000	46%	204	610	4.1%
Other		2%	11	30	
Municipality Total	**30,000**	**100%**	**443**	**1,300**	**4.4%**

Table 2.1. Number of diarrhea cases within Municipality of Paraty by location[1]

Approximately 111 diarrhea cases are treated in the health clinics each month, and approximately 1,330 cases are treated each year, if the incidence of diarrhea is assumed constant throughout the year. Furthermore, each person in the Municipality of Paraty has greater than 4%

[1] Number of diarrhea cases, which were registered at local hospital and health clinics between September 1, 2002 and December 28, 2002.

probability of suffering from diarrhea each year, if each person is assumed to suffer from diarrhea not more than once a year. The probability is greatest for the urban poor, those living in Mangueira and Ilha das Cobras, who have greater than 5% likelihood of suffering from diarrhea in a year.

More importantly, the number of diarrhea cases reported above does not account for all diarrhea cases in Paraty, but only those that received care at the local hospital and health clinics. The actual number of diarrhea cases is expected to be much higher, because many people treat their illnesses at home.

It is expected that the poor and the rural populations are less likely to visit health clinics, due to lack of time and money. Even though basic health services are provided free of charge in Paraty, the time required to go to health clinics can be costly. This cost of time is especially significant for the poor and those living in rural areas, farther away from the health clinics. Therefore, the numbers of diarrhea cases in the poorer areas (Mangueira and Ilha das Cobras), and the rural areas are likely to be much higher than the numbers reported.

The higher proportion of diarrhea cases in the City than in the rural areas suggests that: (i) the disinfection of City's drinking water is often ineffective; and (ii) adequate sanitation is as important as, if not more important than, clean drinking water supply. Although the common sense expects the number of diarrhea cases to be lower for the urban population, which drinks disinfected drinking water, than for the rural population, which does not, the study indicates that this is not so. In fact, the incidence of diarrhea for the urban population is higher at 4.6% than the 4.1% for the rural population. It is likely that the disinfection of City's drinking water with chlorine addition is ineffective and therefore does not benefit the urban population. The test of residual chlorine concentration in City's drinking water, which indicated zero residual chlorine concentration, reinforces this speculation.

It is also likely that environmental pollution, which is more serious in the City than the in rural areas, accounts for larger number of diarrhea cases in the urban population. The City, occupied by half of the Municipality's population, discharges large quantities of untreated sewage everyday, thereby severely polluting its waters. In contrast, the rural areas have smaller population density, and their sewage disposal is likely to be in better control. Therefore, the more polluted environment in the City could account for its higher diarrhea incidence, suggesting furthermore that adequate sanitation is as important as the supply of clean drinking water.

Morbidity

As much as 9% of diarrhea cases studied were serious, with two or more signs of serious dehydration, which can be life threatening without proper and timely treatment (See Table 2.2). Approximately 7% of diarrhea cases showed two or more signs of dehydration that were less serious, and 57% of the cases were mild with no sign of dehydration. The seriousness of these diarrhea cases was determined from the types of medical treatment (i.e. "Plans") received by the patients. The age distribution of the patients was not studied.

	Plan A	Plan B	Plan C	Plan Ign.	Sum
Total Number of Cases (by Plan Type) =	254	31	41	117	443
Percent of Cases (by Plan Type) =	57%	7%	9%	26%	100%
Plan A: No sign of dehydration					
Plan B: Two or more signs of dehydration					
Plan C: Two or more signs, including one which shows serious dehydration					

Table 2.2. Number of diarrhea cases within Municipality of Paraty by morbidity

Conclusion

Diarrhea, a widely studied indicator of water and sanitation-related diseases, is prevalent in both the urban and the rural areas of Paraty. According to this study of diarrhea incidence in Paraty, the most severely affected areas are Mangueira and Ilha das Cobras, the more densely populated, low-income areas within the City of Paraty.

It is assumed that a significant proportion of diarrhea cases is caused by waterborne pathogens, although it is difficult to estimate the exact proportion that is caused by the consumption of poorly disinfected drinking water, or by the contact with polluted surface waters (Payment and Hunter, 2001). For the City of Paraty, it is speculated that both the ineffectively disinfected drinking water, and the highly polluted surface waters are the causes of diarrhea and other water and sanitation related diseases.

2.8. Other Problems

In addition to the problems associated with potable water supply, wastewater disposal, and related health consequences, Paraty suffers from the following problems that are typical and common in many developing areas: (i) commercial and financial problems; and (ii) technical and operational problems (World Bank qtd. in US Dept. of Commerce, 1999).

Commercial and Financial

The commercial and financial problems observed in the City of Paraty are: (i) limited consumption metering; (ii) billing based on property value or lot size, regardless of the amount of water consumed; (iii) under-priced water; and (iv) commercial losses that reflect the high levels of unaccounted-for water.

The City of Paraty, which provides connections to public water supply to nearly 100 % of its population, has water meters connected to only 44% of those water connections (Prefeitura, "Laudo," 2002). In addition, these water meters, which were read in the past, are no longer read. The City claims that it lacks personnel to read the water meters, and that many water meters are broken or malfunctioning.

The City currently sets tariffs for water and sewage according to property size, since the consumption metering has been discontinued. On average, small houses in Mangueira or Ilha das Cobras, are billed approximately R$3 to R$5 per month, and larger houses in the Historical Center and Jabaquara are billed approximately R$7 per month. Commercial entities are billed much more; a bakery would be billed R$100 each month, for example. On the other hand, farms, which are often the largest users of water, are supplied with water free of charge (Reis, 2003).

The City's current tariff for domestic and agricultural water consumption is under-priced. For example, monthly billing of R$7 per month per household is much lower than R$0.73 per m^3 of water consumed, and R$0.87 per m^3 of sewage discharged, which are average volumetric tariff charged by CEDAE (US Dept. of Commerce, 1999). Assuming that a household consists of an average of 4 people, and that each person consumes 180 liters of water each day, each household consumes approximately 22 m^3 each month. Therefore, the City's current tariff of R$7 per month per household is equal to R$0.32 per m^3 of water consumed, much lower than the amount that is charged in most of the State.

The City's suffers from commercial loss (unaccounted-for water) due to poorly enforced billing. Currently, approximately 30% of the bills invoiced are not collected, and the uncollected bills amounts to approximately R$190,000 each year (Prefeitura, "Laudo," 2002) (See Table 2.3) The Municipality of Paraty is currently making efforts to increase the percentage of collected bills to 80% over the next 10 years, and to 85% in 5 additional years, by installing water meters and holding every household accountable for its consumption (Reis, 2003).

Year	Tot Collected (R$)	Tot Invoiced (R$)	% Collected	Annual Loss (R$)
2000	505,000	730,000	69	225,000
2001	540,000	750,000	72	210,000
2002	415,000	556,000	75	141,000
Average	**487,000**	**679,000**	**72**	**192,000**

Table 2.3. Tariffs for water and sanitation invoiced and collected by the City of Paraty

Technical and Operational

Inadequate preventive and regular maintenance of water and wastewater infrastructure is the main technical and operational problem that is observed in Paraty. The inadequate maintenance of water supply infrastructure is evident from the large quantities of water loss due to leakage from broken supply pipes. The inadequate maintenance of the few wastewater infrastructures that exist is also observed from the leakage of sewage in the streets.

2.9. Summary of Problems

The City of Paraty currently suffers from poor public health, polluted surface waters, and degraded aesthetics and commercial value of the environment, all of which are the consequences of poor water and sanitation systems. In addition, the City's goal of becoming a UNESCO World Heritage Site has been deferred due to the lack of functioning sanitation system in the Historical Center. In order to mitigate these problems, improve the quality of life, and foster economic growth in the City, the City's water and wastewater infrastructure must be improved.

Areas of improvement in the potable water supply are: (i) treatment of drinking water, (ii) protection of drinking water sources, and (iii) procurement of sufficient drinking water supply. It is evident that the City's potable water must be filtered and better disinfected in order to make it safe for drinking, and that the drinking water sources must be isolated in order to prevent accidental contamination of the source waters. Furthermore, to improve the quality of life for the local population, as well as the tourists, water shortages must be eliminated.

Areas of improvement in wastewater disposal are: (i) collection of wastewater collection, and (ii) treatment of wastewater. New wastewater infrastructure must be put in place to collect sewage, and a new wastewater treatment plant must be constructed in order to treat the wastewater before it can be safely discharged into the surrounding waters.

CHAPTER 3 - WATER AND SANITATION IMPROVEMENTS

This chapter recommends water and sanitation improvements that are necessary to mitigate Paraty's current water and sanitation-related problems, which were identified and described in detail in the preceding chapter. The population/area(s) to service, the type(s) of improvement, and the time(s) of development are considered. The costs of improvements are estimated and the City's capacity to recover these costs is analyzed by estimating new water and sewage tariff, and the people's willingness to pay.

3.1. Initial Considerations

Although the hope is to achieve universal coverage, providing adequate water and sanitation services to all, this cannot be achieved at once. Therefore, it is necessary to determine which community to service first, with which service, and when to develop these services, adhering to Paraty's objectives and priorities.

Population/Area(s) to Service

Wastewater collection infrastructure and treatment plant is to be constructed for the City of Paraty, excluding the Jabaquara area. Jabaquara is excluded from the City's development of wastewater collection infrastructure and treatment plant, due to geographic constraints. Because Jabaquara is located North of Pereque River, separated from the rest of the City by a hill and a narrow band of water, transporting its wastewater to the City's treatment plant, to be located in Ilha das Cobras, would be too costly (See Area 1 in Figure 3.1) Therefore, a separate wastewater system is recommended for Jabaquara.

On the other hand, Jabaquara is to be included in the City's drinking water supply system, to receive treated drinking water from the City's future drinking water treatment plant, which is to be located on a hill, next to the City's existing reservoir (See Area 2 in Figure 3.1) Although Jabaquara currently brings its drinking water directly from the Caboclo intake, rather than from the City's reservoir, a supply pipe could be constructed to connect Jabaquara to the future treatment plant. The water would flow downhill by gravity from the future treatment plant to Jabaquara, which has an elevation near sea level.

Development Priorities

Due to high capital costs involved with water and sanitation developments, it is often economical to divide the development projects into a number of stages, and undertake one project, or one section of a project, at a time. A project of the highest priority would be developed in stage 1, followed by projects of lower priority (i.e. those projects, the time of completion of which are of less consequence).

Water Supply vs. Sanitation

In the City of Paraty, the need of wastewater collection and treatment is considered more serious and imminent than the need for better drinking water treatment, for the following two reasons: (i) a functioning wastewater collection and treatment system at the Historical Center is necessary in the near future for the qualification of UNESCO World Heritage Site; and (ii) while there is a drinking water alternative, the bottled water, there is no alternative for wastewater collection and treatment. Therefore, in a situation where the undertaking of both water and wastewater projects is not economically feasible, the City is to commence its wastewater project first.

Wastewater Collection Infrastructure vs. Wastewater Treatment Plant

The construction of wastewater collection infrastructure and the wastewater treatment plant is to be undertaken concurrently, since one is useless without the other.

3.2. Recommendations

The previous chapter identified the following improvements, which are essential in the City of Paraty: (i) a wastewater infrastructure and a treatment plant for the collection and treatment of wastewater; and (ii) a drinking water treatment plant with filtration and disinfection, for better treatment of drinking water.

Wastewater Collection System

A gravity sewer system is recommended for the collection of wastewater (Choi, 2003). In a gravity sewer system, wastewater is transported by gravity flow to treatment facilities. The gravity flow is maintained by the slopes of the sewer pipes, which are designed to maintain the minimum "self-cleansing" velocity of approximately 0.6 m/s. Due to the slopes required and the depth of the sewer pipes, gravity sewers often require lift station pumps to transport wastewater from low to high points, so that the flow can proceed by gravity again. Gravity sewer systems generally require less maintenance than other sewer collection systems, such as a low-pressure force main system. In addition, a gravity system can handle large variations in flow, and is readily adaptive for growth and change within the sewer district (Pleasanton, 2001).

Wastewater Treatment Plant

A chemically enhanced primary treatment (CEPT) plant is recommended for the treatment of the City's wastewater (Kfouri and Kweon, 2003). CEPT is the process by which chemical coagulants are added to primary sedimentation basins in order to enhance the treatment efficiency (i.e. removal of solids, organic matter, and nutrients from the wastewater). CEPT costs minimally more than primary treatment, and half as much as secondary treatment, but its efficiency is highly competitive with biological secondary treatment. "CEPT is ideal for a coastal city since the removal of total suspended solids is very high, and the decrease in biochemical oxygen demand is sufficient so as not to impact oxygen concentrations in the ocean" (Chagnon, 2002).

The CEPT plant is to be located in an empty lot in Ilha das Cobras (See Area 1 in Figure 3.1)

Figure 3.1. Possible location of wastewater treatment plant and drinking water treatment plant[1]

Drinking Water Treatment Plant

Different alternatives of filtration and disinfection are to be considered by the City of Paraty, for the treatment of the City's drinking water. Some of the treatment options include conventional filtration, direct filtration, slow sand filtration, and diatomaceous earth (DE) filtration. The descriptions of each follow:

Conventional Filtration

The conventional filtration consists of rapid mix coagulation, flocculation, sedimentation, and gravity filtration. Common filter media include sand, dual-media and tri-media. Conventional filtration is the most widely used technology for treating surface water supplies for turbidity and microbial contaminants, and has the advantage that it can treat a wide range of water qualities. However, it has the disadvantage that it requires advanced operator skill and has high monitoring requirements (US EPA, "Small System," 1997).

[1] (1) = Location of wastewater treatment plant; (2) = Location of drinking water treatment plant.

Direct Filtration

Direct filtration is conventional filtration minus the sedimentation step. In-line filtration is the simplest form of direct filtration and consists of filters preceded by direct influent chemical feed and static mixing. In general, direct filtration requires low turbidity raw water and is attractive because of its low cost relative to conventional treatment. However, similar to conventional filtration, direct filtration requires advanced operator skill and has high monitoring requirements. The performance of direct filtration is extremely sensitive to the proper management of the coagulation chemistry, and if the coagulation step is disrupted or improperly executed, the removal efficiencies for turbidity and microbial contaminants decrease dramatically in a matter of minutes (US EPA, "Small System," 1997).

Slow Sand Filtration

Slow sand filtration employs a sand filter with a large cross-sectional area, which results in a low filtration rate. Slow sand filtration also employs a biological slime layer, called the "schumutzdecke," which develops over time on top of the sand. The schumutzdecke assists in the removal of suspended organic materials and microorganisms, by biodegradation and other biological processes, instead of relying solely on simple filtration or physico-chemical sorption. An advantage of slow sand filtration is that no backwashing is necessary for slow sand filters. When a predetermined duration, headloss or effluent turbidity is reached, the top few centimeters of the sand are scraped off. Other advantages of slow sand filtration include its low maintenance requirements (since it does not require backwashing and requires less frequent cleaning) and the fact that its efficiency does not depend on actions of the operator. A disadvantage of slow sand filtration is that large systems have large land requirements. Slow sand filters are simple, and easily used by small systems (US EPA, "Small System," 1997).

Diatomaceous Earth (DE) Filtration

Diatomaceous earth (DE) filtration involves a filter cake build-up on a fabric filter element or septum. The DE is a powdery, siliceous material that, on a particle level, is porous, multi-shaped, angular, and varies in width between 5 and 60 microns. The DE filter cake is subject to cracking and must be supplemented by a continuous body feed of diatomite to maintain porosity of the filter. Problems inherent in maintaining the filter cake have limited the use of DE filtration. The advantage of DE is that it does not require coagulants. A disadvantage is that advanced operator skill is required for filtration efficiency (US EPA, “Small System,” 1997).

Summary

Land area permitting, the slow sand filtration would be the optimal system for the City of Paraty, since it is cost-effective and does not require advanced operator skills. However, the City should compare the different alternatives of filtration, described above, and select a system that best satisfies the City’s needs. In the cost analysis, which is to follow, the conservative costs of a conventional filtration plant are used.

The most convenient location for the drinking water treatment plant is next to the City’s reservoir, since this is where the waters from two sources, Pedra Branca and Caboclo, are combined, disinfected, and distributed to the City (See Area 2 in Figure 3.1)

Development Sequence

The wastewater collection infrastructure and treatment plant are to be constructed concurrently in three stages for the City of Paraty, excluding the Jabaquara area. The Historical Center is to be developed in the first stage; Mangueira and Ilha das Cobras in the second stage; and the Old City and rest of the City in the third stage. Each development stage is to last approximately 2 years. The incremental development of the CEPT plant is made possible by its ease of implementation and expansion.

The drinking water treatment plant, with the capacity for the entire City of Paraty including Jabaquara, is to be constructed in one stage, since its expansion is likely to be more difficult. The drinking water treatment plant will be constructed after the completion of the wastewater collection infrastructure and treatment plant. However, since there is an immediate need for a more precise method of chlorination, the drinking water disinfection system is to be upgraded immediately, with a flow meter and an automated chlorinator, for example. In addition, the drinking water intake points are to be fenced around the perimeter, in order to protect the integrity of the drinking water.

The earliest feasible time for the construction of drinking water treatment plant is to be determined by comparing the costs of constructing the drinking water treatment plant at different years after the completion of the wastewater infrastructure developments. Four scenarios of development sequence are considered, as shown in Table 3.1:

<table>
<tr><th></th><th>Year0</th><th>Year1</th><th>Year2</th><th>Year3</th><th>Year4</th><th>Year5</th><th>Year6</th><th>Year7</th><th>Year8</th><th>Year9</th><th>Year10</th></tr>
<tr><td>Scenario 1</td><td>WW 1</td><td>WW 2</td><td>WW 3</td><td>DW</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Scenario 2</td><td colspan="2">WW 1</td><td colspan="2">WW 2</td><td colspan="2">WW 3</td><td colspan="2">DW</td><td></td><td></td><td></td></tr>
<tr><td>Scenario 3</td><td colspan="2">WW 1</td><td colspan="2">WW 2</td><td colspan="2">WW 3</td><td></td><td colspan="2">DW</td><td></td><td></td></tr>
<tr><td>Scenario 4</td><td colspan="2">WW 1</td><td colspan="2">WW 2</td><td colspan="2">WW 3</td><td></td><td></td><td colspan="2">DW</td><td></td></tr>
</table>

Table 3.1. Four scenarios of development sequence of wastewater and drinking water infrastructure[1]

Scenario 1 assumes an accelerated project, in which all development is completed in a four-year period, each development stage lasting one year. Scenarios 2, 3 and 4 estimate that each development stage lasts two years. Scenario 2 assumes that all developments will be completed in 8 years, during which time the completion of each development stage is immediately followed by the development of the subsequent stage. Scenario 3 assumes one year of no development between the completion of the development of wastewater infrastructure and the development of drinking water treatment plant, and Scenario 4 assumes two years of no development.

3.3. Design Parameters

Two important parameters in the design of the wastewater collection infrastructure and treatment plant, and the drinking water treatment plant are the population in the City of Paraty, and an average consumption of water per capita. The flow demand for the wastewater infrastructure and treatment plant, and the drinking water treatment plant are estimated from these two parameters:

Daily flow = (Daily water consumption per capita) x (Population)

[1] WW1 = development stage 1 of wastewater infrastructure and treatment plant; WW2 = development stage 2; WW3 = development stage 3; DW = development of drinking water treatment plant.

Population

The population in the City of Paraty is assumed to increase in the summer. The rough estimates of the average annual population, and summertime population are listed in Table 3.2 below:

Area	Average	Summertime Increase	Peak (Summer)
Jabaquara (excluded from WW design)	1,500	3x	4,500
Historical Center	3,000	3x	9,000
Mangueira	4,500	1x	4,500
Ilha das Cobras	3,000	1x	3,000
Old City	3,000	3x	9,000
Total Urban Population	**15,000**		**30,000**

Table 3.2. Average annual population and the peak summertime population for the City of Paraty

As indicated in the table above, most areas in the City are expected to experience a 3-fold increase in population during summer. However, the population in Mangueira and Ilha das Cobras is expected to remain constant since these areas are primarily residential areas for the local people. The annual population growth rate is approximately 0.8%, estimated from the average growth rate in the State of Rio de Janeiro (CEPIS, 2002).

Consumption

The design flow for the wastewater and the drinking water systems are estimated from the daily potable water consumption of 180 liters per capita (Prefeitura, "Laudo," 2002). The amount of wastewater produced is assumed to be approximately equal to the potable water consumption. The flow demand for different stages of development for the wastewater collection infrastructure and treatment plant and for the one-stage development of the drinking water treatment plant are estimated in Table 3.3 below:

Development Stage	Development Area	Design Flow (m^3/day)
WW 1	Historical Center	1,620
WW 2	Mangueira and Ilha das Cobras	1,350
WW 3	Old City	1,620
DW	City of Paraty including Jabaquara	5,400

Table 3.3. Summertime average daily flow for water and wastewater treatment design for the City of Paraty

3.4. Cost Analysis

Project Cost

The capital cost and operation and maintenance (O&M) costs of the wastewater infrastructure and treatment plant and the drinking water treatment plant are derived from a number sources. All costs are assumed to be linear with flow capacity, and a conversion rate of US$1.00 = R$1.00 is used to convert US costs to Brazilian costs. The exchange rate of US Dollar to Brazilian Real is approximately US$1.00 = R$3.11 (X-rates.com, 2003). However, the cost of equipments and labor in Brazil is assumed to be approximately 1/3 of the cost in the US (Tsukamoto, 2003). Therefore, the true value of US$1.00 is approximately equal to the value of R$1.00.

Wastewater Collection and Treatment

The capital cost of wastewater collection infrastructure includes: piping, pump stations, manholes, and associated construction costs. The capital and O&M costs of wastewater infrastructure are estimated from US costs (Choi, 2003).

The capital cost of wastewater treatment includes: CEPT tanks, chlorination and dechlorination chambers, sludge dewatering units and drying beds, and associated construction costs. The O&M cost includes: chemical costs for CEPT and disinfection, as well as sludge treatment and disposal costs. The costs of CEPT and sludge treatment and disposal are Brazilian costs adapted from Tatui-CEAGESP Wastewater Treatment Facility, Brazil (Cabral et al., 1999). The disinfection cost of the wastewater effluent, including chlorination and dechlorination, is US cost adapted from the US EPA (US EPA, qtd. in Kfouri and Kweon, 2003). The capital cost and O&M cost of wastewater collection infrastructure and treatment plant are summarized in Table 3.4below:

Total Cost for Wastewater Collection Infrastructure and Treatment Plant		
WW Infrastructure CC	2,720	R$1000
WW Treatment CC	1,292	R$1000
Total WW Capital Cost	**4,011**	**R$1000**
WW Infrastructure O&M Cost	436	R$1000/yr
WW Treatment O&M Cost	35	R$1000/yr
Total WW Annual O&M Cost	**472**	**R$1000/yr**

Table 3.4. Total capital cost and O&M cost for wastewater collection infrastructure and treatment plant

Drinking Water Treatment

The capital and O&M costs of a conventional drinking water treatment plant, consisting of rapid mixing, flocculation, sedimentation, chlorination, filtration, contact basin, chemical feed systems, and finished water storage, are adapted from typical US costs estimated by US EPA (US EPA, 1999). The cost for a new finished water storage tank is included since the City's existing reservoir, constructed in 1975, is rundown and approaching the end of its lifetime. The following costs are neglected due to lack of information: (i) current O&M cost for chlorination; (ii) capital cost and O&M cost for interim upgrade of drinking water disinfection system; (iii) all costs associated with drinking water infrastructure.

The capital cost and O&M cost of a new drinking water treatment plant with conventional filtration and chlorination are summarized in Table 3.5 below:

Total Cost for Drinking Water Treatment Plant		
DW Treatment CC	1,057	R$1000
Total DW Capital Cost	**1,057**	**R$1000**
DW Treatment O&M Cost	395	R$1000/yr
Total DW Annual O&M Cost	**395**	**R$1000/yr**

Table 3.5. Total capital cost and O&M cost for drinking water treatment plant

Financial Analysis

The above costs are incorporated into four scenarios of development sequence, shown in Table 3.1, and evaluated assuming a project life of 30 years and annual interest rates of 5% and 10%. Equivalent uniform annual cost (EUAC), defined as the amount of money which, paid in equal annual installments over the life of a project, would pay for the project, is referred as average annual cost in this analysis. Average annual cost and benefit/cost ratio of the projects are computed and used to determine the minimum water and sewage tariff required to fully recover costs, as well as the earliest feasible time for the construction of drinking water treatment plant.

Break-Even Tariff for Water and Sewage

In the following analysis, the break-even tariff for water and sewage, which reflects the minimum amount of revenue required to fully recover the costs, is estimated by setting the City's annual revenue to equal the average annual cost (i.e. by setting the benefit/cost ratio equal to 1). An important consideration in this computation is that the break-even tariffs are computed accounting for the fact that the City collects only 70% of its invoiced tariffs (See Section 2.8). The break-even tariffs for water and sewage, for the four scenarios of development sequence listed in Table 3.1, are summarized in Table 3.6 below:

Development Sequence Scenario	Annual Cost (R$1000)		Annual Revenue (R$1000)		Water and Sewage Tariff (R$/m^3)	
	I = 5%	I = 10%	I = 5%	I = 10%	I = 5%	I = 10%
1	1,086	1,226	1,086	1,226	1.57	1.78
2	976	1,058	976	1,058	1.42	1.53
3	955	1,030	955	1,030	1.38	1.49
4	934	1,004	934	1,004	1.35	1.46

Table 3.6. Equivalent uniform annual cost and break-even tariff for water and sewage

According to this financial analysis, the annual cost is greatest for Scenario 1, in which all developments, including wastewater infrastructure and treatment plant and drinking water treatment plant, are completed within a period of 4 years. Under Scenario 1, an average water and sewage tariff, required to fully recover the project costs, is R$1.57/m^3 at 5% annual interest rate, and R$1.78/m^3 at 10% annual interest rate. The annual cost decreases with extended duration of water and wastewater developments, and the minimum water and sewage tariff decreases correspondingly.

Economic Feasibility of Projects when Water and Sewage Tariff = R$1.60/m^3

The economic feasibility of the projects is also analyzed for the case that uses average water and sewage tariffs previously determined by CEDAE. CEDAE charges an average tariff of R$0.73/m^3 for drinking water, and R$0.87/m^3 for sewage (US Dept. of Commerce, 1999). The combined tariff is R$1.60/m^3. The average annual revenue is estimated from the sum of water and sewage tariffs collected each year, which is approximately 70% of the invoiced tariffs. The benefit/cost ratio, an important indicator of the economic feasibility of the projects, is estimated by dividing revenues by costs. The average annual revenue and the benefit/cost ratios are listed in Table 3.7 below:

Development Sequence Scenario	Annual Cost (R$1000)		Annual Revenue (R$1000)		Benefit/Cost Ratio	
	I = 5%	I = 10%	I = 5%	I = 10%	I = 5%	I = 10%
1	1,086	1,226	1,209	1,185	1.1	1.0
2	976	1,058	1,209	1,185	1.2	1.1
3	955	1,030	1,209	1,185	1.3	1.2
4	934	1,004	1,209	1,185	1.3	1.2

Table 3.7. Benefit/cost ratio for water and sewage tariff = R$ 1.60/m^3

According to this analysis, the water and sewage tariff of R$1.60/m^3 produces an average annual revenue of R$1.2 million at annual interest rates of 5% and 10%, and the benefit/cost ratios that range from 1.0 to 1.3. Therefore, all four scenarios of development sequence are economically feasible, at either interest rates, when the tariff for water and sewage is equal to R$1.60/m^3.

Summary

The minimum water and sewage tariff required for full recovery of costs, which include the costs of operation, maintenance, and administration as well as current debt service obligations, is approximately R$1.80/m^3 when the annual interest rate is 10%. This tariff is approximately equivalent to R$38/household-month for a 4-person household, and about 5 to 10 times the City’s current tariff for residential use. At the same time, it is about 1/10 of the City’s current tariff for commercial use.

3.5. Willingness to Pay Analysis

Although the study of willingness to pay (WTP) for improvements in water and sanitation was not performed in the City of Paraty, due to limited time and resources, it can be estimated based on a number of economic indicators.

Assumptions

The basic underlying assumption in this study is that the WTP is approximately equal to the sum of the existing water and sewage tariff paid, the cost of bottled drinking water purchased, and the minimum wage lost due to water and sanitation-related illnesses:

WTP = existing tariff + cost of bottled drinking water + minimum wage lost to illness

Distribution of Income

Since the WTP is closely related to household income, it is estimated separately for the low-income households in Mangueira and Ilha das Cobras, and for the mid- to high-income households in Historical Center, and Old City. The WTP in the low-income areas is expected to be lower than that in the high-income areas. The WTP in Jabaquara, which is a relatively high-income community, is estimated separately, since its sanitation system will not be connected with the City's public sewer system.

Mangueira and Ilha das Cobras

The average current tariff for water and sewage in Mangueira and Ilha das Cobras is approximately R$3/household-month.

It is assumed that half of the Mangueira and Ilha das Cobras population buys bottled water for drinking. Or, it is assumed that the entire Mangueira and Ilha das Cobras population buys bottled water for approximately half of the month, on average. Additionally, it is assumed that each person drinks 2 liters of water each day. Therefore, in Mangueira and Ilha das Cobras, a 4-person household, which consumes 240 liters of water each month for drinking, buys 120 liters of the bottled water each month. Since a 20-liter bottle of water purchased and delivered to individual households costs R$3 in Paraty, the cost of bottled water is approximately R$18/household-month.

Due to a comparatively high diarrhea incidence in Mangueira and Ilha das Cobras, it is assumed that an income-earning member in each household loses a day of work each month due to a water-related illness of his/her own or that of his/her child. Assuming that the monthly minimum wage in Mangueira and Ilha das Cobras is approximately equal to the monthly minimum wage of R$240 in Brazil, the cost of minimum wage lost to water and sanitation-related illness is approximately R$8/household-month.

WTP (Mangueira and Ilha das Cobras) = 3 + 18 + 8 = R$29/household-month

The WTP, for the Mangueira and Ilha das Cobras population, is approximately R$29/household-month.

Historical Center and Old City

Since the tariff for water and sewage in the City is currently determined from property value (i.e. lot size), and the houses in Historical Center, and Old City are generally larger, the average monthly tariff is higher for the households in these areas. The average monthly tariff for water and sewage in Historical Center, and Old City is approximately R$7/household.

It is assumed that mid- to high-income households drink only bottled water. Therefore, each household in Historical Center and Old City purchases approximately 240 liters of bottled water, and the cost of bottled water is approximately $R36/household-month.

It is assumed that the minimum wage in Historical Center, and Old City is generally higher than the minimum wage in Mangueira and Ilha das Cobras. However, it is also assumed that the population in these areas are less afflicted by water and sanitation-related illnesses. These two assumptions considered, it is estimated that the loss of wage due to water and sanitation-illnesses in these areas is also approximately R$8/household-month.

WTP (Historical Center, and Old City) = 7 + 36 + 8 = R$51/household-month

The WTP, for the Historical Center and Old City population, is approximately R$51/household-month.

Jabaquara

Since Jabaquara is a relatively high-income community, with tourism as its major industry, the WTP of its population is expected to be similar to that of the Historical Center and Old City population. However, the WTP of the Jabaquara population is assumed to be approximately half of that for the Historical Center and Old City population, since it will be provided with only half of the service, which is the supply of treated drinking water.

WTP (Jabaquara) = WTP (Historical Center, and Old City)/2 = R$26/household-month

The WTP, for the Jabaquara population, is approximately R$26/household-month.

Willingness to Pay

The WTP is approximately R$29/household-month for the low-income population in Mangueira and Ilha das Cobras, $51/household-month for the mid- to high-income population in Historical Center and Old City, and R$26/household-month for the Jabaquara population, who will receive only the treated drinking water.

The WTP varies widely between the low-income population and the mid- to high-income population, and the difference is approximately R$22/household-month, almost 80% of the WTP of the low-income population. The WTP of the low-income population is approximately R$9/household-month lower than the break-even water and sewage tariff, and the WTP of the mid- to high-income population is approximately R$13/household-month higher.

3.6. Water and Sewage Tariff

The water and sewage tariff must be designed to reflect the people's WTP, which varies with income distribution, because the WTP of the low-income population is below the minimum water and sewage tariff required for full cost recovery. Examples of income-based tariffs include "lifeline" tariffs, and lump-sum credits provided to qualifying low-income households. Lifeline tariffs, which are reduced tariffs applicable to low-income consumers, provide the low-income consumers with a predetermined amount of service to meet a minimum quality of life.

Lifeline tariffs or other income transfers to low-income households are motivated and justified by a goal to achieve "fairness," even though they are in conflict with "equity." Tariffs are fair when they are perceived to be just and equitable by consumers and the general public. Many members of the public believe that it is fair to charge lower prices to low-income households, even though equity precludes non-cost-related differences in tariff as well as any other arbitrary distinctions among users (Boland, 1992).

In this study, a separate tariff for water and sewage is designed for each income group based on the study of WTP. For example, water and sewage tariff of R$1.40/m^3, corresponding to R$29 /household-month, is charged for the low-income households in Mangueira and Ilha das Cobras; R$2.40/m^3, corresponding to R$51/household-month, is charged for the mid- to high-income households in Historical Center and Old City; and R$1.20/m^3, corresponding to R$26/household-month, is charged to mid- to high-income households in Jabaquara. This design of water and sewage tariff is feasible as shown in Table 3.8:

Area	Population	Adjusted Water and Sewage Tariff				Annual Revenue		Target Annual Revenue	
Mangueira, and Ilha das Cobras	7,500	29	R$/hh-mo	1.40	R$/m^3	475	R$1000		
Historical Center, and Old City	6,000	51	R$/hh-mo	2.40	R$/m^3	626	R$1000		
Jabaquara	1,500	26	R$/hh-mo	1.20	R$/m^3	78	R$1000		
Total						**1,228**	**R$1000**	**1,226**	**R$1000**

Table 3.8. Water and sewage tariff adjusted according to income distribution[1]

The above tariffs are substantially higher than the existing tariffs of approximately R$3/household-month in Mangueira and Ilha das Cobras, and R$7/household-month in other parts of the City. Sudden increase in water and sewage tariffs of this magnitude is likely to "shock" the users, and thus appropriate interim tariffs must be designed for one or more steps to phase in the final design tariff.

3.7. Benefits

The benefits associated with water and sanitation improvements are numerous and substantial, although it is difficult to associate these benefits with monetary values for cost-benefit analysis. Some of the benefits include:

(i) Disease reduction and improved human productivity;

(ii) Healthier environment, improved aesthetics, and associated increase in amenities, economic values, and intrinsic values of the environment;

(iii) Encouraged tourism, poverty alleviation, and general economic growth; and

(iv) UNESCO World Heritage Site candidacy, and associated distinction and merit.

3.8. Summary

Following improvements are proposed for the mitigation of the City's current water and sanitation-related problems:

(i) Gravity sewer system for the collection of wastewater;

[1] hh=household; mo=month

(ii) Chemically enhanced primary treatment (CEPT) plant for the treatment of wastewater; and

(iii) Drinking water treatment plant for a better treatment of potable water.

The wastewater collection infrastructure and treatment plant are to be constructed concurrently in three stages for the City of Paraty, excluding the Jabaquara area. The Historical Center is to be developed in the first stage; Mangueira and Ilha das Cobras in the second stage; and the Old City and rest of the City in the third stage. Each development stage is to last approximately 2 years, and the completion of each stage is to initiate an immediate start of the subsequent stage.

The drinking water disinfection system is to be upgraded immediately, with a flow meter and an automated chlorinator, and the drinking water intake points are to be fenced around the perimeter, in order to protect the source waters. The drinking water treatment plant, with the capacity for the entire City of Paraty including Jabaquara, is to be constructed in one stage, immediately following the third stage of wastewater infrastructure development.

The total capital costs and O&M costs associated with the above improvements are as follows:

Total Capital Costs and O&M Costs for Water and Sanitation Improvement Projects	
Total WW Collection Infrastructure and Treatment Plant CC	R$ 4 million
Total WW Collection and Treatment Annual O&M Cost	R$ 0.5 million/yr
Total DW Treatment Plant CC	R$ 1 million
Total DW Treatment Annual O&M Cost	R$ 0.4 million/yr

Table 3.9. Total capital cost and O&M cost for water and sanitation improvement projects

The total annual cost is approximately R$1.2 million, with the capital cost amortized over a 30-year project life at 10% annual interest rate. The minimum water and sewage tariff required for full recovery of this annual cost is approximately R$1.80/$m^3$ or R$38/household-month.

The willingness to pay (WTP) varies between different areas of the City according to household income. WTP is approximately R$29/household-month for the low-income population in Mangueira and Ilha das Cobras, $51/household-month for the mid- to high-income population in Historical Center and Old City, and R$26/household-month for the Jabaquara population, who will receive only the treated drinking water.

Designing a separate water and sewage tariff for each income group, based on the study of WTP, water and sewage tariff is R$1.40/m^3 for Mangueira and Ilha das Cobras population, R$2.40/m^3 for Historical Center and Old City population, and R$1.20/m^3 for Jabaquara population. Since these tariffs can be seen as a substantial increase from the existing tariffs, appropriate interim tariffs are to be designed and implemented in one or more steps to phase in the final design tariff.

Finally, the construction of wastewater collection infrastructure and treatment plant, and drinking water treatment plant is expected to bring substantial benefits in public health, environmental quality, and aesthetics in the city, and hence provide large economic gains.

CHAPTER 4 - PROPOSED POLICY

The City of Paraty currently suffers from inadequate water and sanitation systems, the consequences of which include: poor public health; polluted surface waters; damaged aesthetics; loss of amenities; and depreciated commercial and intrinsic value of the environment. In addition, the City's objective of becoming a UNESCO World Heritage Site has been deferred due to the lack of functioning sanitation system in the Historical Center.

Problems

The potable water supply system for the City of Paraty has a number of problems that must be addressed, including: (i) shortage of water supply in the summer; (ii) ineffective disinfection; (iii) inadequate protection of water sources; and (iv) substandard water quality.

Numerous water quality analyses revealed that the quality of City's potable water is heavily influenced by the quality of surface waters, from which it is derived, and often fails to comply with international drinking water standards due to high turbidity after rainstorms, and bacterial contamination. These analyses also indicated that the City's present method of disinfection is ineffective, and that filtration of drinking water before disinfection is necessary in order to remove suspended particulate matter, and the harmful pathogens adsorbed on those particles, from water.

Due to the lack of wastewater collection and treatment, the City of Paraty suffers from serious environmental degradation and associated health consequences. The environmental degradation in the City results from the direct discharge of untreated sewage into surrounding water bodies, and from the tidal inflows that flood the streets with sewage and seawater mixture.

Four surface water bodies, Jabaquara Beach, Matheus River, Pereque River, and an open ditch of sewer stream, were tested for water quality. According to the water quality analyses, Jabaquara Beach was found to be unsafe for swimming, and Matheus River and Pereque River unsafe for all aquatic sports, due to high fecal contamination. In addition, Sewer stream was found to have the water quality of a diluted sewage.

The uncontrolled disposal of wastewater damages the aesthetics of the rivers, and reduces the commercial value of the environment. The source of pollution must be controlled in order to preserve the environment from further degradation, and therefore an appropriate treatment and discharge of the City's wastewater is critical.

Poor public health is a direct consequence of inadequate potable water quality and polluted environment. Diarrhea, a widely studied indicator of water and sanitation-related diseases, was found to be prevalent in both the urban and the rural areas of Paraty, especially in Mangueira and Ilha das Cobras, the more densely populated, low-income areas within the City of Paraty.

It is assumed that a significant proportion of diarrhea cases is caused by waterborne pathogens, although it is difficult to estimate the exact proportion that is caused by the consumption of poorly disinfected drinking water, or by the contact with polluted surface waters. For the City of Paraty, it is speculated that both the ineffectively disinfected drinking water, and the highly polluted surface waters are the causes of diarrhea and other water and sanitation related diseases.

Improvements

Following improvements are proposed for the mitigation of the City's current water and sanitation-related problems identified above:

(i) Gravity sewer system for the collection of wastewater;

(ii) Chemically enhanced primary treatment (CEPT) plant for the treatment of wastewater; and

(iii) Drinking water treatment plant for a better treatment of potable water.

The wastewater collection infrastructure and treatment plant are to be constructed concurrently in three stages for the City of Paraty, excluding the Jabaquara area. The Historical Center is to be developed in the first stage; Mangueira and Ilha das Cobras in the second stage; and the Old City and rest of the City in the third stage. Each development stage is to last approximately 2 years, and the completion of each stage is to initiate an immediate start of the subsequent stage.

The drinking water disinfection system is to be upgraded immediately, with a flow meter and an automated chlorinator, and the drinking water intake points are to be fenced around the perimeter, in order to protect the source waters. The drinking water treatment plant, with the capacity for the entire City of Paraty including Jabaquara, is to be constructed in one stage, immediately following the third stage of wastewater infrastructure development.

In order to fully recover costs of water and sanitation improvements, annual revenue of R$1.2 million must be collected from water and sewage tariffs. The following water and sewage tariffs, which are based on willingness to pay (WTP), are to be billed for each income group: R$1.40/$m^3$ for Mangueira and Ilha das Cobras population; R$2.40/$m^3$ for Historical Center and Old City population; and R$1.20/$m^3$ for Jabaquara population. Since these tariffs can be seen as a substantial increase from the existing tariffs, appropriate interim tariffs are to be designed and implemented in one or more steps to phase in the final design tariff.

Finally, the construction of wastewater collection infrastructure and treatment plant, and drinking water treatment plant is expected to bring numerous and substantial benefits to the City, which include: improvements in public health, environmental quality, and aesthetics in the city, as well as increases in productivity and economic value of the environment. It is also expected that these water and sanitation improvements will encourage tourism and promote general economic growth, providing large economic returns.

CHAPTER 5 – DESIGN OF WASTEWATER COLLECTION SYSTEM

This chapter provides a conceptual design of a wastewater collection system for the first development stage of wastewater infrastructure (i.e. development of the Historical Center area). In the first part of this chapter, a methodology of creating a design, which includes project understanding, comparison of different wastewater collection system alternatives based on a number of selection criteria, and selection of an optimal collection system is described. In the second part of this chapter, the design of a collection system, which includes, wastewater flow, overall layout, cost, and wastewater treatment plant location, is described. In the last part of the chapter, some recommendations for the wastewater collection system are stated. The robustness of the design of the wastewater collection system is analyzed in Appendix E.

5.1. Project Understanding

Investigation of the area in need of sewerage is important for design and construction. Paraty's sewer design is based on a field visit of the proposed sewer area, a review of the city's mapping, and a preliminary analysis of the different alternatives of collection.

Field Visit

The land of the Historical Center, and the rest of the City, is generally flat, at an elevation that seems to be no more than a half-meter above sea level. Consequently, tides flood the streets of the Historical Center, in the area closer to the water. The streets of the Historical Center, the width of which range between 4 and 7 meters, are in poor condition. The streets are lined with uneven cobblestones, which were placed improperly during road excavations in the past and consequently became severely weatherworn. Due to the high water table, the buildings do not have basements. The buildings are a mix of one- and two-story buildings.

The existing underground structures include a water distribution system, a telephone wire system and an old, incomplete and nonfunctional sewer collection system. The existing sewer is a gravity sewer which was implemented 20 years ago in the Historical Center. It was built with a line of short pipes of concrete (1 meter in length) and has its lowest point close to the sea at depths of around 3.5 meters (Rocha, 2003). Because of the material used to build the sewer, the infiltration is too high for use as a sewer.

Paraty's Mapping

The city lacks appropriate and accurate maps of the existing wastewater infrastructure. Therefore, most of the surveying was performed by interviews with people and by observation. An aerial photo was obtained (Klink, 2003) and the map was then digitized and georeferenced using ArcView GIS. The coordinates were based in Universal Trans Mercator (UTM) projection. The scale of the map is 1:2000 (See Figure 5.2). This map is provided as a means to plan a collection system with accurate spatial data.

5.2. Design Alternatives for Wastewater Collection Systems

Combined sewers are commonly used in the older parts of many major cities to collect wastewater from residential, commercial, institutional, and industrial sources as well as storm water. Storm water is generally less polluted than wastewater, and the treatment of combined wastewater and storm water is difficult during heavy rainfalls, resulting in untreated overflows (commonly termed combined sewer overflow, CSO) (Heaney, 1999). Old combined sewers often discharge untreated wastewater into receiving waters, and a separate sewer system is needed to reduce or eliminate pollution problems derived from combined sewers. In a city like Paraty, where there are large influxes of storm water, a separate sewer system would be best suited. The separate sewer system is also attractive because undiluted sewage is easier and less expensive to treat than combined sewage. Four alternatives for a wastewater collection system for the city of Paraty, discussed below are discussed below, and they are: (i) gravity sewers; (ii) pressure sewers; (iii) vacuum sewers; and (iv) small diameter gravity sewers.

Conventional Gravity Sewers

Conventional gravity sewers transport wastewater by gravity flow from high to low points (Metcalf and Eddy, 1981). They are designed so that the slope and size of the pipe is adequate to maintain flow towards the discharge point without surcharging manholes or pressurizing the pipe. Conventional gravity sewers remain the most common technology used to collect and transport domestic wastewater. Properly designed systems can handle grit and solids in sanitary sewage as well as maintain a minimum velocity, which reduces the production of hydrogen sulfide and methane. The need for a self-cleansing slope can require deep excavations and/or additions of pumping or lift stations.

Several different types of wastewater collection systems have been developed as alternatives to conventional sewers. The network of piping for an alternative collection system can be laid in much shallower and narrower trenches. The pipes are usually of a smaller diameter than those used in a conventional system (100 mm compared to 300 mm in diameter) (US EPA, 1999). They also do not need to be laid in a straight line nor with a uniform gradient. This means they can be laid in such a manner as to easily avoid obstacles. The three main types of alternative collection systems are pressure sewers, vacuum sewers and small-diameter gravity sewers (SDGS).

Pressure Sewers

Pressure sewers use the pressure force supplied by pumps to deliver wastewater to a central location from each property (US EPA, 1991). A pressure system is a small diameter pipeline (typically 100mm), shallowly buried, and following the contour of the land. The systems eliminate the need for lift stations of a conventional system and also infiltration is eliminated because manholes are not required, thus piping materials are not exposed to groundwater fluctuations. There are two types of pressure systems distinguished by the type of pump used. A septic tank effluent pump (STEP) uses septic tanks to capture the solids, grit, grease and stringy material that allows for smaller diameter piping. The effluent pump then provides the necessary pressure to move the wastewater through the system. The second type of pump is a grinder pump (GP), which grinds the solids in the wastewater into tiny particles. The slurry is then pumped into the sewer system that requires a pipe diameter slightly larger than in the STEP system because of the mixture. In the GP system, each household requires a tank containing the pump with grinder blades. Both pump systems require periodic cleaning of local tanks as well as localized electrical supply for each pump.

Vacuum Sewers

Vacuum sewer systems take wastewater from a holding tank (US EPA, 1999). When the wastewater reaches a certain level, sensors within the holding tank open a vacuum valve that allows the contents of the tank to be sucked into the network of collection piping. The vacuum within the system is created by a vacuum station at a central location. Vacuum stations are small buildings that house a large storage tank and a system of vacuum pumps.

Small Diameter Gravity Sewers

Small diameter gravity sewers provide primary treatment at each connection and convey only the effluent (US EPA, 1999). This system is similar to the STEP system in that it would require homeowners to maintain their existing septic tank. Grit, grease and other troublesome solids, which might cause obstructions in the collector mains, are separated from the flow and retained in the septic tanks. Effluent from each tank is discharged to the collector sewer via gravity. There is a lower required velocity in the sewers because solids are not transported through the system. Therefore the pipes do not have to be as large or as sloped.

5.3. Selection Criteria and Preliminary Analysis

The analysis of the different types of collection systems and the selection of the apparent best-fit system for the Historical Center and the City is based on the following four criteria: economics, adaptability, expandability, and simplicity. Addressing the issues associated with these criteria is essential to the selection of a sewer system most appropriate for the Historical Center.

Economics

Capital Costs

Cost is a major deciding factor for any project, and the primary cost trade-offs are discussed here. The capital costs for the sewer collection alternatives include the costs of house connections, sewer mains, and pumping stations. A summary of their comparative costs are provided in Table 5.1 below:

	House Connections ($/household)	Sewer Mains ($/meter)	Pump Stations ($)
Gravity Sewer	2,500	230-330	200,000
Vacuum Sewer	5,000	130-200	400,000
Pressure Sewer	7,000	115-165	None

Table 5.1. Typical costs for sewer systems (Harrington, 2003)

House Connections. Gravity sewers are the simplest and have the lowest typical cost of about $2,500 for a house connection (Harrington, 2003). Vacuum sewers require a vacuum valve station at each property with typical costs starting at $5000 per household. Pressure sewers are the most expensive option, with pump costs approaching $7,000 per household. SDGS have a house connection cost of installing and maintaining the interceptor tanks. Similar to pressure sewers, the cost of installing interceptor tanks is a significant cost. Typically, existing septic tanks cannot be used as interceptor tanks because they are not watertight and cannot be inspected and repaired cost-effectively. Pressure and small diameter gravity sewers are both well-suited for communities with houses that are far apart. The Historical Center of Paraty, where the houses are close together, is therefore not conducive for pressure and small diameter gravity systems.

Sewer Mains. The conventional gravity system has slope requirements to maintain gravity flow. This requires deep excavations and/or additions of pumping or lift stations, which increases construction costs. Pressure sewers are the most cost-effective sewer mains to implement. It does not require deep excavation and its typical cost per meter ranges from $115-165 (Harrington, 2003). Small diameter gravity sewers have a small diameter (~100mm) and can be also busied at a relatively shallow depth. Vacuum sewers are typically 15-25% higher in cost than pressure sewers and gravity sewers are generally more than 100% higher in cost than the lower-cost pressure sewers. The Historical Center of Paraty is not a large area (14 hectares), and therefore, deep excavation is not a large concern. Where the required length of sewer between service connections is comparatively short, the cost of providing conventional sewers is usually affordable.

Pump Stations. Pump systems must have sufficient head to transfer wastewater all the way to the treatment plant. Therefore there is no need for a pump station with these systems. Standard pumping stations for gravity systems begin at $200,000 (Harrington, 2003), while vacuum stations for the same design flows can cost up to 100% more than a sanitary pump station. The average number of customers per station in vacuum systems is about 200-300 (Hassett, 1995). Although it is possible to have a station serve the entire Historical Center, with approximately 700 customers, more pump stations are required to serve the entire city of Paraty, which means higher costs.

Operation and Maintenance Cost

The operation and maintenance cost for pressure systems tend to be high due to the pumps. In areas where the supply of electricity is not reliable, these systems could be more troublesome than not, since constant monitoring, or an addition of backup power supply, is needed at each household. There is less risk with a vacuum system since the vacuum station has a central location, so just one backup power system is required. There is also the cost of cleaning and maintaining each tank at each home. In higher density areas, this could prove to be more costly than the savings from pipe network installations fees for pressure sewers and SDGS.

Case Study

Table 5.2 shows the capital cost as well as O&M cost estimates of different collection systems for a project in Sarasota, Florida. Preliminary design and cost information for low pressure and vacuum sewer systems were obtained from various equipment manufacturers. Table 5.2 provides the estimated annual costs per connection for each collection system alternative based on different population densities selected for analysis. The densities were categorized as low (>0.5 acre average lot size), medium (0.25-0.5 acre average lot size), or high (<0.25 acre average lot size) (Saratosa County, 2000). The Historical Center falls into the high-density category for this analysis. The analysis reiterates the economic impracticality of a pressure system for Paraty. Based on the comparison, the vacuum system is the most cost-effective alternative for a “high” density area like the Historical Center.

	Collection Alternatives	Capital Cost	Annualized Capital Cost	Annual O&M Cost
LOW DENSITY				
	Low Pressure	$10,400	$1,140	$190
	Vacuum	$12,800	$1,370	$140
	Gravity	$18,200	$1,800	$ 90
MEDIUM DENSITY				
	Low Pressure	$8,100	$ 920	$180
	Vacuum	$7,100	$ 820	$ 70
	Gravity	$9,000	$1,010	$ 50
HIGH DENSITY				
	Low Pressure	$8,000	$ 910	$180
	Vacuum	$6,100	$ 730	$ 60
	Gravity	$7,700	$ 890	$ 50

Table 5.2. Summary of estimated annual costs per connection (Saratosa County, 2000)[1]

A further support for the cost-effectiveness of the vacuum system is found in a study by Alan Hassett in Virginia (Hassett, 1995). Hassett provides a comparison of costs for different collection systems for an actual project location in Virginia. The service area was assumed flat with a depth of 1 meter to ground water, an area of 750 acres (300 hectares), and approximately 750 residential units housing 3,000 people. The density was then varied to provide the construction cost information presented in Figure 5.1 below.

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[1] (1) Annualized capital costs were based on an interest rate of 7% over 20 years, and include a capacity fee of $1,642. (2) Replacement costs are based on an interest rate of 7%.

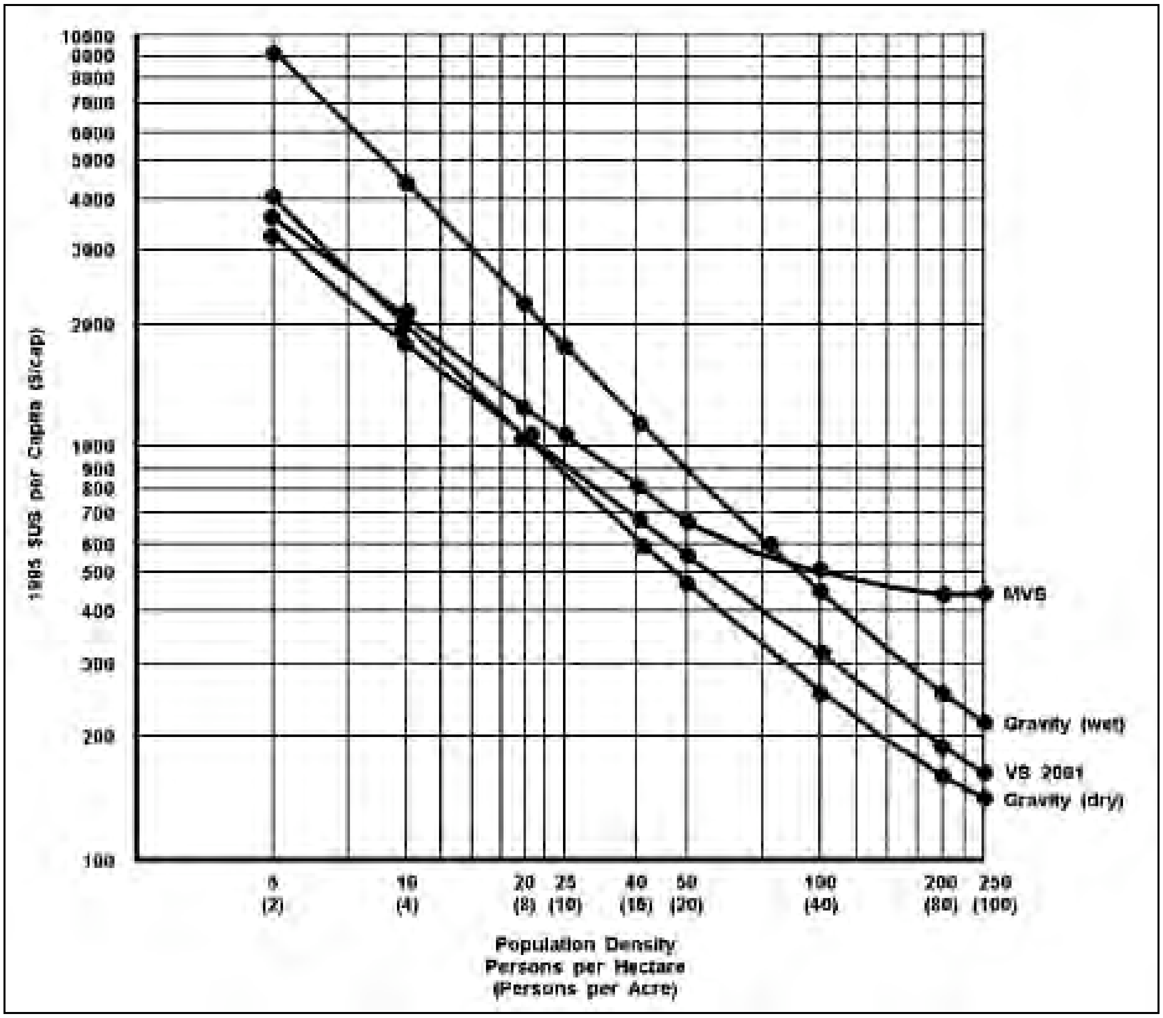


Figure 5.1. Per capita construction costs for different sanitary sewer systems at various population densities (Hassett, 1995)[1]

The population density in Paraty is around 200 persons/ha (Refer to section 5.5 for population data). In the above figure, the vacuum system costs about $60 dollars less than the wet gravity system at that population density. A wet gravity system is a system that includes lift stations and is below the water table. The graph is used for comparative purposes so the exact dollar amount cannot be taken literally. This suggests that for a city like Paraty, a vacuum system can be slightly more cost effective than a gravity system.

[1] MVS means modern vacuum system and VS 2001 represents 21st century vacuum system. Wet means that the system includes lift stations and is below the water table.

Expandability

Although this chapter designs a wastewater collection system for the Historical Center only, its future expansion must be considered since Paraty's long-term goal is to collect and treat all of its wastewater. Expandability is therefore an important criterion for the selection of a wastewater collection system for Paraty. Being comparatively large in diameter and capacity, conventional sewers are often seen as being growth inducing. Both pressure sewer and vacuum technology have less flexibility than gravity sewers in accommodating future increases in flow. For example, a disadvantage of the vacuum system is that the length and amount of pumping possible is restricted by the head limitations (US EPA, 1991).

Adaptability

A third criterion for the selection of wastewater collection system is adaptability and flexibility to seasonal fluxes. Paraty is a tourist city and has a large flux in population as well as rainfall in the summer months. All four systems can handle such a variation in flow, but the gravity system could adapt to these peak flows the best with its comparatively larger size and capacity.

Simplicity

The final criterion is simplicity. Given the history and political climate in Paraty, a simple system is needed. Having a widely-used system enables easier transition from changing administrations. An advantage to conventional collection is that the technology is well established with relatively simple operation and maintenance. Pressure sewer and SDGS systems involve maintenance of septic tanks, and pressure sewers require even more operation and maintenance due to the addition of pumps at each household. Vacuum systems require a full-time system operator with the appropriate training, because possible vacuum leaks can render the whole system inoperable. At the present, Paraty lacks the expertise and resources required for high operation and maintenance systems such as the pressure, vacuum, and small diameter gravity sewers. Also the irregular supply of electricity in the city, especially during the summer, could be problematic for the pressure and vacuum systems.

A gravity system is the simplest alternative, especially since there are plans to build a gravity collection system in the Mangueira section of Paraty (Appendix C). It is desirable to have a consistent collection system for the entire city, since the system becomes more difficult to operate and maintain when the system mixes different types of collection systems and become more complex. A uniform collection system would be the simplest and most desirable in Paraty, where technical support is limited.

5.4. Choice of System: Conventional Gravity Collection

A conventional gravity sewer system is selected for the Historical Center of Paraty based on the preliminary analysis of collection systems. Although the conventional sewer is slightly more expensive than vacuum sewer, its use may be preferred as conventional sewerage is an old and mature practice. Vacuum sewers are not well established in Brazil and are found mostly in large cities (Craveiro, 2003). Paraty needs a system that is easy to maintain and does not require much technical support. The overall plan for a treatment and collection system needs to be expandable, adaptable and centralized. This report covers the design of only the Historical Center of Paraty, but the entire city is in need of an adequate wastewater treatment and collection system. A conventional gravity sewer system is more easily expandable than the alternate systems of collection. Paraty also needs an overall collection and treatment system that is adaptable and robust to the different fluxes in seasonal population and rain. A conventional gravity system coupled with Chemically Enhanced Primary Treatment (CEPT) is ideal for these types of seasonal fluxes. CEPT is a type of wastewater treatment that can handle such variations (Kfouri and Kweon, 2003). Finally a simple system that is adaptable and expandable would be ideal for Paraty, because it would minimize the personnel needed to handle operation and maintenance.

5.5. Technical Approach

The first step in developing a plan is to identify the alternatives for a preliminary design evaluation, and the next step in this process is to evaluate the feasibility for the selected system. This section discusses a conceptual design of a gravity sewer collection system for the Historical Center of Paraty.

Treatment Plant Location

Before sewer networks can be drawn, the location for a potential treatment plant, which determines the layout of pipes, must be selected. Two potential locations were considered – the first near the city's hospital (location 1 in Figure 5.2) and the second in Mangueira (Location 2 in Figure 5.2)

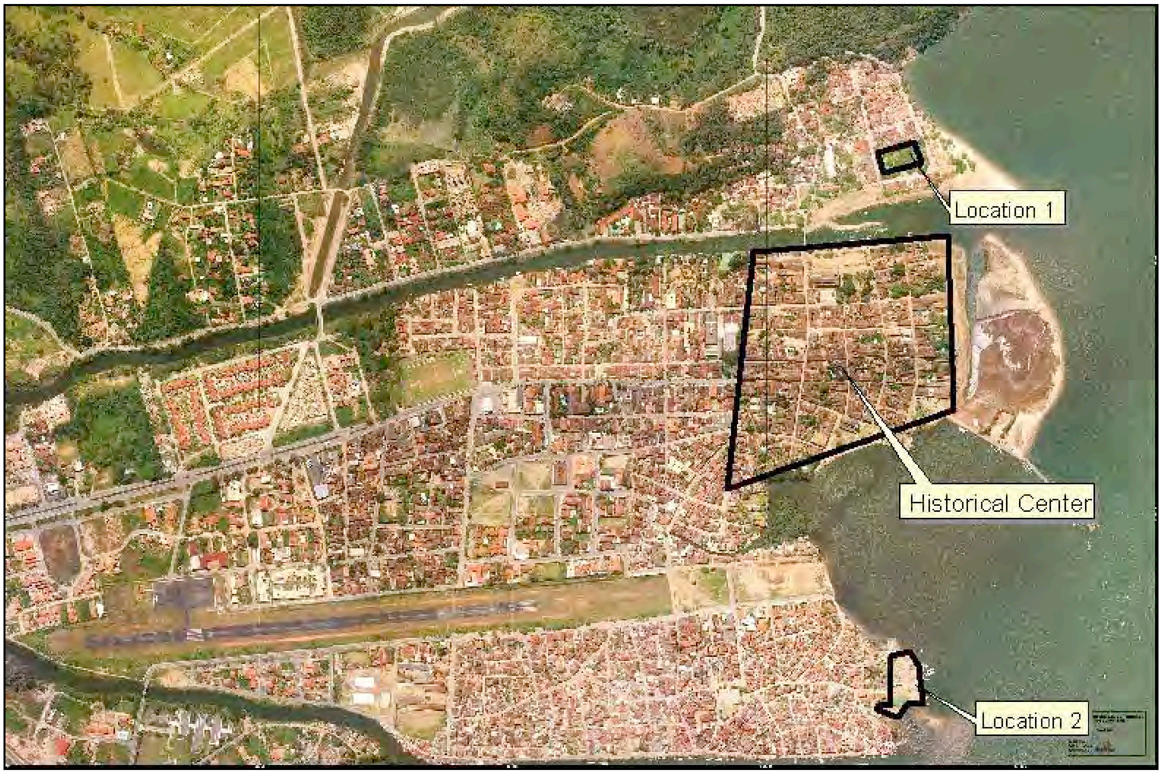


Figure 5.2. Potential locations of treatment plant (Klink, 2003)

Both locations are situated strategically near the Historical Center as well as close to the bay. Location 1 is approximately 3000 square meters in area and Location 2 is approximately 6500 square meters. The proximity to the water allows the discharge of treated wastewater through a marine outfall. Although the site near the hospital is closer to the Historical Center, it is an undesirable location because of its proximity to a beach in the Jabaquara area. The site on Mangueira is more desirable because since there is already a plan to build a wastewater treatment plant there (Appendix C), and the cost of upgrading the plant is less than building another treatment plant. Also the area is larger for Location 2, allowing space for future expansion. In conclusion, Location 2 in Mangueira is chosen as the treatment plant site for the wastewater in the Historical Center.

Overall Layout

ESRI's ArcView GIS is used to lay out the general pipe network in the Historical Center. GIS allows for easy "management, analysis, and mapping of infrastructure and geographic information and descriptive data with cartographic accuracy" (Shamsi, 2002). A line is drawn to represent the proposed sewer in each street to be served, and each line has an arrow indicating the direction in which the wastewater is to flow. Two different pipe networks are designed (See Figure 5.3). Design 1 places the trunk line along the edge of the Historical Center and Design 2 places the trunk line through the middle of the Historical Center. Design 2 is chosen because it could potentially reduce excavation costs. Gravity sewers need to be sloped in order to create velocities large enough to convey wastewater. As pipe segments increase in length, the downstream depth of the pipe also increases. By having the trunk line in a more central location for the network, sewer lines do not have to go as deep because sewer line segments are not as long. Therefore, there is a decrease in installation/excavation costs because of the decrease in the depth of pipe. Manholes are placed at: i) changes in direction; ii) changes in slope; iii) pipe junctions; and iii) the upper ends of all laterals, for cleaning and flushing the lines. The catchment areas are established and quantified in ArcView. The catchment areas, manholes, and pipe segments between manholes are all assigned with labels.

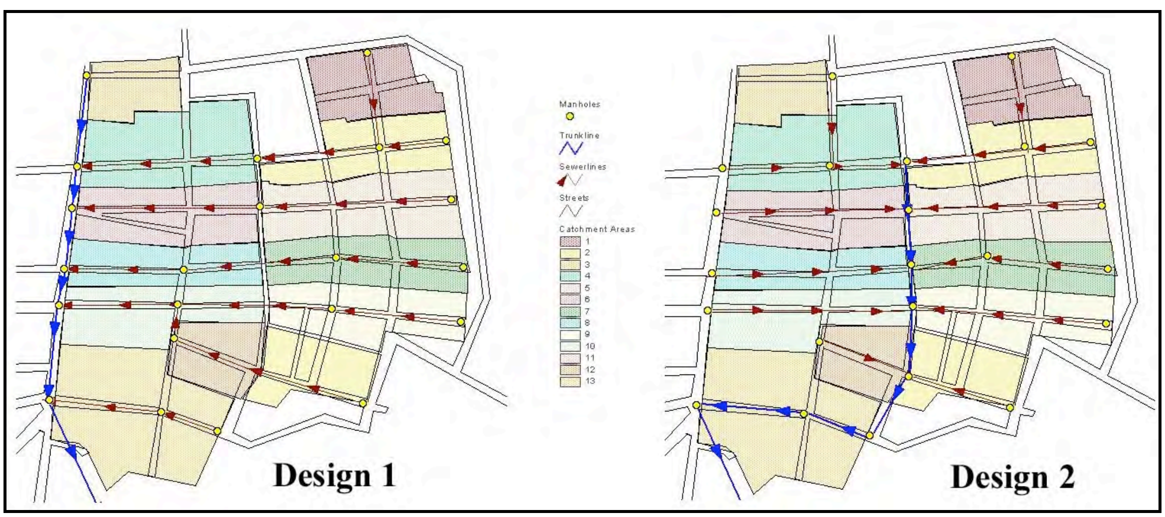


Figure 5.3. Sewer network layout designs

Gravity Sewer Design

Gravity flow sanitary system design involves reviewing design considerations and selecting basic design data and criteria. Once these factors are set, the system is designed, which includes the preparation of a preliminary sewer system design and design of the individual sewers. The system was designed for peak hourly flow of the base population.

Design Parameters

Average Daily Flow. The wastewater flow in Paraty consists of wastewater from residential, commercial and institutional sources and infiltration. An accurate estimation of the wastewater flow rate is crucial in the design of a collection system. A common indicator of wastewater flow is the consumption and use of potable water. The average potable water consumption is 180 liters/person*day, according to the City of Paraty, (Prefeitura, 2001). Few assumptions are made about the population in the Historical Center. The base population of the Historical Center is assumed to be 3,000. This is reasonable considering the size of the whole urban area (15,000 people) and the known population size of another section of the city, Mangueira (5,000 people) (Prefeitura, 2001). Mangueira is a densely populated residential area. The Historical Center is slightly smaller in area and less populous than Mangueira, and therefore a base residential population of 3,000 is a reasonable estimate. The total base flow, calculated as the product of the base population and the average water consumption per person, is approximately 540,000 liters per day. The average summertime population is assumed to be approximately 9,000, three times the base population. The total summertime flow is therefore approximately 1.6 million liters per day. The base and summertime population, and wastewater flow rate are summarized in Table 5.3 below:

Average Base Population	Average Summertime Population	Average WW Flow Rate (L/capita*day)	Total Base Flow (L/day)	Total Summertime Flow (L/day)
3,000	9,000	180	540,000	1,620,000

Table 5.3. Estimate of annual average and summertime average wastewater flow for the Historical Center

Loading. The average wastewater flow is inputted into the network as loads at different manholes. Each catchment area contributes a load to a predetermined manhole. An average flow per hectare is used under the assumption that the different types of property are evenly distributed throughout the Historical Center. The average summer flow per hectare (39,000

L/ha*day) is determined from the total base flow (540,000 L/day) divided by the total area of the Historical Center (14 ha). The load to each manhole is estimated by finding the load contribution from the corresponding catchment area. Table 5.4 displays the distribution of wastewater loads to each manhole and the associated catchment area.

Manhole Number	Catchment Areas	Area (m^2)	Area (ha)	Catchment Load (L/day)
MH1	HC1	8,377	0.8	33,000
MH2	HC2	9,505	1.0	37,000
MH4	HC3	5,610	0.6	22,000
MH5	HC4	13,956	1.4	54,000
MH8	HC5	13,016	1.3	51,000
MH9	HC6	11,360	1.1	44,000
MH11	HC7	10,725	1.1	42,000
MH13	HC8	9,011	0.9	35,000
MH15	HC9	10,862	1.1	42,000
MH16	HC10	10,846	1.1	42,000
MH18	HC11	7,552	0.8	29,000
MH19	HC12	6,338	0.6	25,000
MH20	HC13	20,133	2.0	79,000

Table 5.4. Wastewater loads

Peak Flow. The sewers are designed for peak hourly flows during the non-summer months. Peak hourly flow should be the design average daily flow in conjunction with a peaking factor. In Brazil, the common peaking factor is 1.8 (Tsukamoto, 2003). The peak flow rate is then 1.8 times the mean flow rate. The peak hourly flow is therefore on the order of 1 million liters per day (= 540,000 L/day * 1.8).

Infiltration. In the design, allowance is made for unavoidable infiltration in addition to the expected wastewater flow. One source indicates an infiltration rate of 0.02 L/day/mm diam/m for pipes with a diameter in the range of 200-675mm (8-27in) (City of Arvada, 2001). This infiltration rate does not significantly change the total flow and the summer peak hourly flow is still around 1 million liters per day.

Sewer pipe material and sizes. The proposed pipe material is Polyvinyl chloride (PVC). PVC is favored because it is lightweight but durable. It is also smoother than other materials (Mannings n of 0.010) and highly resistant to corrosion. Other types of pipes, such as concrete pipes (n=0.013), are susceptible to corrosion due to acid and hydrogen sulfide attack (Metcalf and Eddy, 1981). Sewer pipes must have a minimum diameter to account for large objects that may enter the sewers. The minimum pipe size is 150 mm in diameter. The pipe sizes used for the Historical Center range from 150 to 375 mm in diameter.

Depth of cover. The depth of a sewer depends upon the depth of existing underground structures, specifically water lines and basements. In Paraty there are no basements so the depth of the basement is of no concern. The water distribution lines are close to the surface as well as the sidewalks. Therefore the minimum pipe depth of sewers for this design is 0.4 meter below ground surface within the Historical Center. In Brazil the typical standards for minimum cover is 0.6 meters, but 0.4 meter is acceptable for the Historical Center since there is no vehicle traffic. In addition, a relatively inexpensive geotextile can be applied above pipes to absorb pressure and allow for the shallower depth of cover.

Depth of Excavation. A maximum excavation depth is set because it is expensive and impractical to excavate deeper than a certain level, especially in the Historical Center where the water table is high. The maximum excavation depth is set at 1 meter below the mean sea level. This value is based upon input from various engineers working in areas with a high water table much like Paraty.

Velocity. The flow within the sewers must maintain a sufficient velocity in order to flush out any solids that deposit during low flow. The typical minimum velocity for gravity pipes in Brazil, as well as in the U.S., is 0.6 m/s (Metcalf and Eddy, 1981 and Tsukamoto, 2003). Table 5.5 lists a recommendation for PVC pipe slopes at corresponding pipe sizes. It is based on a minimum velocity of 0.6 m/s, when the pipes are 75% full with wastewater flow.

Diameter (mm)	Slope
200	0.004
250	0.003
300	0.002

Table 5.5. Recommended slopes for different PVC pipe diameters (City of Arvada, 2001)

These slopes act as a guide in designing sewer networks. The velocity will be less than 0.6 m/s when the pipes are less than 50% full with wastewater flow.

Elevation. The land in the Historical Center is flat. The tidal range, the difference between highest and lowest tide, in the region is 0.9 m in the sea (Rosman, 2003). Groundwater levels therefore range between 0.5 m above mean sea level (msl) to 0.4m below msl. Since tidal flooding has been observed in the Historical Center the elevation of the ground is probably around 0.3 m above msl. The elevation was assumed to be 0.5 meters above mean sea level for modeling purposes.

Profiling and Modeling

After the design factors and constraints are set, a more detailed profile and model of the sewer network is created. A spreadsheet is prepared in Microsoft Excel to record the data and the computation steps for each section of sewer between manholes. In conjunction with Haestad Method's SewerCAD, the sewer invert elevations, pipe diameters, pipe slopes and velocities are determined by trial and error to find the best-fit design given the design factors and constraints.

SewerCAD is a powerful design and analysis tool that allows the layout of a collection system, computation of sanitary loads, and simulation of the hydraulic response of the entire system - including gravity collection piping and pressure force mains (Haestad, 2002). SewerCAD has features such as steady-state analysis using various standard peaking factors, extended-period simulations of complete collection systems, and advanced automatic system design. The program provides import and export wizards to transfer data between GIS and the model in SewerCAD. This enables an initial layout within GIS, an import of that layout into SewerCAD, and an export of the model back into GIS. Figure 5.4 provides a SewerCAD layout of the pipe network and Figure 5.5 displays the profile of the main trunk line of the optimal preliminary design. All other profiles for the network can be found in Appendix D.

Using the loads, the sewer network components (elevation, pipe diameter, slope, velocity) are designed for a peak hourly flow and a maximum excavation depth of one meter. Table 5.6 presents the network component data for each pipe segment. All elevations are relative to the mean sea level. The maximum depth of a sewer segment is the most downstream segment, L22, with an invert elevation of approximately -1 m relative to mean sea level. The length in pipe ranges from 40 to 200 meters and all average pipe depth of cover re above the minimum constraint of 0.4 meter. The contribution of local infiltration is negligible compared to the total flow. The total flow of the entire system is found at the most downstream point of 1 million liters/day. The velocities at the upstream point of each pipe do not meet the minimum velocity requirement of 0.6 m/s. The flow within all of the pipes is less than 50% full.

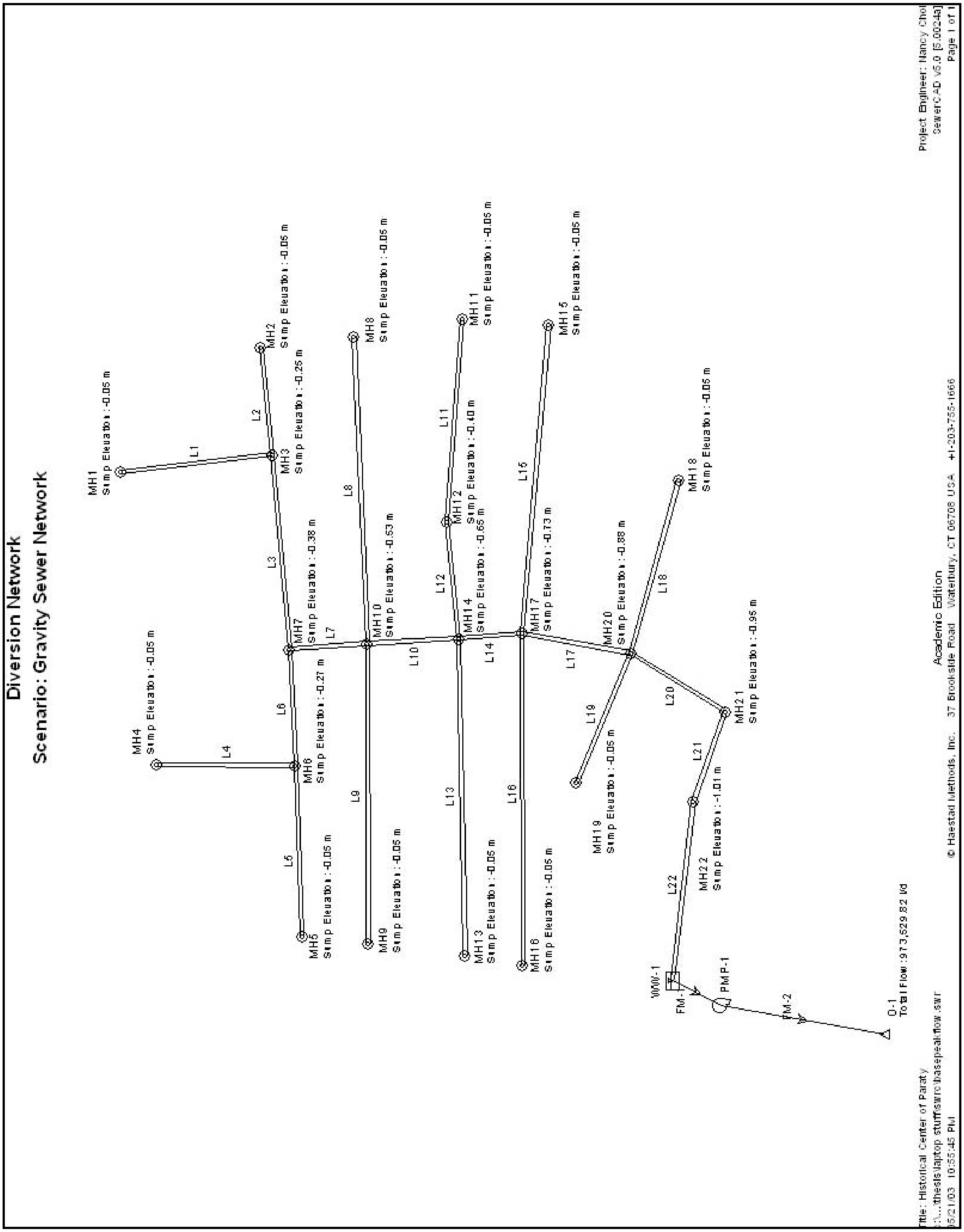


Figure 5.4. Gravity sewer network for the Historical Center

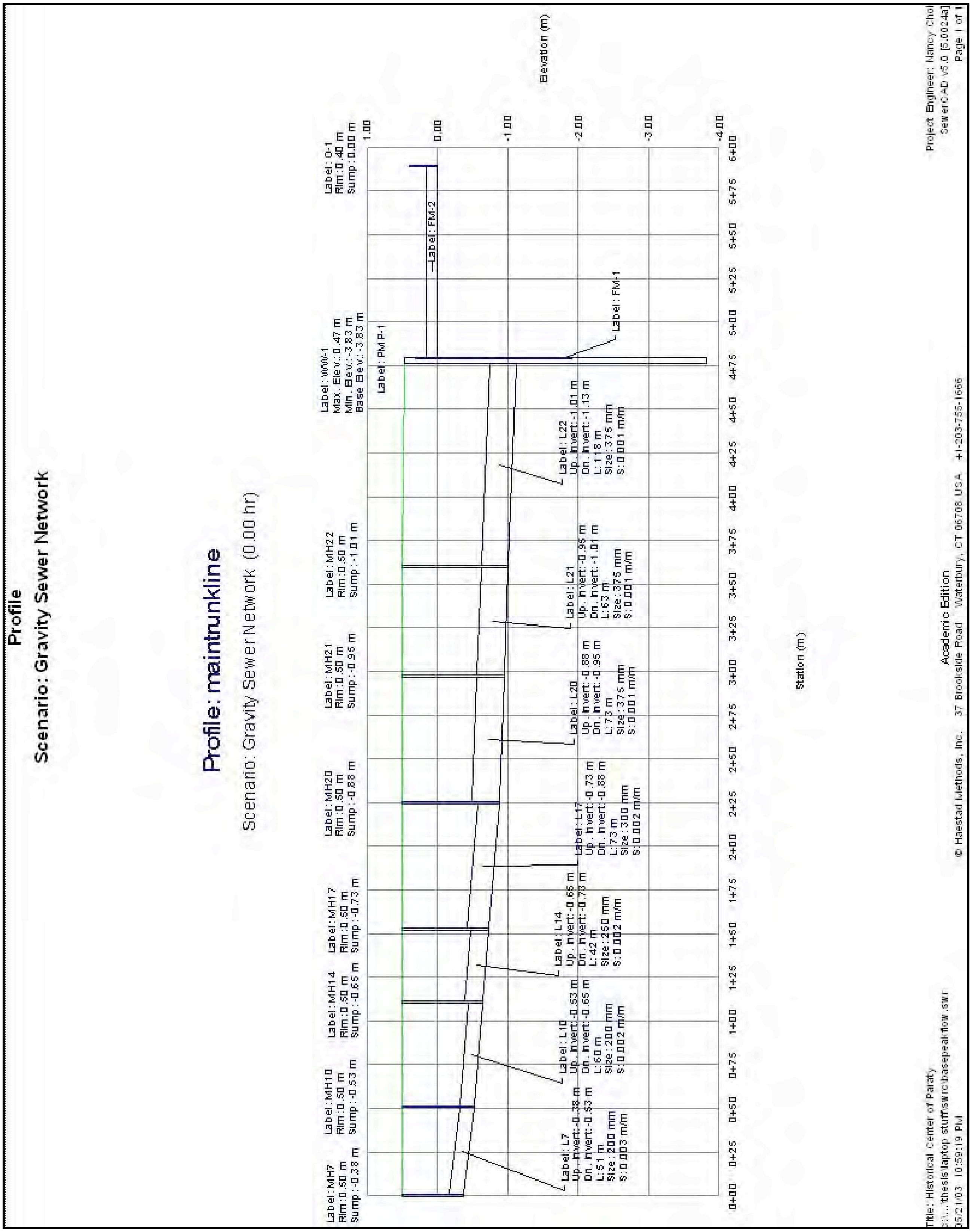


Figure 5.5. Profile of the main trunk line

Pipe No.	Upstream Node	Upstream Invert Elevation (m)[1]	Downstream Node	Downstream Invert Elevation (m)	Constructed Slope (m/m)	Length (m)	Pipe Size (mm)	Local Infiltration (L/d)[2]	Total Flow (L/d)	Average Pipe Cover (m)[3]	Velocity In (m/s)
L1	MH1	-0.05	MH3	-0.25	0.002	100	150	300	60,000	0.5	0.2
L2	MH2	-0.05	MH3	-0.19	0.002	71	150	220	67,000	0.5	0.3
L3	MH3	-0.25	MH7	-0.38	0.001	128	200	520	127,000	0.6	0.2
L4	MH4	-0.05	MH6	-0.23	0.002	92	150	280	40,000	0.5	0.2
L5	MH5	-0.05	MH6	-0.27	0.002	112	150	340	98,000	0.5	0.3
L6	MH6	-0.27	MH7	-0.35	0.001	76	200	310	138,000	0.6	0.2
L7	MH7	-0.38	MH10	-0.53	0.003	51	200	210	265,000	0.8	0.4
L8	MH8	-0.05	MH10	-0.53	0.002	201	200	820	93,000	0.6	0.3
L9	MH9	-0.05	MH10	-0.53	0.002	197	200	800	80,000	0.6	0.3
L10	MH10	-0.53	MH14	-0.65	0.002	60	200	240	438,000	0.9	0.4
L11	MH11	-0.05	MH12	-0.40	0.003	134	150	410	76,000	0.6	0.3
L12	MH12	-0.4	MH14	-0.65	0.003	77	200	310	76,000	0.8	0.3
L13	MH13	-0.05	MH14	-0.65	0.003	208	200	850	64,000	0.7	0.3
L14	MH14	-0.65	MH17	-0.73	0.002	42	250	210	578,000	0.9	0.4
L15	MH15	-0.05	MH17	-0.73	0.003	203	200	820	76,000	0.7	0.3
L16	MH16	-0.05	MH17	-0.73	0.003	217	200	880	76,000	0.7	0.3
L17	MH17	-0.73	MH20	-0.88	0.002	73	300	440	732,000	1.0	0.5
L18	MH18	-0.05	MH20	-0.88	0.007	118	150	360	53,000	0.8	0.4
L19	MH19	-0.05	MH20	-0.88	0.009	92	150	280	45,000	0.8	0.4
L20	MH20	-0.88	MH21	-0.95	0.001	73	375	550	972,000	1.0	0.4
L21	MH21	-0.95	MH22	-1.01	0.001	63	375	480	973,000	1.1	0.4
L22	MH22	-1.01	WW-1	-1.13	0.001	118	375	900	974,000	1.2	0.4

Table 5.6. Pipe segment data

[1] All elevations are relative to mean sea level ("0" datum)

[2] Infiltration rate = 0.02 L/d/mm-m

[3] Ground level assumed to be +0.5 m (MSL); minimum depth of cover +0.4 m (MSL)

5.6. Cost Estimates

The gravity sewer system consists of 2,500 meters of gravity sewer, 22 manholes and 1 pump station. The cost estimates for the construction of this gravity sewer system is summarized in Table 5.7 below:

	Unit Amount	Unit Cost	Units	Capital Costs	O&M Costs
Total Pipe Length (m)	2506	320	US$/meter	US$801,920	
Number of manholes	22	5,400	US$/manhole	US$118,800	
Number of Lift Stations	1	135,000	US$/station	US$135,000	
Total				**US$1,055,720**	**US$150,000/yr**

Table 5.7. Estimated capital cost and O&M cost for wastewater collection infrastructure[1]

The cost data for this study are derived from different sources. Costs for pipes and manholes are estimated based on similar projects in communities in the United States. The costs of lift stations are based on estimates reported by the EPA. Annual operation and maintenance costs are estimated at approximately $150,000. These cost estimates are based on situations in the US and may not accurately reflect the costs in Brazil. Since Brazil has undergone two devaluations in currency, the scales of price are different from the prices in the US. After personal communication with engineers in the US and Brazil[2], it is agreed that the costs in the US are much higher than the costs in Brazil. Other than the currency exchange between the two countries, the costs of labor in Brazil is typically cheaper than in the US as is the cost of construction due to less stringent regulations.

5.7. System Recommendations

Several options exist for the construction of a wastewater collection system to serve the Historical Center of Paraty. Based on a review of current and future service areas, projected wastewater flows, topography, collection system and transport options, capital costs, and operation and maintenance costs, a conventional gravity collection system is recommended. The

[1] Estimates based on information from Engineer Sylvia Lee and EPA (2000)

[2] Personal Communication, Fernando Craveiro, Ricardo Tsukamoto, and Flygt Corporation

capital cost is estimated to be around $1.1 million with an annual operation and maintenance cost of $150,000.

The general schematic for the wastewater collection and treatment system is as follows:

Gravity Sewer → Treatment Plant → Marine Outfall

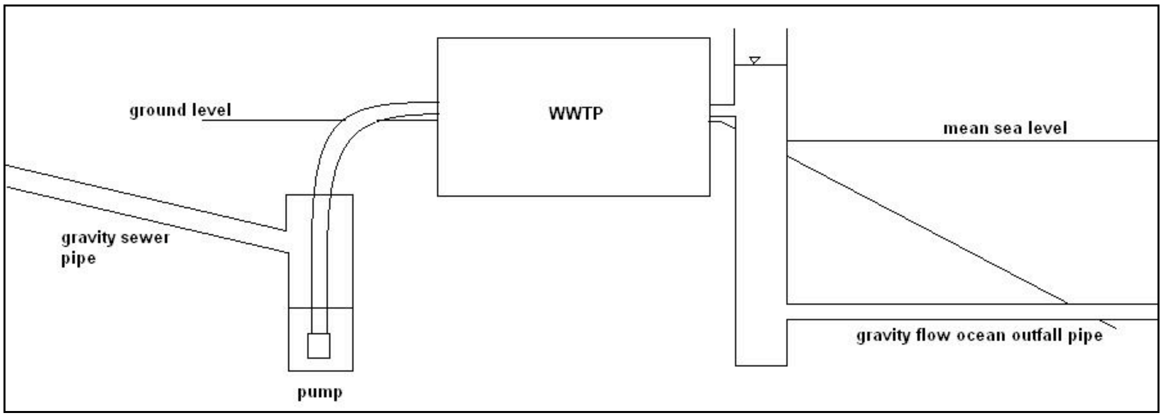


Figure 5.6. General picture of wastewater infrastructure plan

Figure 5.6 above is a schematic representation of the basic plan for wastewater collection and treatment. The wastewater is conveyed through gravity sewers to a wet well for temporary storage before being pumped to the treatment plant. After treatment, the disinfected effluent is discharged through an ocean outfall. Chemically Enhanced Primary Treatment (CEPT) is an attractive treatment process because it can easily adapt to the seasonal changes in population in Paraty. It is important to note that the head of the effluent from the treatment plant must be high in order to maintain a gravity flow ocean outfall. The collection system is designed for peak hourly flow of 1 million L/day. The wastewater treatment plant should be sized for the average daily flow of 500,000 L/day.

There are three possibilities for installation of the new gravity sewers: (i) restoration and reuse of existing infrastructure, (ii) noninvasive, nondestructive installation, and (iii) trench excavation.

Restoration and reuse of existing structures. It is recommended to review and explore the condition of the existing collection infrastructure in the Historical Center. Although the system itself probably could not be used directly as a wastewater collection system, it may be cleaned and used for the new collection system to be laid within the existing structures. The existing collection system consists of 1 meter in length sections of concrete pipe with a diameter of about 1 meter, making it more than possible for the new pipes to fit inside.

Noninvasive, nondestructive installation. Microtunneling is a process that uses a remotely controlled Microtunnel Boring Machine (MTBM) combined with the pipe jacking technique to directly install product pipelines underground in a single pass (www.huxtedtunnel.com). This process avoids the need to have long stretches of open trench for pipe laying, which causes extreme disruption to the community. This process could be highly cost-effective for a place like the Historical Center, where the water table is high and the buildings are old.

Trench excavation. Open trench excavation is the traditional method of installing sewer pipes. This installation process could be favored over the two aforementioned options, because it can provide opportunities for the rehabilitation of the roads and the burial of electrical lines. The streets within the Historical Center are in a bad condition due to road renovations in the past where the stones in the roads were placed back misaligned. Given this project, excavation would be favored to provide an opportunity to renovate the existing roads as well as place all electrical wires underground. Another qualification for the UNESCO recognition is to place all electrical wires underground. If these additional projects were to be taken, much planning would be needed to coordinate these projects.

This project provides a conceptual design of a gravity sewer collection system for the Historical Center of Paraty, which may be used as a model in future expansion for the rest of the City. A discussion and analysis of the feasibility of this design is provided in Appendix E. More in-depth studies are needed for the design of a gravity sewer collection system as well as the three installation possibilities.

5.8. Conclusions

This chapter proposed a conceptual design of a gravity sewer collection system for the Historical Center of Paraty, Brazil. The report investigated wastewater flow requirements, wastewater collection and transport alternatives, possible wastewater treatment plant locations, capital requirements, and operation and maintenance costs.

Gravity sewer system was selected as the system of wastewater collection in the Historical Center. The four criteria used in the selection were economics, expandability, adaptability and simplicity. The results of the analysis concluded that the two major systems to consider would be a vacuum system and a gravity system. Although the study revealed that a vacuum system may be less expensive to construct, other factors favored the gravity collection system. For example, the vacuum system, which was a relatively new technology, required high operation and maintenance skills that are not readily available in Paraty. It has been concluded that a uniform, consistent, simple collection system would be the most appropriate for the City of Paraty.

It is recommended that the City of Paraty pursue the construction of a gravity sewer system, pumping stations, and a wastewater treatment plant, according to the results of the feasibility study. Paraty is in need of infrastructure development, and the construction of wastewater collection facilities will allow Paraty to minimize the impact of untreated wastewater on public health and environmental resources.

CHAPTER 6 – DESIGN OF WASTEWATER TREATMENT PLANT

This chapter provides an introduction to chemically enhanced primary treatment (CEPT) of wastewater, followed by a preliminary design of CEPT plant, which includes the design of CEPT tanks, chlorination basins, and chemical storage tanks. The characteristics of raw sewage, as well as the analysis of jar tests are important in the design of a wastewater treatment plant. The design of wastewater treatment plant for Paraty is based on jar tests performed with wastewater samples collected from Paraty, Brazil and compared to the results from similar jar tests conducted in Boston, U.S. (See Appendix G and H). It is important to note that seawater was tested for its efficiency at acting as a coagulation enhancement tool. The results from these jar tests are also included in Appendices G and H. A discussion of disinfection of treated wastewater effluent is included in Appendix I, and the discussion of various options for sludge treatment and disposal are discussed in Appendix J.

6.1. Introduction to Chemically Enhanced Primary Treatment

Chemically Enhanced Primary Treatment (CEPT) is a wastewater treatment method that is 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 achieves coagulation and flocculation by chemical addition, similar to conventional potable water treatment, and accomplishes large removal of biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), and total phosphorous (TP) from the influent wastewater. The main advantage to CEPT therefore is that it generates an effluent that can be efficiently and economically disinfected 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 adding 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 6.1 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.

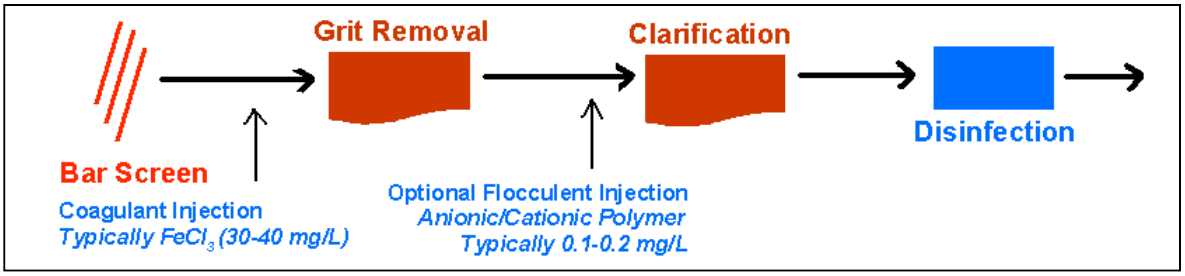


Figure 6.1. CEPT vs. conventional primary treatment

The injected chemicals act as coagulants/flocculants forming large heavy flocs that settle to the bottom of the clarifier and form a sludge layer, which can be appropriately removed. Particulate and colloidal settling are the processes responsible for the formation and settling of flocs. 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).

Theory of CEPT

Colloidal particles found in wastewater typically have a net negative surface charge. The size of colloids (about 0.01 to 1 mm) is such that the attractive 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).

Coagulation

Coagulation encompasses all 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 mmeters 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 6.2 of the following page shows typical floc in chemical treatment.

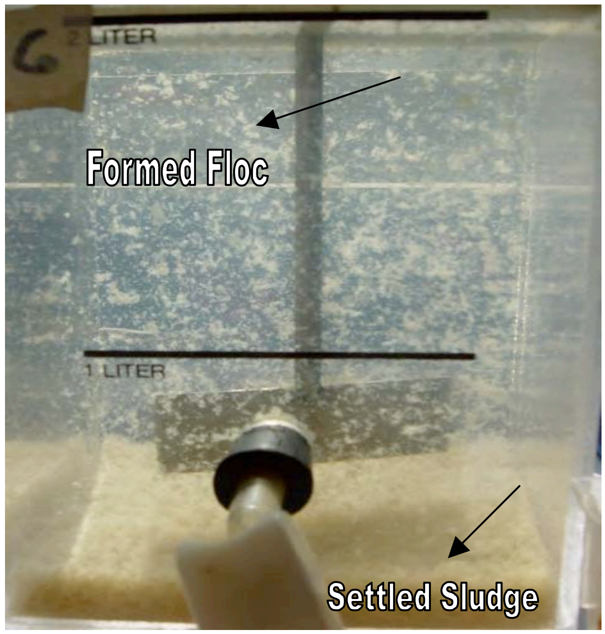


Figure 6.2. Floc formation due to chemical addition

Typical coagulants and flocculants include: natural and synthetic organic polymers; metal salts, such as alum, ferric sulfate, and ferric chloride; and prehydrolized 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 dosage 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.

Flocculation

Flocculation is the process in which the size of particles increases as a result of particle collisions. The two types of flocculation are: i) *microflocculation* (or perikinetic flocculation), in which particle aggregation is brought about by the random thermal motion of fluid molecules known as Brownian motion and ii) *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 6.3 below shows the typical difference in treated effluent quality compared to the raw wastewater influent. Beaker 1 on the left of Figure 6.3 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 6.3. Difference in effluent quality

Advantages of CEPT

The advantages to using chemically enhanced primary treatment revolve mainly around large reductions in the volume and concentrations of required chemicals, ecological effects downstream and maintenance and operation labor demands, all of which translate into huge 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, as discussed in detail below. The footprint of the treatment plant's infrastructure can therefore be significantly smaller, reducing capital costs. Since CEPT can be easily used to upgrade 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.

<u>Removal Efficiencies</u>

Chemically enhanced primary treatment enhances conventional primary treatment and achieves significantly higher removal rates at lower costs compared to secondary treatment. Table 6.1 below is a summary of expected removal rates in conventional primary, chemically enhanced and secondary wastewater treatment (NRC, 1996). These removal rates, graphed in Figure 6.4 and 6.5 below, and coupled with the financial estimates for the three treatment alternatives in Table 6.2, 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 6.1. Relative removal efficiencies[1]

Table 6.1 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. Figure 6.4 and 6.5 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.

[1] TSS = total suspended solids; BOD = biochemical oxygen demand; TP = total phosphorous; TN = total nitrogen; FOG = fat, oil, and grease.

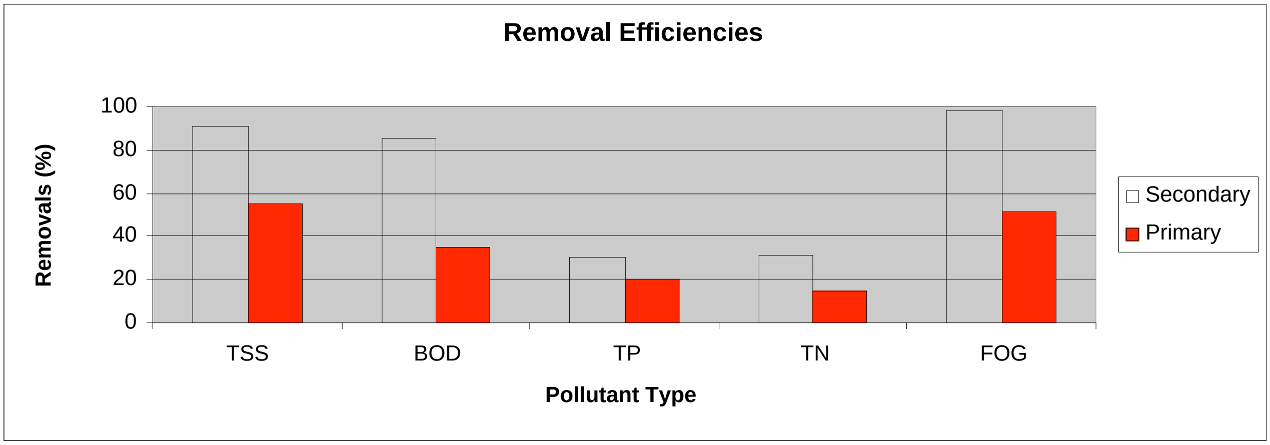


Figure 6.4. Secondary treatment vs. Primary treatment removal efficiencies

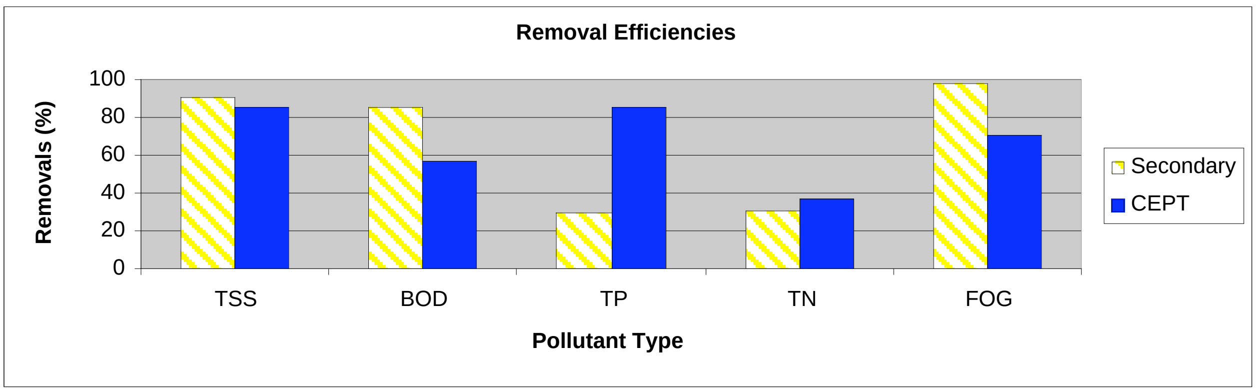


Figure 6.5. Secondary treatment vs. CEPT removal efficiencies

<u>Relative Costs</u>

Table 6.2 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 6.2. Relative treatment costs (NRC, 1996)

6.2. Design of a Chemically Enhanced Primary Treatment Plant

Design Parameters

Based on the results of jar tests included in Appendices G, H, and I, the characteristics of raw Paraty's wastewater, and the required dosages of chemicals were determined for the design of the CEPT plant, as summarized in Table 6.3:

Raw Wastewater Characteristics	
Influent SS	200 mg/L
Influent COD	350 mg/L
Chemical Doses	
Ferric Chloride	40 mg/L
Seawater Volume	5%
Polymer	0.1 mg/L
Chlorine	3 mg/L
Sulfur Bisulfate	0.5 mg/L
Expected Removal	
SS	85%
COD	55%
Fecal Coliform	100%

Table 6.3. Design parameters for wastewater treatment plant for the City of Paraty

Design of CEPT Tanks

The CEPT plant is to be built in three stages in conjunction with the construction of wastewater collection system, as recommended in the proposed policy for Paraty (See Chapter 4). The first CEPT plant is to be built for the Historical Center during the first stage, for Manguera and Ilha das Cobras during the second stage, and for Old city during the third stage. The population, average wastewater flow rate, and peak wastewater flow rate are summarized for each stage in Table 6.4 below:

	Stage 1	Stage 2	Stage 3
	Historical Center	+ Mangueira and Ilha das Cobras	+ Old City
Base Population	3,000	7,500	3,000
Summertime Population	9,000	7,500	9,000
Water Consumption (L/day-capita)	180	180	180
Peak Factor	1.8	1.8	1.8
Qavg (m^3/day)	540	1,890	2,430
Qp (m^3/day)	972	3,402	4,374
Qsp (m^3/day)	1,620	2,970	4,590
Qspp (m^3/day)	2,916	5,346	8,262

Table 6.4. Average and peak wastewater flow rate for each wastewater development stage[1]

The CEPT tanks must be designed with the minimum width-to-length, and height-to-length ratios of 1 to 5 (W:L ≥ 1:5, and H:L ≥ 1:5), to ensure plug flow of wastewater in tanks, and for the capacity to serve the peak flow rate during the summer season (Qspp). With these constraints, CEPT tanks in Paraty are designed based on overflow rate (OFR) of 30 m/day. OFR of 90 m/day is used for Qspp, since the efficiency of CEPT is constant up to OFR of 90 m/day. Table 6.5 below shows the estimated flow rate of CEPT with overflow rates of 30, 60, 90 m/day in a 15m x 3m x 3m CEPT tank.

Design of CEPT Tank and Estimated Flow Rates				
Dimension of 1 CEPT Tank	Width (m)	3		
	Height (m)	3		
	Length (m)	15		
Stage		1	2	3
Number of CEPT Tanks		1	1	2
Footprint (m^2)		45	45	90
Flow Capacity (m^3/day)	Minimum	1350	1350	2700
	Median	2700	2700	5400
	Maximum	4050	4050	8100
Expected Daily Flow (m^3/day)	Non-Summer Season	540	1890	2430
	Summer Season	1620	2970	4590

Table 6.5. Design of CEPT tank, and estimated flow rates

[1] Qavg = average flow rate; Qp = peak flow rate; Qsp = average flow rate during summer = Qavg * seasonal factor; Qspp = peak flow rate during summer

According to the table above, the capacity of one tank meets 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 15m x 3m x 3m are required for Paraty at the end of the third stage. Since one more tank is needed for maintenance, a total of three CEPT tanks, with the total footprint of 135 m^2 (45 m^2 * 3), is recommended.

Design of Chlorination Basins

The chlorination basins must be designed with the minimum width-to-length ratio of 1 to 20 (W:L ≥ 1:20) to maintain an acceptable level of disinfection efficiency (Metcalf & Eddy, 2002). In addition, the detention time should be 30 to 120 min for an average flow, and 25 to 90 min for a peak flow. Table 6.6 shows the acceptable dimensions of chlorination basins and detention times associated with various flow rates, determined using trial and error method.

Design of Chlorine Contact Basin and Estimated Flow Rates				
Dimension of 1 Chlorine Contact Basin	Width (m)	1		
	Height (m)	2		
	Length (m)	20		
Stage		1	2	3
Number of Chlorine Contact Basins		1	1	2
Footprint (m^2)		20	20	40
Expected Daily Flow (m^3/d)	Non-Summer Season	540	1890	2430
	Summer Season	1620	2970	4590
Detention time (min)	Non-Summer Season	107	30	47
	Summer Season	36	19	25

Table 6.6. Design of chlorine contact basin, and estimated flow rates and detention times

As shown above, detention time of flow during summer season (Qsp) of the third stage is in the acceptable range, 25 to 90 min with two basins (Metcalf & Eddy, 2002). Since one more contact basin is required for maintenance, a total of three basins, with the total footprint of 60 m^2 (20 m^2 * 3), is recommended.

Design of Chemical Storage Tanks

Ferric Chloride Storage Tanks

Ferric chloride (40 mg/l) and small amounts of polymer will be used for wastewater treatment. If the concentration of ferric chloride is 40%, the volume of ferric chloride stored for a period of 10 days is:

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

Assume 20% more storage for the case of higher amount of chemical dosage:

$$3.3m^3 \text{¥} 1.2 \text{ @} 4.0m^3$$

The volume of storage tank for ferric chloride is 4.5 m^3, to ensure sufficient storage capacity. The footprint of the storage tank is 3 m2, when the height of the tank is 1.5 m. The footprint of storage tank for polymer is negligible since amount of polymer used is much smaller than that of ferric chloride.

Sodium Hypochlorite (Chlorination Agent) Storage Tanks

Liquid sodium hypochlorite (3 mg/) 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 %, the volume of sodium hypochlorite stored for a period of 10 days is:

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

Assume 10% more storage for the case of higher chlorine demand:

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

The volume of storage tank for sodium hypochlorite is 1.2 m3, to ensure sufficient storage capacity. The footprint of the storage tank is 1 m^2, when the height of the tank is 1.2 m.

<u>Sulfur Bisulfate (Dechlorination Agent) Storage Tanks</u>

If the concentration of sulfur bisulfate (SBS) is 25 %, the volume of SBS stored for a period of 10 days is:

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

It is expected that the amount of SBS will not change much considering the proper dosage of chlorine. The volume of the tank will be 0.1 m^3, to ensure sufficient storage capacity. The footprint of the storage tank will be 0.1m^3, when the height of the tank is 1 m.

The volume, footprint of the CEPT tanks, chlorine basins, and chemical storage tank are summarized in Table 6.7 below:

	CEPT Tank	**Chlorine Contact Basin**	**$FeCl_3$ 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 6.7. Volume and footprint of CEPT facilities

In addition, bar screen, grit chamber, parshall flume, and pumps will be added.

6.3. Conclusions

This chapter proposed a preliminary design of a chemically enhanced primary treatment (CEPT) plant for Paraty, Brazil. Included in this chapter are recommended chemical doses and the use of seawater as a coagulation enhancement mechanism, which are based on jar test experiments, performed in Paraty, Brazil and Boston, MA. According to the results of jar tests, 40 mg/l of ferric chloride and 5 % of seawater addition are recommended. Furthermore, 0.1 mg/l of polymer can be added to enhance the SS and COD removal efficiencies. For chlorination and dechlorination, 3 mg/l of chlorine and 0.5 mg/l of sulfur bisulfate are recommended. Expected removal efficiencies are 85 % for SS, 55 % for COD, and 100% for fecal coliform. To obtain expected removal efficiencies, 2 CEPT tanks with dimensions of 15m x 3m x 3 m, and 2 chlorination basins with dimensions of 20m x 1m x 2 m are required. For maintenance, one additional tank of each is recommended. This wastewater plant requires a total footprint of approximately 180 m^2.

CHAPTER 7 – CONCLUSIONS

Paraty is a beautiful coastal city in the State of Rio de Janeiro that is thriving as a tourist city with its abundant natural beauty and cultural charm. With its esteemed Historical Center, which has well-preserved colonial architecture of considerable historical and cultural significance, the City is currently making efforts to qualify for a UNESCO World Heritage Site, which encompasses improving its existing sanitation system. In congruence to the City's objectives, this project is undertaken for a general purpose of assessing the condition of the City's existing water and sanitation infrastructure and associated public health problems, and of providing a preliminary design of wastewater collection system and treatment plant for the City.

The inadequacy of the City's existing potable water quality and supply system, well-known to and fully felt by the local population, include: (i) shortage of water supply in the summer; (ii) ineffective disinfection; (iii) inadequate protection of water sources; and (iv) substandard water quality (Chapter 2). In order to address these problems, the existing disinfection system for the potable water is to be upgraded immediately, with a flow meter and an automated chlorinator, and the potable water intake points are to be fenced around the perimeter, in order to protect the source waters. A new drinking water treatment plant, with the capacity for the entire City of Paraty including Jabaquara, is to be constructed in one stage, immediately following the completion of the development of wastewater collection system and treatment plant (Chapter 3).

The City's sanitation system, which is almost completely lacking, has greater impact on the humans and the environment, and the need for its development is therefore more imminent. Public health problems related to water and sanitation, reflected in the high incidence of diarrhea in the City, are representative of not only the poor potable water quality, but also the direct consequence of discharging untreated wastewater into nearby surface water bodies, with which people easily come into contact (Chapter 2). It is therefore evident that the City must construct a wastewater collection system and infrastructure and a wastewater treatment plant, in order to reduce the environmental pollution and associated public health risks.

It is proposed that a gravity sewer system is used for the collection of wastewater, and that a chemically enhanced primary treatment (CEPT) plant is used for the treatment of wastewater. The wastewater collection system and infrastructure and the treatment plant are to be constructed concurrently in three stages for the City, in order to fully utilize the easily upgradeable characteristics of the CEPT plant and thereby reduce the cost of developments. The Historical Center is to be developed in the first stage; Manguera and Ilha das Cobras in the second stage; and the Old City and rest of the City in the third stage. Each stage is expected to last approximately two years, and the wastewater treatment plant is to serve the developed areas immediately after the completion of each development stage (Chapter 3).

The conceptual design of the wastewater collection system is limited to the Historical Center, but applicable to the whole city. A gravity sewer system is proposed as the system of wastewater collection, based on four selection criteria: economics, expandability, adaptability, and simplicity. The feasibility analysis of conventional gravity sewers, pressure sewers, vacuum sewers, and small diameter gravity sewers reveals that the vacuum system is the least expensive alternative. However, the vacuum system is also a relatively new technology, and requires high operation and maintenance skills. Therefore, the study suggests that the gravity sewer system, which is also relatively inexpensive, may be most appropriate for the City since is a well-established, simple technology.

A chemically enhanced primary treatment (CEPT) plant is proposed for the treatment of the City's wastewater. The addition of 40 mg/l of ferric chloride and 5 % seawater are recommended for CEPT, according to the results of jar test experiments. Furthermore, 0.1 mg/l of polymer can be added to enhance the SS and COD removal efficiencies. For chlorination and dechlorination, 3 mg/l of chlorine and 0.5 mg/l of sulfur bisulfate are recommended. Expected removal efficiencies are 85 % for SS, 55 % for COD, and 100% for fecal coliform. Two CEPT tanks with dimensions of 15m x 3m x 3 m, and 2 chlorination basins with dimensions of 20m x 1m x 2 m are required, and one additional tank of each is recommended for maintenance. The wastewater plant requires a total footprint of approximately 180 m^2, excluding the area required for the treatment of sludge.

The total cost of wastewater treatment collection system and treatment plant is approximately R$ 4 million for capital, and R$ 0.5 million per year for operation and maintenance (O&M). The total cost of a new drinking water treatment is approximately R$ 1 million for capital, and R$ 0.4 million per year for O&M. In order to fully recover these costs, an annual revenue of R$1.2 million must be collected from water and sewage tariffs. The following water and sewage tariffs, which are based on willingness to pay (WTP), are to be billed for each income group: R$1.40/$m^3$ for Mangueira and Ilha das Cobras population; R$2.40/$m^3$ for Historical Center and Old City population; and R$1.20/$m^3$ for Jabaquara population. Since these tariffs can be seen as a substantial increase from the existing tariffs, appropriate interim tariffs are to be designed and implemented in one or more steps to phase in the final design tariff.

The construction of a wastewater collection infrastructure and treatment plant, and drinking water treatment plant is expected to bring numerous and substantial benefits to the City, which include: improvements in public health, environmental quality, and aesthetics in the city, as well as increases in productivity and economic value of the environment. It is also expected that these water and sanitation improvements will encourage tourism and promote general economic growth in the City, providing large economic returns.

APPENDIX A - INTRODUCTION TO WATER AND SANITATION

The provision of safe drinking water and the proper treatment and disposal of human waste can achieve large gains in human health, and environmental quality, and hence provides substantial economic returns. Therefore naturally, the provision of adequate drinking water supply and sanitation ranks at the top of priority environmental challenges in Paraty, Brazil, as well as in many parts of the developing world. In this report, drinking water supply refers to a system or service of water collection, drinking water treatment, and water distribution for human consumption. Sanitation is defined as the services or systems of collection, transportation, treatment, and sanitary disposal of wastewater, excreta, or other waste.

A.1. Health Consideration

Many studies report that unreliable drinking water quality and supply and the lack of wastewater treatment has a significant impact on health. The use of polluted waters for drinking and bathing causes infectious diseases that kill millions and sicken more than a billion people each year (World Bank, 1992). Thousands of outbreaks of waterborne diseases are caused by the consumption of untreated or improperly treated drinking water (Ford and Colwell, qtd. in Payment and Hunter, 2001).

Water and sanitation-related diseases are transmitted through many pathways, and can be classified into four categories: (i) waterborne diseases, caused by the ingestion of water contaminated by human or animal feces or urine containing pathogenic bacteria or viruses; (ii) water-washed diseases, caused by poor personal hygiene; (iii) water-based diseases, caused by parasites found in intermediate organisms living in water; and (iv) water-related diseases, transmitted by insect vectors that breed in water (Eisenberg et al., 2001). Examples of these diseases are listed in Table A.1.

Category	Disease
Waterborne diseases	Cholera, typhoid, amoebic and bacillary dysentery, and other diarrheal diseases
Water-washed diseases	Scabies, trachoma and flea-, lice-, and tick-borne diseases, in addition to the majority of waterborne diseases, which are also water-washed
Water-based diseases	Dracunculiasis, schistosomiasis, and some other helminths
Water-related diseases	Dengue, filariasis, malaria, onchocerciasis, trypanosomiasis, and yellow fever

Table A.1. Examples of water and sanitation-related diseases (Bradley, qtd. in Eisenberg, 2000)

The direct health consequence of poor water supply and sanitation is huge. According to the World Health Organization (WHO), approximately one child dies every eight seconds from a water-related disease, and more than 5 million people died each year from illnesses linked to unsafe drinking water or inadequate sanitation (Anon, qtd. in Payment and Hunter, 2001). "Unsafe water is implicated in many cases of diarrheal diseases, which, as a group, kill more than 3 million people, mostly children, and cause about 900 million episodes of illness each year. At any one time more than 900 million people are afflicted with roundworm infection and 200 million with schistosomiasis. Many of these conditions have large indirect health effects – frequent diarrhea, for instance, can leave a child vulnerable to illness and death from other causes" (World Bank, 1992).

Children, the poor, and travelers are most at risk of water and sanitation-related diseases, due to undeveloped or degraded immunity for disease-causing environmental pathogens. Children under 5 years of age are the most vulnerable population, because they are "in a dynamic state of growth" (WHO, "Children"). Also, children are "more exposed to unhealthy conditions and to dangerous substances because they live their lives closer to the ground and, especially in the early years, they are frequently exposed through hand-to-mouth activities" (WHO, "Children"). People from low-income areas are more likely to suffer disease due to increased exposure to pathogens from poor living conditions, and are likely to suffer more severely, once affected by disease, "because of inadequate health-care and social support systems, and from poorer general health due to malnutrition" (Eisenberg et al., 2001). Therefore, not surprisingly, poor children suffer the most, and approximately "one in five children in the poorest parts of the world will not live to their fifth birthday, mainly because of environment-related diseases" (WHO, "Children"). For the third group of vulnerable population, travelers, the risk of infection is higher because they are exposed to new environmental pathogens, to which they do not have acquired immunity due to prior exposure. Most waterborne pathogens have acquired immunity, the protection conferred to a host after exposure to the agent of disease, that is partial and temporary (Eisenberg et al., 2001).

The reduction in water-related illnesses with improvements in water and sanitation is large. “WHO suggest that if sustainable safe drinking water and sanitation services were provided to all, each year there would be 200 million fewer diarrheal episodes, 2.1 million fewer deaths caused by diarrhea, 76,000 fewer dracunculiasis, 150 million fewer Schistosomiasis cases and 75 million fewer trachoma cases” (Payment and Hunter, 2001). The effects of improved water and sanitation on the occurrence of related illnesses, studied by the U.S. Agency for International Development (USAID), is summarized in Table A.2, and the effects on the morbidity from diarrhea, studied by WHO, is summarized in Table A.3 below. The WHO analysis suggests that the effects of making several kinds of improvements at the same time are roughly additive (Esrey at al., qtd. in World Bank, 1992).

Disease	Millions of people affected by illness	Median reduction attributable to improvement (%)
Diarrhea	900/year	22
Roundworm	900	28
Guinea worm	4	76
Schistosomiasis	200	73

Table A.2. Effects of improved water and sanitation on water and sanitation-related illnesses (Esrey et al., qtd. in The World Bank, 1992)

Type of improvement	Median reduction in morbidity (%)
Quality of water	16
Availability of water	25
Quality and availability of water	37
Disposal of excreta	22

Table A.3. Effects of improved water and sanitation on morbidity from diarrhea (Esrey et al., qtd. in The World Bank, 1992)

Some epidemiological evidence suggests that improvements in sanitation are at least as effective in preventing disease as improved water supply (UNICEF et al., 2000). The improvement in wastewater treatment and disposal interrupts the transmission of much fecal-oral disease at its most important source by preventing human fecal contamination of water and soil.

Water and sanitation-related diseases are prevalent in Brazil, where the delivery of drinking water and sanitation services falls far short of the goal of universal coverage. In Brazil, approximately 75% of the total population is served with domestic water connection, and 48% is served with a connection to public sewer system. Among the urban population, which accounts for 78% of the Brazil's total population of 162 million, 91% is served with domestic water connection, and 59% is served with connection to public sewer system. Among the rural population, 20% has domestic water connection, and mere 6% has connection to public sewer system. More alarmingly, only 10% of the total volume of sewage collected from the sewerage systems receives treatment (CEPIS, 2000).

The prevalence of water and sanitation-related diseases, which corresponds to low water and sanitation service coverage, in Brazil is considerable. As much as 32% of all hospital admissions in 1990, were due to diseases related to inadequate sanitation, according to a 1995 report from the Ministry of Planning and Budget of Brazil titled 'Assessment of the Sanitation Sector: Economic and Financial Study' (Csillag and Zorzetto, 2000). This report revealed that as many as 4.5 million hospital admissions, registered by the Ministry of Health from 1987 to 1992, were caused by sanitation-related diseases. The main group of diseases, labeled "poorly defined enteric infection," caused 92% of the cases, and the remaining 8% comprised what are labeled as "other specific enteric infections," as well as typhoid fever, shigellosis, Schistosomiasis, and amebiasis. Furthermore, this report remarked that infant mortality is two times higher in households with inadequate sanitation than in households with adequate sanitation, revealing a strong correlation between limited service coverage of water and sanitation and poor public health.

A.2 Environmental Quality Consideration

In addition to losses in human health, there are many costs related to environmental degradation, such as losses in productivity, amenity, and the intrinsic value of the environment. The productivity includes both the human productivity that can be lowered by impaired health, and the productivity of many resources, used directly or indirectly by people, that can decline with damage imposed by those uses (World Bank, 1992). Amenity is “a term that describes the many other ways in which people benefit from the existence of an unspoiled environment” (World Bank, 1992). The “intrinsic” value of the environment is separate from its value to human beings, that can only estimated under the notion of amenity values.

The quality of many surface water bodies – such as rivers, streams, and beach waters – have economic values, as fisheries and/or recreational waters, aesthetic value that can add to quality of life, and the intrinsic value, all of which depend on the state of water and sanitation systems.

A.3 Economic Consideration

Water is an economic good, with many competing uses, that can be a driving force for social and economic development. In countries where tourism is an important contributor of foreign exchange and employment, the preservation of attractive environment, through proper management of sanitation infrastructure and wastewater treatment facilities, becomes critical for the development of the industry. Polluted environment, such as a beach contaminated with human wastes, and its associated health risks for tourists and local population can easily pose a threat for the development and survival of the tourism industry (San Martin, 2002).

Tourism

Tourism contributes significantly to the economies of developing countries by achieving “three high-priority goals of developing countries: the generation of income, employment, and foreign exchange earnings” (San Martin, 2002). Tourism, classified as exports, accounts for a significant portion of the GDP earnings in the Latin American and Caribbean countries, although this portion is not fully reflected in the domestic income and product accounts of most countries. In Brazil, tourism accounts for approximately 4% of total exports (World Bank, qtd. in San Martin, 2002). In 1997, the Brazilian exports totaled US$ 53 billion (BIT, n.d.). Thus, tourism accounted for approximately US$2 billion of exports in 1997.

Tourism, which does not require sophisticated technology or much skilled training, is a great generator of employment and income. “[In] hotels, which account for about 75 percent of tourism employment (distribution, transport, finance and insurance, and entertainment make up the other 25 percent), [e]very room in a three- or four-star hotel generates one job, for five-star hotels, each room creates 1.3 jobs” (San Martin, 2002). “Even before the 1990s, one job generated by a hotel generated one more job elsewhere in the tourism trade and two in the rest of the economy; thus one job generated an estimated three others” (IDB, qtd. in San Martin, 2002). “It is estimated that in the Latin American and the Caribbean five-star hotels can generate US$5.4 for each dollar spent in their operation. The figure for three- and four-star hotels averages US$4.2” (San Martin, 2002).

A.4. Social Consideration

In addition to the economic contributions, there are important social contributions associated with water and sanitation, the most significant of which, aside health, is poverty alleviation.

Poverty Alleviation

Water and sanitation infrastructure can promote poverty alleviation by: (i) stimulating economic growth; (ii) converging the poor and rich regions within a country; (iii) increasing agricultural productivity through by improving irrigation; and (iv) improving the health and productivity. “It has been estimated that in Latin America, a 1 percent growth in per capita income reduces the share of the people living in poverty by half a percentage point” suggesting that “any contribution of infrastructure to growth will therefore have a poverty alleviation effect” (San Martin, 2002). “In Argentina and Brazil, recent studies show that lack of access to sanitation and to roads over the last 20 years have been important impediments to convergence [between the poor and the rich regions]” (San Martin, 2002). “With large percentage of the population employed in agriculture in the low-income economies of Latin American countries, investments in irrigation and agriculture more generally and improvements in water management, in particular, can have substantial impacts on rural poverty alleviation” (San Martin, 2002). “Extending coverage rates for water supply and sanitation will affect the living conditions of the poor via better health, and increased potential labor productivity; through considerable cash savings (since their supplies must often be bought extensively, from water trucks, bottled water, etc.); and through reduced time use in bringing the water to the household" (San Martin, 2002).

A.5. Institutional Framework in Brazil

Much of the water and sanitation sector in Brazil currently follows the PLANASA (Plano Nacional de Saneamento) model, which is responsible for 80% of water supply and 32% of sewage services for the urban population. Created in 1971, with the goals of improving water supply and sanitation services, PLANASA required each State in Brazil to create its own State-owned public company, from which the municipalities were able to contract services for water and sanitation. The municipalities had the choice of awarding concession contracts to the public company or establishing their own public services, a right granted by the Brazilian Constitution. However, the Federal National Bank of Housing (Banco Nacional de Habitao), under the Ministry of the Interior, did not finance water and sanitation works unless the municipality had joined PLANASA. Although the Federal National Bank of Housing, and PLANASA were abolished in 1986, the PLANASA model remains operational as the backbone of water and sanitation sector in Brazil (US Dept. of Commerce, 1999).

As the concession period from the municipalities to the State companies reaches their end, changes are actively sought. The State companies had shown inadequate performance and low productivity in many cases and had left many consumers, who often viewed their services as unreliable, discontent with their services. The State companies had some typical and common problems, which the World Bank classified into four groups: (i) technical and operational, (ii) commercial and financial, (iii) human and institutional, and (iv) environmental problems. The municipalities are looking for new models or for a new role of the State in providing public services, with the emphasis on decentralization and privatization, as it has occurred in other parts of the world. The service contracts (for pumping stations, sewage treatment plants, metering and reading, for example), and the discussion of private sector participation are becoming more common (US Dept. of Commerce, 1999).

APPENDIX B – WATER QUALITY ANALYSIS

B.1. Water Quality of Potable Waters

City's Potable Water

The quality of City's potable water is heavily influenced by the quality of surface waters, from which it is derived, and thus is highly variable. As surface waters often do, the City's potable water quality often falls substandard due to high turbidity after rainstorms, and bacterial contamination. The following is the description and analysis of the City's potable water quality.

In order to characterize the quality of water that people drink in Paraty, and the seriousness of the water quality degradation of the source waters, samples of these waters were collected from numerous locations and tested. Some of the parameters measured are pH, turbidity, suspended solids, free chlorine (potable water only), chemical oxygen demand (ambient waters only), total coliform, and fecal coliform.

<u>Sampling Locations</u>

From January 10 to January 23, 2003, twenty-seven samples of potable water were collected from four locations in and near the City of Paraty. The first sampling location, "Caboclo" was an opening at the top of the city's reservoir, through which the raw water from Caboclo discharged into the reservoir from the end of a 3,000 m pipe. Eight samples of raw water from Caboclo were collected at this location. The second sampling location, "Pedra Branca" was a dam located at the Pedra Branca intake, where five samples of raw water from Pedra Branca were collected before the water was chlorinated. The third sampling location, "Reservoir" was a tap located adjacent to the reservoir, which combined and chlorinated the waters from Pedra Branca and Caboclo. The chlorinated reservoir water was sampled eight times at this location. Finally, the sampling location "Tap Water" was a tap located in the Historical Center of Paraty, supplied with chlorinated water from the reservoir. The tap water was sampled six times, during five times of which the pH, turbidity, suspended solids, and total and fecal coliform bacteria were measured. The free chlorine was measured for only four of the six samples.

Water Quality Parameters

The World Health Organization (WHO) specifies acceptable values of various drinking-water parameters that could be used to gauge the quality of a water sample (WHO, "Health criteria," 1998). Some of these parameters are listed in Table B.1 below:

Parameter	Acceptable Level	Reason
PH	6.5-8	Low pH: corrosion high pH: taste, soapy feel, preferably <8.0 for effective disinfection with chlorine
Turbidity	<5 NTU	Appearance; median turbidity 1 NTU for acceptable terminal disinfection
Total coliform bacteria	None detectable	
Fecal coliform bacteria	None detectable	
Residual free chlorine	0.5 mg/l	Effective disinfection

Table B.1. Criteria for acceptable drinking water quality

pH

The waters from all four locations had satisfactory pH, their median pH ranging from 6.6 to 7.0, as illustrated in the Figure B.1. Although this range is acceptable, the pH of tap water measured as low as 5.7, suggesting that the water is slightly acidic and likely to be corrosive. Although there is no health-based guideline proposed for pH, the optimum pH value suggested by the WHO is in the range 6.5-8. It has been shown that the pH should be less than 8 so that chlorination is effective, but greater than 6.5 to prevent corrosion of water mains and pipes in household water systems, which could lead to the contamination of water.

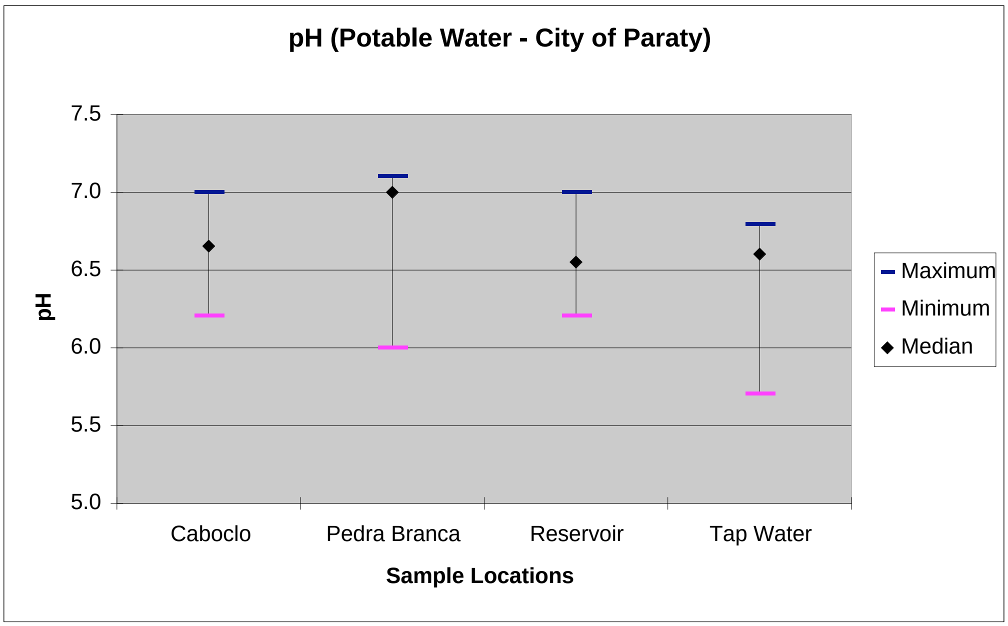


Figure B.1. pH of the potable water in the City of Paraty

Turbidity

The turbidity of the waters, especially at the tap, was highly variable and unsatisfactory. Of the four sampling locations, only Caboclo had water with turbidity lower than 5 NTU (See Figure B.2). Turbidity is a water quality that refers to the cloudy appearance of water that is caused by particles or suspended matter that can adsorb harmful contaminants. Although turbid water is not necessarily harmful, it can be an indicator of more serious problems. The turbidity of 5 NTU is the criteria to avoid filtration, and also the threshold of consumer disapproval (WHO, "Health criteria," 1998). With regard to effective disinfection, an even lower level of turbidity of 1 NTU is recommended. Therefore, needless of much discussion, it is clear that the waters in Paraty are unsuitable for safe drinking.

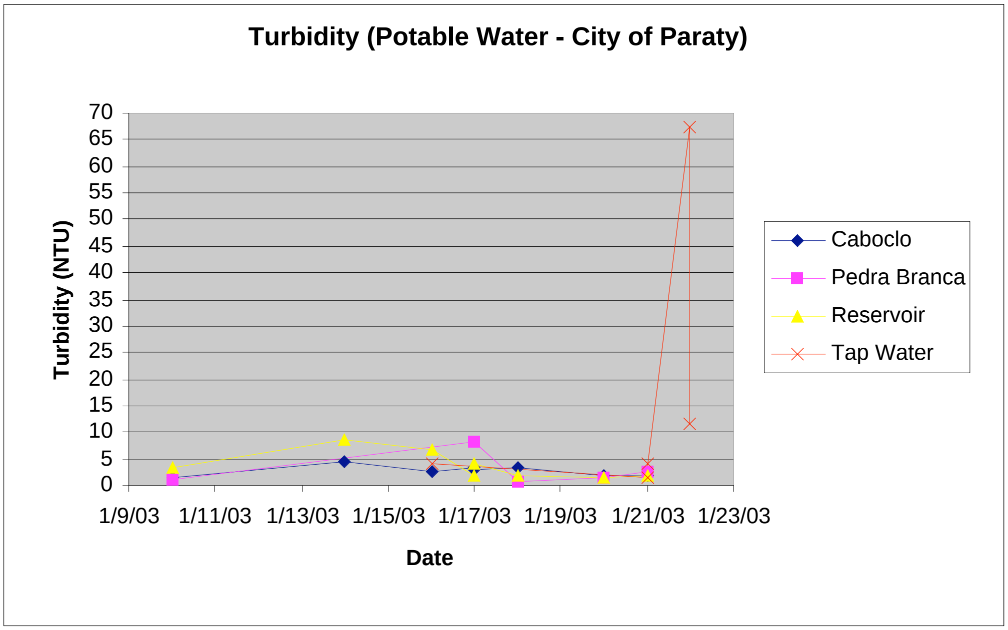


Figure B.2. Turbidity of the potable water in the City of Paraty

<u>Residual Free Chlorine</u>

The concentration of residual free chlorine in tap water, also highly variable, was sometimes lower than 0.5 mg/l, which is the recommended concentration for effective disinfection (See Figure B.3). Four samples of tap water were tested for the concentration of residual free chlorine. As illustrated in the Figure 3.4, on Jan. 22nd, the residual free chlorine concentration was dangerously close to 0 mg/l.

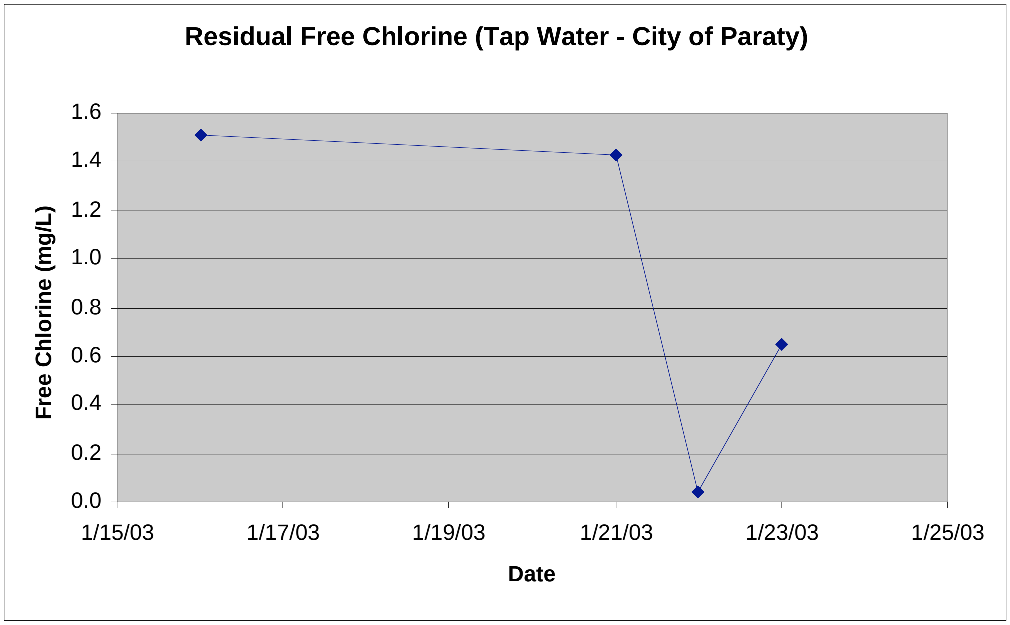


Figure B.3. Residual free chlorine in the tap water in the City of Paraty

According to this data the sudden drop of the concentration of residual free chlorine corresponded to the sudden increase of turbidity in tap water, and the sudden jump of the concentrations of total coliform and fecal coliform were the consequences of this drop in the free chlorine concentration (See Figure B.4). The turbidity of the tap water had values below 5 NTU until the January 22, when it suddenly rose to 68 NTU. (Unfortunately no samples were collected from other locations to enhance this data.) At the same time, the residual free chlorine in this sample dropped to an almost undetectable concentration of 0.04 mg/l. Accordingly on this day, the concentration of fecal coliform peaked at 420 colonies/100ml, and the total coliform at greater than 2,400 colonies/100ml.

There are two plausible causes for the sudden decrease of residual free chlorine concentration in the tap water. First, the sudden increase of suspended solids and organic particles in the water, following rainstorms, could have increased the chlorine demand in water, dramatically reducing the residual free chlorine and consequently causing the disinfection to be ineffective. Second, the chlorine addition could have been overdue at the reservoir. One thing is clear: the current method of chlorination, which fails to account for the inconsistencies in flow rate and chemical composition of the highly variable surface water, is ineffective and unreliable.

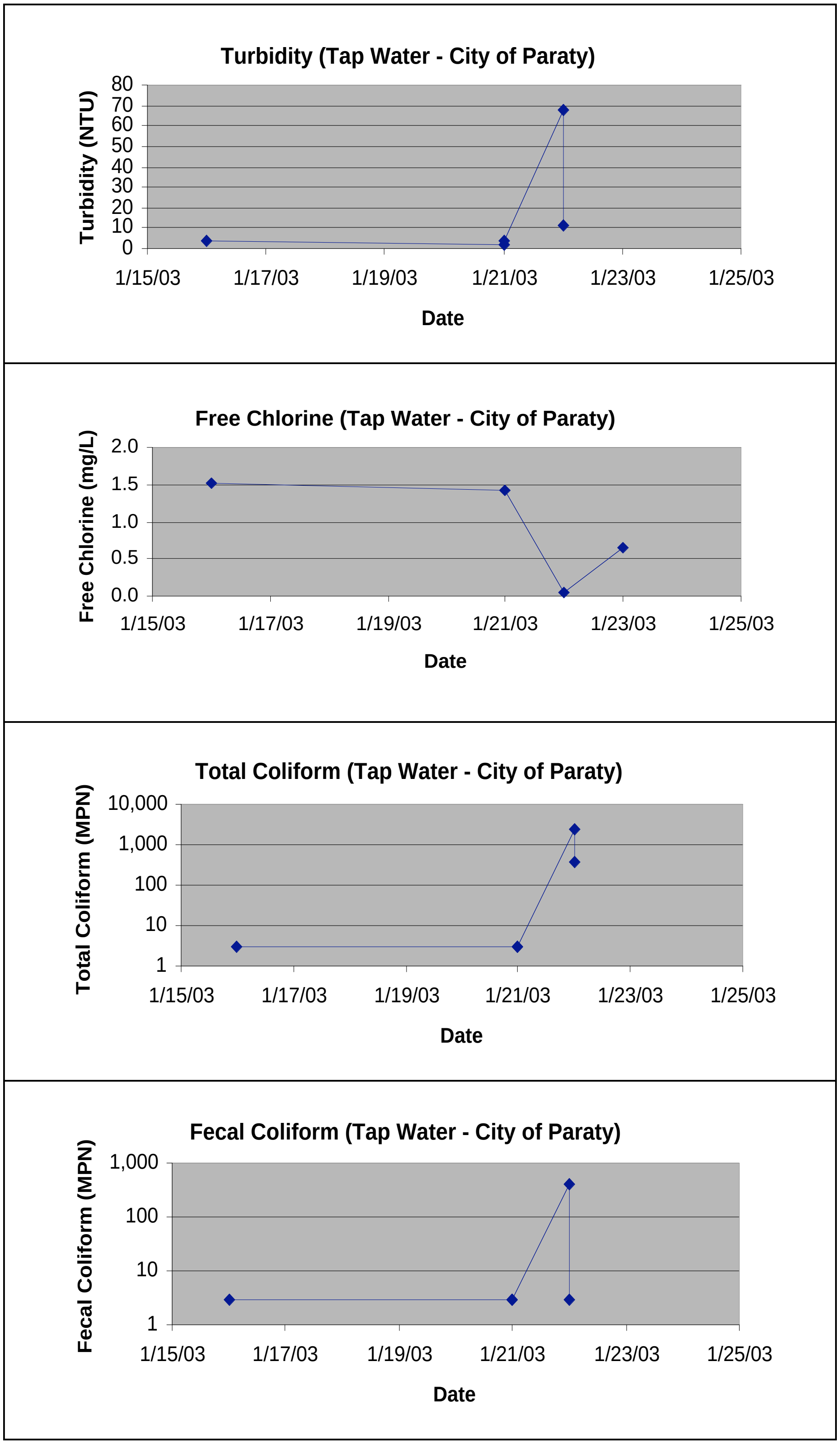


Figure B.4. Correlation of turbidity, residual free chlorine, total coliform, and fecal coliform for the tap water in the City of Paraty

Total Coliform and Fecal Coliform Bacteria

The source water samples from Caboclo and Pedra Branca, had the total coliform concentration of approximately 2,400 colonies/100ml, or greater, and the fecal coliform concentration that was approximately 1/10 of the total coliform concentration (See Figure B.5). It is clear from this data that the potable water sources are heavily contaminated and that they must be disinfected for safe ingestion. The international drinking water standards require that no fecal coliform bacteria be detectable in any 100 ml sample, for all water intended for drinking. In addition, there must not be any total coliform bacteria detectable in any 100 ml sample of treated water entering a distribution system. However, neither the reservoir water sample, collected immediately after disinfection, nor the tap water sample, collected at the end of the distribution system, complied with these standards. The reservoir water samples consistently had detectable concentrations of total coliform, as well as detectable concentration of fecal coliform on January 20. The tap water samples showed significant concentrations of total coliform and fecal coliform bacteria on January 22.

Summary

This water quality analysis not only asserts that City's present method of disinfection is ineffective, but also that filtration of drinking water before disinfection is necessary in order to remove suspended particulate matter, and the harmful pathogens adsorbed on those particles, from water. The turbidity in drinking water that rises as high as 68 NTU makes filtration obligatory. Chlorination, a method of disinfection that kills organic contaminants in water through the oxidizing ability of chlorine, is ineffective against hard-shelled cysts like those produced by Cryptosporitium, although it can effectively treat biological pathogens like coliform bacteria and lelegionella. Filtration, a method of disinfection, physically removes biological contaminants present in water. The benefits of drinking water filtration are extensive and include: (i) removal of suspended particulate matter; (ii) disinfection by the removal of harmful pathogens adsorbed on those particles; and (iii) reduction of disinfection by-products by the removal of natural organic matter, which are their precursors.

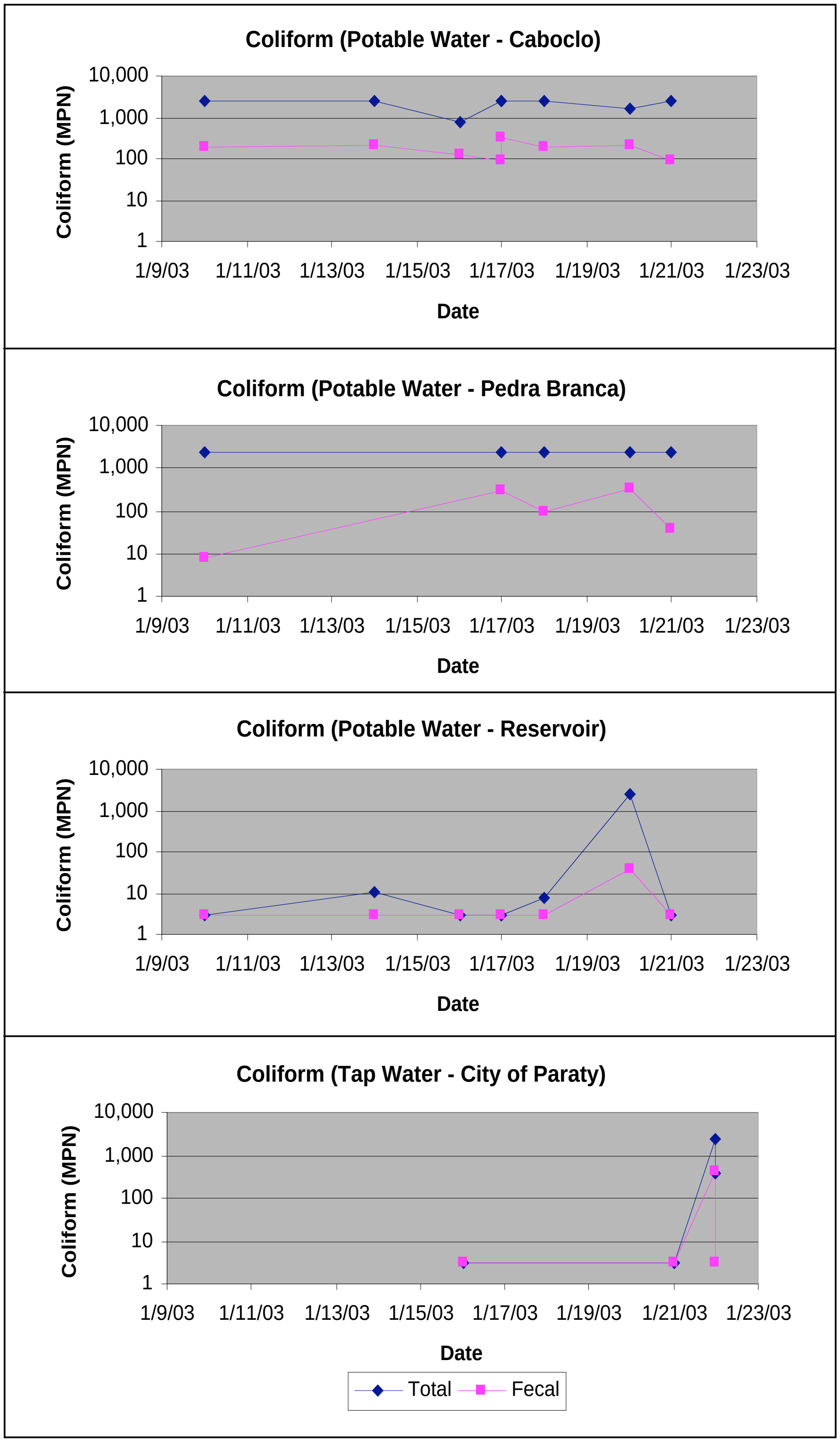


Figure B.5. Total and fecal coliform in the potable water in the City of Paraty

Municipality's Potable Water

<u>Additional Sampling Locations</u>

The drinking water sources for numerous rural communities, in addition those for the City of Paraty, were sampled and tested for similar physical characteristics and microbial contamination. The communities, from where the drinking water sources were sampled, are: Agua Fria, Barra Grande, Corisco, Patrimonio, Sao Goncalo, Sao Roque, Taquari, Tarituba, and Trindade (See Figure 3.2).

The water quality of potable waters used in the rural communities was measured by the same standards used to gauge the potable water quality in the City. In general, the waters in the rural communities had pH within the 6.5-8.0 range, and turbidity less than 5 NTU (See Figure B.6 and B.7). By these parameters, the potable waters in the rural communities were superior to the water in the City. However, these waters had high concentrations of total coliform and fecal coliform bacteria, which made them unsafe to drink (See Figure B.8 and B.9). None of these waters were disinfected. The results of water quality analysis, for the city and the rural communities in the Municipality of Paraty, are summarized in Table B.2:

Community	No. of Households	Water Quality Parameters					Conclusion
		Treatment	pH	Turbidity	Total Coliform	Fecal Coliform	
City of Paraty	3850	Chlorination	Low	High	Present	Present	Unsatisfactory
Agua Fria		None	Normal	Normal	Present	Present	Unsatisfactory
Barra Grande	226	None	Normal	Normal	Present	Present	Unsatisfactory
Corisco	200	None	Normal	Normal	Present	Present	Unsatisfactory
Patrimonio	125	None	Normal	Normal	Present	Present	Unsatisfactory
Sao Goncalo	100	None	Normal	Normal	Present	Present	Unsatisfactory
Sao Roque	250	None	Normal	High	Present	Present	Unsatisfactory
Taquari	300	None	Normal	Normal	Present	Present	Unsatisfactory
Tarituba	107	None	Normal	Normal	Present	Present	Unsatisfactory
Trindade	250	None	Normal	Normal	Present	Present	Unsatisfactory

Table B.2. Drinking water quality results for the City and rural communities in the Municipality of Paraty

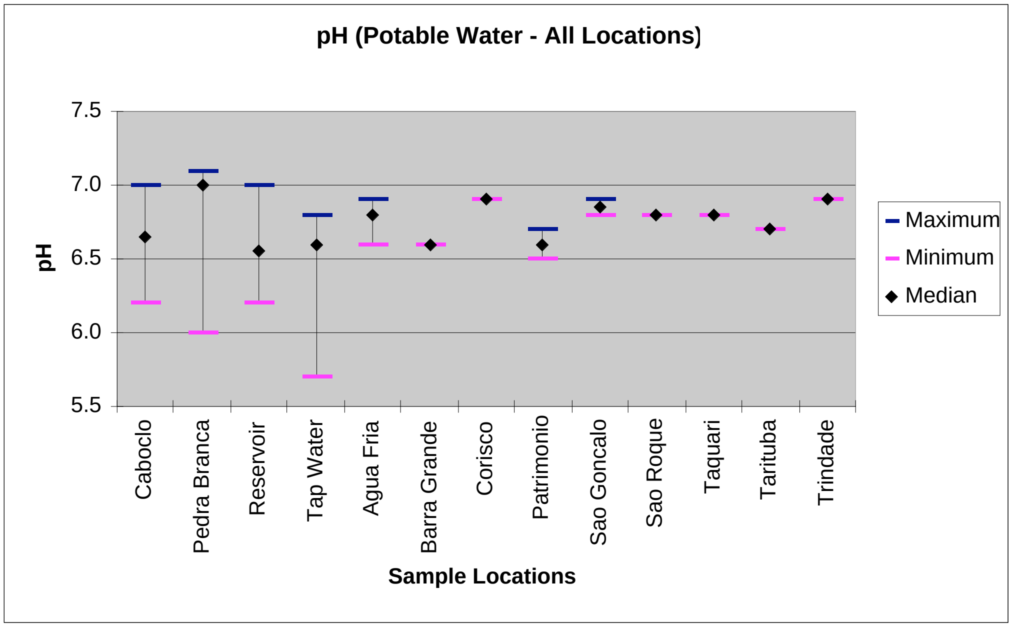


Figure B.6. pH of potable waters in the Municipality of Paraty

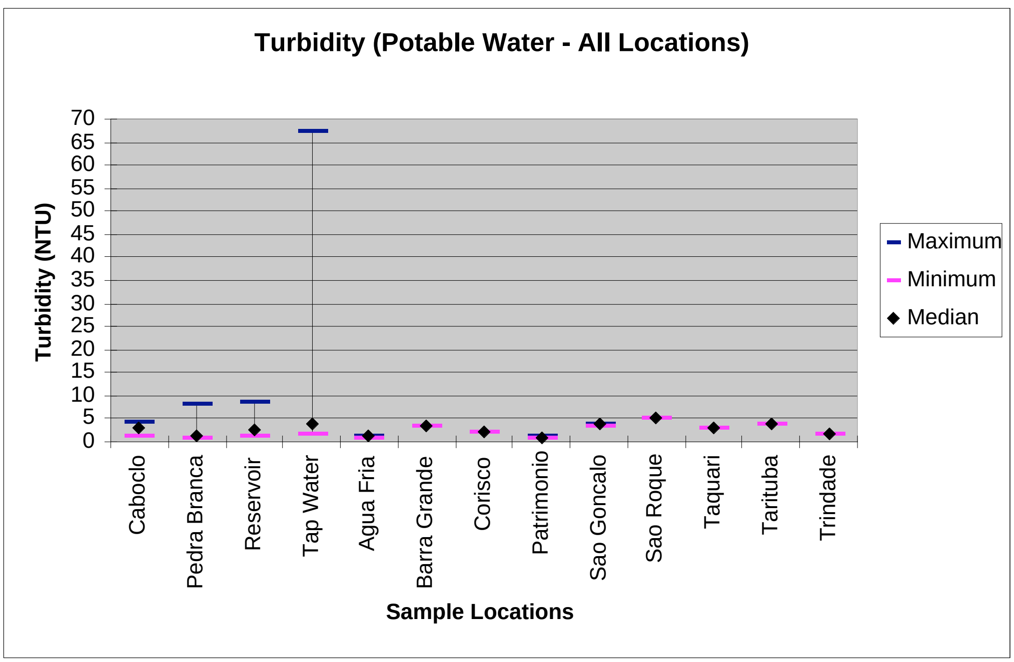


Figure B.7. Turbidity of potable waters in the Municipality of Paraty

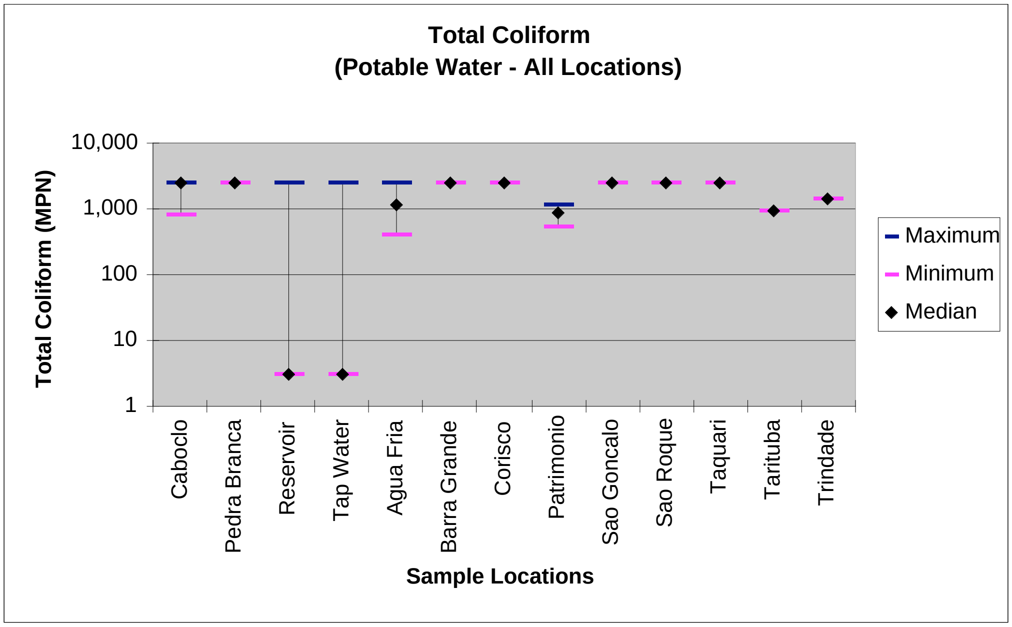


Figure B.8. Total coliform of potable waters in the Municipality of Paraty

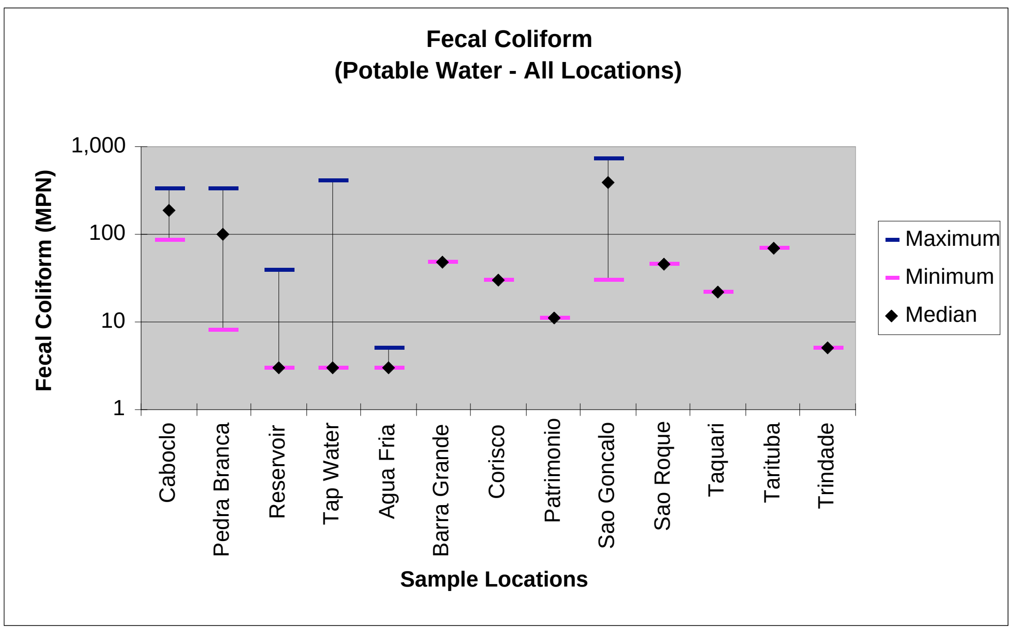


Figure B.9. Fecal coliform of potable waters in the Municipality of Paraty

Other Water Quality Tests

The Municipality of Paraty determined, from a series of laboratory tests that were performed in the past, that many water sources violated the drinking water standards and were in fact unsafe to drink (See Table B.3). Between October 2001 and March 2002, 44 samples of potable water were collected from various locations within the Municipality of Paraty. Three physical characteristics (turbidity, color, and odor), and the tests of total and fecal coliform bacteria were used to determine the quality of the water samples. Of the 44 samples, only 22 samples (17 from the City of Paraty, 3 from Pantanal, and 2 from Ponte Branca) had been chlorinated.

Of the 44 samples, 28 samples (64%) were determined to be of unsatisfactory quality by at least one of these parameters. Ten samples (23%) had high concentration of particulate matter; 1 sample (2%) had yellow color. No sample had any detectable odor. Twenty-five samples (57%) had total coliform bacteria, and 20 of these samples were contaminated with fecal coliform bacteria. The presence of total coliform bacteria, with 89% occurrence, was the principal cause for unsatisfactory water quality.

Of the 28 samples that had unsatisfactory water quality, 6 samples (21%) had been chlorinated for disinfection. Four out of the 6 chlorinated samples were declared unsatisfactory due to the presence of detectable amounts of coliform bacteria, revealing that the disinfection was not effective. Two samples from the City of Paraty had both total and fecal coliform bacteria present, and two had only total coliform present. Two more chlorinated water samples (collected from Pantanal and Ponte Branca) had no coliform bacteria, suggesting that the chlorination had been effective, but were declared unsatisfactory due to the high concentration of suspended solids.

Although 100% of potable water in the City of Paraty was chlorinated, 4 out of the 17 samples collected in the City (24%) were declared unsatisfactory, due to microbial contamination as well as high concentration of suspended solids.

Location	Date (d/m/y)	Time	Treatment	Turbidity/ Suspended Solids (ss)	Color	Odor	Total Coliform	Fecal Coliform	Conclusion
Barra Grande	17/10/01	15:30	None	high ss	none	none	present	present	not satisfactory
Barra Grande	23/01/02	15:55	None	clear	none	none	present	present	not satisfactory
Campinho	06/02/02	16:27	None	clear	none	none	present	present	not satisfactory
Campinho	19/12/01	16:10	None	clear	none	none	present	present	not satisfactory
Corisquinho	06/02/02	17:35	None	cloudy	none	none	present	present	not satisfactory
Corisquinho	19/12/01	17:20	None	clear	none	none	present	present	not satisfactory
Corumbe	17/10/01	16:05	None	clear	none	none	present	present	not satisfactory
Corumbe	23/01/02	16:25	None	clear	none	none	present	present	not satisfactory
Grauna	06/03/02	15:25	None	high ss	none	none	present	absent	not satisfactory
Grauna	17/10/01	15:45	None	clear	none	none	present	present	not satisfactory
Grauna	23/01/02	16:10	None	high ss	none	none	present	present	not satisfactory
Pantanal	06/02/02	18:15	Chlorination	clear	none	none	absent	absent	satisfactory
Pantanal	06/03/02	14:56	Chlorination	clear	none	none	absent	absent	satisfactory
Pantanal	17/10/01	16:28	Chlorination	high ss	none	none	absent	absent	not satisfactory
Paraty City	17/10/01	13:30	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	06/02/02	18:50	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	19/12/01	18:35	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	19/12/01	18:50	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	23/01/02	16:43	Chlorination	clear	none	none	present	present	not satisfactory
Paraty City	06/02/02	18:25	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	19/12/01	18:25	Chlorination	high ss	none	none	present	absent	not satisfactory
Paraty City	17/10/01	15:10	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	06/03/02	17:45	Chlorination	clear	none	none	present	absent	not satisfactory
Paraty City	17/10/01	15:00	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	23/01/02	15:35	Chlorination	clear	none	none	present	present	not satisfactory
Paraty City	06/03/02	17:10	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	23/01/02	17:05	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	06/02/02	18:40	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	06/03/02	14:35	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	17/10/01	14:35	Chlorination	clear	none	none	absent	absent	satisfactory
Paraty City	23/01/02	16:55	Chlorination	clear	none	none	absent	absent	satisfactory
Pastiba	19/12/01	18:00	None	high ss	none	none	absent	absent	not satisfactory
Patrimonio	06/02/02	16:34	None	clear	none	none	present	present	not satisfactory
Patrimonio	19/12/01	15:40	None	clear	none	none	present	present	not satisfactory
Pedras Azuis	06/02/02	16:12	None	clear	none	none	present	present	not satisfactory
Pedras Azuis	19/12/01	16:40	None	cloudy	yellow	none	present	present	not satisfactory
Ponte Branca	06/02/02	17:55	Chlorination	clear	none	none	absent	absent	satisfactory
Ponte Branca	19/12/01	17:40	Chlorination	high ss	none	none	absent	absent	not satisfactory
Taquari	06/03/02	15:45	None	clear	none	none	present	absent	not satisfactory
Taquari	18/10/01	7:00	None	clear	none	none	present	present	not satisfactory
Tarituba	06/03/02	16:00	None	clear	none	none	present	absent	not satisfactory
Tarituba	18/10/01	7:40	None	clear	none	none	present	present	not satisfactory
Trindade	06/02/02	16:55	None	clear	none	none	present	present	not satisfactory
Trindade	19/12/01	15:20	None	high ss	none	none	present	present	not satisfactory

Table. B.3. Results of drinking water quality analysis performed by the Municipality of Paraty from October 2001 to March 2002

Conclusion

Numerous water quality analyses reveal that many rural communities in the Municipality of Paraty, as well as the City of Paraty, consume drinking water that fails to comply with international drinking water regulations. Two principal causes of substandard water quality are high turbidity and bacterial contamination. The rural communities, which currently do not treat their drinking water, must disinfect their drinking water at the least, with chlorine addition for example.

The City of Paraty must adopt various measures to improve the quality of its drinking water. In addition to procuring a sufficient supply of drinking water to meet demand at all times, the City must better protect its drinking water at the sources, and treat the water by filtration and disinfection. The drinking water must be filtered in order to reduce the turbidity in water, which frequently rises to unacceptable levels after rainstorms, and a more precise method of chlorination must be adopted in order to make disinfection of drinking water more effective.

B.2. Water Quality of Surrounding Water Bodies

Surface water bodies near the City of Paraty are heavily polluted from human activities. In order to characterize the quality of these surface water bodies, samples were collected from numerous locations and tested. The following is the description and analysis of the surrounding surface water bodies in the City.

Sampling Locations

Water samples were collected from Jabaquara Beach, Matheus-Nunez River, and Pereque-Acu River (referred as "Jabaquara Beach," "Matheus River," and "Pereque River," respectively), and tested. Samples were also collected from an open ditch (designated "Sewer Stream") that carries raw sewage through Mangueira and discharges into the Paraty Bay. Jabaquara Beach water was sampled 11 times, at the knee level near the most populated places. Matheus River, Pereque River, and Sewer Stream waters were sampled 7, 9, and 4 times, respectively. The Matheus River water was sampled at the riverbank, near small boats. The Pereque River water was sampled from a bridge, at the center of the river's cross-section. The Sewer Stream water was sampled similarly at the middle of the cross-section, from a walkway crossing the ditch.

Water Quality Parameters

The water quality parameters tested are pH, turbidity, suspended solids, chemical oxygen demand (COD), total coliform, and fecal coliform bacteria concentrations. The water quality measurements for Jabaquara Beach samples are compared against surface water criteria for coastal waters designated for aquatic life, recreation, navigation, and industrial water supply (See Table B.4). Similarly, the water quality measurements for Pereque River and Matheus River samples are compared against surface water criteria for waters designated for aquatic life, recreation, navigation, and industrial and agricultural water supply (See Table B.5). The Sewer Stream samples, on the other hand, are compared to the raw sewage sampled in the City of Paraty (See Table B.6).

Beach	
	Coastal water standards, EPA Connecticut
Designated Use	Habitat for marine fish and other aquatic life and wildlife; shell fish harvesting; recreation; navigation; and industrial water supply
pH	6.8-8.5
Turbidity (NTU)	None other than of natural origin
Total suspended solids (mg/l)	None other than of natural origin
Fecal coliform bacteria (colonies/100 ml)	Geometric mean of 200/100 ml for summer primary contact recreation

Table B.4. Beach water quality criteria

River			
	Interim national river water quality standards, Malaysia	**Water quality constituents and standards, EPA Kansas**	**Surface water standards, EPA Connecticut**
Designated Use	Aquatic life; recreation	Aquatic life; recreation	Habitat for fish and other aquatic life and wildlife; recreation; navigation; and industrial and agricultural water supply
pH	6.0-9.0	6.5-8.5	6.5-8.0
Turbidity (NTU)	50	9.9	<5 NTU over ambient conditions
Total suspended solids (mg/l)	50		<10 mg/l over ambient conditions
COD (mg/l)	25		
Total coliform bacteria (colonies/100ml)	5,000		
Fecal coliform bacteria (colonies/100ml)	100	Geometric mean of 200/100 ml for summer primary contact recreation; 2000/100 ml for winter primary contact recreation or secondary contact recreation	

Table B.5. River water quality criteria

Raw Sewage	
pH	6.8
Turbidity (NTU)	128
Suspended solids (mg/l)	117
COD (mg/l)	412
Total coliform bacteria (colonies/100ml)	3,280,000
Fecal coliform bacteria (colonies/100ml)	460,000

Table B.6. Quality of raw wastewater in the City of Paraty (Kfouri and Kweon)

pH

All four water samples had pH near 7 (See Figure B.10). The samples from Jabaquara Beach and Pereque River fluctuated significantly from 6 to 8, while the samples from Matheus River and Sewer Stream stayed within the much narrower range of 6.5 to 7.4. The acceptable range of pH for beach waters is 6.8-8.5, and the pH of the water samples from Jabaquara Beach was at the lower end of this range. The more strict range of pH for the surface waters is 6.5-8.0. The samples from Matheus River and Sewer Stream were safely within this range, while the samples

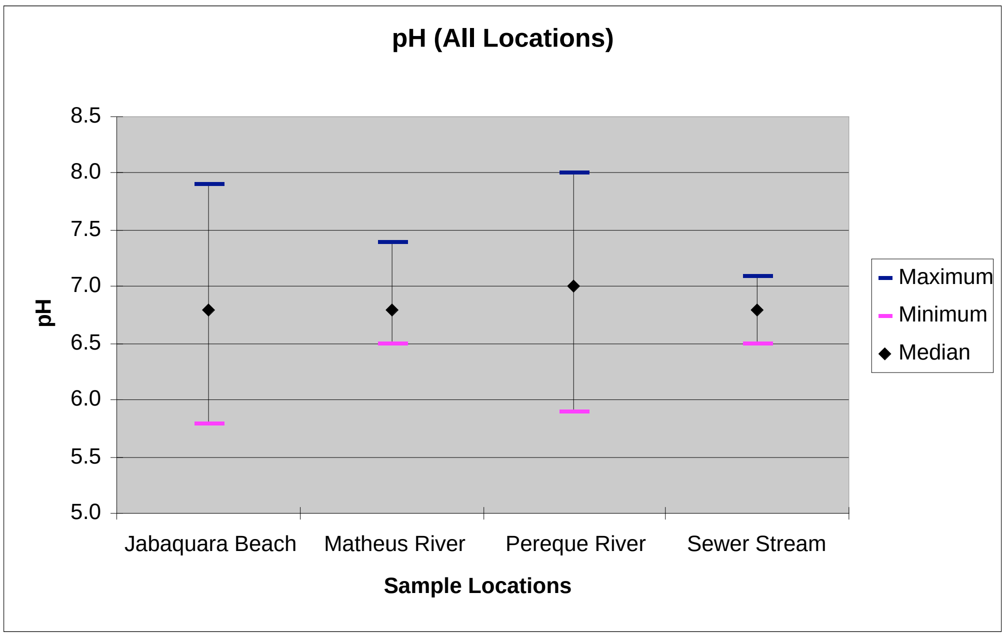


Figure B.10. pH of the surrounding water bodies

Turbidity

The turbidity of the waters from Jabaquara Beach, Matheus River, and Pereque River were safely below 50 NTU (See Figure B.11). The median turbidity was approximately 20 NTU for Jabaquara Beach, and approximately 10 NTU for Matheus River and Pereque River. The acceptable level of turbidity for safe aquatic life and recreation is approximately 10 NTU (State of Kansas). The median turbidity for the two rivers suggests that they are often, but not always, acceptable for safe aquatic life and recreation. The turbidity of Sewer Stream ranged from 30 NTU to 90 NTU, with the median of 41 NTU. The Sewer Stream had turbidity that is much higher than those of the rivers or the beach, but smaller than the turbidity of the raw sewage, which was 128 NTU.

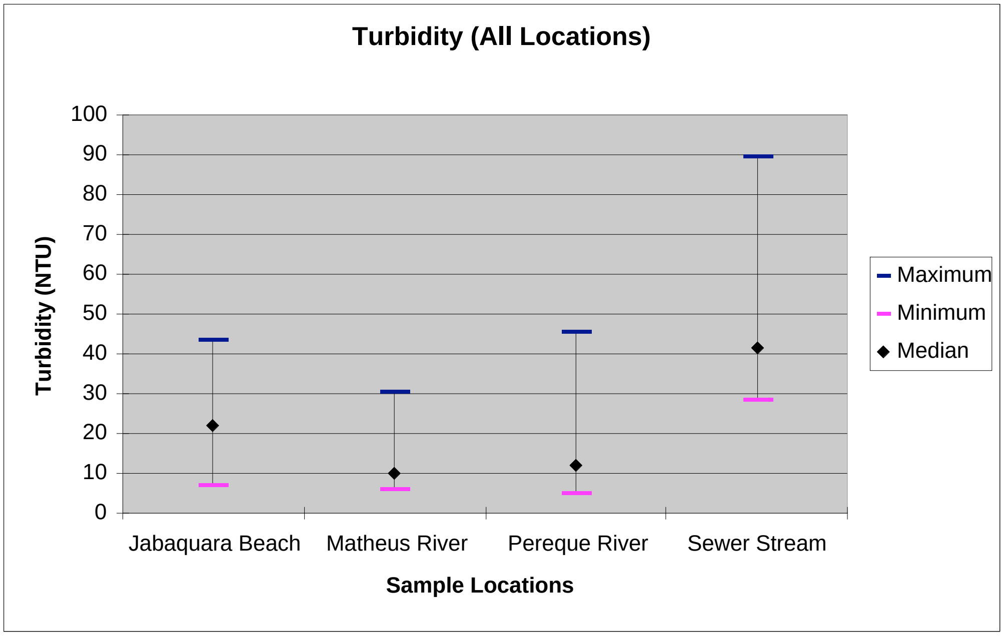


Figure B.11. Turbidity of the surrounding water bodies

Suspended Solids

Similar observations were made with the analysis of suspended solids in the water. The amounts of suspended solids in the water samples from Jabaquara Beach, Matheus River, and Pereque River ranged from 5 mg/l to 40 mg/l, with the median of approximately 20 mg/l. This level of suspended solids in water is acceptable for all aquatic life and recreational activities under Malaysian standards. The US EPA standards are more stringent and require that the suspended solids do not exceed 10 mg/l over the ambient condition. If the upstream river waters, Caboclo and Pedra Branca, which are also potable water sources, represent the "ambient condition," the ambient suspended solids concentration is approximately 5 mg/l, and could be as high as 10 mg/l. Therefore the median concentration of suspended solids in the surface waters must not exceed 15 mg/l, and the maximum concentration of suspended solids must not exceed 20 mg/l. According to these standards, Jabaquara Beach, Matheus River, and Pereque River water quality are unsatisfactory, with their single sample maximums of 42 mg/l, 29 mg/l and 38 mg/l. The Sewer Stream showed levels of suspended solids that are unacceptable for aquatic recreation, indicating heavy contamination from domestic sewer discharge (See Figure B.12). The raw sewage had approximately 120 mg/l of suspended solids, and the samples of Sewer Stream had approximately 56 mg/l of suspended solids, that could be as high as 102 mg/l.

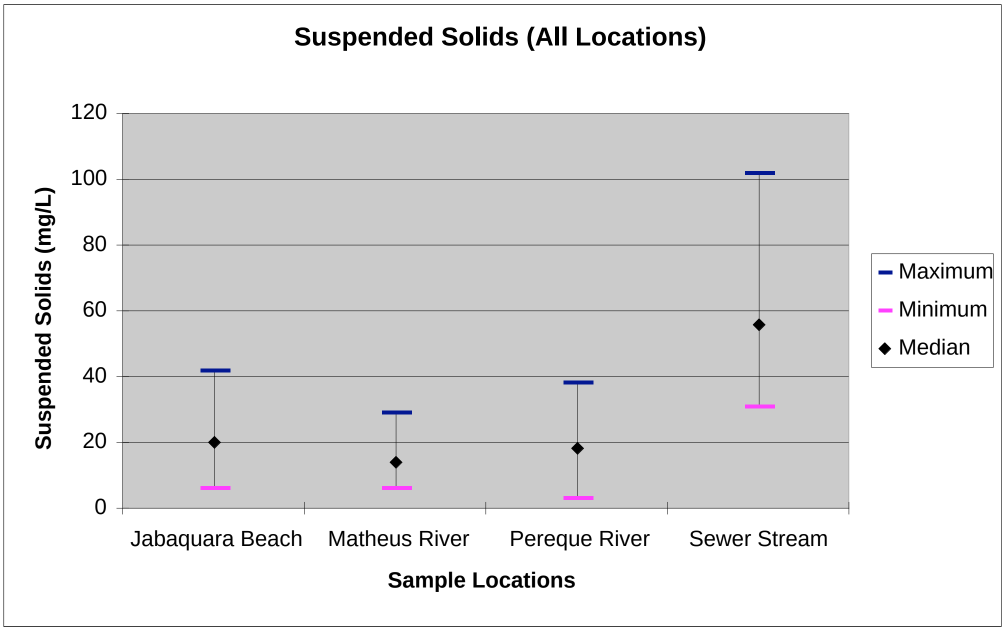


Figure B.12. Suspended solids of the surrounding water bodies

As the similar values of turbidity and suspended solids suggest, there is a strong correlation between turbidity and suspended solids (See Figure B.13 and B.14). The correlation can be explained by the fact that both turbidity and suspended solids were measured using photometric method, which measures the amount of light scattered by the impurities present in water.

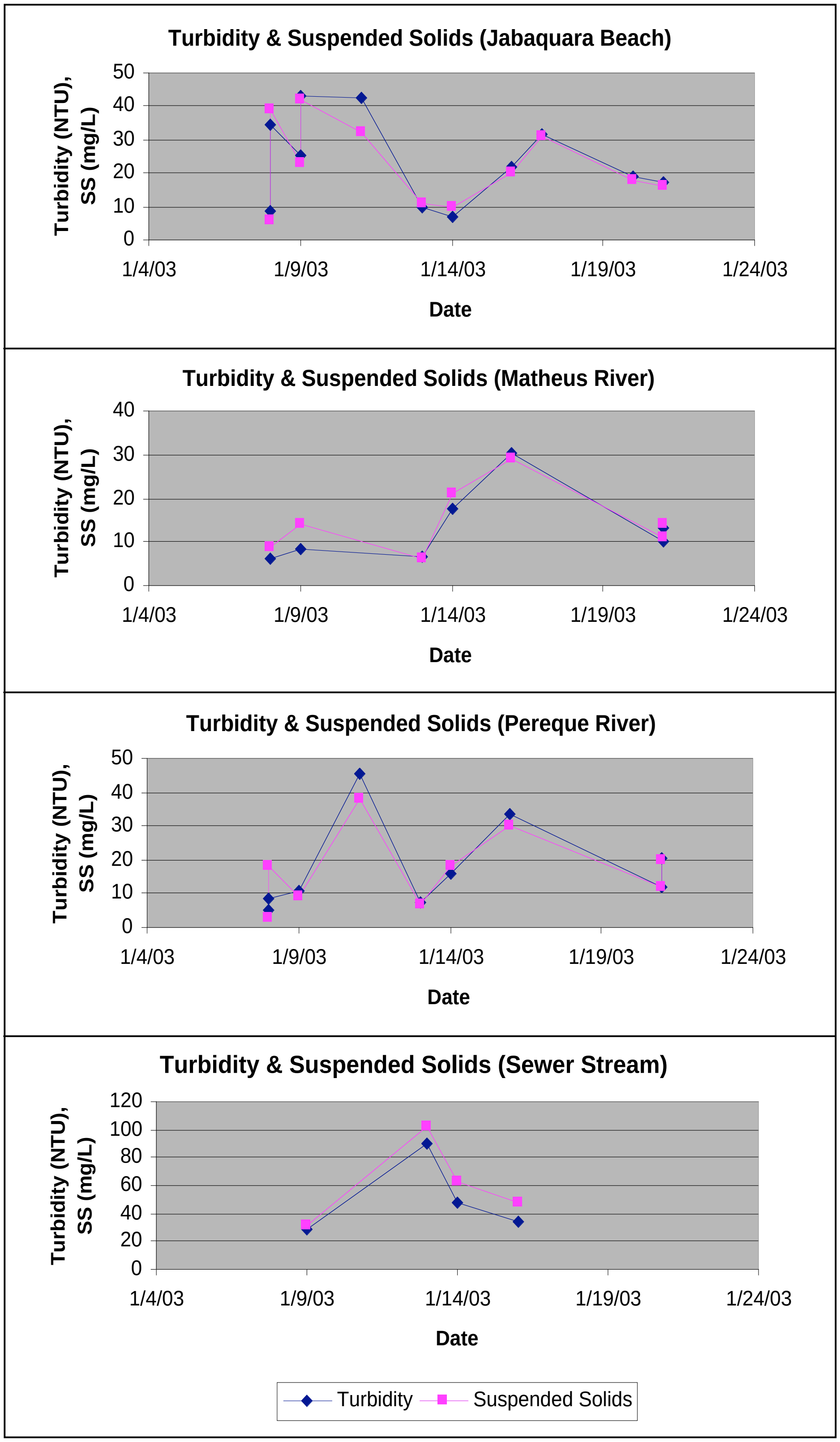


Figure B.13. Correlation of turbidity an suspended solids

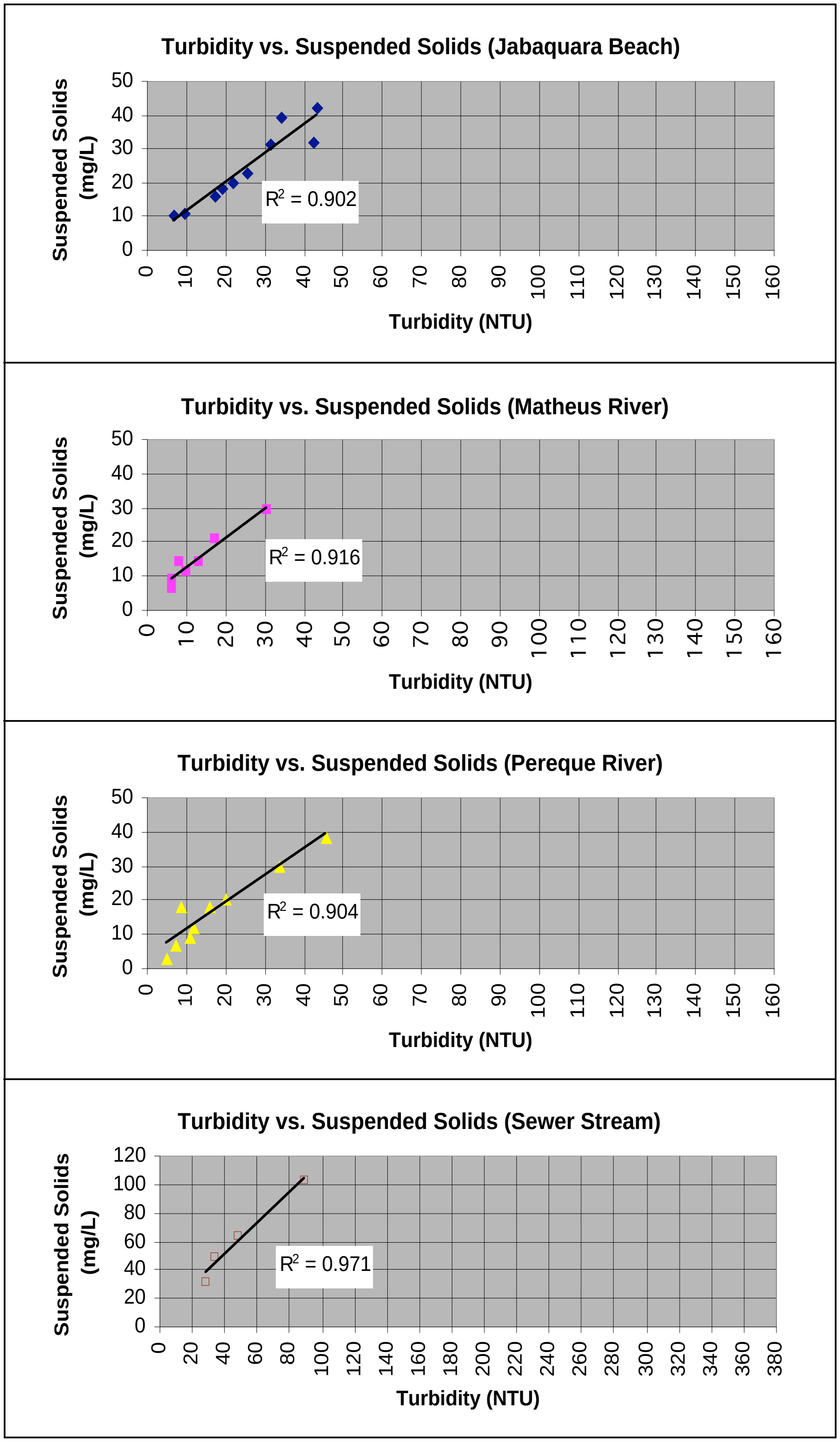


Figure B.14. Correlation of turbidity and suspended solids

COD

All water samples had unacceptable levels of chemical oxygen demand (COD). According to the Malaysian river water quality standards, maximum COD level for aquatic life and recreational activities is 25 mg/l. However, the median COD concentrations in water samples from Jabaquara Beach, Matheus River, Pereque River, and Sewer Stream are 120 mg/l, 85 mg/l, 21 mg/l, and 280 mg/l, respectively (See Figure B.15). The maximum COD level in Matheus River is as high as 800 mg/l, most likely due to oil spills from small boats anchored at the riverbank. Although the median COD level in Pereque River is less than 25 mg/l, its maximum COD level is as high as 230 mg/l. The COD level in Sewer Stream is a bit lower than that of raw sewage, which is approximately 400 mg/l. The US EPA does not list maximum COD level acceptable for aquatic life because the dissolved oxygen (DO) is deemed more applicable.

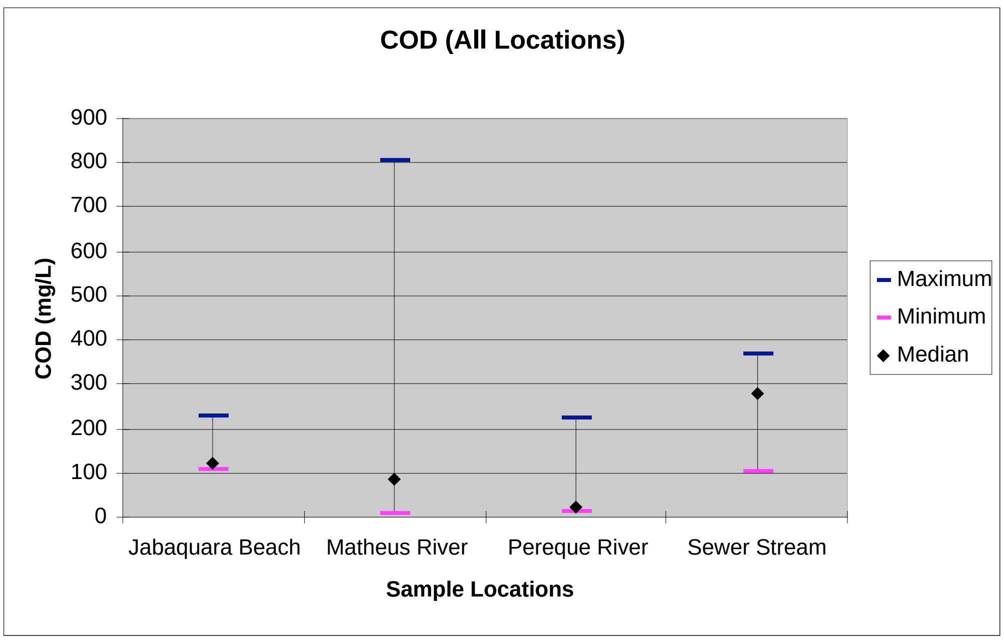


Figure B.15. COD of the surrounding water bodies

Fecal Coliform bacteria

The concentration of fecal coliform bacteria is an important water quality parameter in determining the feasibility of the intended uses of the water bodies, especially for those water bodies intended for primary contact recreation. Primary contact recreation is defined as when the body is immersed in surface water to the extent that some inadvertent ingestion of water is probable such as boating or swimming. Secondary contact recreation is defined as recreation where ingestion of the surface water is not probable such as wading, fishing, or hunting (KDHE, 2001). A geometric mean of 200 colonies/100ml of fecal coliform is acceptable for waters intended for summer primary contact recreation, and 2,000 colonies/100ml for winter primary contact recreation and secondary contact recreation. Jabaquara Beach, which is intended for summer primary contact recreation, has median fecal coliform concentration of 160 colonies/100ml, and maximum of 600 colonies/100ml. Therefore Jabaquara Beach is not adequate for primary contact recreation (See Figure B.16). Pereque River and Matheus River have median fecal coliform concentration of 36,000 colonies/100ml and 6,300 colonies/100ml, respectively, reflecting heavy fecal contamination caused by direct discharge of domestic sewage into these rivers. Neither river is adequate for secondary contact recreation. The sewer stream has fecal coliform concentration of 1,600,000 colonies/100ml, which is typical of raw sewage. Unsurprisingly, all waters exceed the maximum total coliform concentration of 5,000 colonies/100 ml is allowed in surface waters (See Figure B.17).

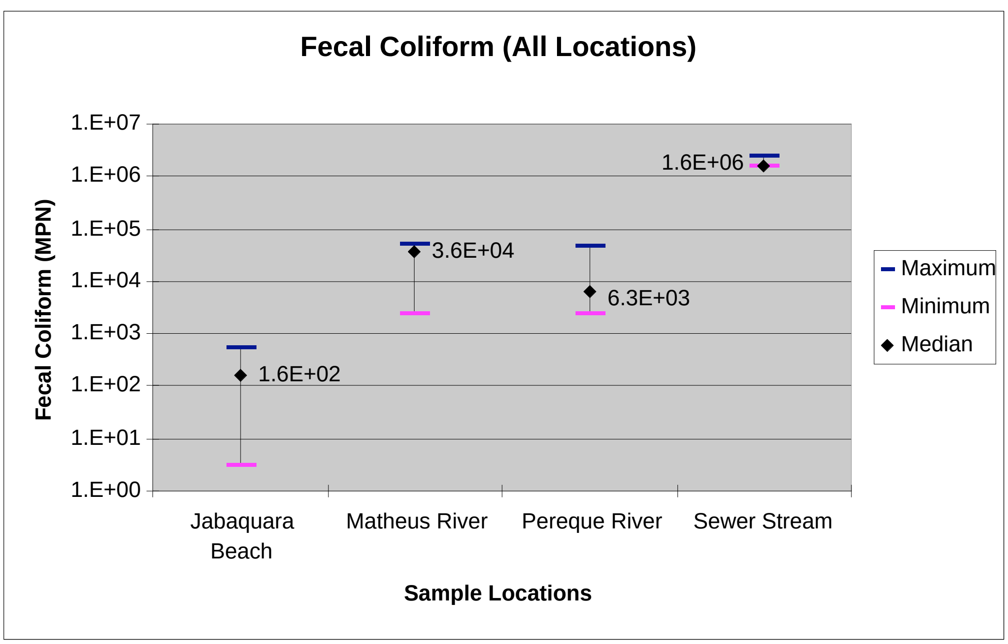


Figure B.16. Fecal coliform of the surrounding water bodies

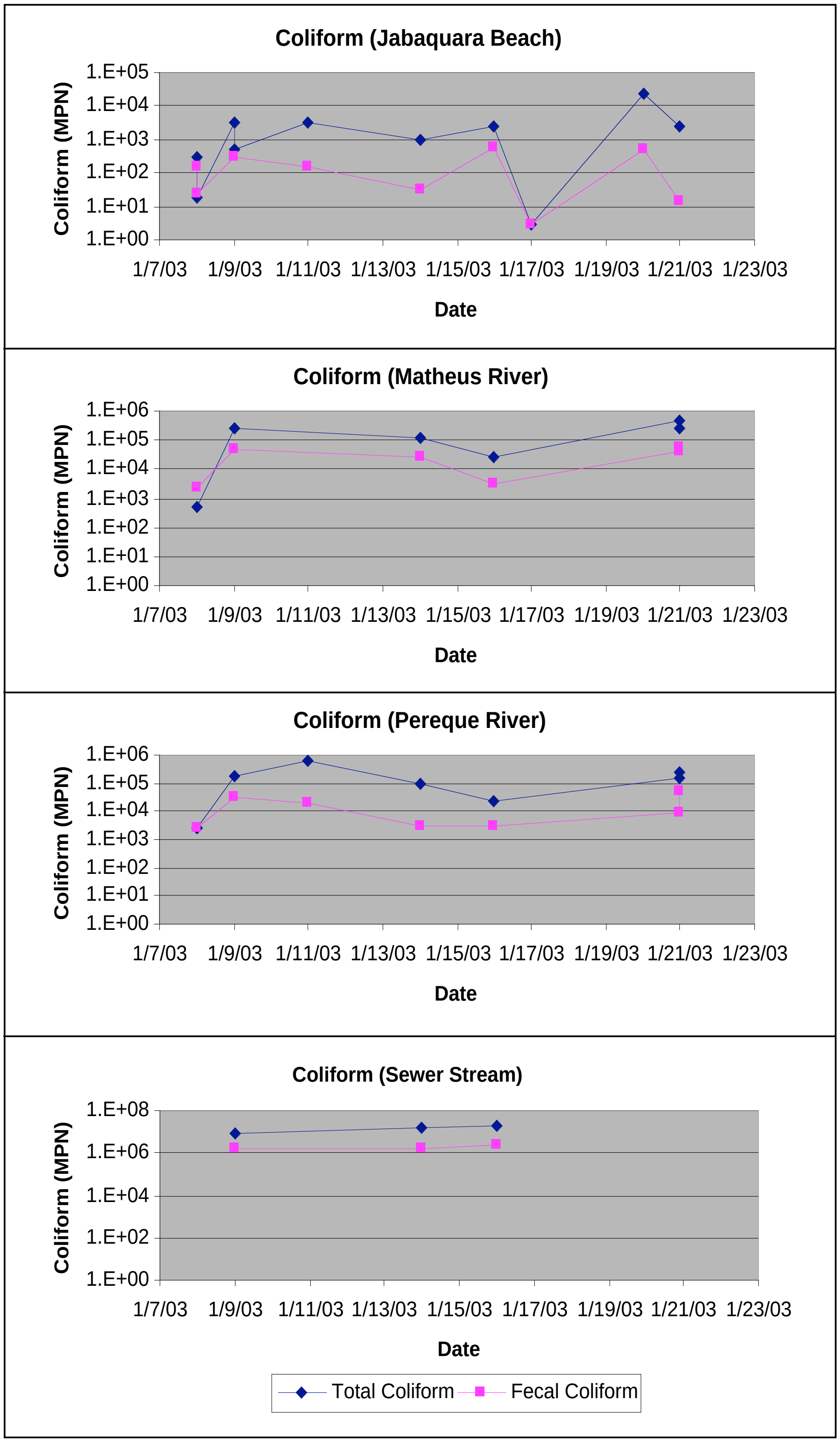


Figure B.17. Total coliform of the surrounding water bodies

Summary

All four surface water bodies show fecal coliform concentrations that suggest contamination from sewer discharge. Among the four, Jabaquara Beach shows the least amount of contamination, most likely benefited by tidal dilution. Matheus River and Pereque River are approximately equally contaminated, and Sewer Stream shows characteristics of diluted raw sewage.

Jabaquara Beach, a popular recreational water body where people swim, that is within walking distance from the City of Paraty, is inadequate for primary recreation, which includes swimming. Jabaquara Beach water has a slightly low pH, adequate levels of turbidity and suspended solids, and high COD.

Neither Matheus Rivers nor Pereque River is adequate for secondary recreation, due to high levels of fecal contamination. Matheus River showed acceptable pH, but especially high COD level that is most likely due to oil spills from small boats anchored at the riverbank. Pereque River had pH that is in the lower end of the acceptable range, and low COD that is within acceptable range most of the time. The turbidity and suspended solids for both Rivers suggest that they are often, but not always, in the safe range for aquatic life.

Conclusion

From the water quality analysis above, it is evident that the City's current mode of wastewater disposal degrades its surface waters, rendering Jabaquara Beach unsafe for swimming, and Matheus River and Pereque River unsafe for all aquatic sports. The uncontrolled disposal of wastewater damages the aesthetics of the rivers, and reduces the commercial value of the environment. The source of pollution must be controlled in order to preserve the environment from further degradation, and therefore an appropriate treatment and discharge of the City's wastewater is critical. The collection and treatment of wastewater is expected to limit pollution of the surface waters, as well as the streets, in the City of Paraty.

APPENDIX C – WASTEWATER COLLECTION PLAN FOR MANGUEIRA

This chapter provides a summary and rough translation of a wastewater collection plan described in “Programa Morar Melhor – Acao Saneamento Basico Plano de Trabalho: Tronco Coleter Estacao de Tratamento de Esgotos” (Prefeitura, 2001).

Cost of project: R$643200.00

The work consists of the construction of 1,800 meters of a gravity main collector of 200 mm in diameter of PVC. There are 25 manholes with a medium height of 2.5 meters and a wastewater treatment plant using slime with a capacity for 5,000 inhabitants (60 m^3/hour). The intent is to handle approximately 16.6% of the urban population of the municipality.

The Station of handling as well as the log collector that will be built and maintained by the Municipal city Hall of Paraty, specifically from the Municipal Office of the secretary of Works.

Type	Material	Unit	Quantity	Cost (R$)	Construction Time (Months)
Trunk Collector	PVC 200 mm	meter	1800	114,000	3
Manholes	Concrete	NA	25	13,000	3
Pump Station	Pre-made	NA	1	26,000	3
Treatment Plant	Capacity: 60 m^3/hr	NA	1	398,000	3

Table C.1. Estimated cost

Population to benefit from project:

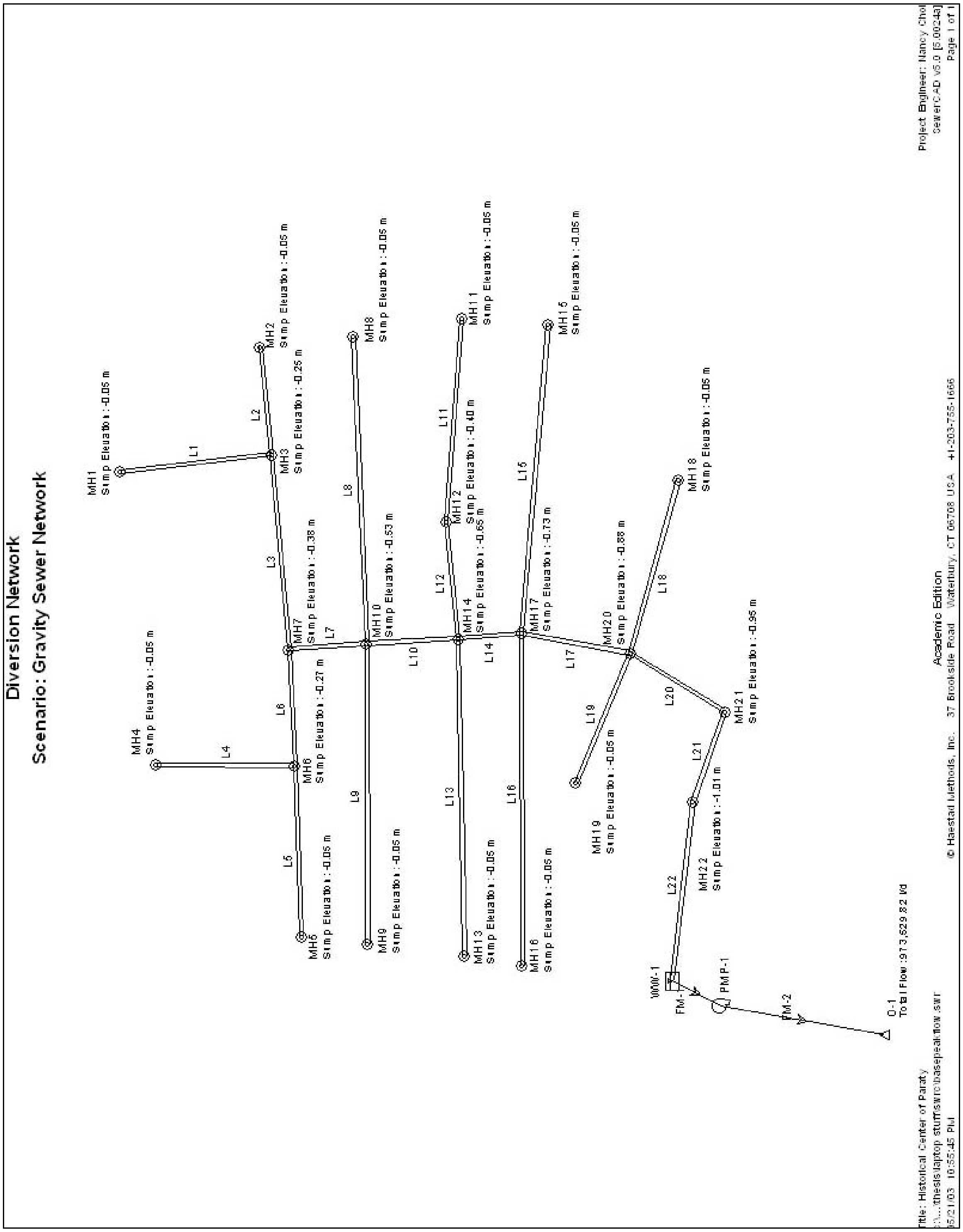
Actual population - 4000, 800 families
Projected population and design - 5000, 1000 families

Figure C.1 Trunk Sewer Layout

APPENDIX D – PROFILE DRAWINGS

This chapter provides profile drawings of sewer pipes for the wastewater collections network in the Historical Center of Paraty. These profile drawings were created using Haestad Method's SewerCAD.

Figure D.1. Sewer network



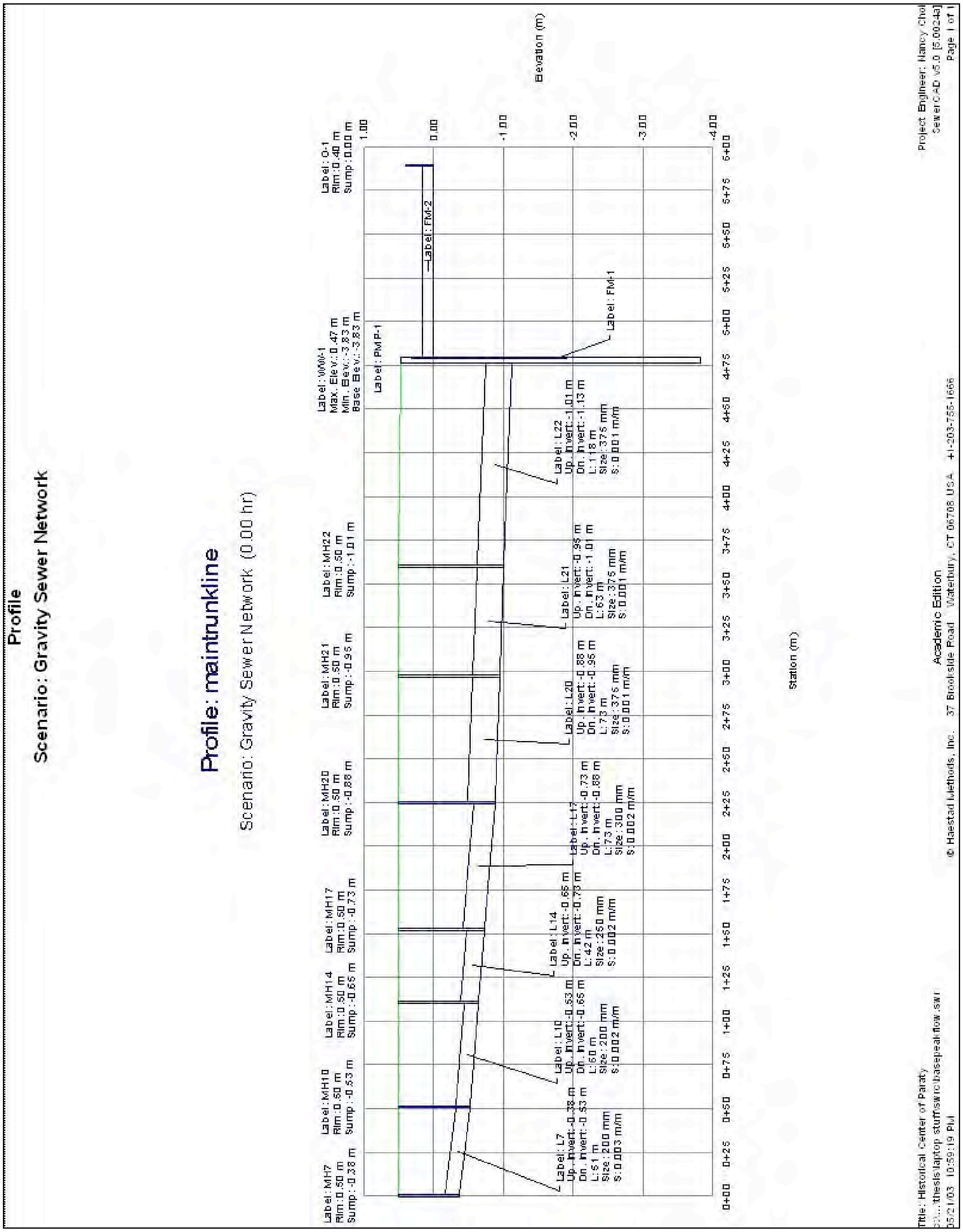


Figure D.2. Profile: Main trunk line

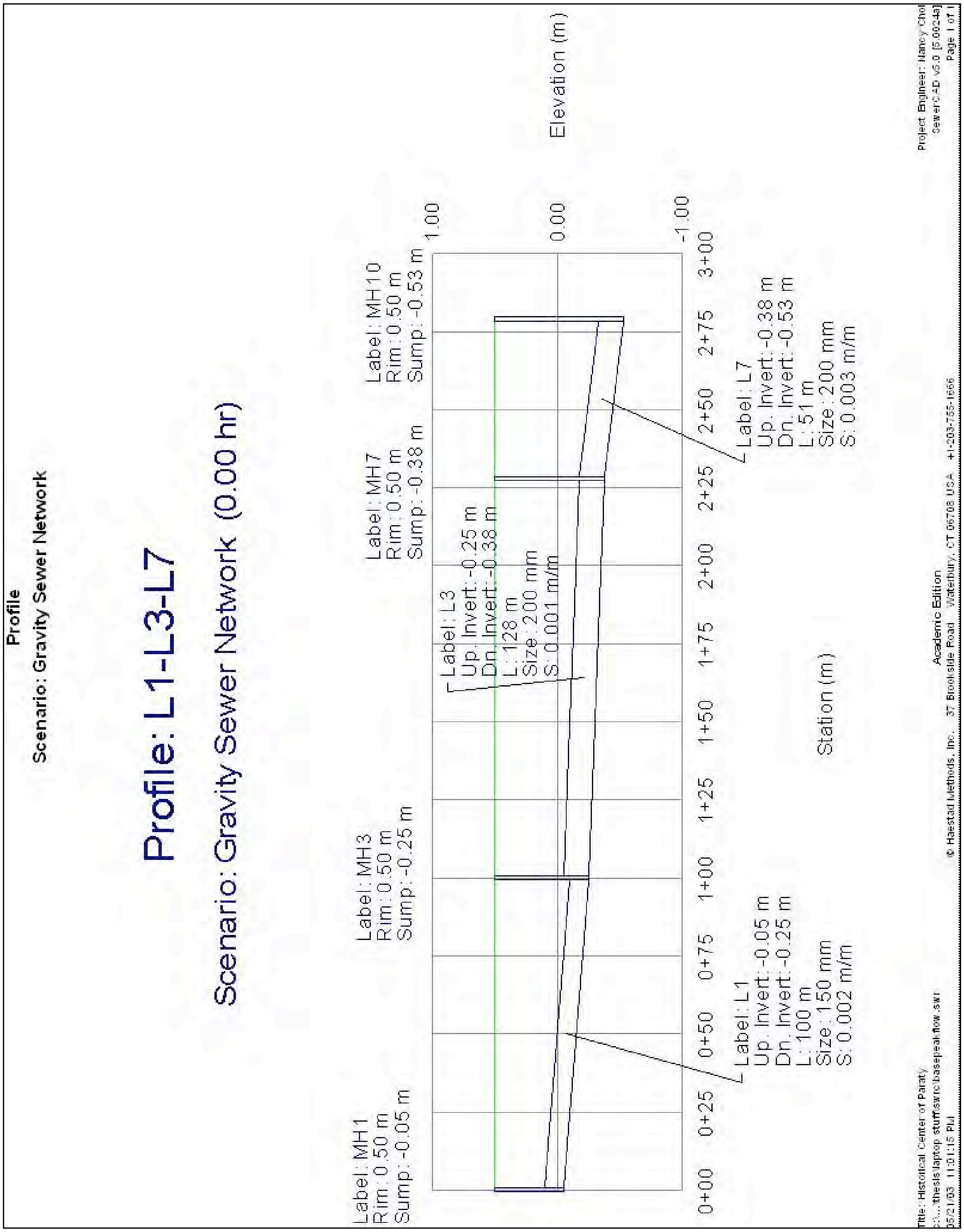


Figure D.3. Profile: L1-L3-L7

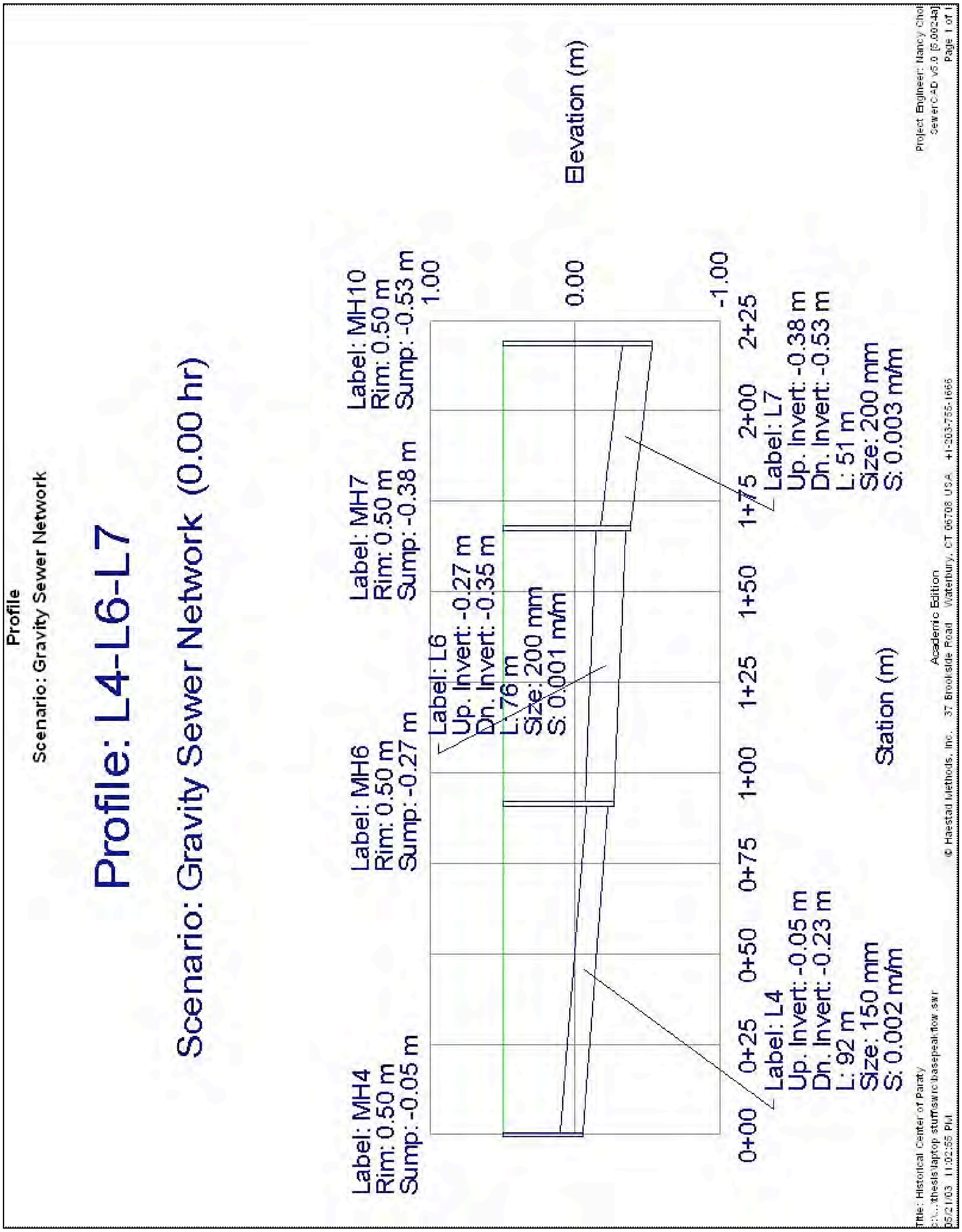


Figure D.4. Profile: L4-L6-L7

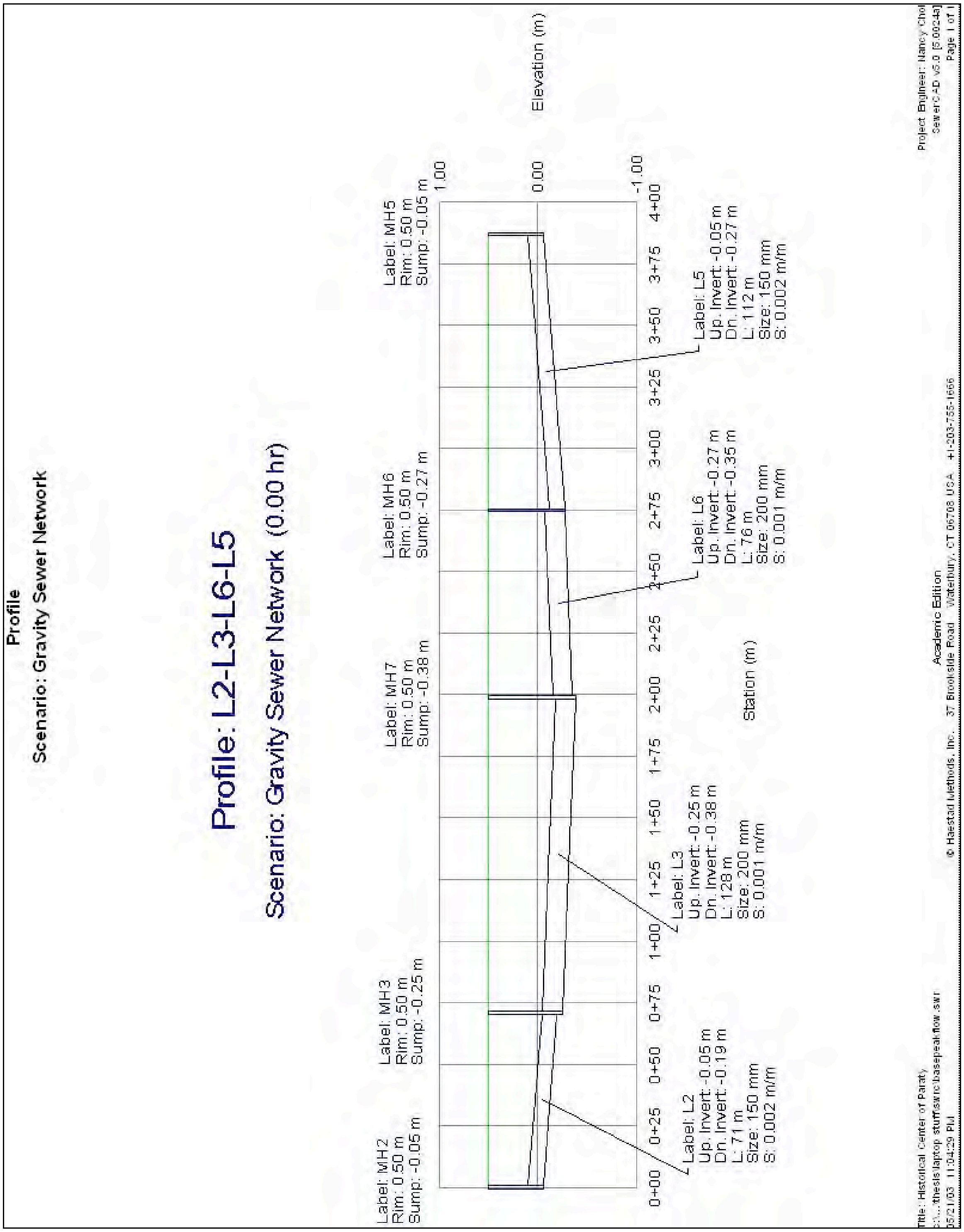


Figure D.5. Profile: L2-L3-L6-L5

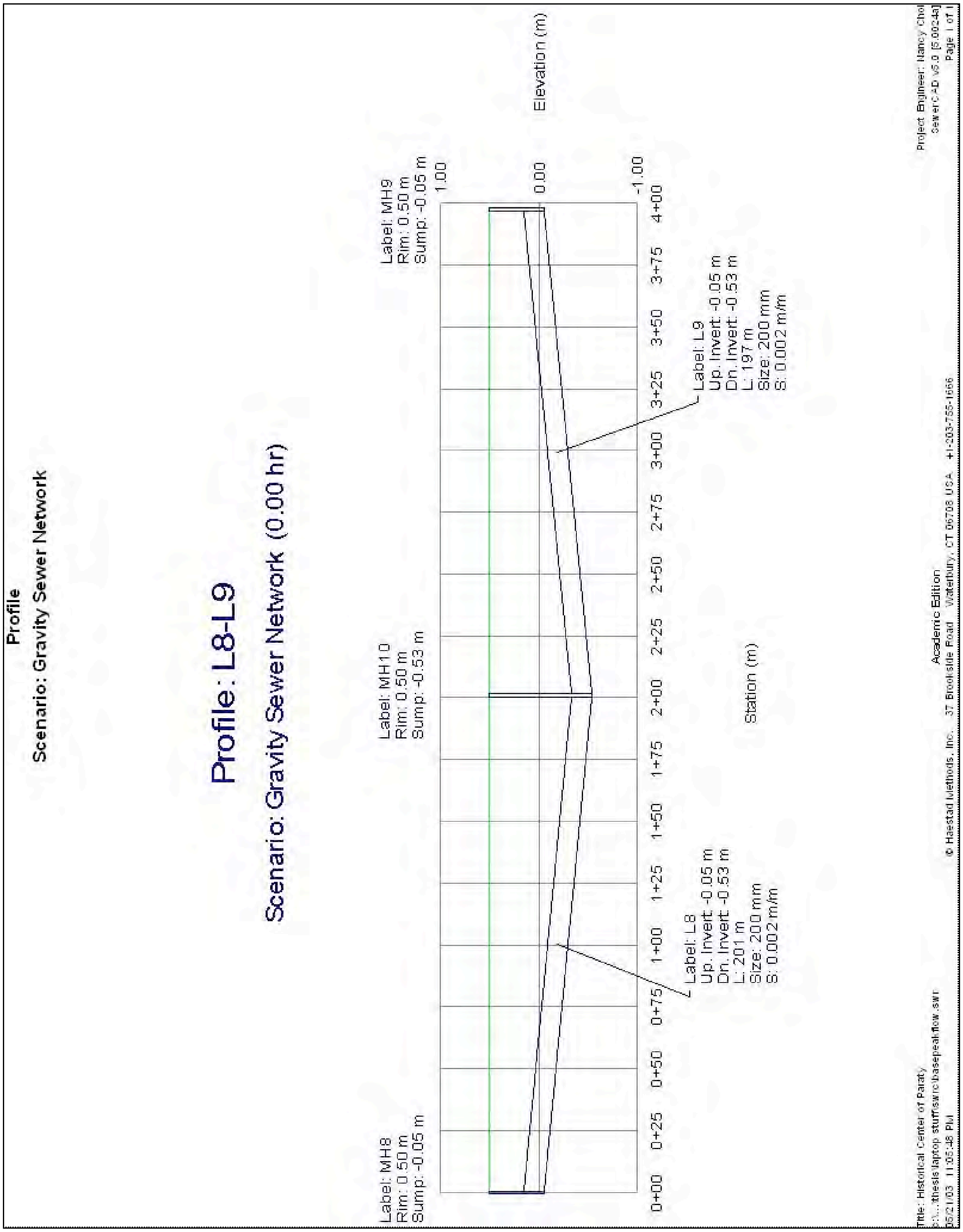


Figure D.6. Profile: L8-L9

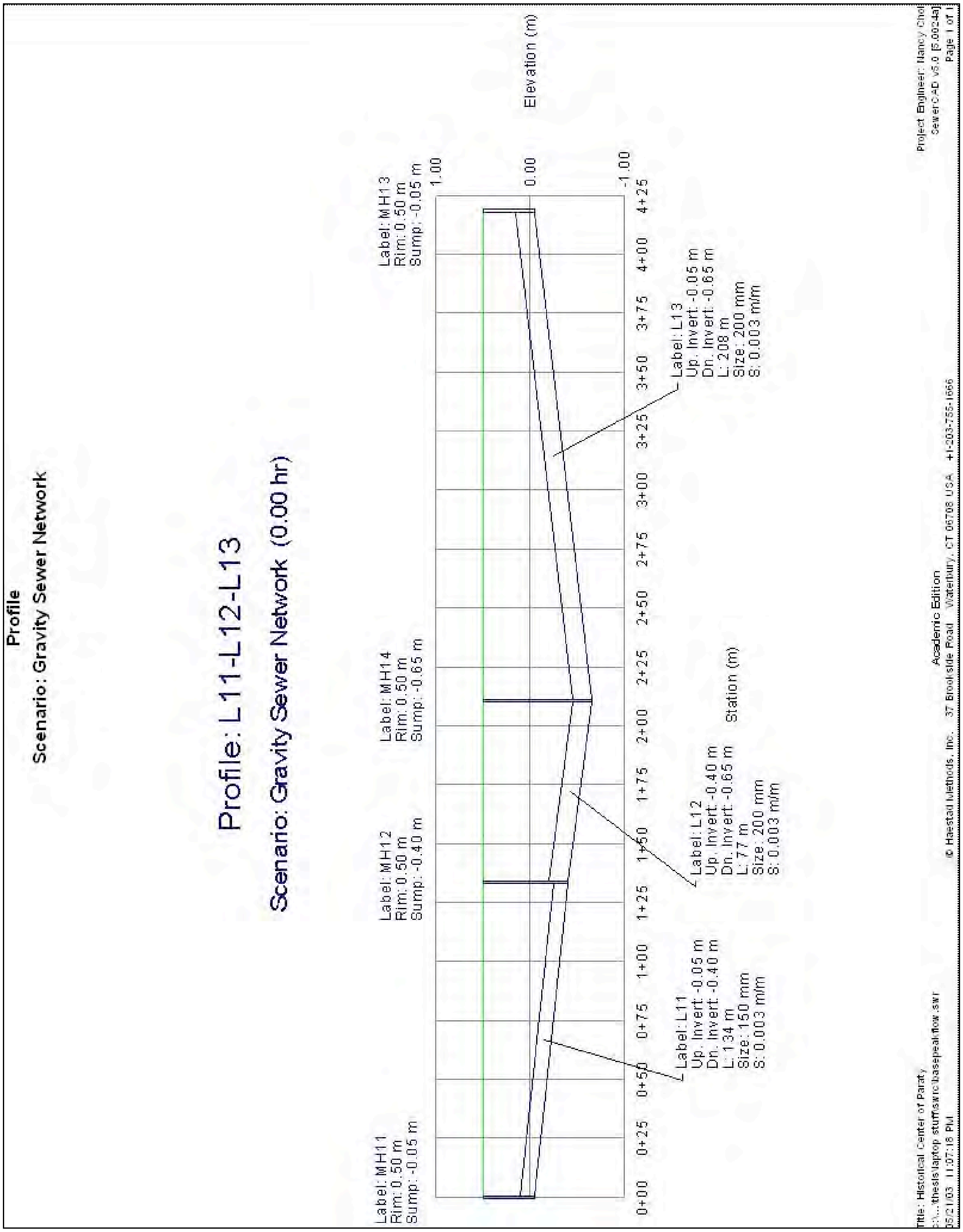


Figure D.7. Profile: L11-L12-L13

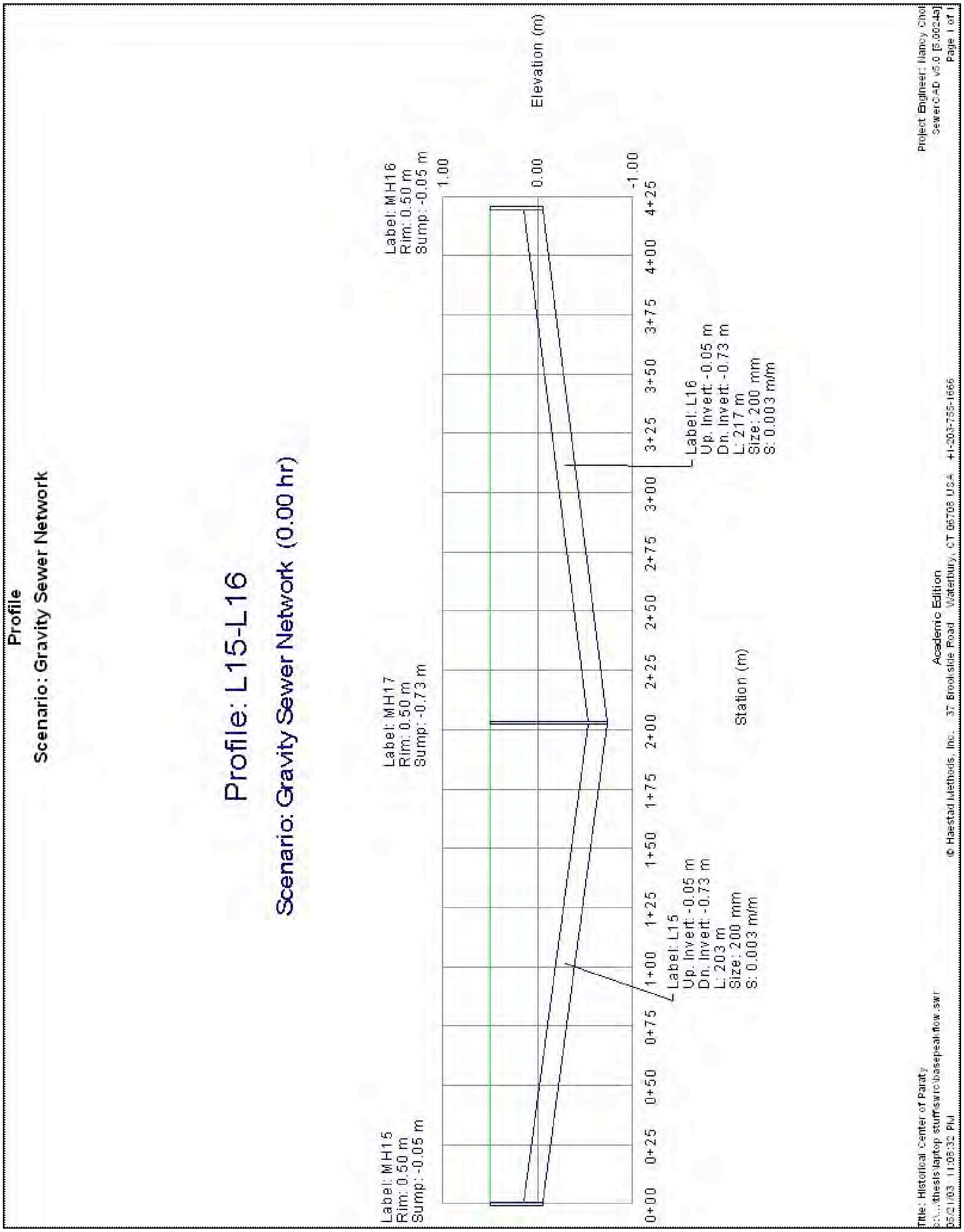


Figure D.8. Profile: L15-L16

APPENDIX E – DISCUSSION AND ANALYSIS OF GRAVITY SEWER DESIGN

Due to inadequate source of information, numerous assumptions and extrapolations of data are made in the gravity sewer design, leaving a large margin of error. This section discusses these numerous assumptions, and evaluates the robustness of the design through a sensitivity analysis. The main assumptions discussed are those made for the population, wastewater flow, peaking factor, infiltration rate, ground elevation, and pumping.

Wastewater Flow

The collection system is designed for peak hourly flow, the estimation of which involves making several assumptions. Although Paraty lacks data on wastewater flows, the city has information on potable water use, which is often a valid indicator of wastewater flow. The potable water use is estimated from the daily consumption per capita, so the total population in the Historical Center is an important factor in its approximation. The base yearly population and the three-fold increase in the summer tourist season are both approximated according to observations made by the city's local people. In general, the population is assumed to be evenly distributed within the historical center, in order to separate the total wastewater flow into individual catchment area.

Another method of quantifying the wastewater flow is to use typical wastewater flows for different types of property. There are approximately 700 individual properties within the Historical Center, according to a survey conducted by the city (Prefeitura, 2002). Of these, 400 are residential, 200 are commercial, and the rest are other types of property, including vacant lots. In Brazil, the wastewater flow rate is approximately 150 L/person/day for an upper class house, 100 L/person/day for a hotel, and 25 L/meal/day for a restaurant (Tsukamoto, 2003). However, using this method requires making many more assumptions, because detailed breakdown of property types and locations of these properties is not readily available. For this reason, estimating the daily wastewater flow using the known average daily potable water consumption per capita (180 L/person/day) and the approximated population in the Historical Center is deemed more appropriate.

Velocity

In the proposed gravity sewer system, it is found that pipe flow is less than half full, and consequently the minimum acceptable velocity of its wastewater flow is unknown. However, parameters, such as pipe diameters or pipe roughness coefficient, have substantial impact on flow and velocity, and hence sediment transport in the pipes. Since these parameters were approximated, a sensitivity analysis is performed to look at flows of different magnitudes, and their impact on wastewater velocity within the pipes, as well as on the total flow capacity. While an hourly peaking factor of 1.8 is typically used in Brazil (Tsukamoto, 2003), the peaking factor is varied in Figure E.1 in order to see the effects of different magnitudes of load (i.e. wastewater flow) on wastewater velocities within pipes. Although many points overlap and cannot be seen, the general trend is that velocities inside pipes increase with increasing peaking factors.

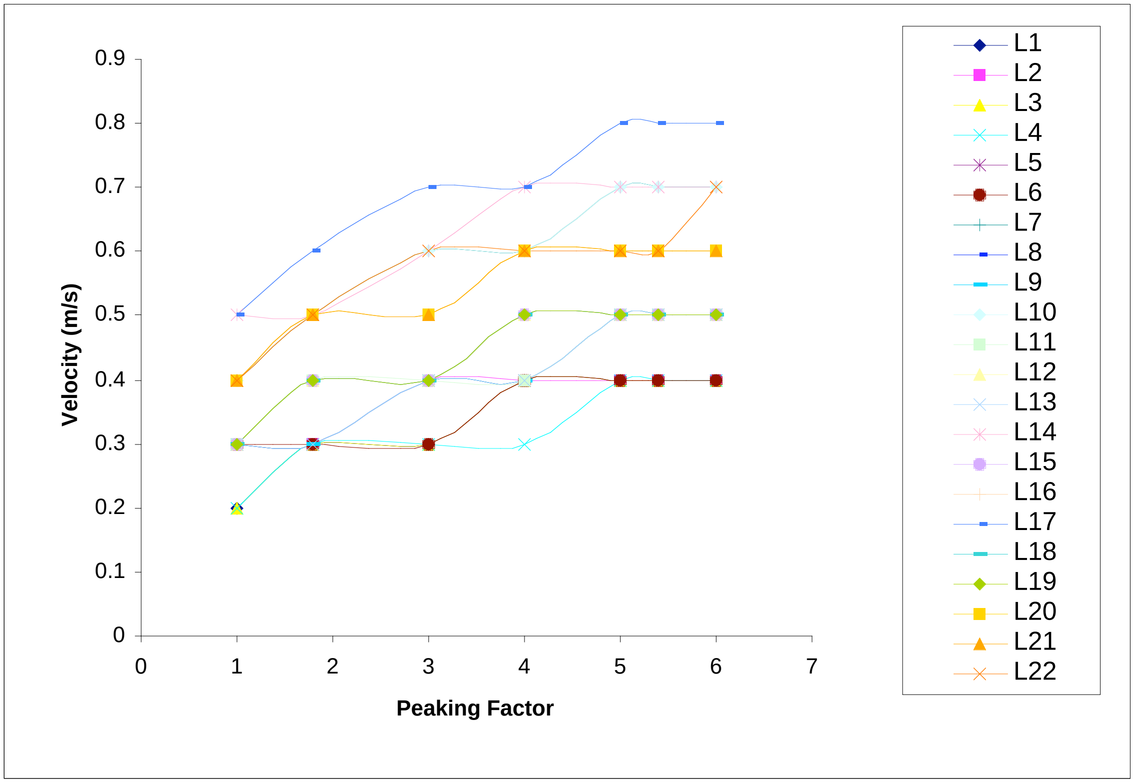


Figure E.1. Velocity versus peaking factor[1]

[1] L1 through L22 indicate different sewer pipe segments.

The figure above shows many pipe segments have velocities less than 0.6 m/s, suggesting that deposition of heavy sediments may occur inside pipes. However preliminary studies in Paraty indicate that Paraty's wastewater does not contain much sediment or suspended solids (Kfouri and Kweon, 2003), and that low velocities may be effective in self-cleansing of pipes.

It is suggested that the system design needs to be further calibrated with an acceptable minimum velocity, and that periodic cleanouts may be needed for the system if deposition of heavy sediments is to occur during the off-peak seasons. The system has the capacity to handle a peaking factor of about 6, above which point the sewers and manholes become flooded and pressurized. This limit may be a reflection of the maximum capacity of the pumping station.

It is observed that the pump operation has a significant effect on the systems operation. Different pump operation curves can produce different flow conditions in pipes. For the purpose of the preliminary model, a single point pump, which has a specified design head of 5 meters and a discharge of 2.5 m^3/min, is used. This basic pump is used to represent a pumping station, which is required due to the depth of excavation, but is not meant to be an exact model for an actual station that needs to be built. Refinement of pump operation rules are needed but is not addressed in this model.

Peaking Factor and Infiltration Rate

Another analysis is performed on the effects of peaking factor and infiltration rate on total flow. The figures below evidently indicate that the total flow increases with increases in either factor (See Figures E.2 and E.3). The impact of changes in the peaking factor on total flow is much greater than the impact of changes in the infiltration rate. There is approximately a four-fold difference between the slopes of the two trends. Since the infiltration rate, within small ranges, does not have a profound effect on the total flow, the accuracy of the assumption of infiltration rate is not as significant as other assumptions. The concern is whether the infiltration rate is extremely underestimated because, at smaller infiltration rates, the amount of infiltration is not a significant portion of the flow. This can be noted in the results for the proposed design in Table 5.6. Although an underestimated infiltration rate has a potentially large impact on the system, it is highly unlikely, especially with a new system with PVC pipes.

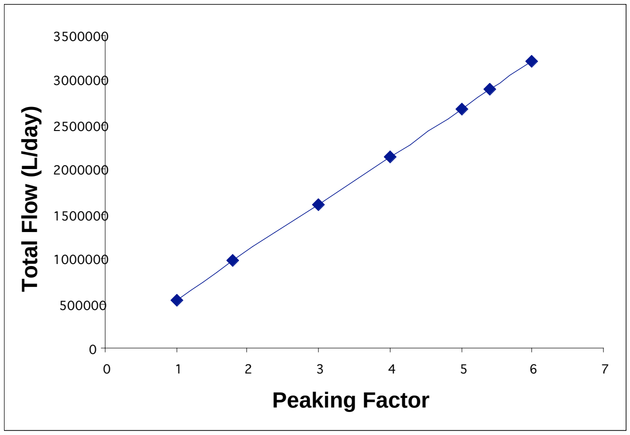


Figure E.2. Total flow versus peaking factor

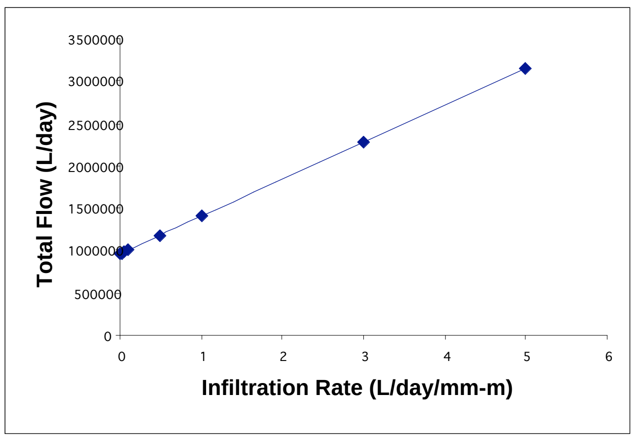


Figure E.3. Total flow versus infiltration rate

Roughness Coefficient

A third sensitivity analysis is performed on the impact of Manning's roughness coefficient, n, of pipes on the velocity. The range of roughness coefficients studied is 0.010 to 0.013. Most manufacturers have advocated the small n values for plastics, but some guides recommend using an n value of 0.013 for a new sewer, regardless of the availability of smoother materials. As seen in Figure E.4, the velocities of flow within the pipes decrease with increasing roughness coefficients.

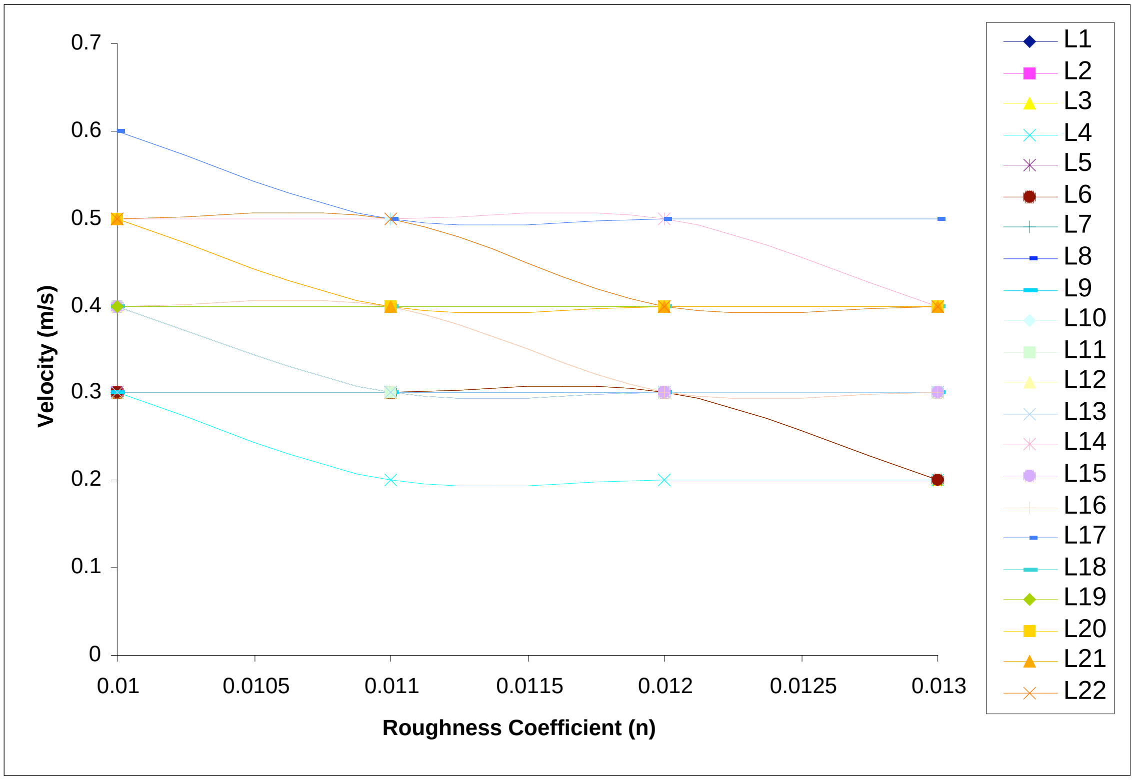


Figure E.4. Velocity versus roughness coefficient

Conclusion

Under the designed system operation, there is no node or branch that has flooding. Also all of the pipes are sloped to maintain a minimum velocity of 0.6 m/s for the designed flow. Overall, the system designed has some flexibility, but needs more calibration for it to become a more robust system, adaptable to variable seasonal flows in Paraty. The results from this study suggest that a gravity collection system is a feasible and viable option for the Historical Center of Paraty.

It is important to remember that this is a conceptual preliminary design. Additional study is needed in the following areas:

(i) Flow conditions and population numbers;

(ii) Acceptable minimum velocities in order to produce a self-cleansing velocity within pipes, which is a main design factor;

(iii) Economic trade-offs between the costs of deeper excavation (in order to increase pipe slopes/velocity) and the continuous O&M costs of pipe cleaning;

(iv) Various pump station schemes, to understand the effects of flow and hydraulic condition in the sewer system, and to find the most suitable operation rule; and

(v) Vacuum sewerage in addition to gravity sewerage.

APPENDIX F – INTRODUCTION TO JAR TESTS

This chapter introduces jar tests, which are essential to the design of wastewater treatment plant, and describes its methodology and theory. The analysis of the jar tests data follows in Appendices G and H.

F.1. Introduction

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

Testing Apparatus

The standard jar testing apparatus shown in Figure F.1 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 F.1 and F.2 below show a complete and typical jar testing setup before and after a typical jar test. Beakers 1 through 6 in Figure F.2 (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.

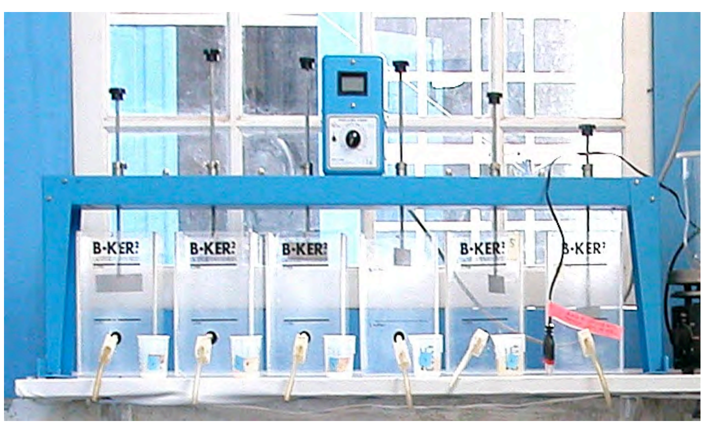


Figure F.1. Typical jar tests apparatus

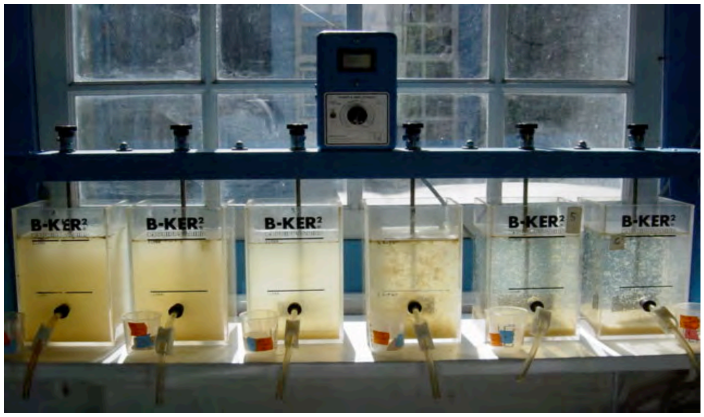


Figure F.2. Difference in effluent quality due to chemical addition

F.2. Theory

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:

$$\mathbf{T = L*W*H / Q} \qquad \textbf{Eq. F-1}$$

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 flow rate in m^3/sec.

Surface Overflow Rate

The surface overflow rate (SOR) is correlated with the percent removal of particulate matter in a settling tank, and it can be expressed as:

$$\mathbf{SOR = Q / (L*W) = H/T} \qquad \textbf{Eq. F-2}$$

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:

$$SOR = h/t \qquad \textbf{Eq. F-3}$$

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 is:

SOR = (8 cm)/(5 minutes) = 23 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). However, the lower surface overflow rate of 23 m/d seen in these experiments can be attributed to the fact that the settling time allowed (5 minutes) is higher than average jar testing settling times of 1 to 2 minutes, which consequently yields 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 is clear at settling times of approximately 2 minutes. Therefore, if the clear effluent sample is collected after the 2 minutes, rather than waiting the unnecessary 5 minutes, the corresponding overflow rate would be equal to the expected 60 m/d.

F.3. Methodology

Measured Parameters

Standard jar test experiments are performed to test the efficiency of chemically enhanced primary treatment in removing pollutants of concern from 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 Appendices G and H, the prime emphasis is on determining the removal rates of suspended solids, chemical oxygen demand, and turbidity.

Total Suspended Solids

Total suspended solids (TSS), for an influent sample, is defined as 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, the suspended solids are measured by the Hach Spectrophotometer (www.hach.com).

Suspended solids test results are used routinely 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 6.4 and 6.5), measuring suspended solids is of utmost importance to gauge the removal efficiency.

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 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 nepholometric turbidity units (NTU) (Metcalf and Eddy, 1991). Figure F.3 below is the Hach Pocket‰ Turbidimeter Analysis System set that is used for the experiments in this project.

Figure F.3. 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 6.1), the following equation can be adopted to relate TSS and turbidity values in chemically enhanced jar tests as well:

TSS, mg/L ª (TSS$_f$)(T) **Eq. F-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).

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:

i) Many organic substances that are difficult to oxidize biologically (lignin, for example) can be oxidized chemically;

ii) Inorganic substances that are oxidized by dichromate increase the apparent organic content of the sample; and

iii) High COD values may occur because of the presence of inorganic substances, with which 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 F.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 F.1. BOD/COD ratios

In chemically enhanced primary treatment (CEPT) plants, the BOD removal rates have been observed to be very close to the COD removal rates (Harleman, 2003). Since the BOD test is a 5-day test and the COD test is a 2-hour test, COD removal rates are commonly measured to represent the wastewater characteristics pre- and post-treatment in time-constrained laboratory settings. Figure F.4 below shows the Hach COD Reactor (www.hach.com) used in this project to incubate the COD vials containing effluent samples.

Figure F.4. Hach COD reactor

<u>Effects of Chlorides on COD Readings</u>

In this project, seawater is added to the influent wastewater to test its efficiency as a coagulant enhancement mechanism. Seawater is naturally very abundant in chlorides that constitute slightly more than 50% by weight of dissolved ions (Table F.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_3BO_3)	0.07	0.0026
Strontium	(Sr^{2+})	0.04	0.0013
Fluoride	(F^-)	0.002	0.0001
Total		**99.992**	**3.4482**

Table F.2. Dissolved ions in seawater (http://www.guilford.edu/original/academic/geology/Seawater.html)

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

Vial Type Used	Max. Cl- Conc. in sample (mg/L)	Suggested Cl- Conc. in diluted sample (mg/L)	Max. Cl- Conc. in sample w/ 0.5 HgSO4 added (mg/L)
Ultra Low Range (0.7 - 40 mg/L)	2,000	1,000	N/A
Low Range (3 - 150 mg/L)	2,000	1,000	8,000
High Range (20 - 1500 mg/L)	2,000	1,000	4,000
High Range Plus (200 - 15,000 mg/L)	20,000	10,000	40,000

Table F.3. Recommended Chloride concentrations for accurate COD testing

If sample dilution causes 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 raises the maximum chloride concentration allowable to the level given in column four of Table F.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 does not produce misleading COD results when using Hach equipment. Two approaches are adopted to determine the concentration of chloride (Cl-) in different seawater volumes used for the 2-liter jar tests described in Section F.3. Both methods are based on the values from Table F.3 above and are important in showing the sensitivity of COD removal readings to the presence of chlorides.

<u>% Weight of Chlorides</u>

The percent of chlorides in seawater by weight is 1.898 % (Table F.2). This means that 1 gram of seawater contains 0.019 grams of chlorides. The mass of seawater can therefore be calculated knowing the density of 1.0250 g/cm^3 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 is 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 of seawater

10 ml of seawater = (10 g seawater) * (0.018 g Cl^-)/(1 g seawater) = 0.1845 grams Cl^-

So the concentration of chlorides in the 10ml = 18,450 mg/L.

When 10 ml of seawater is added to the 2-liter beaker, the total chloride concentration is:

Concentration = (18,450 mg/L)*(10 ml)*(1 L/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 F.4 below:

Seawater by Vol.	Vol. Seawater Added	Mass Seawater	Cl- mass	Cl- conc.	Cl- conc.
%	ml	g	g		mg/L
0.5	10	10.3	0.185	18,500	92
1	20	20.5	0.369	36,900	185
2	40	41.0	0.738	73,800	369
5	100	103	1.85	185,000	923
10	200	205	3.69	369,000	1,850
15	300	308	5.54	554,000	2,770

Table F.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 F.3. The COD readings for 15 % seawater additions are therefore incorrect.

% 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 details 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:

Assume 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.

NaCl Æ Na^+ + Cl^-

1 mole NaCl Æ 1 mole Cl^-

(35 + 23) g/mol NaCl Æ 35 g/mol Cl^-
36 g/liter NaCl Æ X g/liter Cl^-

X = 22 g Cl^-/liter = 22000 mg Cl^-/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:

22000 mg Cl^- Æ 1000 ml

Y Æ 10 ml

Y = 220 mg Cl^- / liter

The values of chloride concentrations added to the wastewater in the jar tests for various concentrations of seawater by volume are calculated and summarized in Table F.5 below:

% Seawater Added (by volume)	Volume of Seawater Added	Chlorides (mg/L)
0.5%	10 mL	220 mg/L
1%	20 mL	440 mg/L
2%	40 mL	880 mg/L
5%	100 mL	2000 mg/L
10%	200 mL	4000 mg/L
15%	300 mL	6000 mg/L

Table F.5. Chloride concentrations in seawater volumes Method 2

Method 2 used above also shows that the volumes of seawater added to the two-liter beakers cause the chloride concentrations to approach the 2,000 mg/L maximum chloride concentration and therefore interfere with the COD readings using Hach equipment.

Method 2 can be considered less reliable than method 1 however, because Method 1 encompasses all possible sources of chlorides in the seawater and does not limit the analysis to the salt concentrations in the seawater. 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.

APPENDIX G – ANALYSIS OF JAR TESTS DATA FROM PARATY, BRAZIL: USE OF $FeCl_3$, SEAWATER AND POLYMERS

G.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 $FeCl_3$ 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, $FeCl_3$, 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 was a critical examination point for the jar tests results. These experiments are summarized in Table G.1 below and are very effective at comparing and contrasting the effect of $FeCl_3$ 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 G.1. Summary of Experiments in Paraty, Brazil

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.

Figures G.1 and G.3 below show the effects of dilution on SS and COD removal. 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):

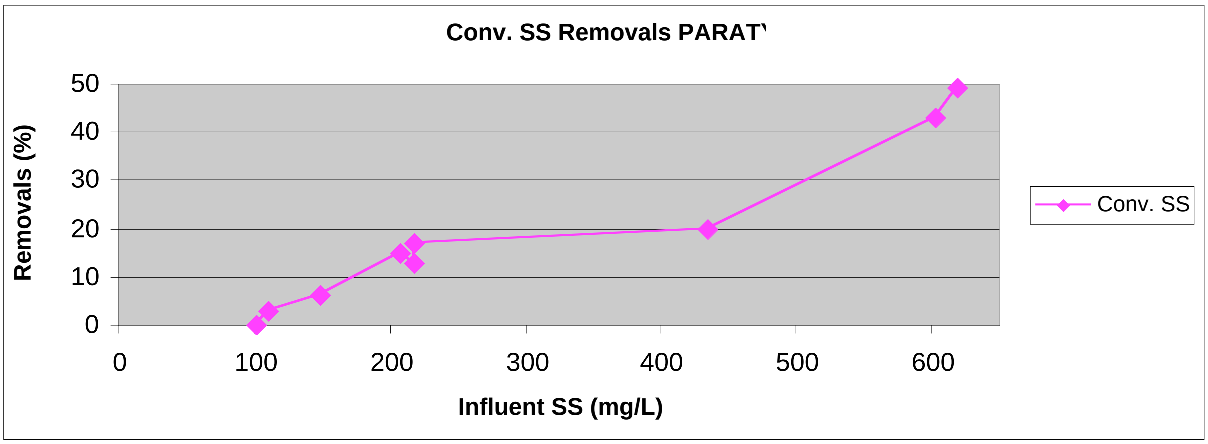


Figure G.1. SS removals with increasing influent concentration

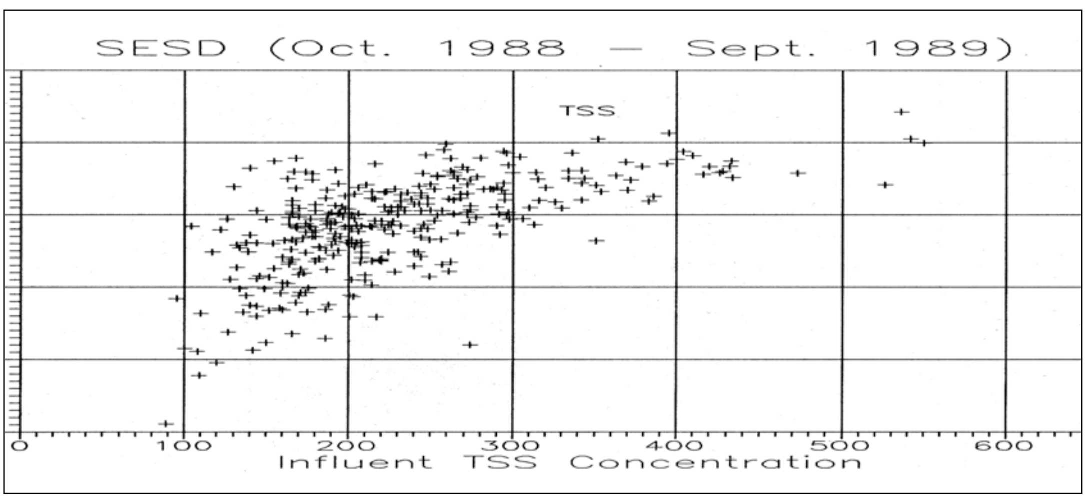


Figure G.2. TSS removals with increasing TSS concentration

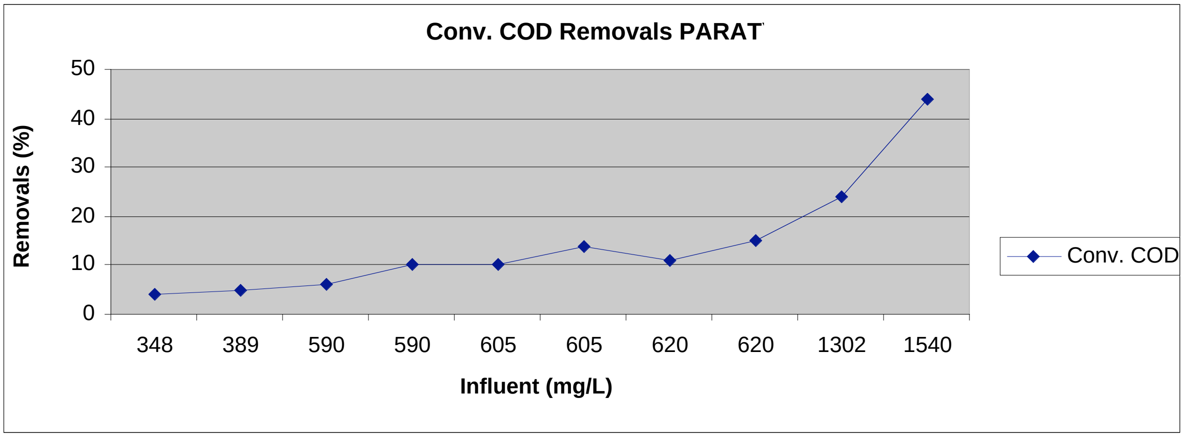


Figure G.3. COD removals with increasing influent concentration

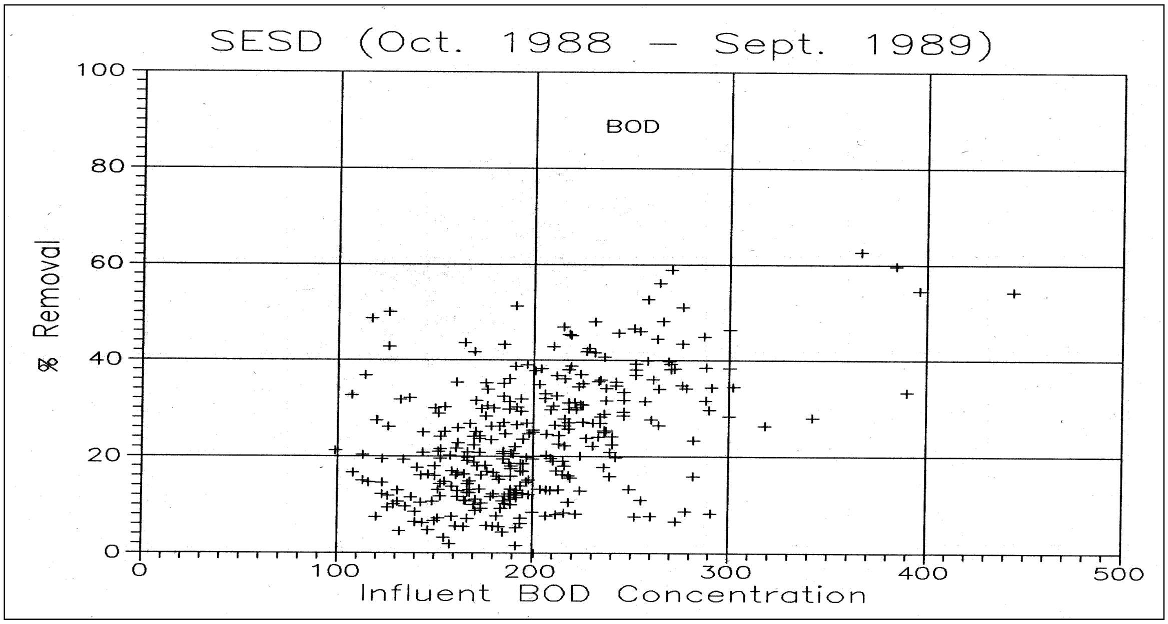


Figure G.4. BOD removals with increasing BOD concentration

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.

G. 2. Experiment One: Ferric Chloride ($FeCl_3$) Tests

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.

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 G.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 (Appendix F). 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	1,650	COD	5	34	47	45	5	57
	435	SS	20	54	81	82	94	97
	440	Turbidity	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	1,540	COD	44	49	49	48	58	59
	603	SS	43	40	53	53	63	90
	350	Turbidity	36	50	60	62	63	62

Table G.2. Experiment 1A: Summary of removal rates

Suspended Solids Removal

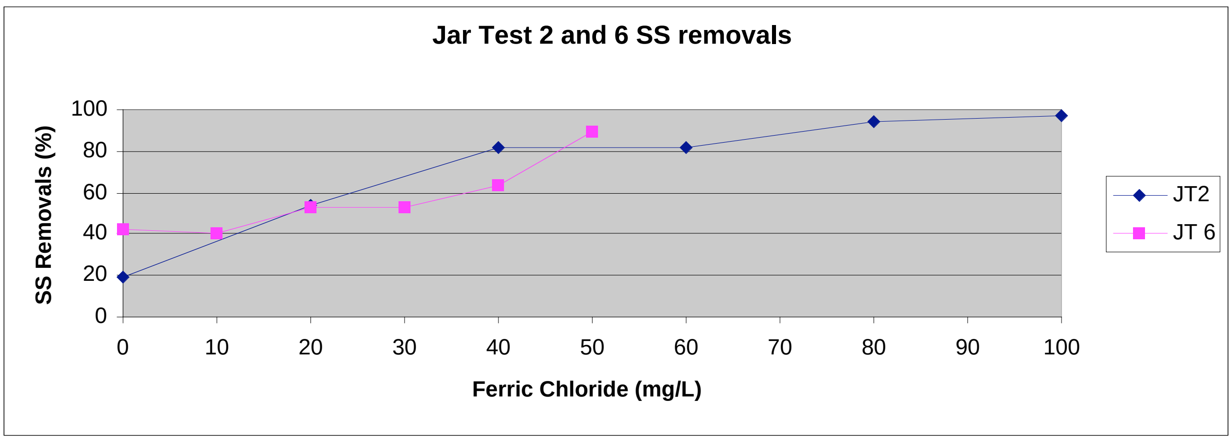


Figure G.5. Experiment 1A: Suspended solids removals

The suspended solids removal rates were higher in Jar Test 2 peaking at 97% for 100mg/L of $FeCl_3$. 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 $FeCl_{3)}$ were also lower than the removals in jar test 6 because of the dilution effect (Refer to Figure G.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 $FeCl_3$ 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 $FeCl_3$ doses between 40 and 65 mg/L. The optimal dose of $FeCl_3$ was therefore determined to be 40 mg/L.

Turbidity Removals

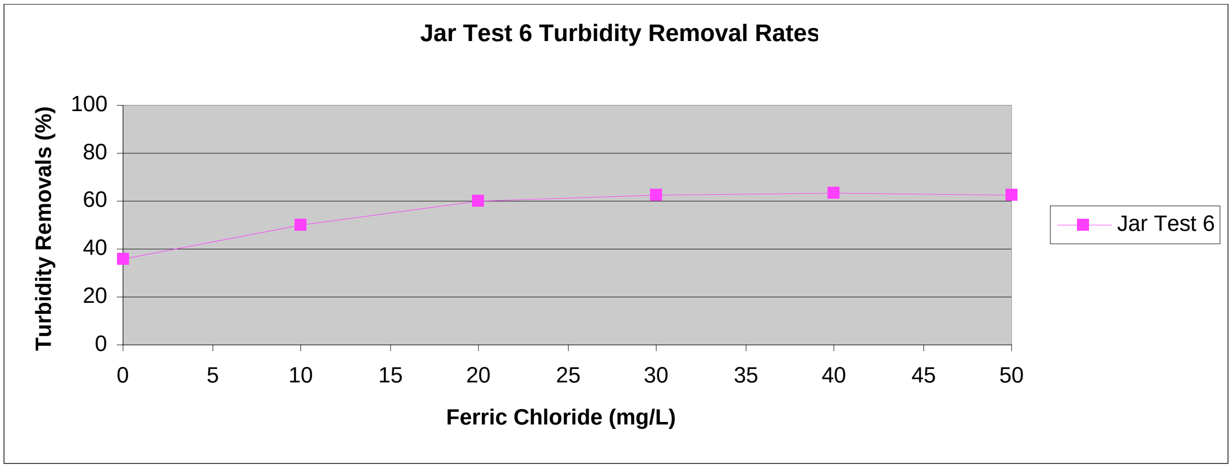


Figure G.6. Experiment 1A: Turbidity removals

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.

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).

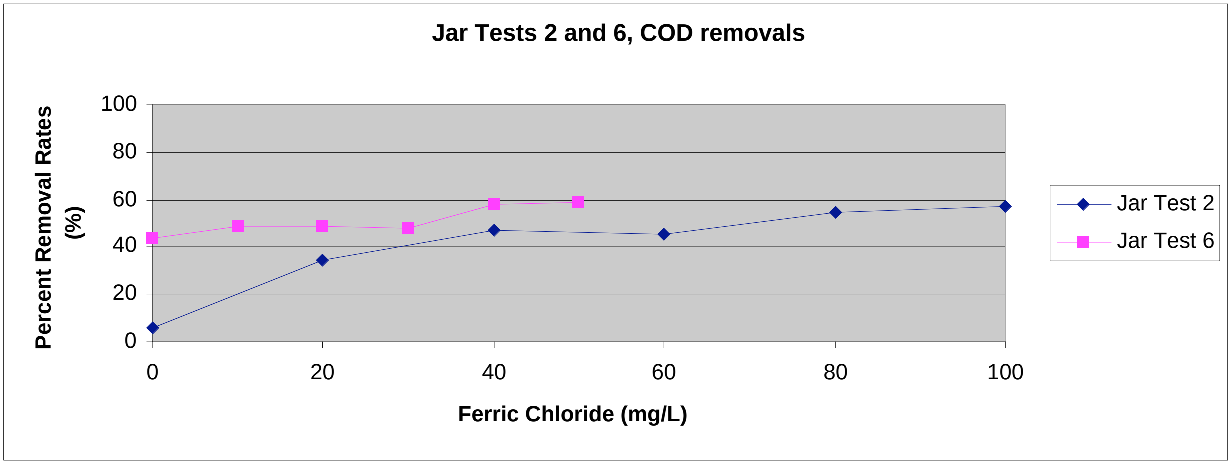


Figure G.7. Experiment 1A: COD removals

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.

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 $FeCl_3$ 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 above. Removal rates can therefore be expected to be lower.

The summary of raw waste characteristics and removal rates are shown in Table G.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	Turbidity	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	Turbidity	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	Turbidity	34	29	42	50	54	56

Table G.3. Experiment 1B: Summary of removal rates

Suspended Solids removals

The results for the three different jar tests showed a clear sensitivity to initial suspended solids concentrations: At 40 mg/L of $FeCl_3$, 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 $FeCl_3$ 40 mg/L concentration.

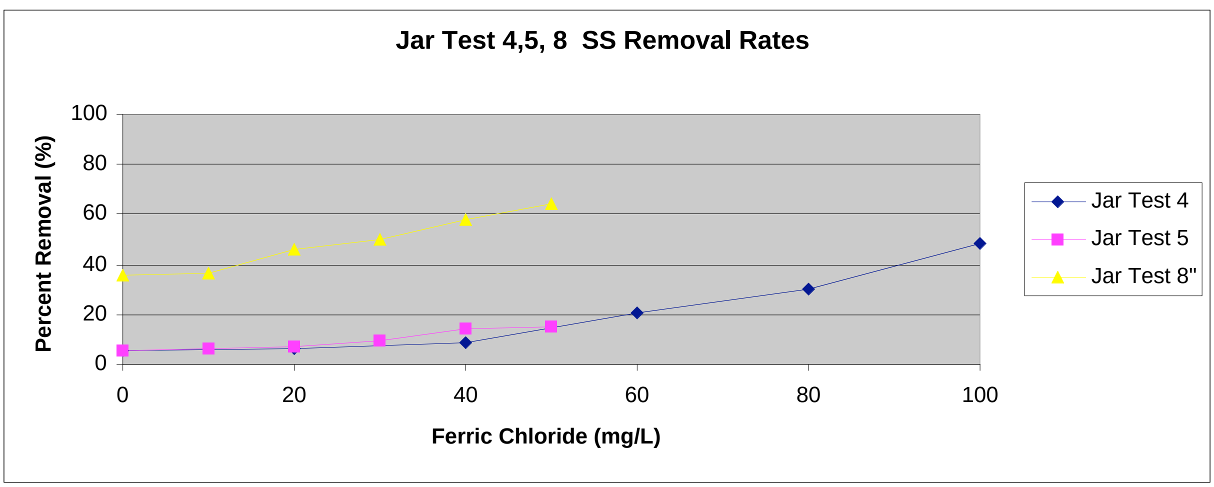


Figure G.8. Experiment 1B: Suspended solids removals

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 $FeCl_3$ 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.

Turbidity Removal

The turbidity removals were more consistent between jar tests 4,5 and 8. Close examination of Figure G.9 below shows that at a $FeCl_3$ dose of 40 mg/L, turbidity removal rates for jar tests 5 and 8 were 50% and 40% in jar test 4.

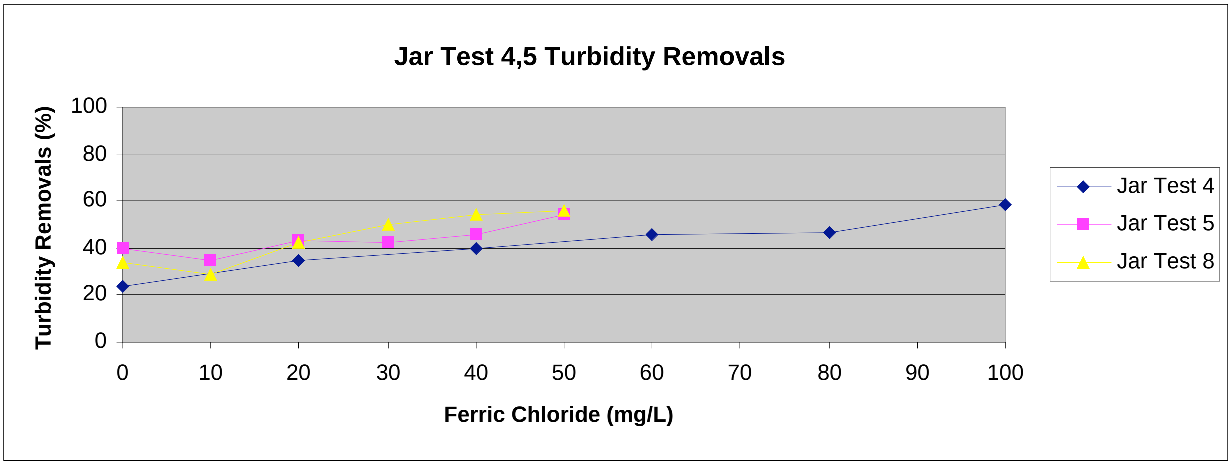


Figure G.9. Experiment 1B: Turbidity removals

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.

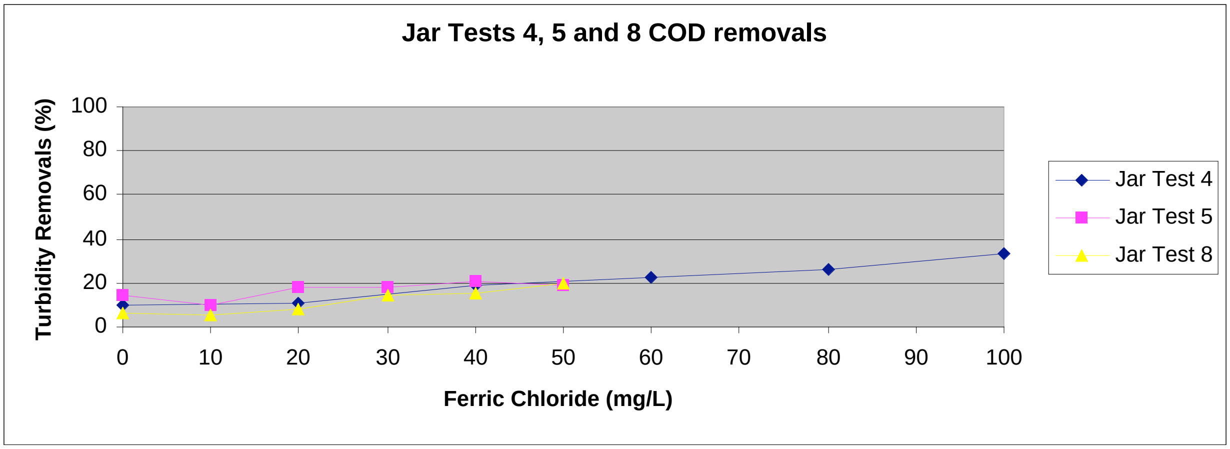


Figure G.10. Experiment 1B: COD removals

A summary table of the removals achieved with injection of 40 mg/L $FeCl_3$ alone is provided below. The average removals expected from $FeCl_3$ 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	Turbidity	N/A
Jar Test	Raw	40 mg/L $FeCl_3$(mg/L)	% Removal
6	1540	COD	58
	603	SS	63
	350	Turbidity	63
Jar Test	Raw	40 mg/L $FeCl_3$(mg/L)	% Removal
4	605	COD	20
	149	SS	14
	200	Turbidity	43
Jar Test	Raw	40 mg/L $FeCl_3$(mg/L)	% Removal
5	605	COD	19
	149	SS	16
	200	Turbidity	54
Jar Test	Raw	40 mg/L $FeCl_3$(mg/L)	% Removal
8	590	COD	16
	208	SS	58
	166	Turbidity	54

Table G.4. Experiment One: Summary of removal rates

G.3. Experiment Two: Seawater Tests

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).

Experiment 2A: Seawater at 0.5% by volume.

The raw wastewater used for Jar test 8 in Section G.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. 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 G.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 (No SW)	590	COD	6	6	8	15	16	20
	8	SS	36	37	46	50	58	64
	166	Turbidity	34	29	42	50	54	56
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
9 (0.5% SW)	590	COD	10	13	17	28	31	30
	8	SS	37	43	55	70	60	75
	166	Turbidity	42	46	57	64	69	68
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
10 (0.5% SW)	1302	COD	24	25	30	35	41	37
	619	SS	49	59	62	65	68	73
	395	Turbidity	31	48	51	59	71	76

Table G.5. Experiment 2A: Summary of removal rates

<u>Suspended solids removal</u>

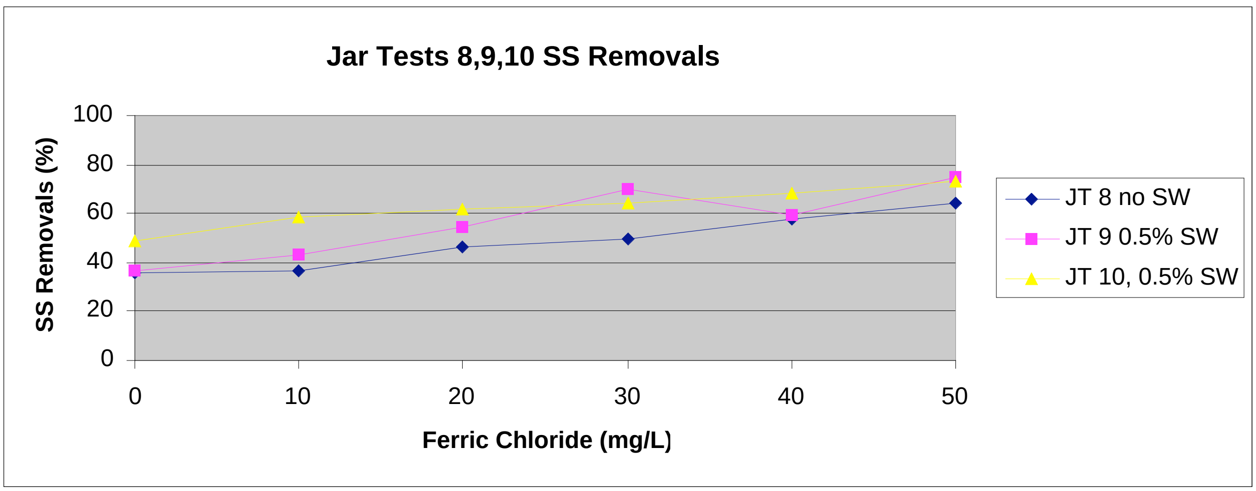


Figure G.11. Experiment 2A: Suspended solids removals

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 the suspended solids removals increased to 75% marking a 17% increase in suspended solids where:

% Increase in solids = [final (mg/L) – initial (mg/L)]/[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:

(i) Reduce the amount of ferric chloride needed to achieve a specified SS removal rate; and

(ii) Increase the suspended solids removal rate, for the same concentration of ferric chloride.

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

Turbidity Removals

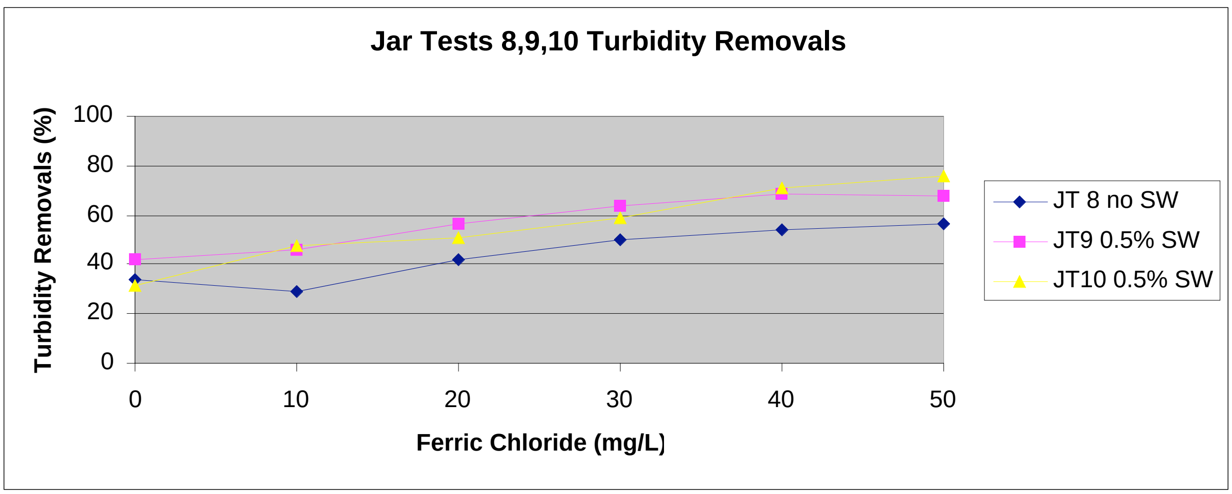


Figure G.12. Experiment 2A: Turbidity removals

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 suspended solids removal above, the difference between removal efficiencies with and without seawater, in jar tests 8 verses 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.

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%.

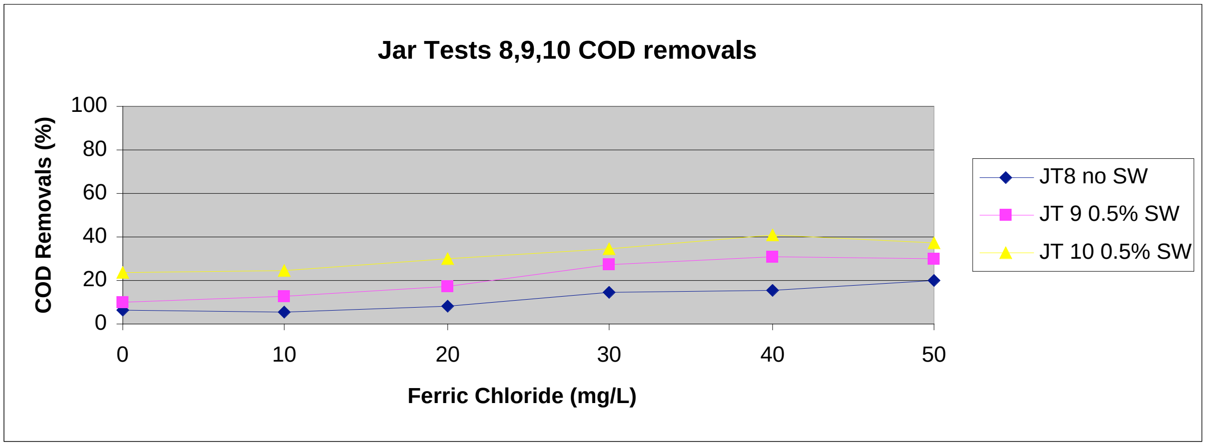


Figure G.13. Experiment 2A: COD removals

It is important here to note again, that because of the initially diluted raw wastewater samples (especially for that used for 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.

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 G.6 below summarizes the raw water characteristics and the observed removal efficiencies.

			Removal Rates (%)					
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
8 (No SW)	590	COD	6	6	8	15	16	20
	208	SS	36	37	46	50	58	64
	166	Turbidity	34	29	42	50	54	56
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
11 (1% SW)	620	COD	11	25	34	36	39	44
	218	SS	13	27	41	50	64	69
	131	Turbidity	8	16	48	51	59	59
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
12 (2% SW)	620	COD	15	28	32	38	46	50
	218	SS	17	33	47	61	70	65
	131	Turbidity	10	33	45	60	68	68

Table G.6. Experiment 2B: Summary of removal rates

Suspended Solids Removals

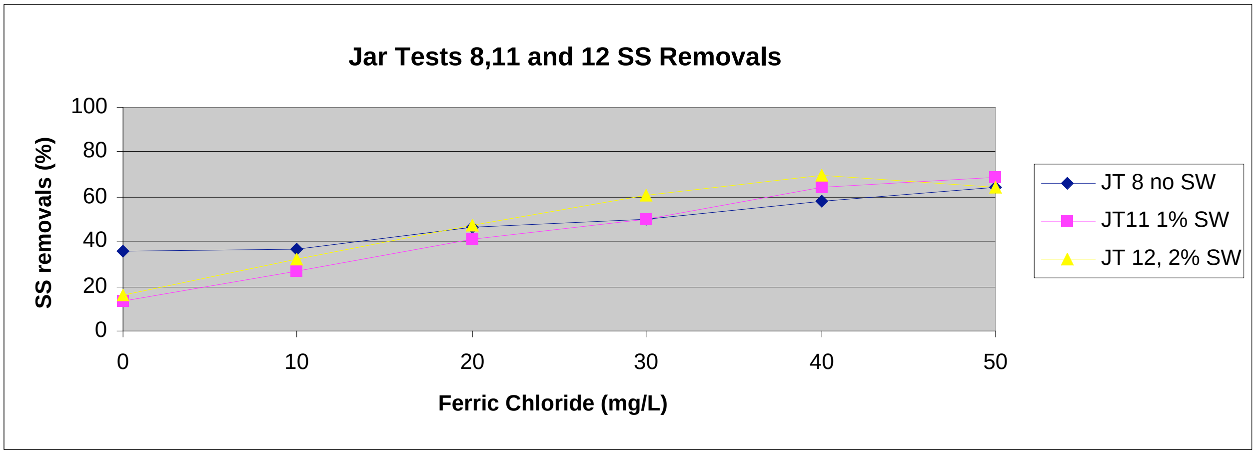


Figure G.14. Experiment 2B: Suspended solids removals

Increases in suspended solids removal efficiencies caused by the addition of seawater occurred after the addition of 30 mg/L $FeCl_3$ 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 $FeCl_3$ concentrations of 10 and 20 mg/L (See Figure G.14 above). The increase in SS removal efficiency was 20% with a 2% seawater addition at 30 mg/L $FeCl_3$ and 21.5% at 40 mg/L $FeCl_3$. The increases in SS removals due to seawater addition seemed to stabilize in excess of 50 mg/L $FeCl_3$ indicating a potential limit to the level at which seawater and $FeCl_3$ can be effectively mixed and used as coagulants.

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 $FeCl_3$ 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 $FeCl_3$ whereby a 1% seawater addition caused a 15% turbidity removal increase and 28% increase with the addition of 2% seawater.

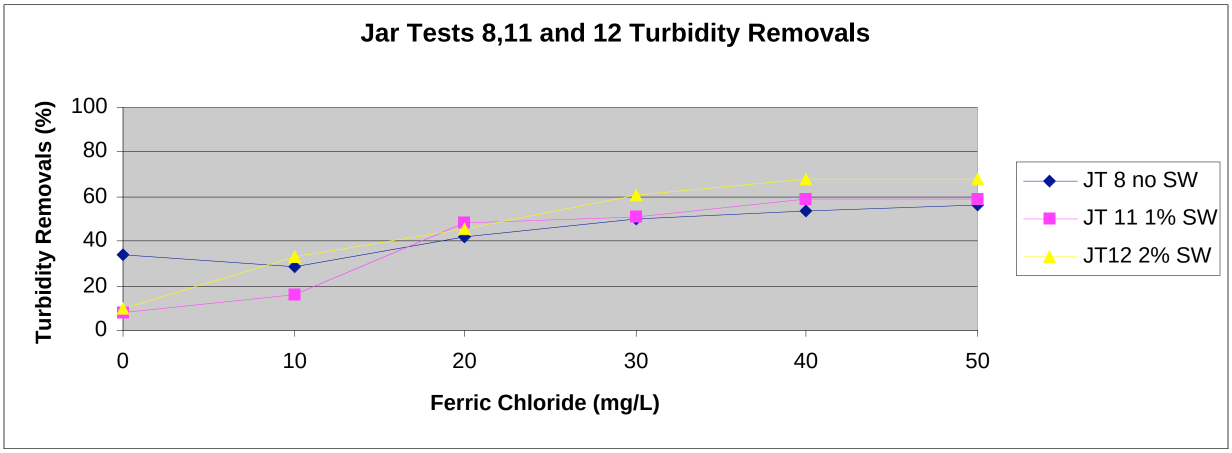


Figure G.15. Experiment 2B: Turbidity removals

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.

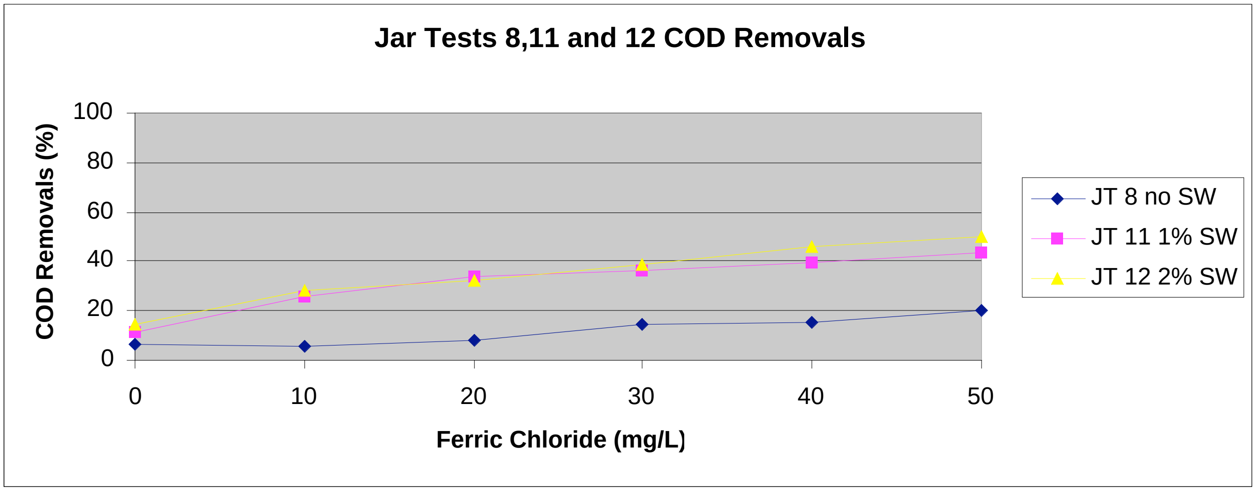


Figure G.16. Experiment 2B: COD removals

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 $FeCl_3$ 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 G.7 below:

			Removal Rates (%)					
Jar Test	Raw	$FeCl_3$ (mg/L)	0	10	20	30	40	50
8	590	COD	6	6	8	15	16	20
(No SW)	208	SS	36	37	46	50	58	64
	166	Turbidity	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	389	COD	5	0	0	2	1	3
(5% and	101	SS	0	25	42	11	66	68
10% SW)	84	Turbidity	5	19	38	4	63	68
Jar Test	Raw	$FeCl_3$ (mg/L)	0	30	30(5)	30(10)	40(5)	40(10)
26	348	COD	4	20	14	25	14	22
(5% and	110	SS	3	9	58	60	65	72
10% SW)	143	Turbidity	8	7	53	58	62	70

Table G.7. Experiment 2C: Summary of removal rates

Suspended Solids Removal

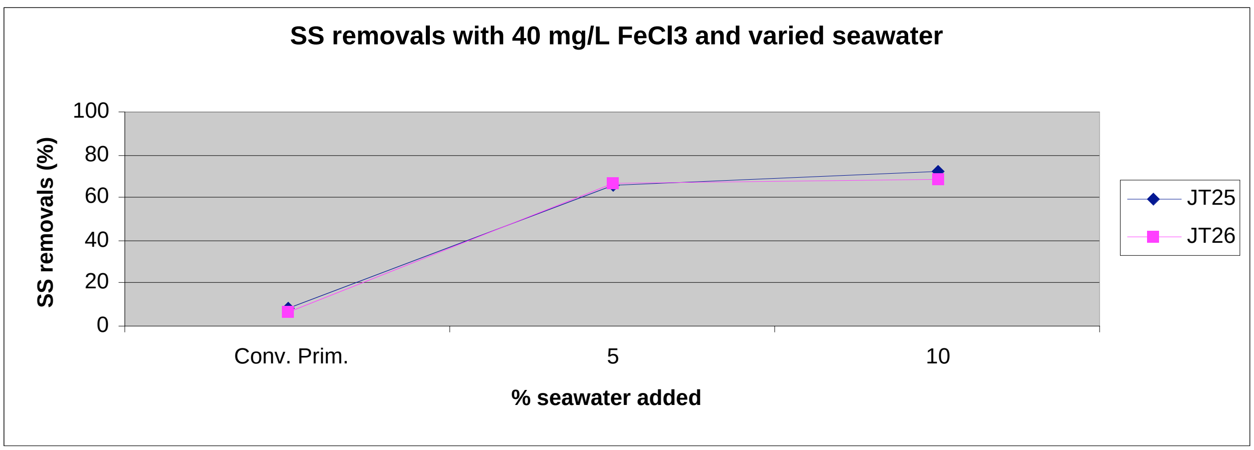


Figure G.17. Experiment 2C: Suspended solids removals

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.

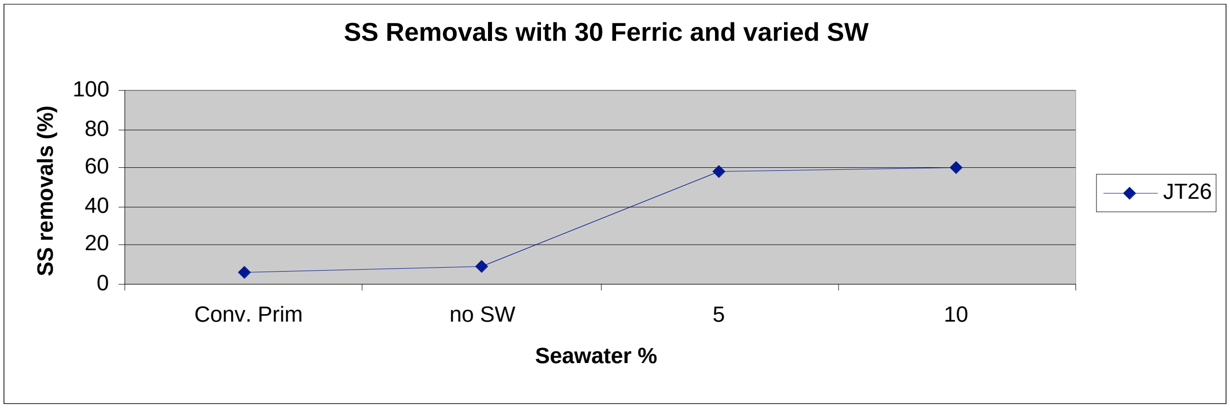


Figure G.18. Experiment 2C: Suspended solids removals (2)

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 G.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 $FeCl_3$ concentration, the SS removal rate achieved was 60% this 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 $FeCl_3$ to the previous sample where 40 mg/L $FeCl_3$ 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 $FeCl_3$ with 5% seawater is not the optimally economic coagulant dose.

<u>Turbidity Removals</u>

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).

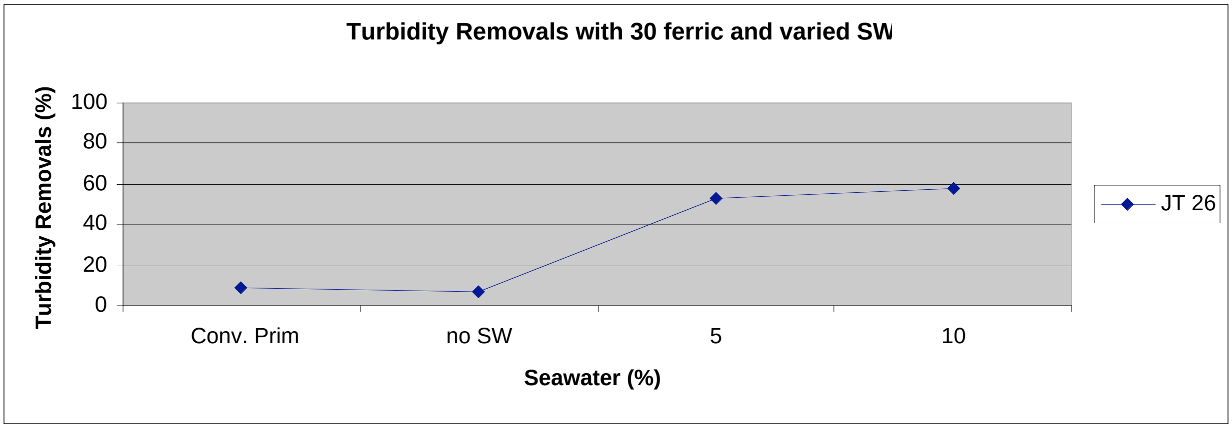


Figure G.19. Experiment 2C: Turbidity removals

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.

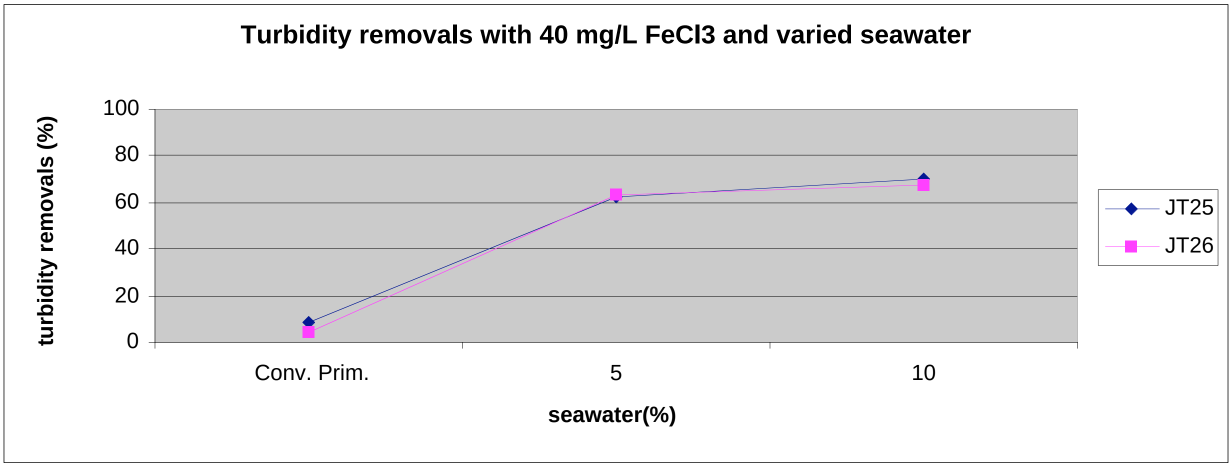


Figure G.20. Experiment 2C: Turbidity removals (2)

Figure G.20 above shows the results from an identical experiment, in which a $FeCl_3$ 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.

<u>COD Removals</u>

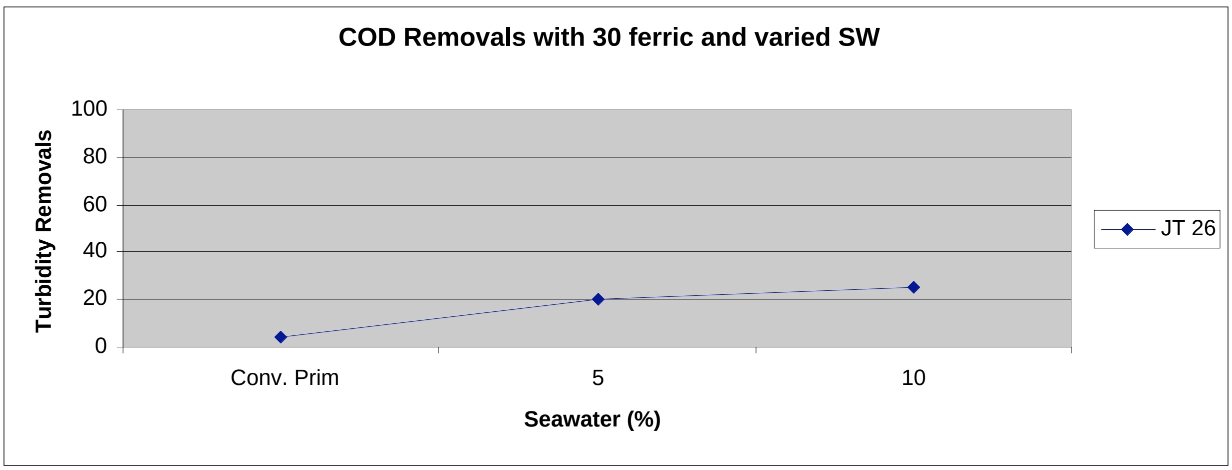


Figure G.21. Experiment 2C: COD removals

COD removals were, as expected, lower than the suspended solids and turbidity removals for the same $FeCl_3$ 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 $FeCl_3$, the removal efficiency increased to 25% from the initial 6% in conventional primary treatment marking a 316% in removal efficiency.

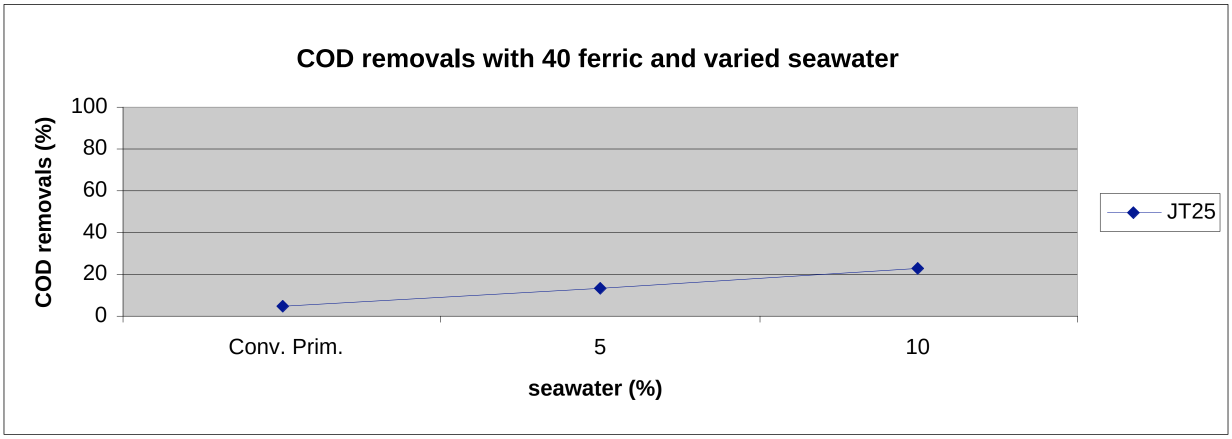


Figure G.22. Experiment 2C: COD removals (2)

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 Figure G. 22 above where only 30 mg/L $FeCl_3$ 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 $FeCl_3$ were 21% and approximately 24% for 30 mg/L.

G.4. General Conclusion and Dosage Recommendation for Paraty

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

(i) Seawater has a positive effect on the removal efficiencies in diluted wastewater samples;

(ii) At $FeCl_3$ doses higher than 50 mg/L, the effect of seawater decreases significantly;

(iii) Minimum $FeCl_3$ doses for seawater to take effect are approximately 20-25 mg/L;

(iv) Large seawater additions do not necessarily yield large increases in removal efficiencies;

(v) COD removals are mostly affected with small seawater volume additions; and

(vi) 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:

(i) 40 mg/L $FeCl_3$, or

(ii) 30 mg/L $FeCl_3$ with 5% seawater.

G.5. 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. The first set of jar tests therefore included $FeCl_3$ doses combined with polymer doses. $FeCl_3$ was then tested with seawater alone. Finally, a combination of $FeCl_3$, polymer and seawater at varying concentrations were combined to test for the most efficient and economically optimal dosage to treat the wastewater in Paraty.

Experiment 3A: Ferric Chloride and Polymers

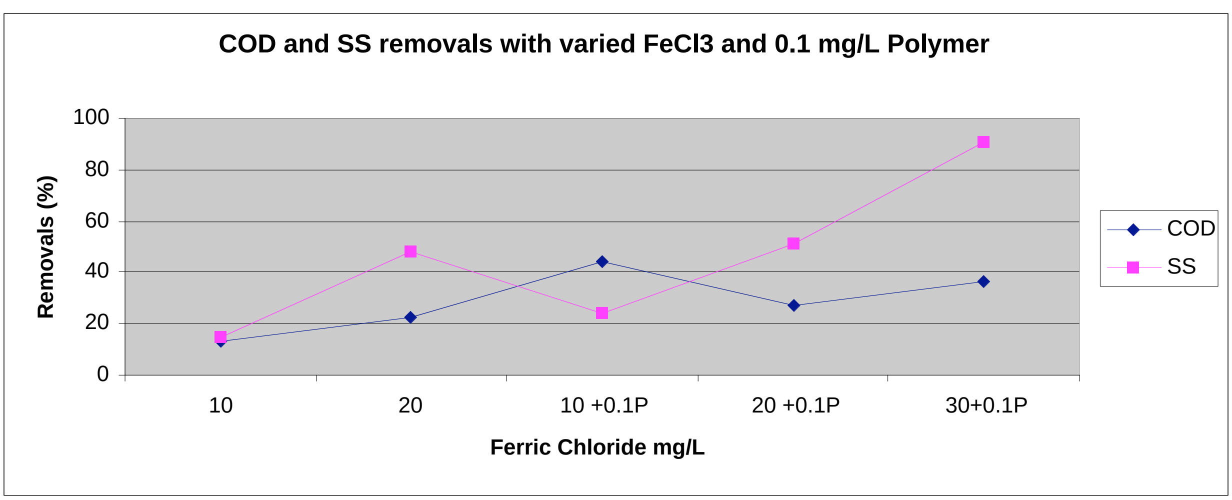


Figure G.23. Experiment 3A: COD and SS removals with $FeCl_3$ and 0.1mg/L polymer

In Figure G.23 above, the $FeCl_3$ doses were varied while the polymer dose was kept constant at 0.1 mg/L. Suspended solids 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 $FeCl_3$ and 0.1 mg/L Polymer which yield approximately 38% COD removal and 90% SS.

It is also important to note that at relatively low $FeCl_3$ 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 $FeCl_3$ was used that the polymer displayed an effect in enhancing removal efficiencies.

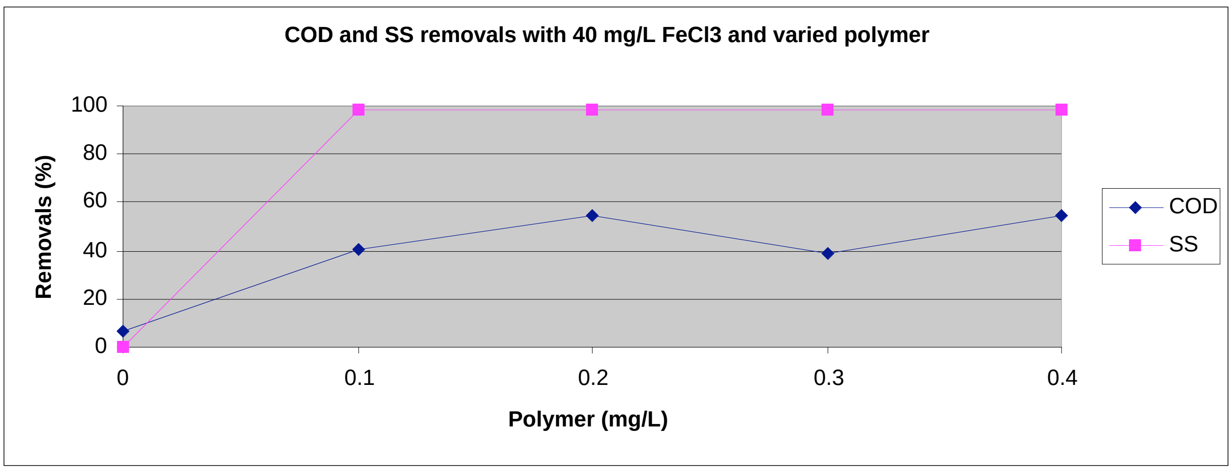


Figure G.24. Experiment 3A: COD and SS removals with 40 mg/L $FeCl_3$ and polymer

In Figure G.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 $FeCl_3$ 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 $FeCl_3$ with 0.1 mg/L of polymer.

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 $FeCl_3$ varied between 30 and 40 mg/L, both figures remain within acceptable range.

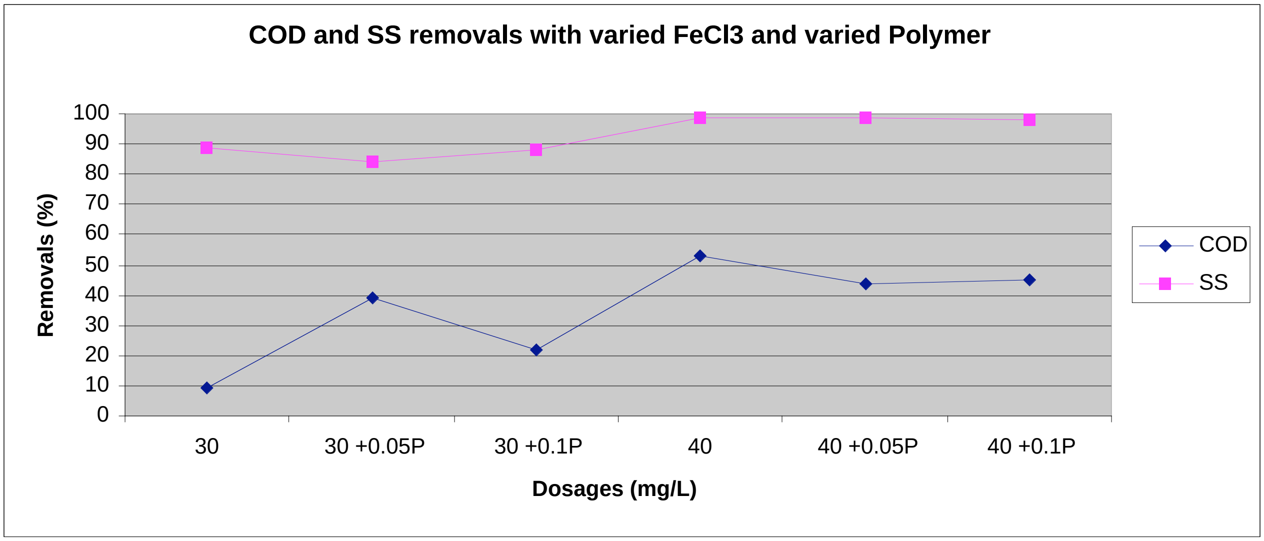


Figure G.25. Experiment 3A: COD and SS removals with varied $FeCl_3$ and polymer

The jar test above was used to try to confirm the $FeCl_3$ and polymer doses as shown in Figure G.24 before. Both $FeCl_3$ and polymer concentrations here were varied, in particular, to look for trends in the addition of polymers to the raw wastewater and 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 $FeCl_3$. The COD removals at 40 g/L $FeCl_3$ however were significantly high (50%) and only decreased with the addition of polymers. This test therefore pointed to using 40 mg/L $FeCl_3$ 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 $FeCl_3$. General conclusions can be made however regarding the general performance and effect of polymers on jar tests with $FeCl_3$ as the only coagulant:

(i) As FeCl3 doses increase, suspended solids removals increase. This effect is not as noticeable if SS removals are already very high;

(ii) The use of small polymer doses seems to display better COD removals with larger doses of FeCl3 (i.e. polymers used with concentrations smaller than 20 mg/L did not show large increases in removal efficiencies);

(iii) 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; and

(iv) Increasing polymer concentrations does not necessarily increase removals.

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 $FeCl_3$.

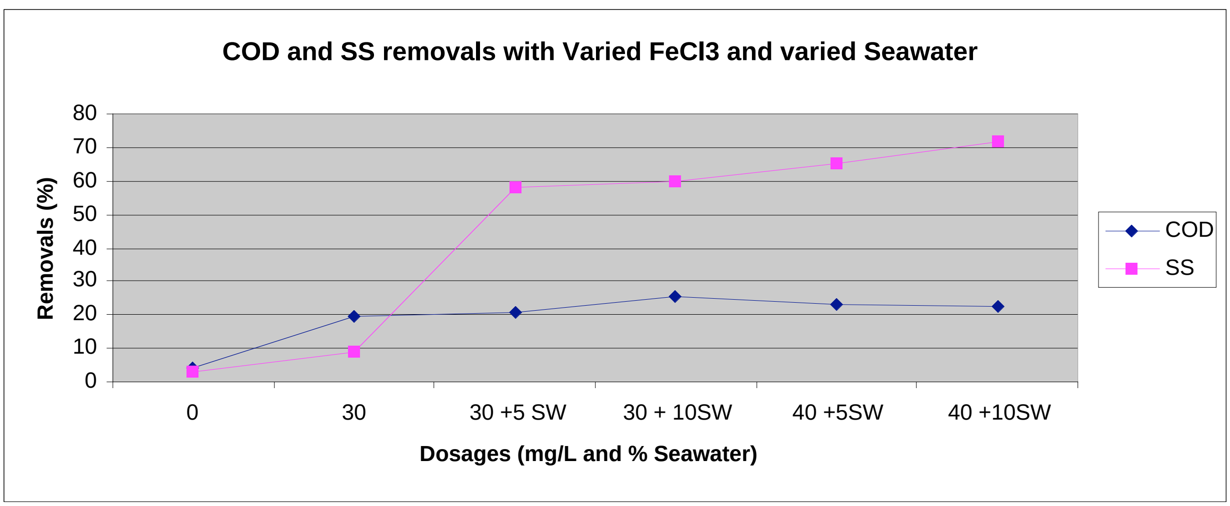


Figure G.26. Experiment 3B: COD and SS removals with varied $FeCl_3$ and seawater

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 F.3.

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

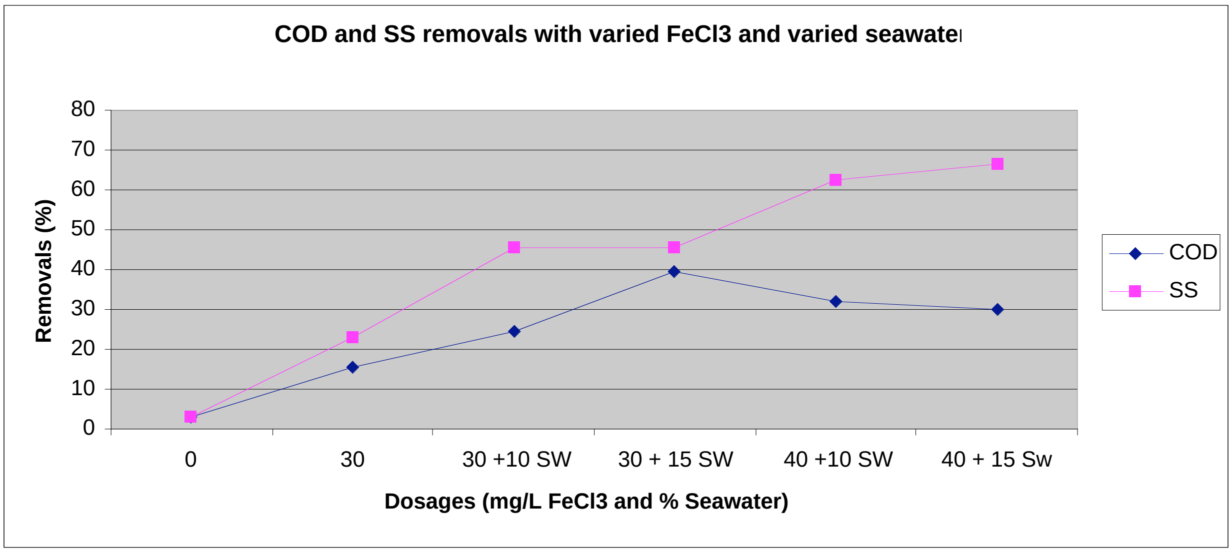


Figure G.27. Experiment 3B: COD and SS removals with varied $FeCl_3$ and seawater

In this test, various $FeCl_3$ concentrations were again tested with different seawater volumes and the SS and COD removals were consequently observed.

When $FeCl_3$ was tested again with the use of seawater, the observed removal efficiencies were lower than in previous tests (See Figure G.26 above). At a $FeCl_3$ 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 $FeCl_3$ and 5% seawater in Figure G.26 above. The addition of seawater only enhanced COD removals at relatively low $FeCl_3$ doses (i.e. less than 30 mg/L). When larger doses of $FeCl_3$ 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 F.3 on chloride interference with the COD Hach vial readings).

The general conclusions on the use of seawater and $FeCl_3$ in conjunction therefore are:

(i) Higher volumes of seawater seem to cause larger increases in SS and COD removals with smaller concentrations of $FeCl_3$ (i.e. 30 mg/L $FeCl_3$ and 15% Seawater yielded higher results than 40 mg/L $FeCl_3$ and 15% seawater);

(ii) COD removals seem to decrease as the percentage of seawater increases; and

(iii) 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).

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.

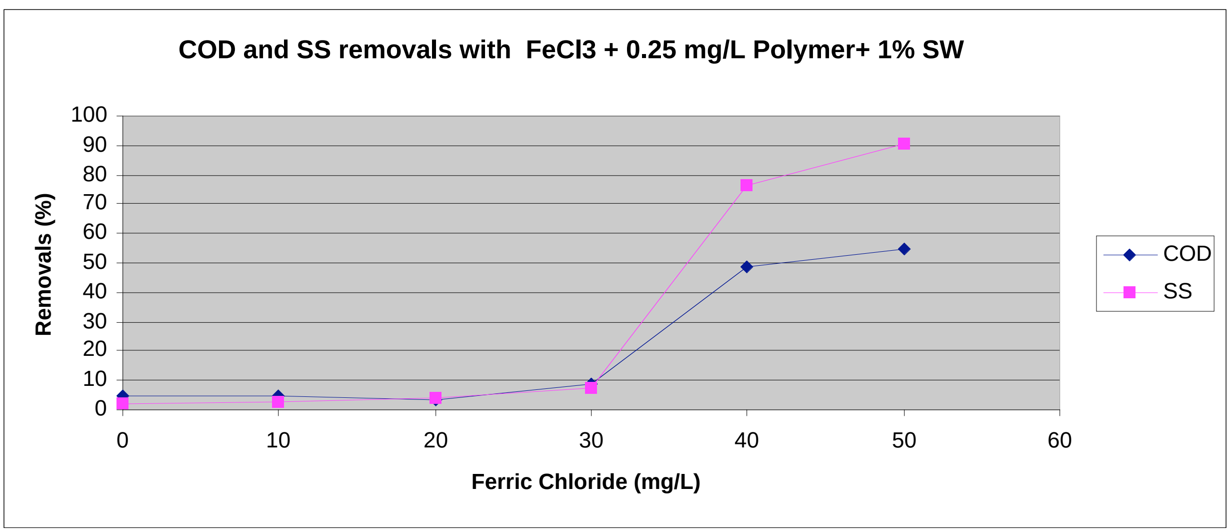


Figure G.28. Experiment 3C: COD and SS removals with varied FeCl3, 0.25 mg/L polymer and 1% seawater

In this jar test, varying $FeCl_3$ concentrations (ranging from 10 to 60 mg/L) were tested in conjunction with 0.25 mg/L 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 G.5 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 cause for the low removals in there jar tests.

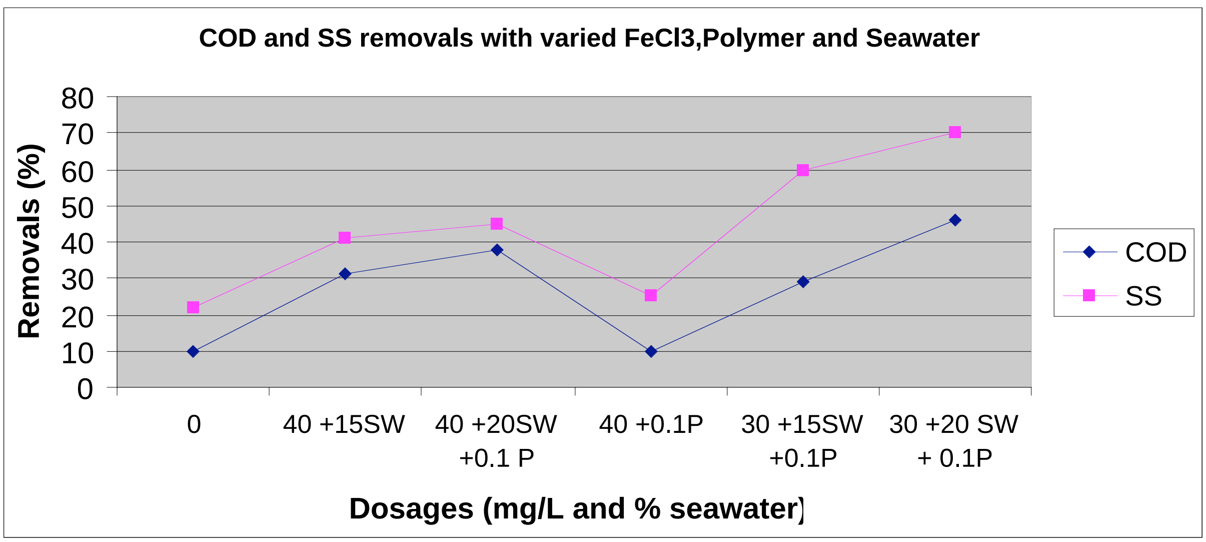


Figure G.29. Experiment 3B: COD and SS removals with varied $FeCl_3$, polymer and seawater

In this jar test, all of $FeCl_3$, seawater and polymer concentrations were varied to observe for trends in COD and SS removals.

When 0.1 mg/L polymer alone was added to the 40 mg/L $FeCl_3$, SS and COD removals were low compared to when 15% seawater was used with the same $FeCl_3$ dose. The use of seawater seems to be more effective in this jar test, therefore, at increasing SS and COD removals.
Using 40mg/L $FeCl_3$ with 20SW and 0.1 P instead of 40 mg/L $FeCl_3$ with 15% SW also did not yield significantly higher SS and COD removals. In fact, using 30mg/L $FeCl_3$, 0.1 mg/L P 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 $FeCl_3$, seawater and polymer are as follows:

(i) Polymer alone with $FeCl_3$ is not as efficient as seawater acting with $FeCl_3$;

(ii) When $FeCl_3$, seawater and polymer were used together, the use of smaller $FeCl_3$ concentrations performed as well as larger $FeCl_3$ concentration dosages; and

(iii) It is also important to note that the results from using 40 mg/L $FeCl_3$ 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.

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%.

Design Parameters for CEPT Plant

Based on the jar tests results displayed in Sections G.2 through G.5 above, and taking from the conclusions on the general trends that ferric chloride, seawater and polymers, the following Table G.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 G.7. Design parameters for CEPT plant

APPENDIX H – ANALYSIS OF JAR TESTS DATA FROM DEER ISLAND WASTEWATER TREATMENT PLANT IN BOSTON, U.S.

H.1. Introduction

The Boston Deer Island wastewater treatment plant (Figure H.1 below) is the second largest wastewater treatment plant in the United Sates 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 H.1. Deer Island Wastewater treatment plant in Boston

(http://www.bryant-engrs.com/projects/deer.htm)

H.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 H.2 through H.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 FeCl3	61	53	58	45
30 mg/L FeCl3	61	54	57	50

Table H.1. Summary of removal rates from pilot plant

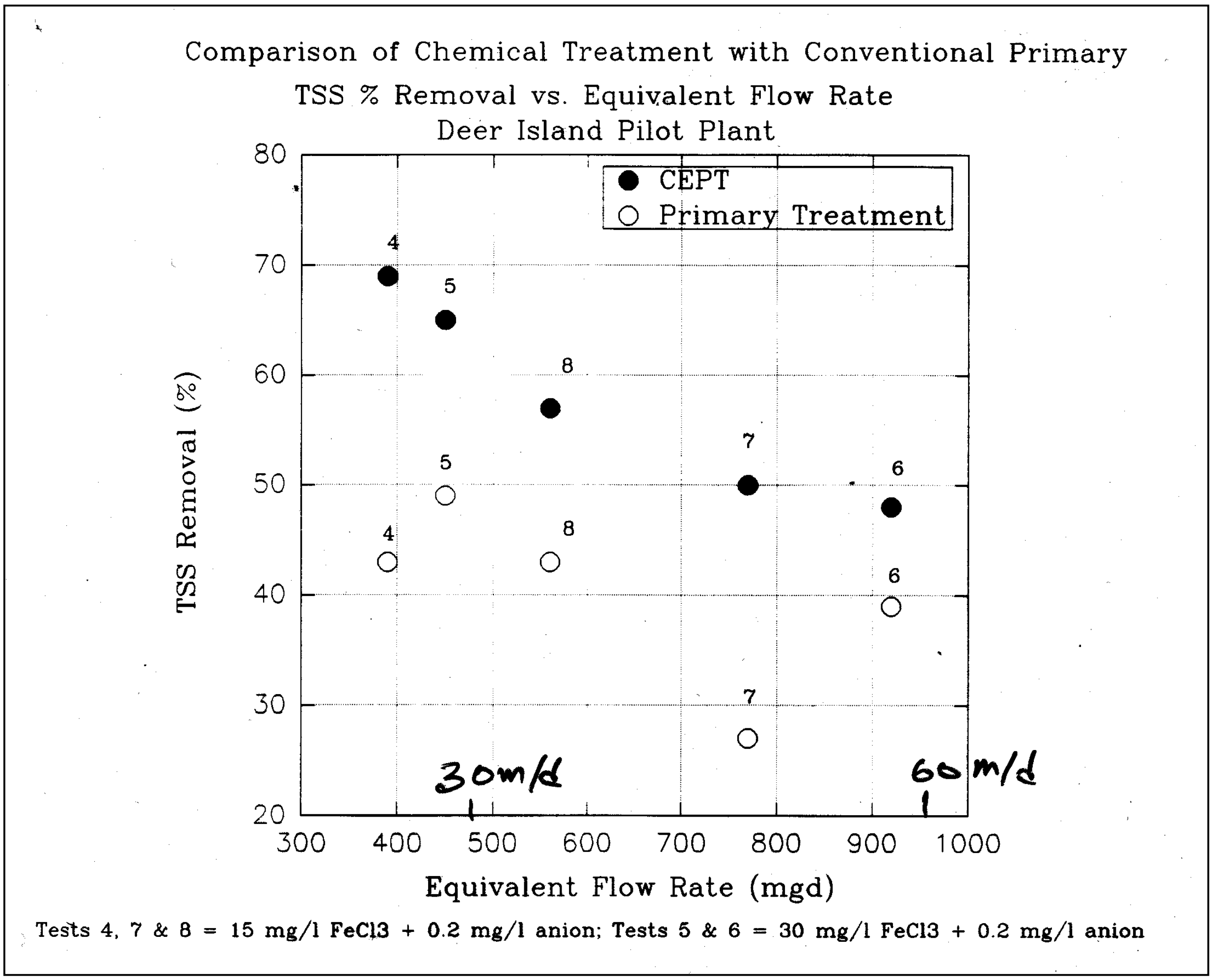


Figure H.2. TSS removals with conventional and CEPT treatment

Figure H.2 above therefore 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.

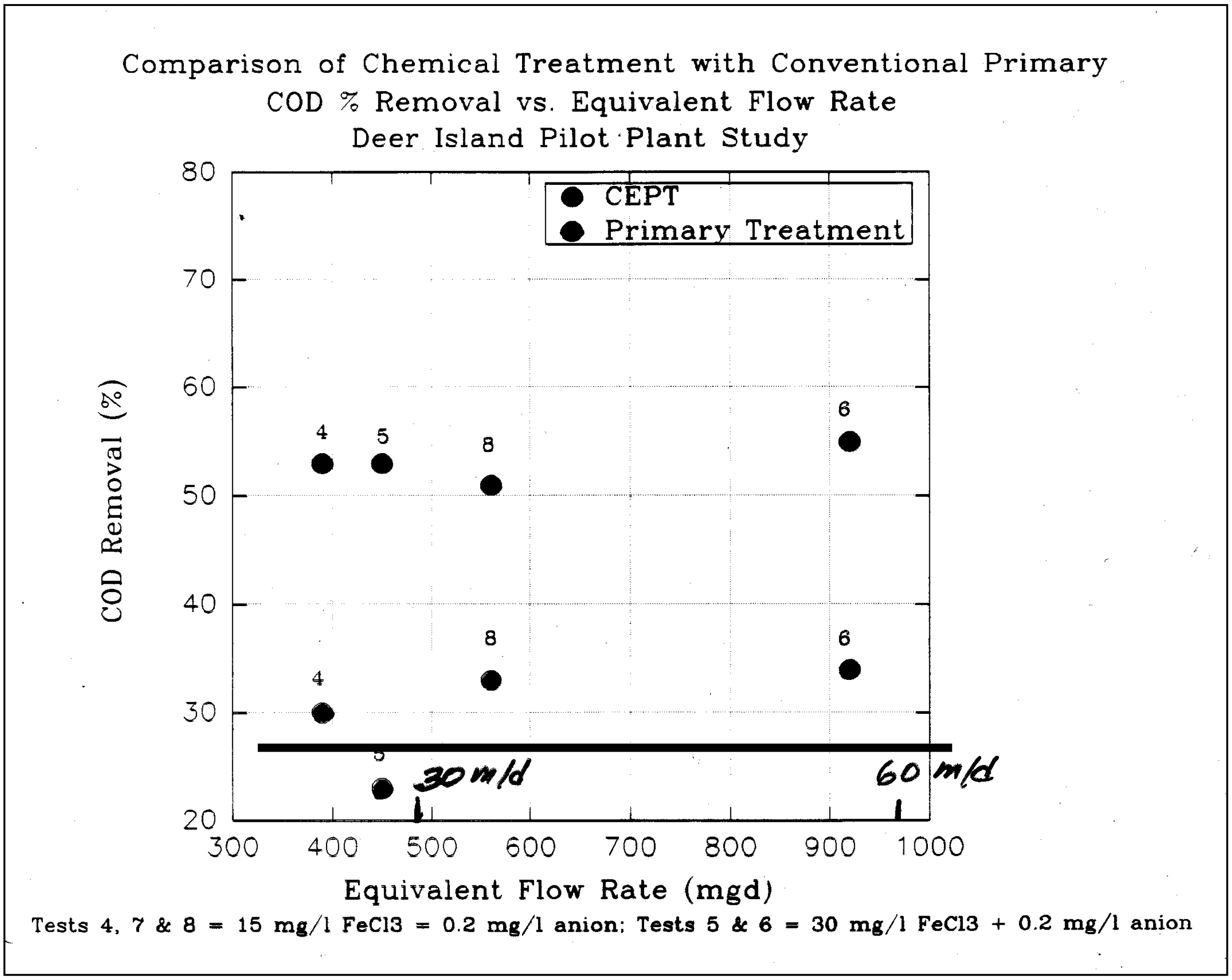


Figure H.3. COD removals at the Deer Island pilot scale test

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

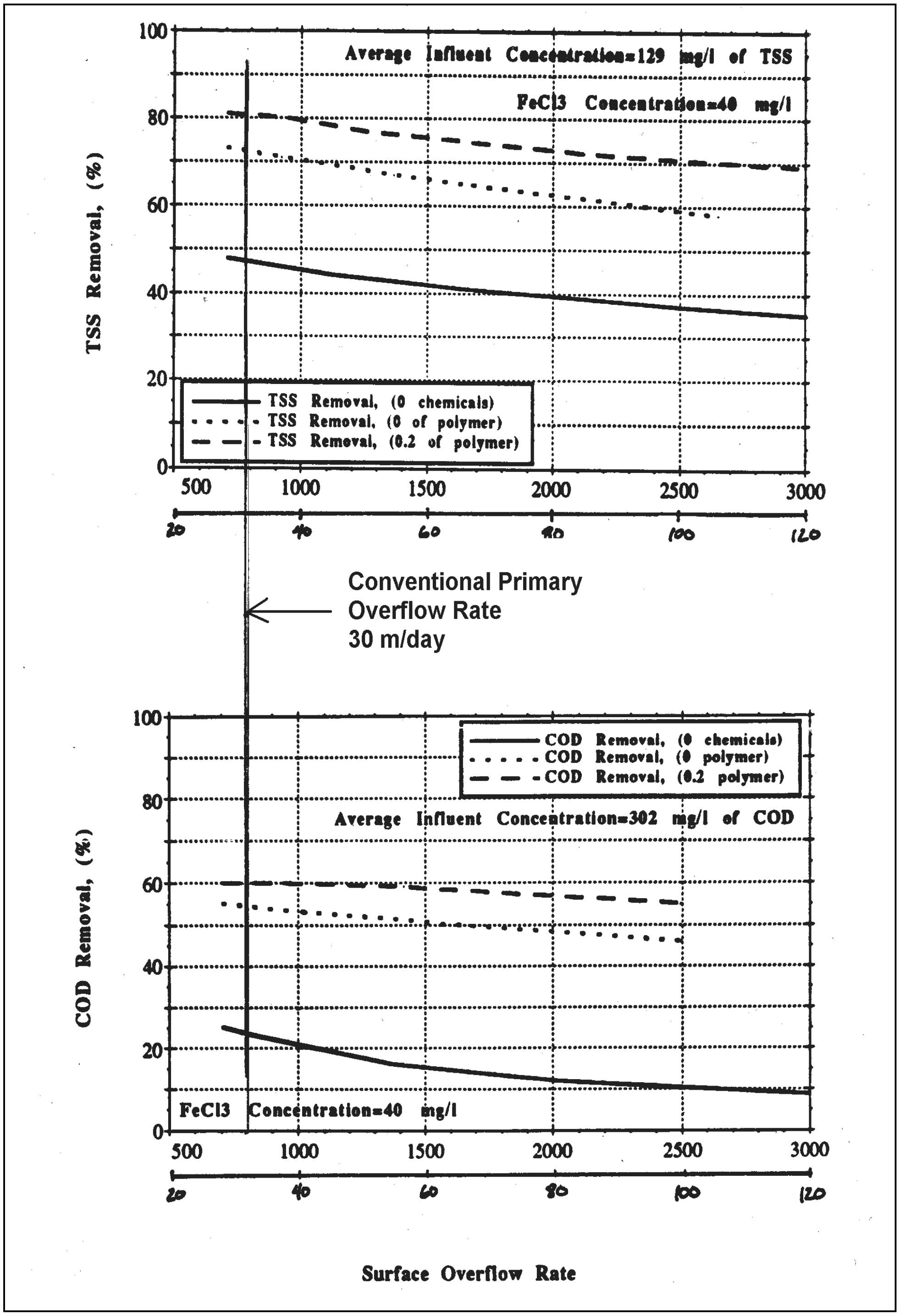


Figure H.4. Pilot scale removal rates

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.

H.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:

Conductivity

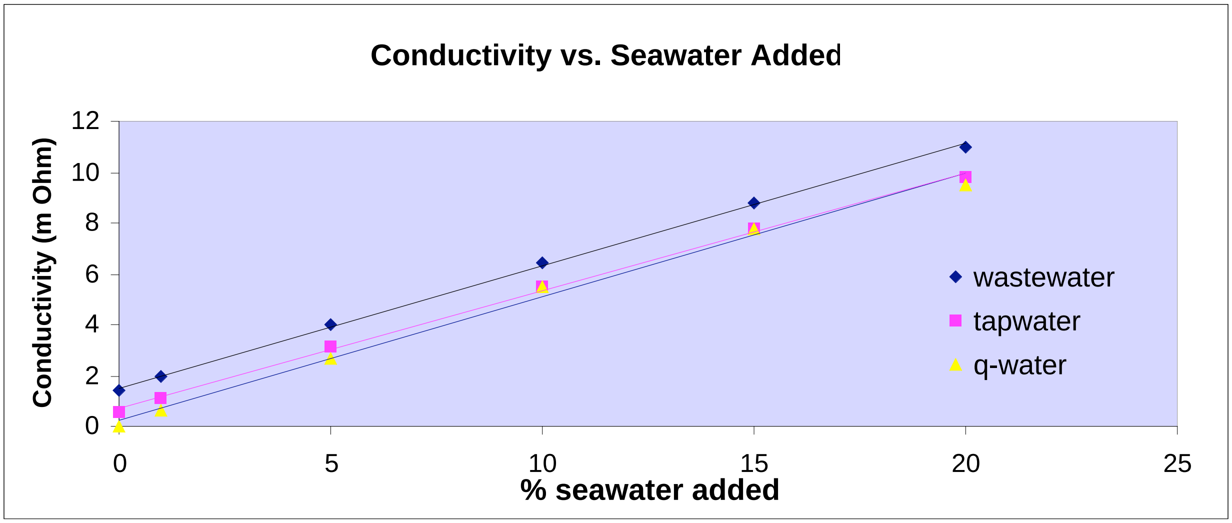


Figure H.5. Conductivity versus seawater added

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 H.5 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%.

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:

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

Where:

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

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

H.4. DEER ISLAND Data Analysis

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 Figures H.6, H.7 and H.8 below.

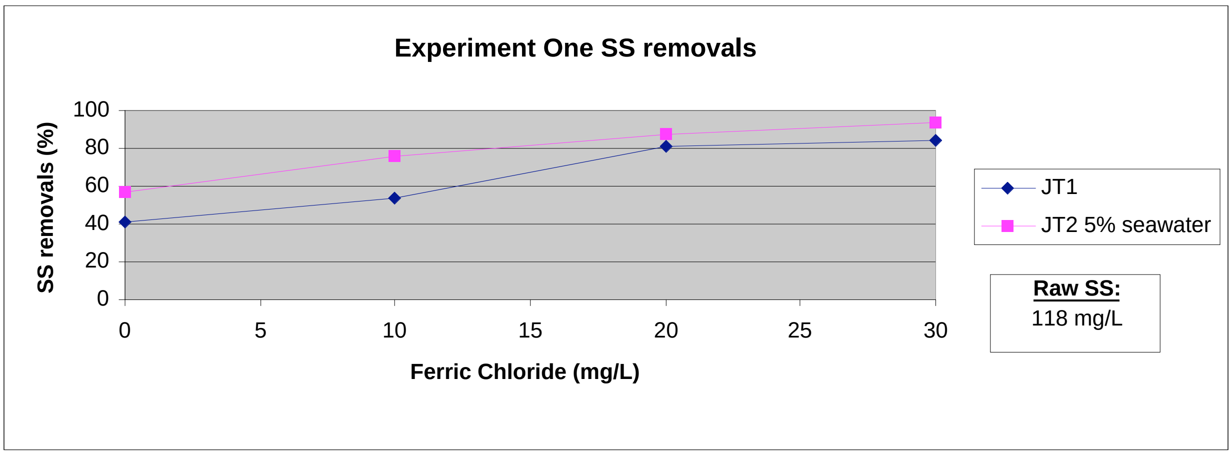


Figure H.6. Experiment One: SS Removals

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 H.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.

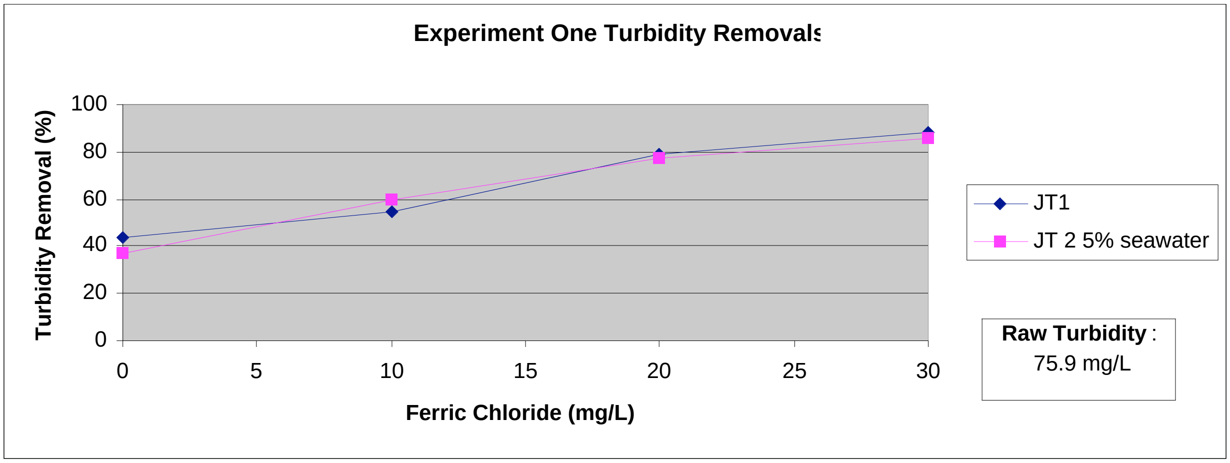


Figure H.7. Experiment One: Turbidity removals

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.

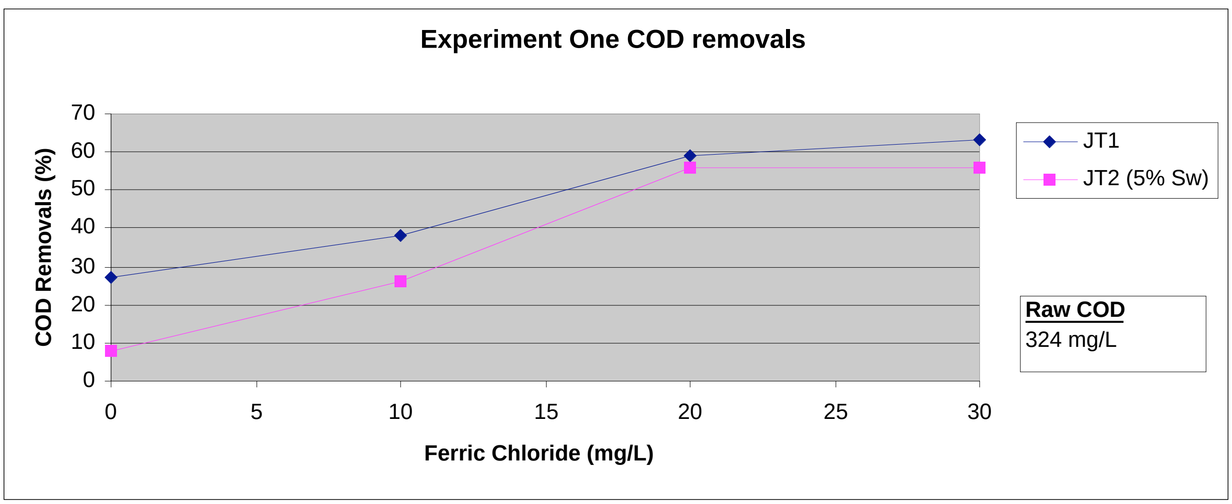


Figure H.8. Experiment One: COD removals

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 $FeCl_3$ was used and the difference in removals remained relatively approximate with doses of $FeCl_3$ higher than 20 mg/L. This points to a potential sensitivity of the effect of seawater for flows treated with low doses of $FeCl_3$. 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 $FeCl_3$ 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.

Experiment Two

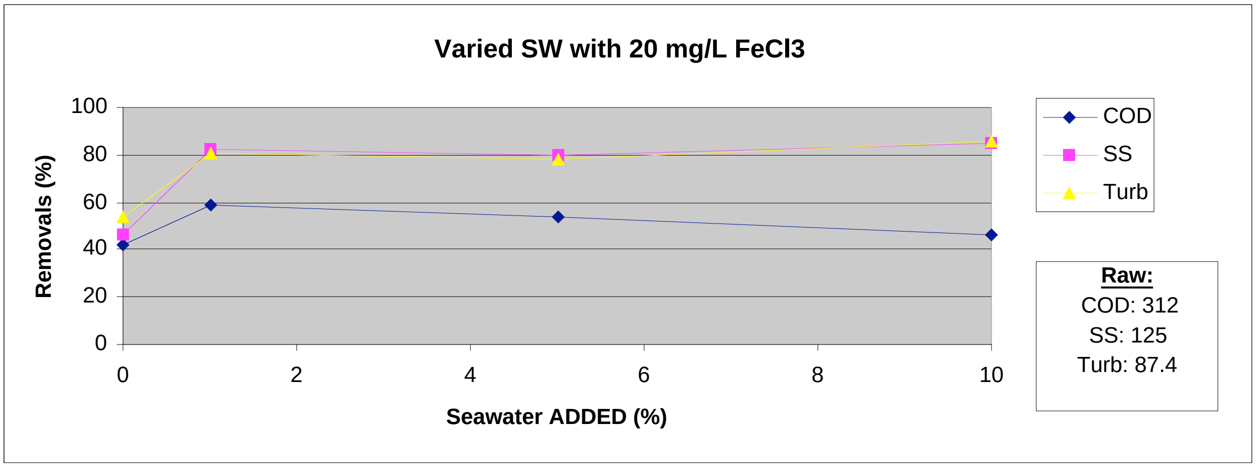


Figure H.9: Experiment Two: COD, SS, and turbidity removals

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 H.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%.

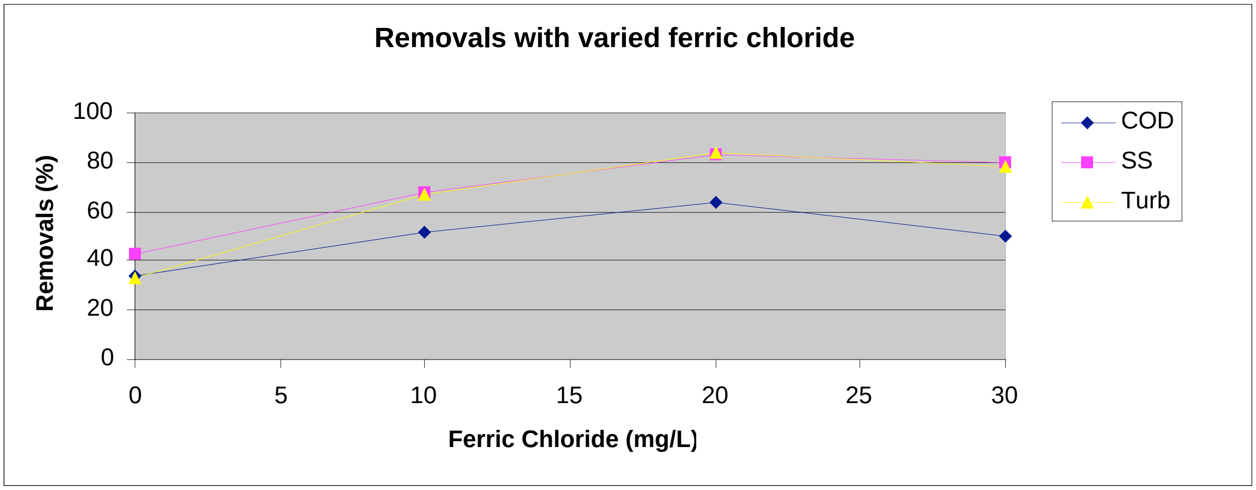


Figure H.10. COD, SS, and turbidity removals with varied $FeCl_3$

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 a specified volume of seawater (See Section H.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 $FeCl_3$. However, at 30mg/L $FeCl_3$, 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 $FeCl_3$ as a coagulant.

It is also important to compare the results from this test to those from Section H.4 above which help to show that the presence of seawater is responsible for higher SS and COD removals at relatively lower $FeCl_3$ concentrations.

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.

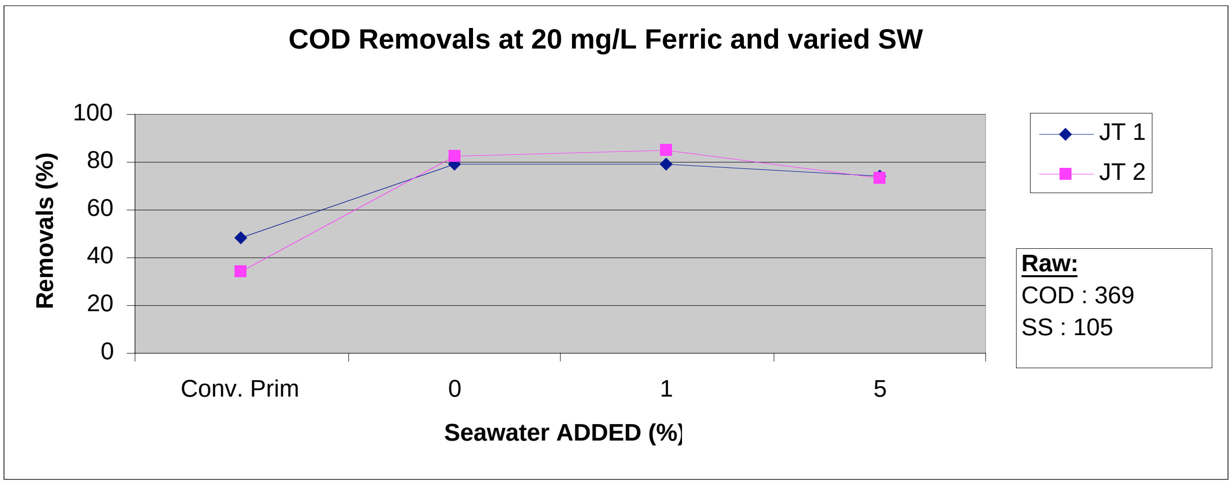


Figure H.11. COD removals with varied $FeCl_3$ and seawater

In this jar test, 20 mg/L $FeCl_3$ 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 H.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 $FeCl_3$, 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 sensitivity to lower $FeCl_3$ doses.

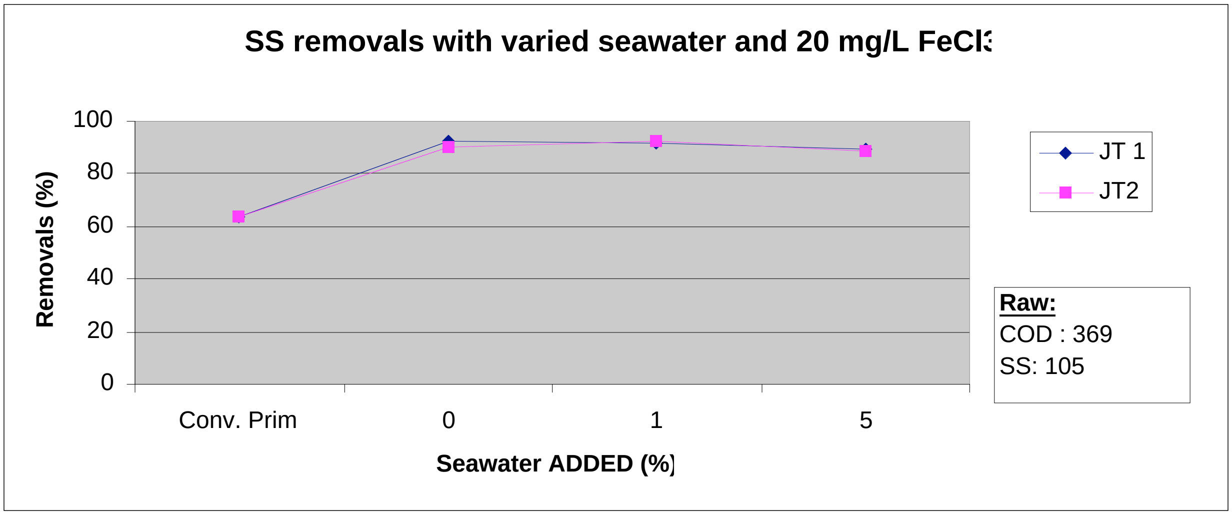


Figure H.12. SS removals with varied seawater and 20 mg/L $FeCl_3$

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 which remained constant at 93%.

Experiment Four

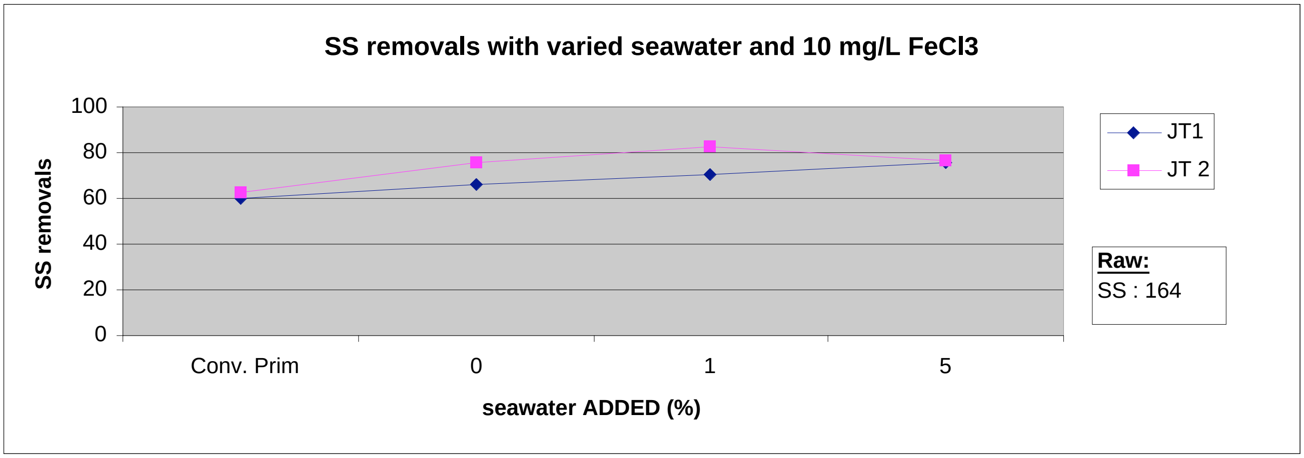


Figure H.13. SS Removals with varied seawater and 10 mg/L $FeCl_3$

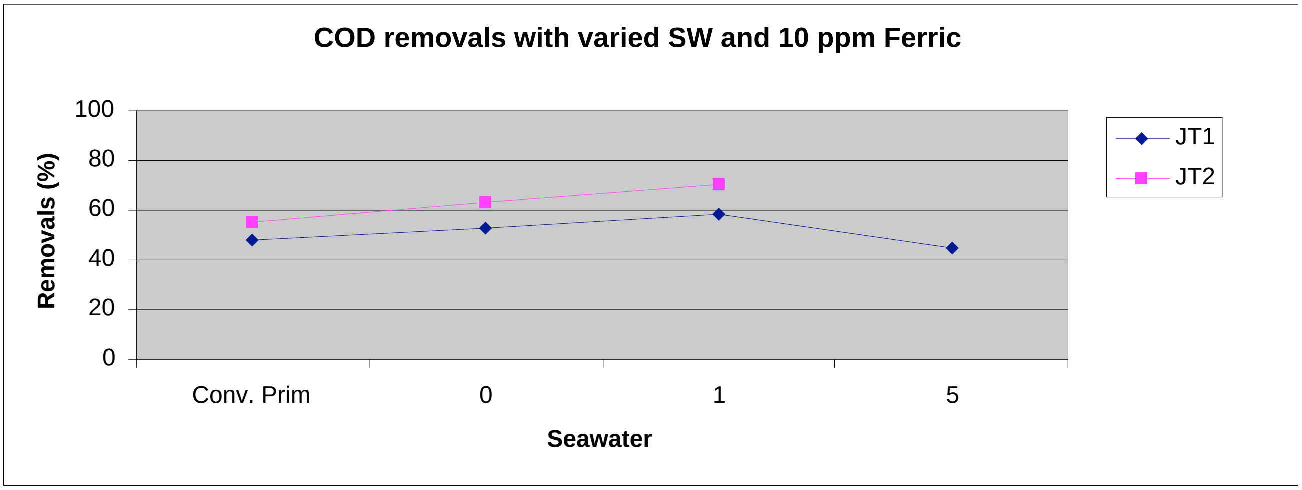


Figure H.14. COD Removals with varied seawater and 10 mg/L Ferric Chloride

APPENDIX I: ANALYSIS OF DISINFECTION DATA FROM PARATY, BRAZIL AND BOSTON, U.S.

I.1. Introduction

Constraints

Treated wastewater effluent is commonly discharged to a natural surface water body, such as a river or an ocean. Since surface waters generally sustain human life and are ecological habitat for large numbers of species, treated wastewater effluent must be disinfected to remove disease-causing organisms before it is discharged into 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 along with suspended solids during the sedimentation process, the settling process alone does not produce treated wastewater effluent that meets the regulations. 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):

(i) Ability to destroy infectious organisms under normal operation conditions;

(ii) Safety and ease of handling, storage, and shipping;

(iii) Absence of toxic residuals and harmful byproducts; and

(iv) Affordability of capital, and 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:

(i) Be present when fecal contamination is present;

(ii) Be present in equal or greater number than target pathogenic organisms;

(iii) Have same or greater survival characteristics in the environment as the target pathogenic organisms;

(iv) Not reproduce during the culturing procedure;

(v) Be cheap and easy to cultivate compared to the target pathogenic organisms; and

(vi) Be a member of the intestinal microflora of warm-blooded animals (Metcalf & Eddy, 2002).

Regulations in the U.S. and Brazil

Although no ideal indicator organism has been found, coliform is commonly used as an indicator of pathogenic organisms. Humans discharge approximately one hundred billion coliform bacteria per day per capita on average. Thus water is considered free from disease-producing organisms when there are no detectable coliform bacteria in 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, maximum fecal coliform concentration of 200 MPN/100 ml is the most common standard. According to state ocean water quality standards in California, which enforces some of the strictest regulations in the United States, the maximum 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 coliform in treated wastewater effluent discharged into natural waters is 1000MPN/100ml. Thus, maximum fecal coliform concentration of 200 MPN/100ml can be adopted as the effluent quality standard, considering the proposed locations of CEPT effluent discharge, which are near a public beach, and the regulations in United States and Brazil.

I.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, disinfectants would have to cover a wide range of wastewater quality. The characteristics for an ideal disinfection agent are shown in Table I.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
Non-corrosive and non-staining	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 I.4. Characteristics of an ideal disinfectant (Metcalf & Eddy, 2002)

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

I.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 processes are compressed chlorine 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 I.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_2Cl	51.5	2	69	138

Table I.5. Actual and available chlorine in compounds containing chlorine (Metcalf & Eddy, 2002)[1]

The disinfection efficiency of chlorine is dependent on the characteristics of wastewater, as summarized in Table I.3. 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)	Interferes with disinfection; and 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 trihalomethanes (THMs)
pH	Affects distribution between hypochlorous acid and hypochlorite ions, and among the various chloramine species
Total Suspended Solids (TSS)	Shields embedded bacteria and increases chlorine demand

Table I.6. Wastewater characteristics affecting chlorination performance (EPA, 1999)

As mentioned in the table above, the level of suspended solids in treated wastewater affects the performance of chlorination. Evidently, suspended solids and soluble organic compounds are important in disinfection efficiency (Robert Armon et al., 1995). Since suspended solids surround and shield microorganisms, disinfection agents cannot penetrate through suspended solids, and consequently cannot inactivate the target microorganisms. Moreover, low suspended solids removal efficiencies can indicate that the concentration of coliform in treated wastewater effluent is not much different from the concentration in the influent, since coliform is adsorbed in the suspended solids (Metcalf & Eddy, 2002; Water Quality and Treatment, 2000).

[1] Actual Chlorine = % of Cl_2 in compounds, w/w; available Chlorine = Actual Chlorine * Chlorine Equivalent

CEPT produces a treated wastewater effluent that can be effectively disinfected. As shown in Section 6.1, suspended solids (SS) removal efficiency in CEPT reaches approximately 85 %, which is an acceptable level for the disinfection of treated wastewater effluent. Moreover, since seawater addition increases the efficiency of SS removal, chlorine may be more effective when seawater is added to the raw wastewater in CEPT. Typical chlorine dosages are showed in Table I.4 below:

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 I.7. Typical chlorine dosages (Metcalf & Eddy, 2002)[1]

According to the table above, the concentration of fecal coliform is rarely reduced by primary treatment. Due to the high levels of fecal coliform, the effluent of primary treatment cannot be disinfected cost-effectively by chlorination. On the other hand, the activated sludge treatment process reduces the concentration of fecal coliform by three-orders of magnitude, and its effluent can be disinfected cost-effectively. Since the suspended solids removal efficiency of CEPT is as high as that of activated sludge treatment, similar reduction of fecal coliform is expected from CEPT. Consequently, CEPT effluent can be disinfected cost-effectively.

[1] Typical chlorine dosages are based on combined chlorine, unless otherwise indicated, required to achieve different effluent total coliform disinfection standards for various wastewaters, and 30 minute contact time.

Types of Chlorine

Molecular Chlorine (Cl_2)

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 (Mecalf & Eddy, 2002; Water Quality and Treatment, 2000).

Sodium Hypochlorite (NaOCl)

Sodium hypochlorite can be supplied in liquid form, and available chlorine is usually 12.5 to 17 percent, at the time of manufacturing. The decomposition rate of the solution depends on its concentration, and exposure to light and heat. Therefore it must be stored in a cool location inside 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 below. Although it is possible to generate sodium hypochlorite from sodium chloride (NaCl) or seawater, the use of onsite generation systems is limited due to high cost of electric power.

The hydrolysis reaction of sodium hypochlorite is as follows:

$$\mathbf{NaOCl + H_2O \_ HOCl + NaOH} \qquad \textbf{Eq. I-1}$$

Calcium Hypochlorite ($Ca(OCl)_2$)

Calcium hypochlorite is available in dry or wet form, and is commonly used to treat the wastewater effluent from textile and paper mills under controlled conditions (PPG Industires, 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 and dry location, separated from other chemicals in corrosion-resistant storage containers. Calcium hypochlorite is more expensive than molecular chlorine, and its strength is reduced on storage. Handling of calcium hypochlorite can be difficult, since metering pumps, piping, and valves can be clogged due to calcium hypochlorite, which is likely to crystallize.

The hydrolysis reaction of calcium hypochlorite is as follows:

$$\mathbf{Ca(OCl)_2 + 2H_2O \_ 2HOCl + Ca(OH)_2} \qquad \textbf{Eq. I-2}$$

Reactions of Chlorine

Hydrolysis of Chlorine

When molecular chlorine is added to water, it equilibrates with aqueous chlorine, which is hydrolyzed to form hypochlorous acid, chloride ion, and proton as described in Equation I-3 below.

$$\mathbf{Cl_{2(aq)} + H_20 \_ HOCl + H^+ + Cl^-} \qquad \textbf{Eq. I-3}$$

Hypochlorous acid, a weak acid, dissociates into the hypochlorite ion and to a proton, as follows:

$$\mathbf{HOCl \_ OCl^- + H^+} \qquad \textbf{Eq. I-4}$$

The concentration of hypochlorous acid and hypochlorite ion is determined by the dissociation constant (pKa @7.6, at 25℃) 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).

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, as follows:

$$\mathbf{NH_4^+ + HOCl \_ NH_2Cl + H_2O + H^+}$$
$$\mathbf{NH_2Cl + HOCl \_ NHCl_2 + H_2O + H^+} \qquad \textbf{Eq. I-5}$$
$$\mathbf{NHCl_2 + HOCl \_ NCl_3 + H_2O + H^+}$$

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 (NH_2Cl), dichloramine ($NHCl_2$), and trichloramine (NCl_3)) 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 when it is added in water. When these reactions are completed, the additional chlorine reacts with ammonia to form chloramine, between points A and B (See Figure I.1).

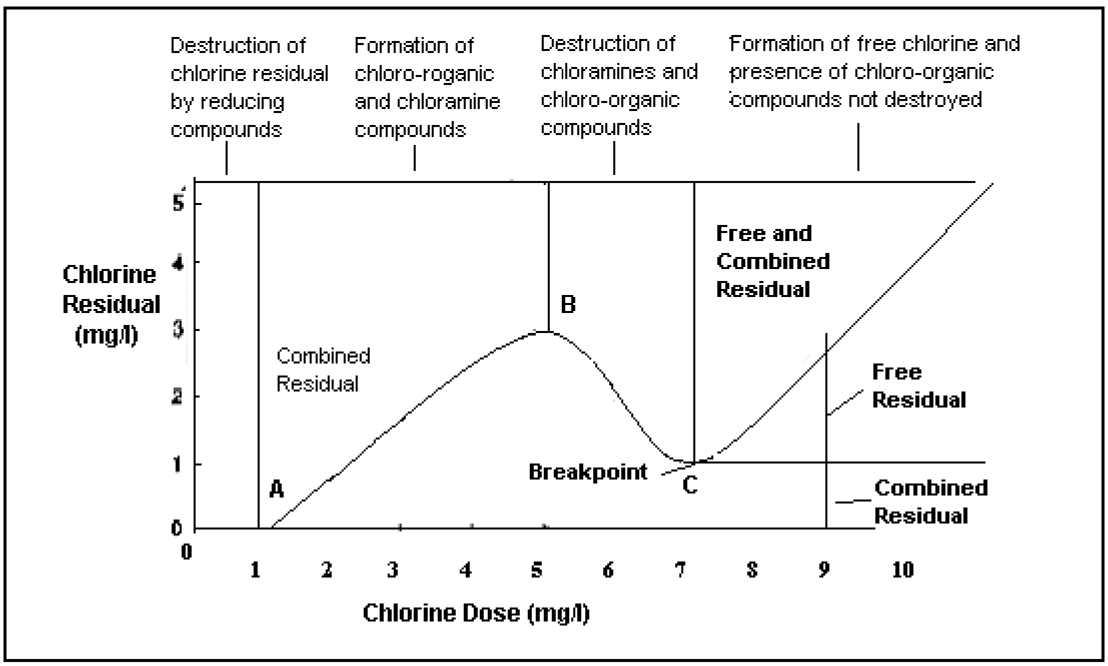


Figure I.1. Chlorine breakpoint (Metcalf & Eddy, 2002)

Between point B and point C, the breakpoint, chloramine is 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, effective disinfection can be assured. Therefore, the breakpoint dosage is the minimum amount of chlorine to be added to water. The amount of chlorine required to achieve a desired level of residual is called the “chlorine demand” (Metcalf & Eddy, 2002).

Chlorine Dioxide (ClO_2)

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

$$\mathbf{ClO_2 + 5e^- + 4H^+ \_ Cl^- + 2H_2O} \qquad \textbf{Eq. I-6}$$

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 2.63 g/L of chlorine (Water Quality and Treatment, 2000). Based on this information and Table I.1 from Section I.2 above, the required chlorine dioxide dosage for disinfection can be calculated. Because the data on the appropriate dosage of chlorine dioxide is limited, however, site-specific testing is recommended to determine appropriate dosage range.

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

The disinfection byproducts (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 and the reduction of the chlorine dioxide. The chlorate ion is produced by the oxidation and the photolysis of chlorine dioxide, and the impurities in the sodium chlorite, which is a source of chlorine dioxide generation.

Dechlorination

Chlorine is one of the common disinfectants for pathogenic organisms that also endangers human health, and 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 concentrations with acute and choric effects are 0.013 mg/l and 0.0075 mg/l, respectively. Since chlorine disinfection normally produces a total residual chlorine concentration of 1.0 to 5.0 mg/l in the effluent, the disinfected wastewater must be dechlorinated before it can be discharged safely into the receiving surface water.

The most common dechlorination agent is sulfur dioxide (SO_2). Sodium sulfite (Na_2SO_3) and sodium metabisulfite ($Na_2S_2O_5$) and activated carbon have also been used as dechlorination agents. Table I.5 shows dechlorination reaction associated with each dechlorination agent and theoretical ratio of residual chlorine to dechlorination agent.

Chemical	Reaction	Chemical Use Ratio
Sulfur Dioxide	SO_3+ Cl_2 + $2H_2O$ _ H_2S0_4 + 2HCl	1.1
Sodium Sulfite	Na_2SO_3+ Cl_2 + H_2O _ Na_2S0_4 + 2HCl	1.8
Sodium Metabisufite	$Na_2S_2O_5$ + $2Cl_2$ + $3H_2O$ _ $2NaHSO_4$ + 4HCl	1.5
Sodium Bisulfite	Na_2HSO_3+ Cl_2 + H_2O _ $NaHS0_4$ + 2HCl	1.5
Hydrogen Peroxide	H_2O_2+ Cl_2 _ 2HCl + 0_2 (g)	0.5

Table I.8. Dechlorination agents and reactions (The Dow Chemical Company, 2000)

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 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 increases by approximately 30 to 50% if dechlorination is added, although chlorination is the most inexpensive way of disinfection.

Environmental impacts associated with using chlorination

After the dechlorination process of 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 the treatment. Regrowth also occurs because of the lack of predators such as protozoa. Due to this problem, it is important to maintain an adequate concentration of residual chlorine in effluent discharging into the nature. Typical residual concentration is from 0.1 to 0.5 ppm for free available chlorine, and 2 ppm for combined chlorine.

Another source of environmental impact is disinfection byproducts (DBPs). It has been reported that very small amounts of DBPs can negatively affect humans as well as aquatic lives. Residual chlorine in dechlorinated wastewater produces chlorinated organic byproducts by reacting with organic compounds. Among these 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 rate of formation of DBPs depends on the presence of organic substances, free chlorine concentration, bromide concentration, pH, and temperature. The principal means of controlling the formation of DBPs in wastewater is not to add free chlorine directly, since the reactivity for the formation of byproducts is higher for free chlorine as compared to chloramine. Although the use of chloramine can prevent forming high levels of DBPs, alternative disinfection means, such as UV radiation, should be considered if specific precursors of DBPs, such as humic materials, are present in water.

I.4. Alternative Disinfection Agents

Although chlorine is a highly effective disinfectant, alternative disinfection methods are considered for the following concerns associated with its use:

(i) The high risk of transportation of chlorine;

(ii) Potential health risks to treatment plant operators due to the high toxicity of chlorine;

(iii) Formation of odorous compounds by the reaction with organic compounds in wastewater;

(iv) Formation of carcinogenic DBPs by the reaction with organic compounds; and

(v) Toxicity of residual chlorine in the treated wastewater effluent, and its negative impact on aquatic lives.

Ozone, ultraviolet (UV) radiation, and peracetic acid are alternative disinfection agents, discussed in below.

Ozone

Effectiveness

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

$$\begin{aligned} &\mathbf{O_3 + H_2O \ \_\ HO_3^+ + OH^-} \\ &\mathbf{HO_3^+ + OH^- \ \_\ 2HO_2} \qquad\qquad \textbf{Eq. I-7} \\ &\mathbf{O_3 + HO_2 \ \_\ HO + 2O_2} \\ &\mathbf{HO + HO_2 \ \_\ H_2O + O_2} \end{aligned}$$

The free radicals, HO_2 and HO, are very good oxidation agents and are highly active in the disinfection process. These radicals also oxidize other impurities in water. The typical values of ozone demand are shown in Table I.6 below:

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 I.9. Typical ozone dosages[1] (Metcalf & Eddy, 2002)

The table above shows the same figures as Table I.4. Although ozone is more effective with the inactivation of viruses and bacteria as compared to chlorine, the effluent of primary treatment cannot be disinfected efficiently with ozone. However, the contact basins for disinfection by ozone could be smaller than the chlorine contact basins, since ozone can destroy chlorine-resistant organisms with relatively short contact time of approximately 10 to 30 minutes (EPA, 1999).

Advantages and Disadvantages

The primary advantage of disinfection by ozonation is that ozone does not produce halogenated organic matter, in contrast to chlorine compounds. Moreover, taste, odor, and color of the treated effluent can be controlled, and the concentration of dissolved oxygen in the treated effluent can be elevated during ozonation, since ozone readily decomposes into water and oxygen.

The primary disadvantage of disinfection by ozonation is that ozone can produce bromate, which is harmful to human health, when the treated effluent contains raw bromide at high pH. Ozone may also produce oxygenated byproducts and assimilable organic carbon, which can be used by

[1] Typical ozone dosages, based on 15 minute contact time, required to achieve different effluent coliform disinfection standards for various wastewaters.

bacteria for regrowth. The toxic byproducts of ozonation are usually unstable, and dissociate quickly in water, persisting only for minutes.

Cost

Ozonation is more expensive than chlorination, in terms of capital and O&M expenses, and its use is not appropriate for treated effluent with high levels of suspended solids, BOD, or COD. In general, ozonation is not appropriate in areas where economical treatment of wastewater is desired. The costs of various disinfection methods is summarized and compared later in Table I.7.

Ultraviolet (UV) Radiation

Effectiveness

The range of an ultraviolet (UV) wave is between 40nm and 400nm, and the germicidal range of UV radiation 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 suspended solids.

Advantages and Disadvantages

The main advantage of UV radiation as a disinfectant is that UV radiation neither forms disinfection byproducts nor have toxic residuals, in contrast to chlorine compounds. UV radiation 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, the main disadvantage of disinfection by UV radiation is that it is ineffective for treated wastewater effluent with high levels of turbidity and suspended solids. It has been shown that disinfection with low intensity UV lamps is not effective for the treated effluent with TSS levels above 30 mg/l (EPA, 1999). Moreover, since UV radiation does not have residual effects, microorganisms can regrow after UV radiation through a repair mechanism.

Cost

UV radiation is more expensive than chlorination, although its costs have recently decreased with improved technology and competition between suppliers. The majority of the costs of UV radiation are accounted for by the costs of facilities and operation and maintenance (O&M), which includes electric power. However, the cost of UV radiation is comparable to the cost of chlorination, when the costs of dechlorination and fire codes are added to the latter (EPA, 1999).

Peracetic Acid

Effectiveness

Peracetic acid (PAA, CH_3COOOH) is a very strong oxidizing solution containing peracetic acid, glacial acetic acid, hydrogen peroxide, and water at equilibrium.

$$\mathbf{CH_3COOOH + H_2O \_ CH_3COOH + H_2O_2} \qquad \textbf{Eq. I-8}$$

PAA performs better than sodium hypochlorite against vibrio choleral species (Baldry et al., 1995), and is effective for sewage treatment, especially for cholera control in warm climates. Its efficiency is higher at 30 ℃ than at 20 ℃. PAA concentration of 10 ppm with contact time of 30 minutes easily reduces the concentration of fecal coliform in the treated wastewater effluent to 1,000 CFU/100ml, to meet the corresponding guidelines of the World Health Organization (WHO). However, much higher dosage of 400 ppm with contact time of 20 minutes is required to meet the more stringent standards for agricultural reuse (2 CFU/100ml of total coliform), since increased concentration of PAA and contact time do not substantially improve its efficiency against total coliform bacteria (Sanchez-Ruiz et al., 1995).

Advantages and Disadvantages

PAA has been used as a disinfectant for years in various industries, and research regarding its use as a wastewater disinfectant began in the late 1980s. PAA is included among 5 disinfectants by the EPA 1999 report, despite the lack of quantitative information regarding the activity of PAA against the microorganisms in water.

The desirable attributes of PAA listed in the EPA's report are: (i) the absence of persistent residuals and disinfection byproducts (DBPs); (ii) independence of pH; short contact time; and (iii) high effectiveness as a bactericide and virucide. The main advantage of using PAA as a disinfection agent is that PAA hydrolyzes and produces acetic acid and hydrogen peroxide, which are readily biodegradable in water.

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

Cost

Disinfection by addition of PAA is more expensive than chlorination. For example, 1 lb of 5% PAA solution costs US$ 44, which is 10 times more expensive than sodium hypochlorite solution, according to Industrial Water Treatment Bulletin by Houghton Chemical Corporation. Moreover, the total cost of disinfection using PAA is the most expensive including operation and maintenance costs among UV, PAA, and ozone, according to the pilot investigation performed by L. Liberti and M. Notarnicola in 1999. The cost estimation of various methods of disinfection is summarized in Table I.7 below:

Disinfectant	Dose	Flow Rate (m^3/h)	Total Coliform Target Achieved (CFU/100ml)	O&M Costs (US$/1000$m^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 I.10. Cost estimation for disinfection of treated wastewater effluent using UV radiation, chlorination, chlorination/dechlorination, PAA, and O_3 (Liberti and Notarnicola, 1999)[1]

[1] Currency exchange for EURO to US$ is 1 (REGAL-Chlorinators Inc.)

Evidently, PAA cannot be used in Paraty since it is much more expensive than chlorine, and economic feasibility is an important criterion in the selection of a wastewater treatment system most suitable for the city of 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, alkalies, 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 when handling PAA, since it can cause severe health problems such as eye irritation, skin burns, and gastrointestinal tract problems.

I.5. Data Collection and Analysis

Procedure

The disinfection of CEPT effluents by addition of peracetic acid and by addition of chlorine is studied using the effluents of jar tests. To measure the concentration of total coliform and fecal coliform in the treated wastewater effluent, after the disinfection process, samples are injected into ColiPlate™, which has 96 micro-wells, and incubated for 24 hours at 35ºC. Total coliform tests positive when the wells are blue, and fecal coliform tests positive when the wells are fluorescent under UV light (See Figure I.2 below).

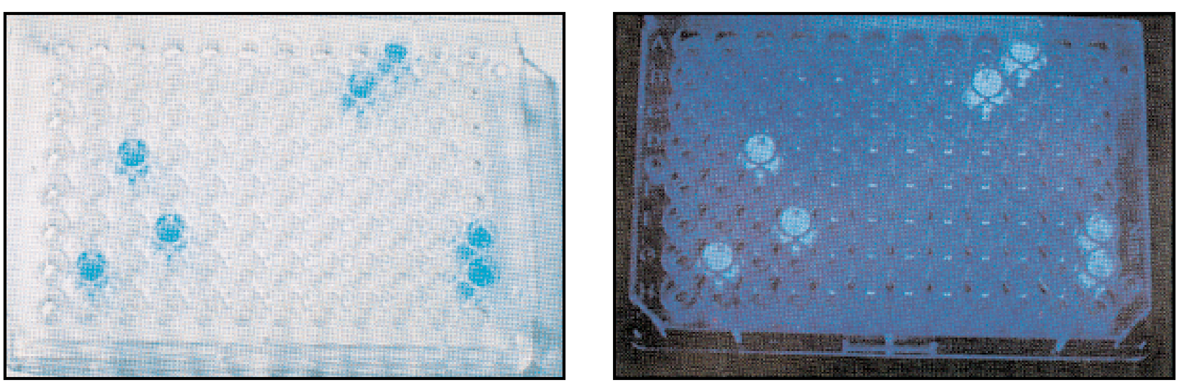


Figure I.2. Blue color indicative of total coliform (left); Fluorescence indicative of fecal coliform (right) (http://www.ebpi-kits.com/)

Analysis of Paraty Data

CEPT effluent treated with 30 mg/l of ferric chloride is used in the experiment. The levels of COD and SS of this effluent are 19.8 mg/l and 9.1 mg/l, respectively. To compare the efficiencies of coliform removal of peracetic acid (PAA) and chlorine, 5, 10, 15, and 20 mg/l of PAA, and 20 mg/l of chlorine are added to the effluent for a contact time of 30 minutes. The reduction of coliform bacteria by the addition of various concentrations of PAA and chlorine are shown in Figure I.3 below:

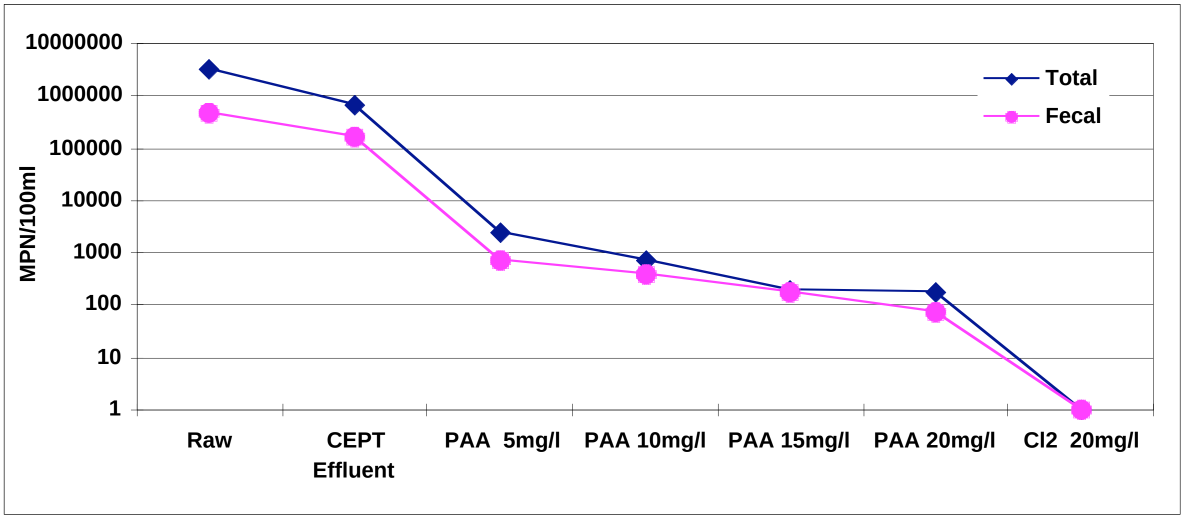


Figure I.3. Coliform reduction in CEPT effluent by PAA and chlorine

As shown in the Figure I.3, coliform concentrations in treated wastewater decreases with increased dosage of PAA, although PAA does not eliminate all coliform bacteria, while 20mg/l of chlorine does. Disinfection with 15 mg/l of PAA achieves a fecal coliform concentration below 200 MPN/100ml, which is the recommended level as discussed in Section I.1.

It is important to note that there is only one set of disinfection test data from Paraty, due to unrepresentative quality of jar-test effluents. Considering the high cost of PAA, however, it is evident that PAA is not a good disinfection agent to be used Paraty.

Analysis of Deer Island Data

For the remainder of disinfection experiments, samples of wastewater from the Deer Island wastewater treatment plant, in Boston, U.S., are used. The raw wastewater is treated with 20 and 10 mg/l of ferric chloride, and the effluent is used for the disinfection tests. To examine the effect of adding seawater during CEPT on the coliform reduction, 1% and 5% of seawater are added during CEPT. The effluent of conventional primary treatment and CEPT is disinfected with 5 and 10 mg/l of Cl_2 for 30 minutes of contact time. The fecal coliform levels in the raw sewage are 800,000 MPN/100ml for the effluent of conventional treatment, and 500,000 MPN/100ml for the effluent of CEPT, and these concentrations are acceptable, based on the average fecal coliform concentration of 918,000 MPN/100ml, in the influent of the Deer Island wastewater treatment plant.

Conventional Effluent vs. CEPT Effluent

According to the collected data, there is a significant difference of fecal coliform reduction between the effluent of conventional primary treatment and the effluent of CEPT. Fecal coliform level is reduced to below 10^5 MPN/100ml by CEPT without the addition of seawater, while it is reduced to below 10^6 MPN/100ml by the conventional treatment. The differences of fecal coliform reduction between conventional treatment and CEPT can be explained by the relationship between SS removal efficiency and coliform removal efficiency. Figures I.4 and I.5 below show the levels of COD, SS, fecal coliform in the effluents of conventional treatment, and CEPT using 20mg/l of ferric chloride.

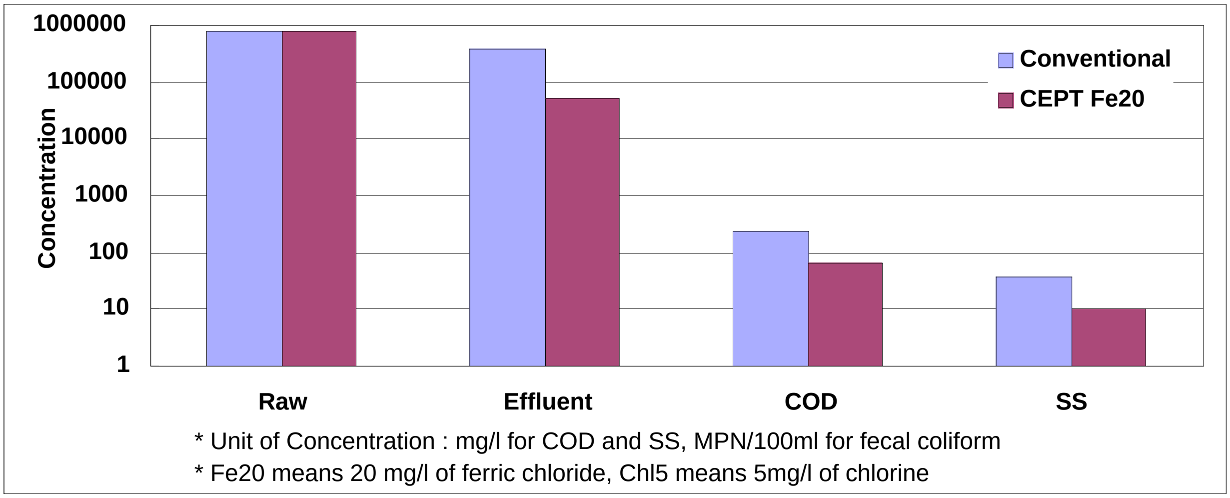


Figure I.4. COD, SS, and fecal coliform in the effluent treated with 20mg/l of FeCl3 (1)

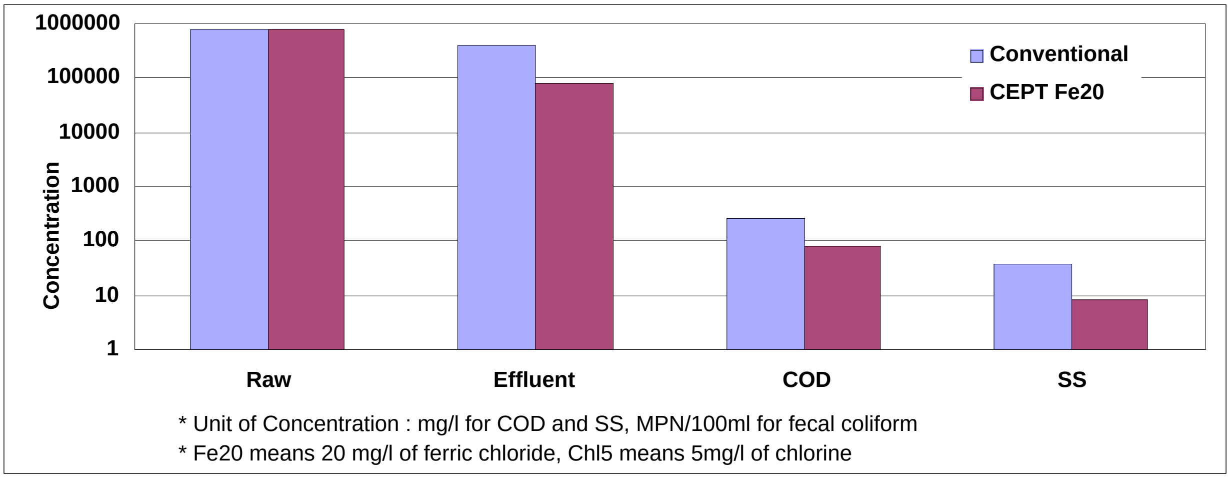


Figure I.5. COD, SS, and fecal coliform in the effluent treated with 20mg/l of FeCl3 (2)

As the figures above show, the quality of CEPT effluent treated by 20 mg/l of ferric chloride is evidently better than that of the conventional effluent.

In contrast, the chemical treatment with 10 mg/l of ferric chloride does not make a significant difference in the removal of SS and fecal coliform (See Figures I.6 and I.7 below). According to Figures I.4 to I.7, it is obvious that 20 mg/l of ferric chloride is more effective than 10 mg/l of ferric chloride for wastewater treatment in Boston.

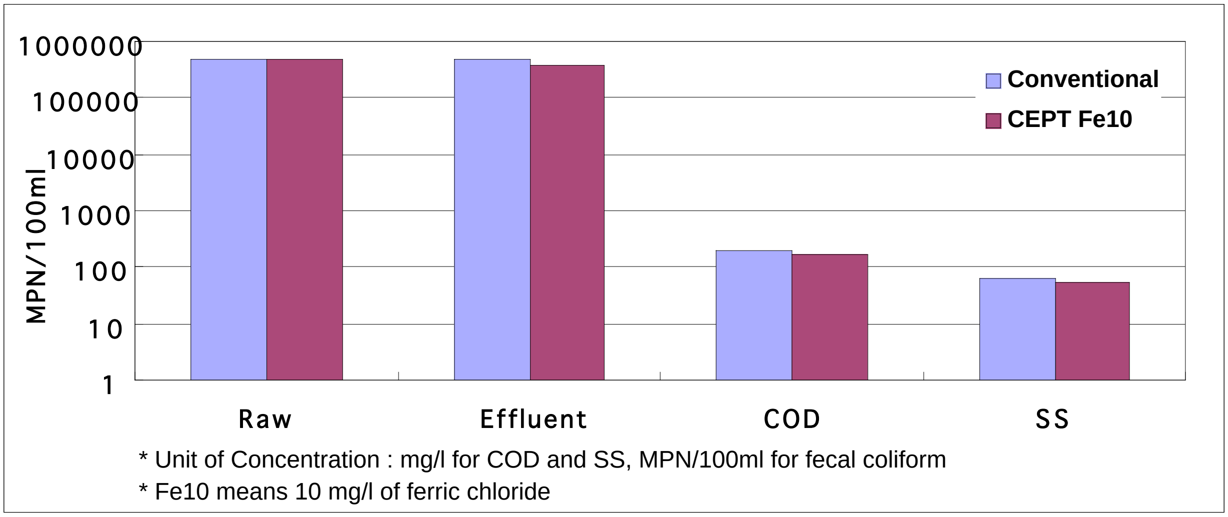


Figure I.6. COD, SS, and fecal coliform in the effluent treated with 10mg/l of FeCl3 (1)

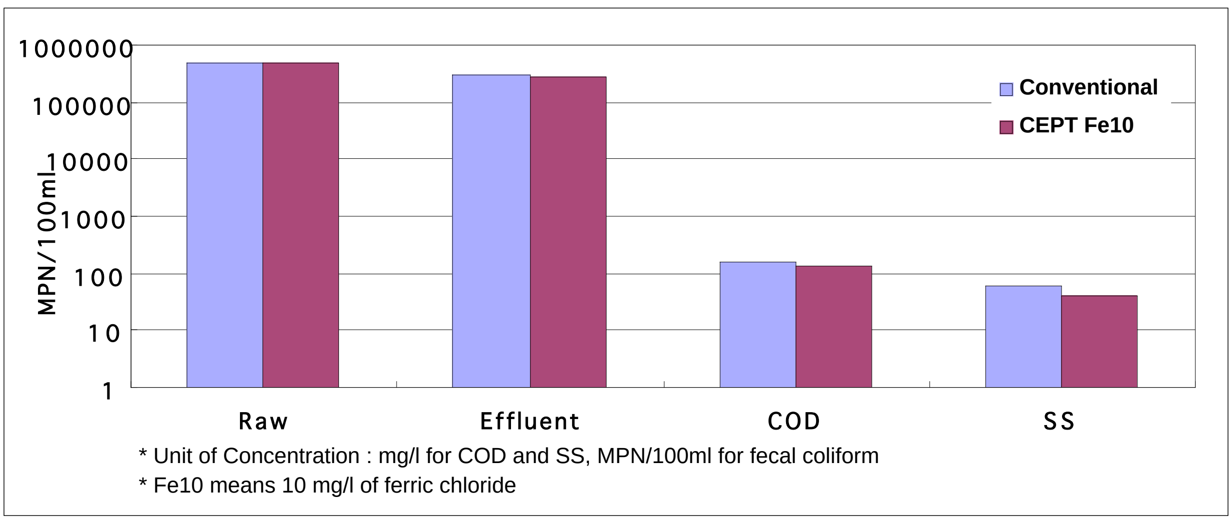


Figure I.7. COD, SS, and fecal coliform in the effluent treated with 10mg/l ofFeCl3 (2)

It is noticed that the trends of SS and COD reduction are similar to the trend of fecal coliform reduction. As discussed above in Section I.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 coliform in the CEPT effluent is much lower than that in the conventional effluent. Similarly, the concentration of coliform in the effluent is lower when seawater is added during CEPT, since seawater enhances the efficiency of SS removal. Figures I.8 and I.9 below show the relationship between concentrations of SS and fecal coliform:

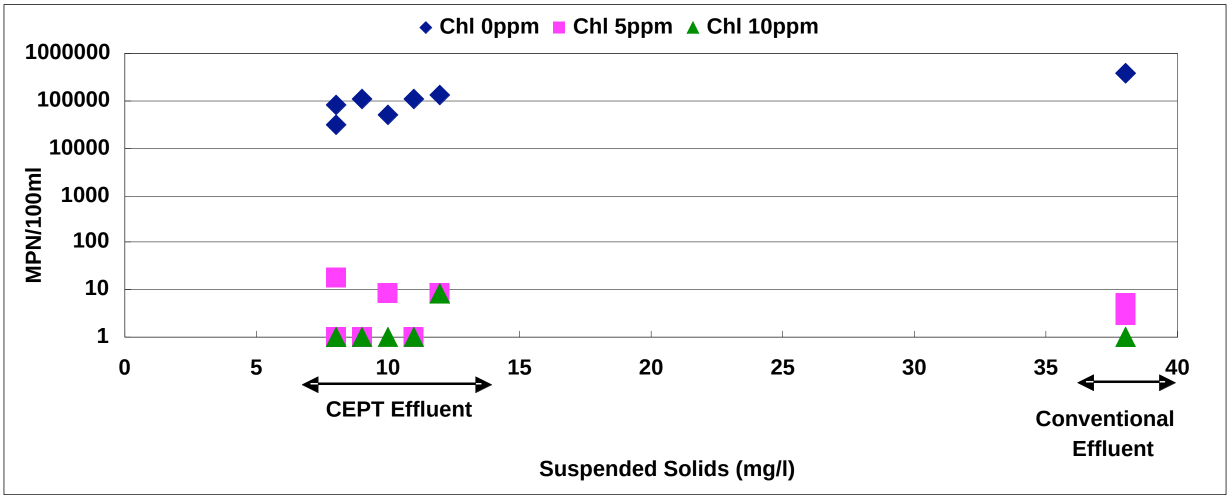


Figure I.8. SS concentration versus fecal coliform concentration in the effluent treated with 20mg/l of FeCl3

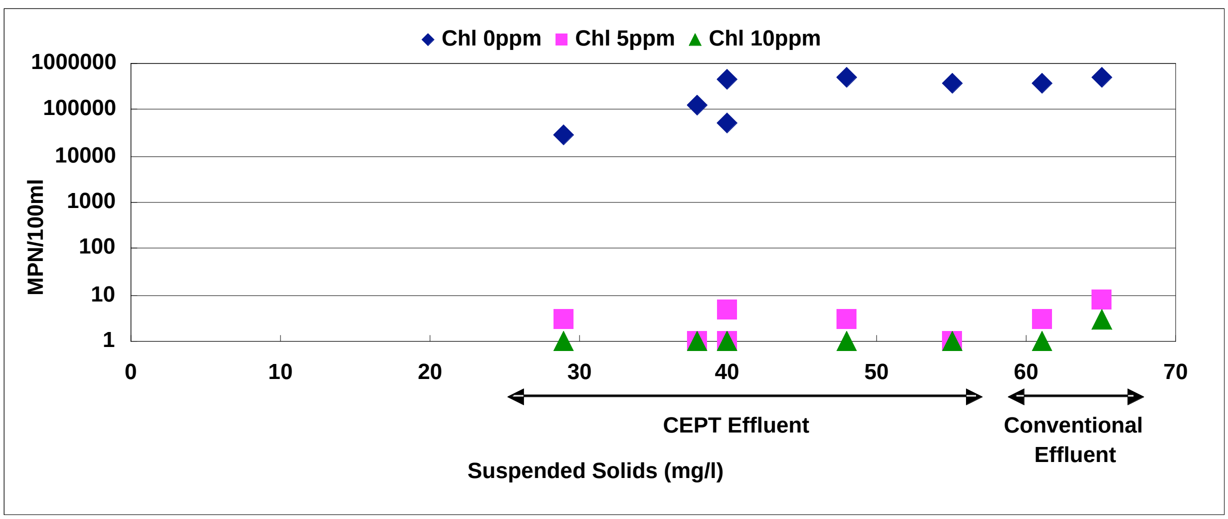
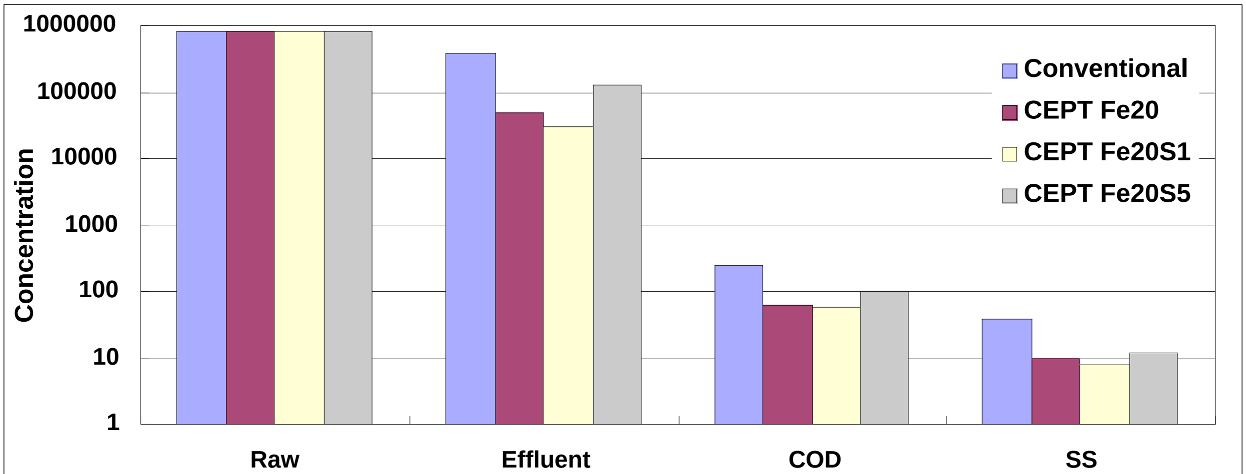


Figure I.9. SS concentration versus fecal coliform concentration in the effluent treated with 10mg/l of FeCl3

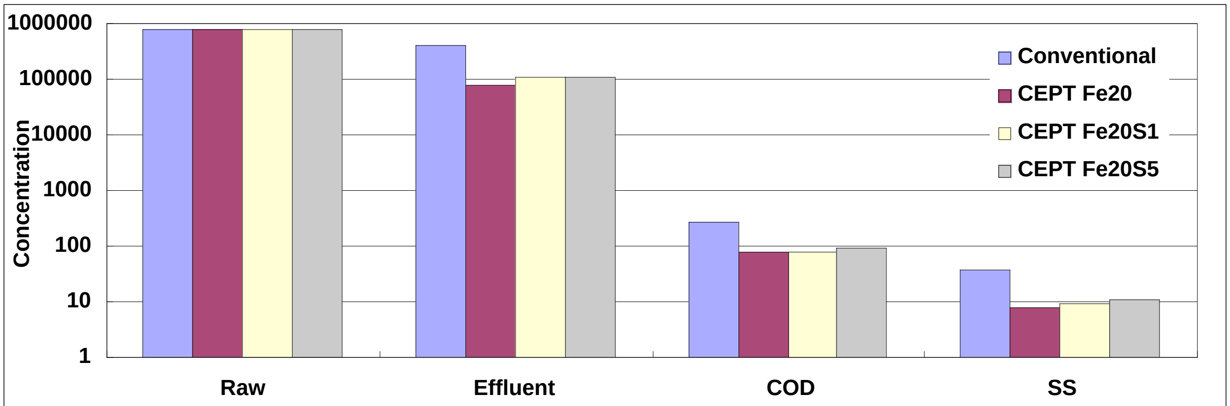
<u>Effect of Additional Seawater on the Fecal Coliform Reduction</u>

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 affects the SS removal efficiency, and 1% of additional seawater makes a little difference from no additional seawater. The effluent qualities of conventional treatment and CEPT are shown in Figures I.10 and I.11 below:



* Unit of Concentration : mg/l for COD and SS, MPN/100ml for fecal coliform
* Fe20S1 means 20 mg/l of ferric chloride and 1% of additional seawater

Figure I.10. Concentration of COD, SS, and fecal coliform (1)



* Unit of Concentration : mg/l for COD and SS, MPN/100ml for fecal coliform
* Fe20S1 means 20 mg/l of ferric chloride and 1% of additional seawater

Figure I.11. Concentration of COD, SS, and fecal coliform (2)

Since raw sewage in Boston already contains approximately 2% of seawater (See Section H.3), its conventional treatment has higher SS removal efficiency than the typical SS removal efficiency (i.e. SS removal efficiency of conventional treatment without added seawater). Therefore, the addition of more seawater produces no significant difference in fecal coliform reduction. However, it is observed that 1% of additional seawater is more effective in reducing the coliform levels than 5% of additional seawater for Boston's wastewater.

For the CEPT effluent treated with 10 mg/l of ferric chloride, neither 1% nor 5% of additional seawater makes significant difference for the fecal coliform reduction, due to the effect of seawater already present in the wastewater. Figure I.12 below shows the effects of additional seawater on pollutant reduction in the wastewater effluents treated with10mg/l of ferric chloride. As can be seen, the SS removal efficiency does not increase with additional seawater.

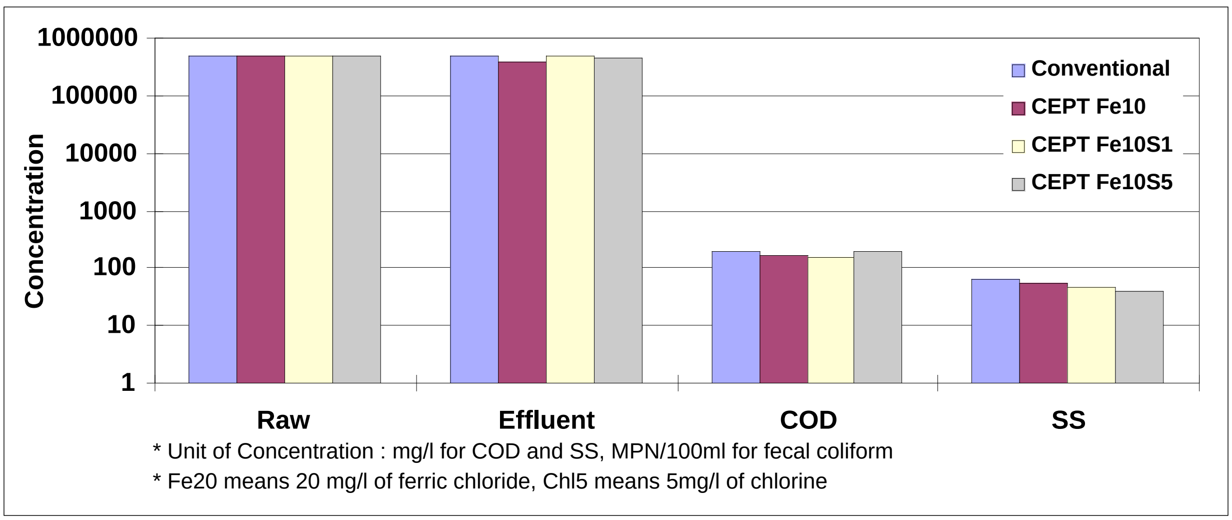


Figure I.12. Concentration of COD, SS, and fecal coliform in the effluent according to the additional seawater

Disinfectability of the Effluent

Typically, the concentration of fecal coliform in the effluent of conventional treatment is 10^7~10^9 MPN/100ml, based on the concentration of 10^7~10^9 MPN/100ml in raw sewage. It means that conventional treatment barely removes fecal coliform, and it is impossible to reduce its coliform level to below 200MPN/100ml with high dosage of chlorine (See Section I.3 and Table I.4). In contrast, CEPT reduces the concentration of coliform in its effluent to approximately 10% of that of raw sewage, similar to the secondary effluent (WPCF, 1986). This suggests that the disinfectability of CEPT effluent is higher than that of conventional effluent.

In the experiments with Boston's wastewater, the effluents of CEPT effluent as well as those of conventional treatment are disinfected with 5 mg/l of chlorine. The reason even the conventional effluent is disinfected with low concentration of chlorine is that the level of fecal coliform in its effluent is approximately 390,000 MPN/100ml, much lower than the typical value. The level of fecal coliform in the conventional effluent of Boston's wastewater is similar to the typical values of the activated sludge effluent. This low level of fecal coliform in Boston's conventional effluent is derived from the low level of fecal coliform, approximately 10^6 MPN/100ml, in its raw sewage, and the relatively high SS removal efficiency of its conventional treatment, previously discussed.

Chlorine Demand for CEPT Effluent of Boston's Wastewater

The coliform reduction in the CEPT effluent with 5 mg/l and 10 mg/l of chlorine is shown in Figures I.13 and I.14 below:

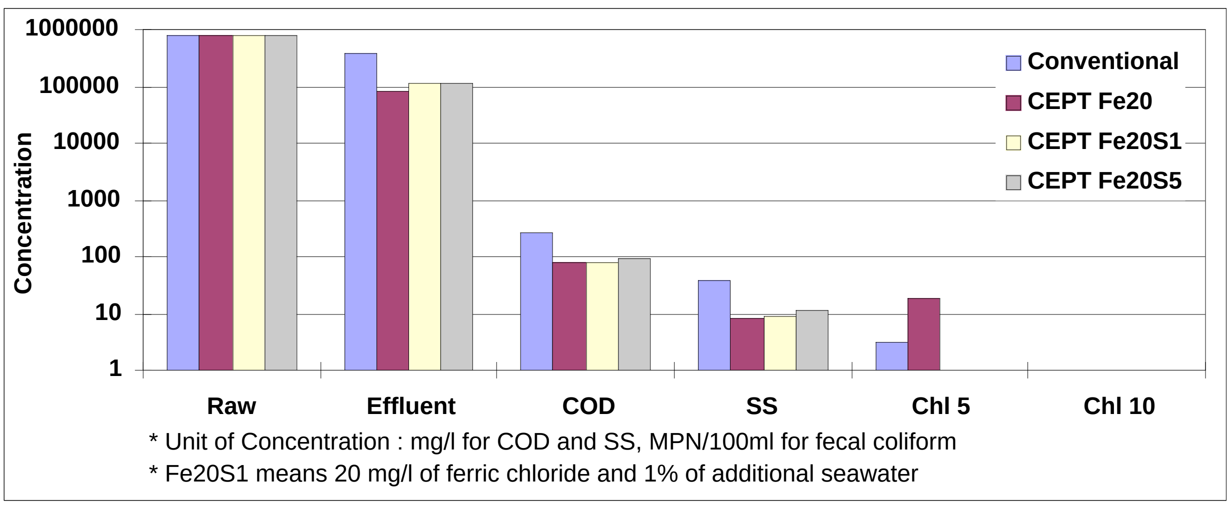


Figure I.13. Fecal coliform reduction in the effluent with 5 mg/l and 10 mg/l of chlorine (1)

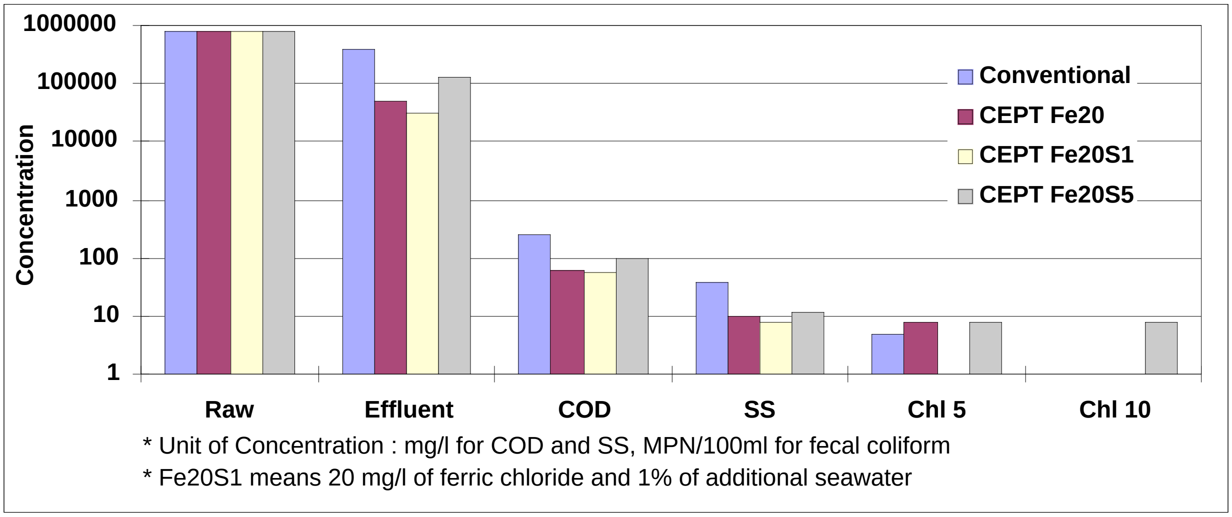


Figure I.14. Fecal coliform reduction in the effluent with 5 mg/l and 10 mg/l of chlorine (2)

As shown in the Figure I.13 and I.14, 5 mg/l of chlorine achieves the same coliform removal efficiency as 10 mg/l of chlorine, which suggests 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 an average contact time of 45 minutes. The average fecal coliform in the effluent of Deer Island wastewater treatment plant is 10 MPN/100ml. Since similar fecal coliform reduction efficiency is achieved in the experiments with 5 mg/l of chlorine with shorter contact time of 30 minutes, and since the disinfection efficiency depends on the chlorine dose and the contact time, it can be concluded that 5 mg/l chlorine is higher than the required chlorine demand.

I.6. Conclusion and Recommendation

In the tests performed for wastewater samples in Paraty, 15 mg/l of peracetic acid (PAA) reduces the concentration of fecal coliform in the treated wastewater effluent to approximately 200 MPN/100ml, which satisfies the Brazilian regulations. However, since the cost of PAA is 10 times higher than that of chlorine, and its disinfection efficiency is lower, chlorine is a more appropriate disinfectant to be used in Paraty.

The disinfection efficiency of a wastewater treatment depends on its suspended solids (SS) removal efficiency, because significant amounts of coliform bacteria is removed with SS during the settling process, and the remaining SS interferes with disinfection by protecting the coliform bacteria from the disinfection agents. Since CEPT has higher SS removal efficiency than conventional primary treatment, it produces an effluent that is easier to disinfect that the effluent of conventional treatment. However, in the experiments performed with wastewater samples from Boston, it is found that the effluents of both CEPT and conventional treatment are disinfectable for the following reasons:

(i) Raw wastewater in Boston is relatively dilute and has lower than typical values of SS; and
(ii) 2% of seawater, already included in the Boston wastewater, enhances the SS removal efficiency and hence the coliform reduction efficiency.

On the other hand, it is found that additional seawater does not enhance the coliform reduction in the CEPT effluent in Boston for the following reasons:

(i) Additional seawater does not make significant differences of SS removal; and
(ii) Initial SS and coliform removal efficiencies by CEPT without additional seawater are already high.

Since raw sewage in Paraty does not contain seawater, however, the level of fecal coliform in the effluent would be significantly reduced with addition of small amounts of seawater, since COD and SS removal efficiency increase with additional seawater.

According to the data from Deer Island wastewater treatment plant, which uses secondary treatment, the average dose of chlorine is 2.2 mg/l with an average contact time of 45 minutes. Since the SS removal efficiency of CEPT is similar to that of secondary treatment, it is expected that the amount of chlorine demand in Paraty would be similar to that of the Deer Island wastewater treatment plant, also. Considering the cases in the summer season, where higher amount of chlorine demand would be required with relatively short contact time, the chlorine dosage of 3 mg/l is recommended (See Table I.3). If chlorine dosage is well-controlled, the residual chlorine will not vary by much, and therefore the amount of dechlorination agent will be constant. Based on the data from the Deer Island wastewater treatment plant, 0.5 mg/l of dechlorination agent is recommended, when sulfur bisulfate is used as a dechlorination agent.

The average dosage of chlorine and sulfur bisulfate in the Deer Island wastewater treatment (Secondary Treatment) and the recommended dosage of respective chemicals for Paraty's future wastewater treatment (CEPT) are summarized in Table I.8 below:

	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 I.11. Recommended dosage of chlorine and sulfur bisulfate (SBS)[1]

[1] Recommended dosage of chlorine and sulfur bisulfate based on SS removal efficiency of CEPT and secondary treatment (Source: Chemical Dosage in Secondary Treatment: Deer Island WWTP)

APPENDIX J – INTRODUCTION TO SLUDGE TREATMENT AND DISPOSAL

This chapter describes a number of treatment technologies and disposal options for sludge that is produced from the wastewater treatment.

J.1. Introduction

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 that contributes chemical precipitates and microorganisms in the sludge.

J.2. Sludge Treatment Technologies

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 J.1:

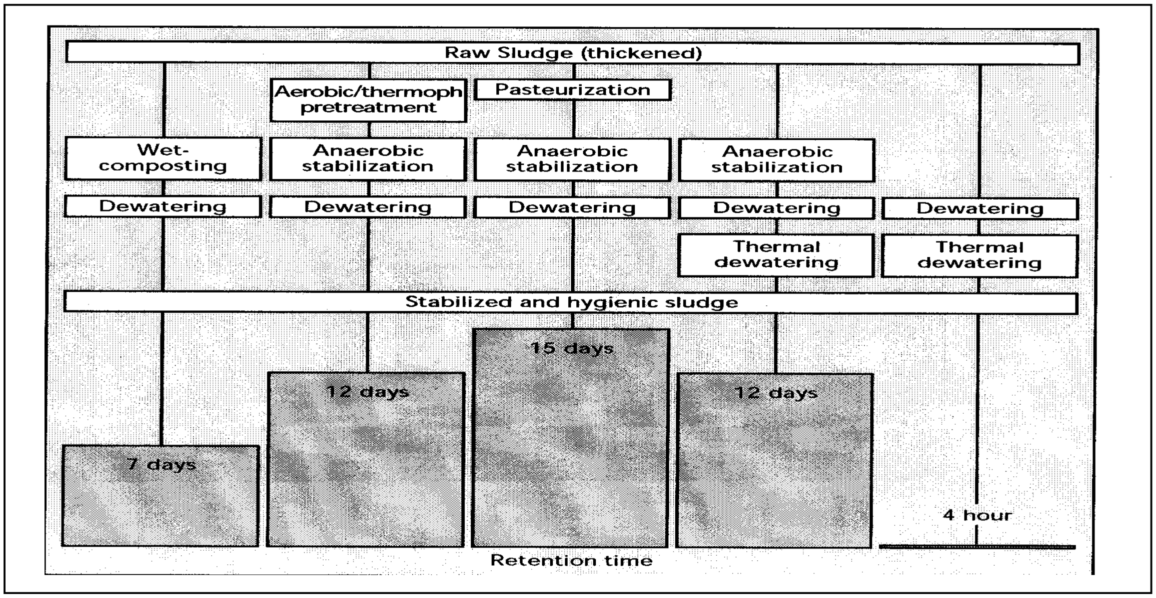


Figure J.1. Sludge pre-treatment options

Figure J.1 above is a summary of the several options for sludge treatment and disposal routes.

The following sections will highlight the different fates of sludge disposal (as depicted in Figure J.2), and expand on the conditions, advantages and disadvantages of each process.

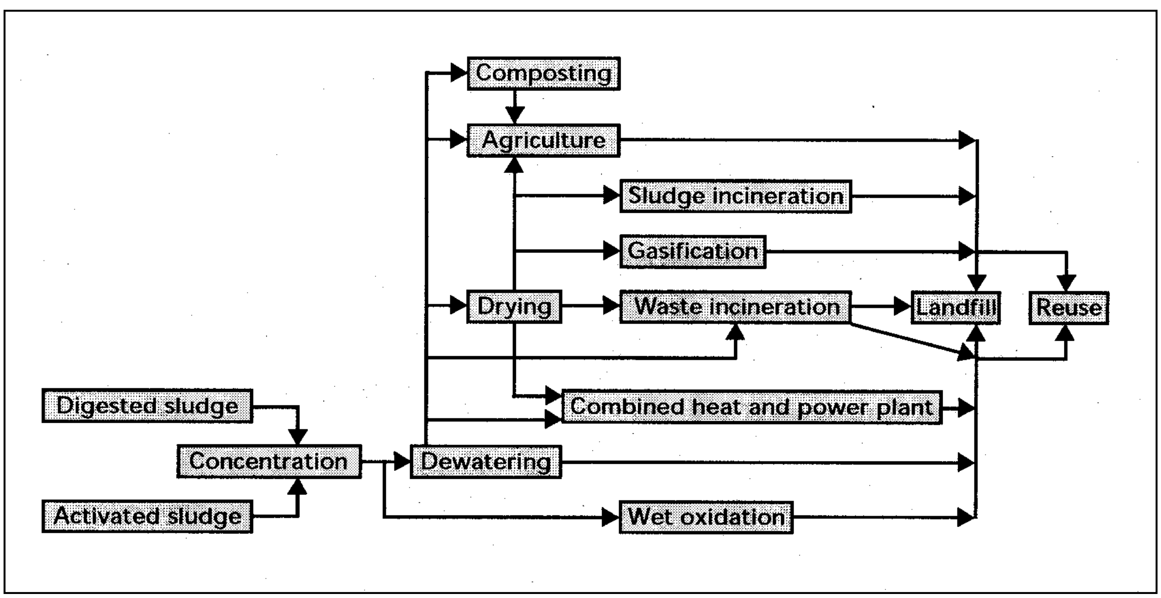


Figure J.2. Sludge treatment options

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 sludge 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:

i) Utilization of nutrients contained in the sludge (mainly phosphorous and nitrogen);

ii) Utilization of organic substances contained in the sludge for the improvement of the humus layer of the soil; and that it is

iii) Often the cheapest route of disposal.

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

i) Major investments in storage facilities since sludge can only be spread a few times a year; and

ii) 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 J.1 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	pH	pH	PH	
	5.0 <5.5	5.5<6.0	6.0-7.0	> 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 (provisional)				15 (provisional)
Molybdenum	4				0.2
Selenium	3				0.15
Arsenic	50				0.7
Fluoride	500				20

Table J.1. Maximum permissible concentration of potentially toxic elements (PTE) in soil after application of sewage sludge, and maximum annual rate of addition (www.fao.org)

Composting

Sludge composting aims at biologically stabilizing sludge in order to develop agricultural outlets that exploit the nutrient or organic value of sludge. 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 composing efficiency (European Environment Agency, 1997).

Sludge can be composted if it has 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 J.3 below depicts a typical in-vessel sludge composter:

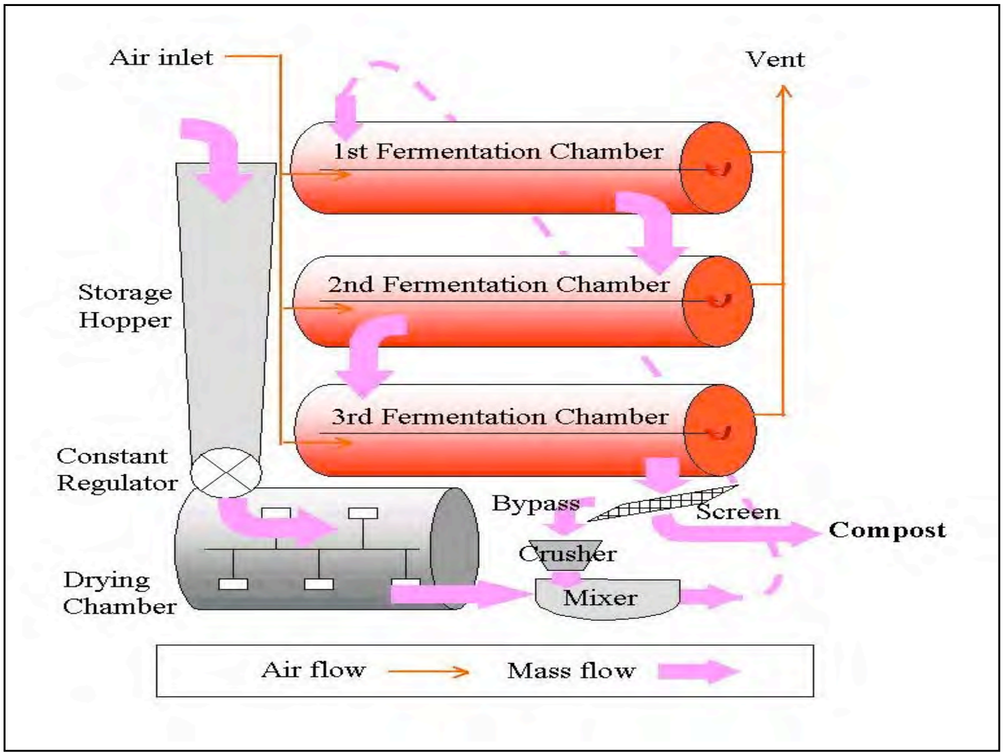


Figure J.3. In-vessel sludge composter process (http://www.emc.or.kr/english/koetv)

The advantages to composting sludge are:

i) Reductions in the volume of sludge to be transported to agricultural fields for example;

ii) Easier storage and spreading capabilities; and

iii) Control of compost materials which leads to a more stable end-product.

The disadvantages however are:

i) Higher treatment costs compared to direct sludge application to agricultural fields;

ii) High energy costs of aeration; and

iii) Need for an outlet market for the compost products.

Drying

The general flow sheet for a typical sludge drying process is shown in Figure J.4 below:

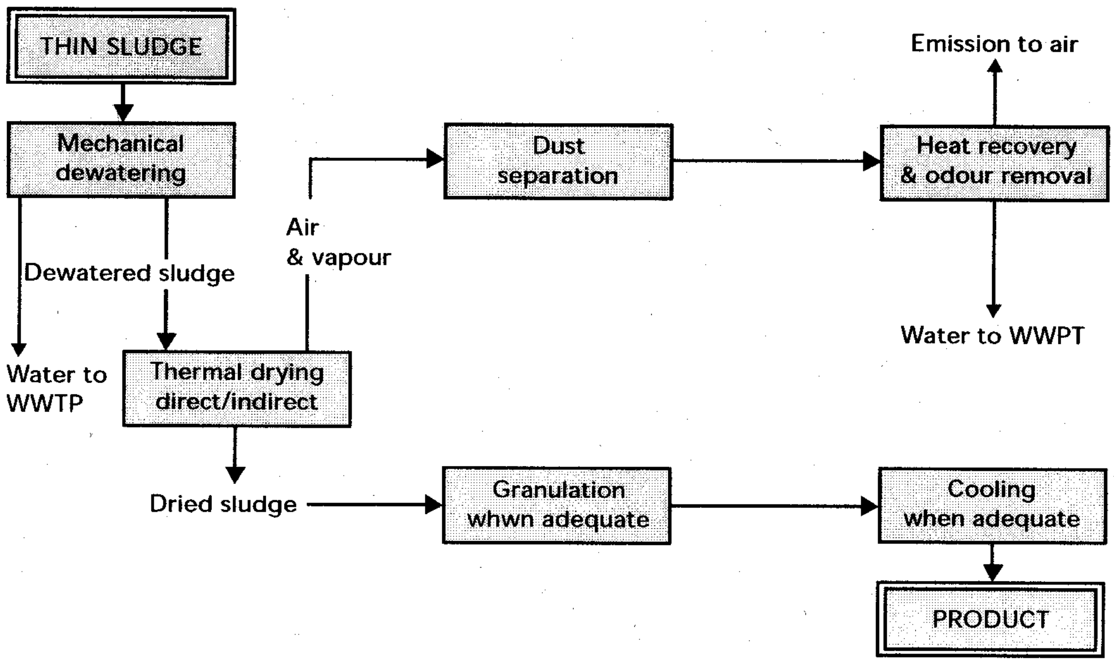


Figure J.4. Typical drying process diagram (European Environment Agency, 1997)

The two distinctly different drying methods are direct drying, and indirect 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 includes granulation, is generally 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).

Incineration

15% of Europe's sludge is currently incinerated (European Environment Agency, 1997). Since the agricultural uses of sludge, by direct application, as well as sludge landfilling are subject to increasingly stringent regulatory control, the incineration of sludge 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 sludge 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, sludge 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:

i) A significant reduction in sludge volume, after incineration;
ii) Energetic valorization of sludge;
iii) 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;
iv) Low sensitivity to sludge composition;
v) Reliable systems; and
vi) Odor minimization due to closed systems and high temperatures (European Environment Agency, 1997).

The disadvantages however are:

i) Incinerators are capital intensive and usually justified only in larger volume situations; and

ii) 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 (European Environment Agency, 1997).

Landfilling

Since sludge 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 sludge be stabilized.

Sludge 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 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 described 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).

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).

J.3. Sludge Decision Making Tree

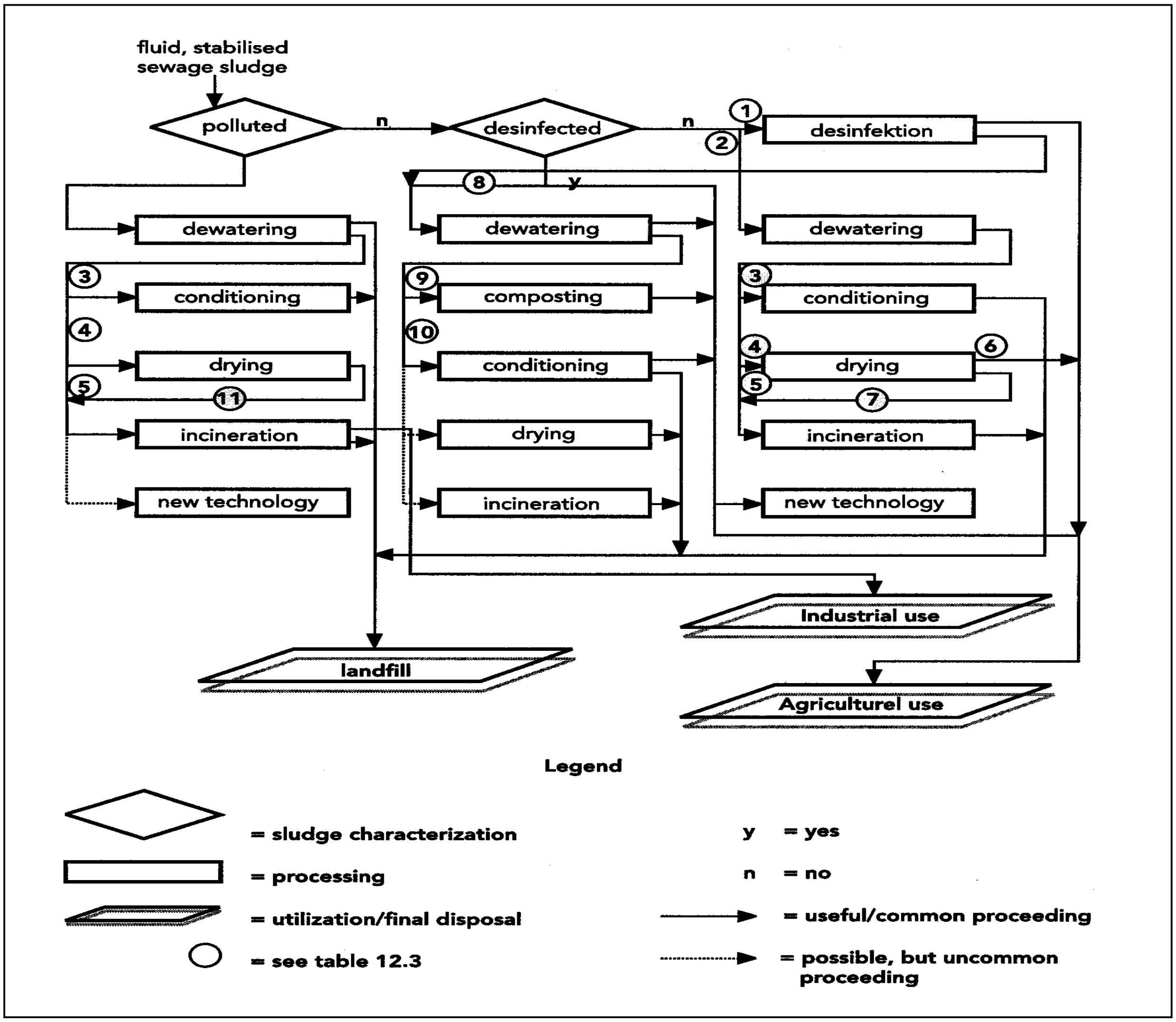


Figure J.5. Sludge decision-making tree (European Environment Agency, 1997)

Figure J.5 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 J.2 below accompanies Figure J.5 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 sludge 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 J.2. Conditions influencing sludge decisions

REFERENCES

Anonymous. “Interim national river water quality standards for Malaysia.” n.p.: n.d <http://agrolink.moa.my/did/river/sgklang/sgklang_wqi.htm>.

Arvada, City of. Minimum Design Standards Section 110 Sanitary Sewers. Arvada, Colorado: 2001, <http://www.ci.arvada.co.us/HTML/Spec_Book/New-S-SEC110-2001.pdf>.

Baldry. M. G. C., et al (1995). “Effluent Disinfection in Warm Climates with Peracetic Acid.” Water Science and Technology. Vol 31(5-6). pp. 161-164.

Barreto, Wilsa Mary S. Epidemiological Study Data of Diarrhea in Municipality of Paraty. 2002.
BIT (Brazillian Incentrive & Tourism). “Business Essentials – Economy: Historical perspective.” n.d. <http://www.bitourism.com/countryinfo/business/business_economy.asp>.

Blumenthal, Ursula J., Jay M. Fleisher, Steve A. Esrey, and Anne Peasey. “Epidemiology: a tool for the assessment of risk.” Water Quality: Guidelines, Standards, and Health: Assessment of Risk and Risk Management for Water-Related Infectious Disease. London, UK: IWA Publishing, 2001. pp. 135-160. World Health Organization.

Blumenthal. U. J., D. Mara, A. Peasey, G. Ruiz-Palacios, and R. Scott (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.

Boland, John J. “Pricing Urban Water: Principles and Compromises.” Water Resources Update. Summer 1992: 7-10.

Brazilian Tourist Office. “International Arrivals and Departures by City.” 2002 <http://www.brazil.org.uk/page.php?cid=1193>.

Cabral, Christian, Frederic Chagnon, and Domagoj Gotovac. Alternative Designs for the Upgrade of the Tatui-CEAGESP Wastewater Treatment Facility using Chemically Enhanced Primary treatment. Master of Engineering Project Report, Massachusetts Institute of Technology: June 1999.

Canaldotempo.com (The Weather Channel). “Tempo Local – Paraty, Brazil.” 2003 <http://br.weather.com/weather/local/BRXX3898>.

CEPIS (Pan American Center for Sanitary Engineering and Environmental Sciences). “Assessment of Drinking Water and Sanitation 2000 in the Americas.” 2000 <http://www.cepis.ops-oms.org/enwww/eva2000/brasil/informe.html>.

Chagnon, Frederic. “An Introduction to Chemically Enhanced Primary Treatment” Cambridge, MA: MIT, 2002 <http://www.efm.leeds.ac.uk/CIVE/Sewerage/articles/Introduction_to_CEPT.pdf>.

Choi, Nancy. Design of a Wastewater Collection System for the Historical Center of Paraty, Brazil. Thesis (M. Eng.) – Massachusetts Institute of Technology, Dept. of Civil and Environmental, 2003.

Colgan. S. and R. Gelhr (November, 2001). “Disinfection.” Water Environment & Technology, vol. 5, pp. 29-33.

Commonwealth of Pennsylvania, Department of Environmental Protection. “Drinking water treatment technologies for groundwater systems under the direct influence of surface water.” Harrisburg, PA: October 2001 <http://www.dep.state.pa.us/dep/deputate/watermgt/wsm/WSM_DWM/Technol/Trt_GUDI.pdf>.

Craveiro, Fernando. Personal communication with Regional Water Business Group Manager CH2M HILL of Brazil. May 2003.

Csillag, Claudio and Ricardo Zorzetto. “Turning Point: Environmental Health in Brazil.” Environmental Health Perspectives. Volume 108. Number 11. November 2000 <http://ehpnet1.niehs.nih.gov/docs/2000/108-11/focus.html>.

De Sigueira Baffo, Sergio Mauricio. Personal Interview. January 2003.

Deer Island Wastewater Treatment Plant (March, 2003). “Disinfection/Dechlorination Performance Overview Report.”

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

Eisenberg, Joseph N. S., Jamie Bartram, and Paul R. Hunter. “A public health perspective for establishing water-related guidelines and standards.” Water Quality: Guidelines, Standards, and Health: Assessment of Risk and Risk Management for Water-Related Infectious Disease. London, UK: IWA Publishing, 2001. 229-256. World Health Organization.

EPA (February 2002), Onsite Wastewater Treatment Systems Manual. EPA. pp. TFS17-TFS23
EPA (September, 1999), Combined Sewer Overflow Technology Fact Sheet; Alternative Disinfection Methods. EPA. 832-F-99-033.

EPA (September, 1999), Wastewater Technology Fact Sheet Chlorination Disinfection. EPA. 832-F-99-062.

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.

Fateen, Shaheerah A., Natalia Olive, and Jennifer K. Stout. Chemically Enhanced Primary Treatment in Alfenas, Brazil: CEPT plant design, sludge management strategy, and reservoir impact analysis. Master of Engineering Project Report, Massachusetts Institute of Technology: June 2002.

Flygt Corporation. Personal communication with Engineer. May 2003.

Furtado, Georgina, Office of Mayor. Personal Interview. January 2003.

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

Haestad Mathods. Computer Applications in Hydraulic Engineering – Connecting Theory to Practice. 5th ed. Waterbury: Haestad Press, 2002.

Harrington, Brian. Personal communication with Engineer at Metcalf and Eddy. March 2003.

Hassett, Alan F. Vacuum Sewerage – Ready for the 21st Century. Proceedings from "Sewers of the Future" Conference, September 10-13, 1995, Houston Texas. Published in Virginia: Water Environment Federation, 1995.

Houghton Chemical Corporation. Water Front: Industrial Water Treatment Bulletin.

Hutton, Guy. "Cosiderations in evaluating the cost-effectiveness of environmental health interventions." Geneva: WHO, 2000 <http://www.who.int/environmental_information/Disburden/WSH00-10/WSH00-10TOC.htm>.

Hutton, Guy. "Economic evaluation and priority setting in water and sanitation interventions." Water Quality: Guidelines, Standards, and Health: Assessment of Risk and Risk Management for Water-Related Infectious Disease. London, UK: IWA Publishing, 2001. 333-359. World Health Organization.

James P. Heaney, Robert Pitt, Richard Field. Innovative Urban Wet-Weather Flow Management Systems. Cincinnati: U.S. Environmental Protection Agency, 1999.

Jiménez. B., A. Chávez, C. Maya, and L. Jardines (2001) "Removal of Microorganisms in Different Stages of Wastewater Treatment for Mexico City." Water Science and Technology. vol. 43(10). pp. 155-162

KDHE (Kansas Department of Health and Environment). "Kansas Water-Water Quality Standards." Kansas: 2001 <http://ks.water.usgs.gov/Kansas/rtqw/parameter.html#TSS>.

Kfouri, Claire and Hyo Jin Kweon. Seawater as a Substitute Coagulant and Effects on Disinfection in Chemically Enhanced Primary Treatment: A Case Study of Paraty, Brazil and Deer Island, Boston. Thesis (M. Eng.) – Massachusetts Institute of Technology, Dept. of Civil and Environmental, 2003.

Klink, Tymur. Personal communication with architect and land owner in Paraty. January 2003.
Lee, Sylvia. Personal communication with Engineer at Montgomery Watson Harza. January – May 2003.

Lemos Padua, Edneia Maria, Director of Public Works (1999-2000). Personal Interview. January 2003.

Liberti. L. and M. Notarnicola (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.

Metcalf and Eddy. Wastewater Engineering: Collection and Pumping of Wastewaster. New York: McGraw-Hill, 1981.

Narkis. N., R. Armon, R. Offer, F. Orshansky, and E. Friedland (1995). Effect of Suspended Solids on Wastewater Disinfection Efficiency by Chlorine Dioxide. Water Resource. vol. 29(1). Pp. 227-236.

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

Payment, Pierre, and Paul R. Hunter. "Endemic and epidemic infectious intestinal disease and its relationship to drinking water." Water Quality: Guidelines, Standards, and Health: Assessment of Risk and Risk Management for Water-Related Infectious Disease. London, UK: IWA Publishing, 2001. 61-88. World Health Organization.

Pleasanton Sewer Feasibility Board. "Bear Lake – Pleasanton Wastewater Feasibility Study." Bear Lake, MI: 2001 <http://www.bearlakemichigan.org/sewer/feasibility1.html>.

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

Prefeitura Municipal de Paraty (Municipal City Hall of Paraty). "Laudo de Avaliacao Tecnica e Economico – Financeiro." Paraty, Brazil: January 2002.

Prefeitura Municipal de Paraty (Municipal City Hall of Paraty). "Paraty: A patrimony of life." Paraty, Brazil: 2003.

Prefeitura Municipal de Paraty (Municipal City Hall of Paraty). "Programa Morar Melhor – Acao Saneamento Basico Plano de Trabalho" Paraty, Brazil: December 25, 2001.

Prefeitura Municipal de Paraty (Municipal City Hall of Paraty). “Vigilancia da Qualidade da Agua de Consumo Humano – VQACH.” Paraty, Brazil: 2001.

Prefeitura Municipal de Paraty (Municipal City Hall of Paraty). Divisao de Vigilancia Sanitaria. Paraty: GPO, 2002.

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

Reis, Renato. Financial Secretary for the Municipality of Paraty. Personal Interview. January 2003.

Rocha, Wilson. Personal interview with past water and sanitation engineer for Paraty. January 2003.

Rosman, Paulo Cesar Colonna. Personal communication with Professor at the Universidade Federal do Rio de Janeiro, 14 April 2003.

San Martin, Orlando. “Water Resources in Latin America and the Caribbean: Issues and Options.” February 2002 <http://www.iadb.org/sds/doc/ENV-OSanMartin-WaterResourcesinLaC.pdf>.

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

Sarasota County Environmental Services. “Evaluation of Onsite Wastewater Treatment and Wastewater Collection System Alternatives.” Phillippi Creek Septic System Replacement Program: September 2000 Executive Summary TM No. 7 (2000).

Secretaria Municipal de Saude (Municipal Secretary of Health). “SIAB – Sistema de Informacao de Atencao Basica: Consolidado das familias cadastradas do ano de 2003 do modelo general.” Paraty, Brazil: January 2003.

Shamsi, U.M. GIS Tools For Water,Wastewater and Stormwater Systems. United States: ASCE Press, 2002.

Solvay Interox (2002). Material Safety Data Sheet: Proxitane® EQ Liquid Sanitizer. pp. 1-10.

State of Connecticut, Department of Environmental Protection. Connecticut Water Quality Standards. Connecticut: 2002 <http://dep.state.ct.us/wtr/wq/wqs.pdf>.

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

Tsukamoto, Ricardo. Personal communication with Engineer and concerned citizen of Paraty. January – May 2003.

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.

U.S. Department of Commerce. "Brazil – Water and Wastewater Concession Opps. – Market Assessment." 13 January 1999. <http://strategis.ic.gc.ca/SSGF/dd73927f.html>.

U.S. Department of Interior, Bureau of Reclamation. "Total Coliform and E-Coli: Factsheet." September 21, 2001 <http://www.usbr.gov/water/media/pdfs/TC&Ecoli.pdf>.

U.S. EPA (Environmental Protection Agency). "Appendix B – Selected Costs of Alternative Disinfection Systems" EPA Guidance Manual: Alternative Disinfectants and Oxidants. EPA, April 1999 <http://www.epa.gov/safewater/mdbp/pdf/alter/app_b.pdf>.

U.S. EPA (Environmental Protection Agency). Collection Systems Technology Fact Sheet: Sewers, Lift Station. Washington D.C.: U.S. Government Printing Office, September 2000.
U.S. EPA (Environmental Protection Agency). Manual: Alternative Wastewater Collection Systems for Small Communities. Washington D.C.: U.S. Government Printing Office, October 1991.

U.S. EPA (Environmental Protection Agency). "Small System Compliance Technology List for the Surface Water Treatment Rule." EPA, August 1997 <http://www.epa.gov/safewater/regs/swtrsms.pdf>.

UNESCO. "Criteria for the inclusion of properties in the World Heritage List." 1997 <http://whc.unesco.org/criteria.htm >.

UNICEF, WHO, and WSSCC. "Global Water Supply and Sanitation Assessment 2000 Report." 2000 <http://www.who.int/water_sanitation_health/Globassessment/GlobalTOC.htm>.

WHO (World Health Organization). "Children are among the most vulnerable to environmental threats." n.d. <http://www.who.int/heca/advocacy/publications/HECIbr2.pdf>.

WHO (World Health Organization). "Cholera and epidemic-prone diarrhoeal diseases." 2003 <http://www.who.int/csr/disease/cholera/en/>.

WHO (World Health Organization). "Health criteria and other supporting information." Guidelines for Drinking-Water Quality. 2nd ed. Vol. 2. 1996. 940-949. and Addendum to Vol. 2 1998. 281-283. <http://www.who.int/water_sanitation_health/GDWQ/Summary_tables/Sumtab.htm>.

World Bank, The. Environmental Priorities for Development and Sanitation and Clean Water. n.p.: 1992.

X-rates.com. “Exchange Rates.” 2003 <http://www.x-rates.com/>.

You, Eun Chu. Policy for Development of Water and Sanitation Infrastructure in Paraty, Brazil. Thesis (M. Eng.) – Massachusetts Institute of Technology, Dept. of Civil and Environmental Engineering, 2003.