Chapter 2 – Treatment Alternatives

2.1 Effluent Regulations

In September 2000, Salud (Ministerio de Salud) issued the publication entitled *"Normas para Aguas Residual"* which outlined the regulations on municipal and industrial wastewater discharges and established biosolids quality criteria and final disposal methods. These national standards and regulations were prepared under the cooperative guidance of the Ministerio De Commercio E Industries, Salud and ANAM. The Reglamento Técnico DGNTI – COPANIT 35-2000 (Agua Descarga De Effluentes Líquidos Directamente A Cuerpos Y Masas De Agua Superficiales Y Subterraneas) define by Panamanian law, the maximum permitted effluent concentrations of biological, chemical and physical constituents allowed to be discharged into receiving waters. These pollutant limits were developed by a technical committee consisting of thirty qualified members representing a cross section of Panamanians from the public and private sectors (ACP, ANAM, ARI, Colegio De Biólogos, DGNTI/MICI, ETESA, IDANN, MIDA/IDIAP, IEA, Laboratorio Central, LACAYA/UP, MINSA, PANAIDIS, USMA, Universidad De Panamá, Universidad Tecnológica, UTP Y Ente Regulador). The maximum permissible effluent discharge values for specific constituents are presented in Table 2.1-1.

Table 2.1-1
Maximum Effluent Discharge Limits to Receiving Water Bodies
Reglamento Técnico DGNTI-COPANIT 35-2000

Parameter	Unit	Symbol	Limit
Grease and Oils	mg/l	A y G	20
Aluminum	mg/l	Al	5
Arsenic	mg/l	As	0,50
Boron	mg/l	B	0,75
Cadmium	mg/l	Cd	0,01
Calcium	mg/l	Ca	1.000
Cyanide	mg/l	CN	0,2
Residual Chlorine	mg/l	Cl	1,5
Chlorides	mg/l	Cl_2	400
Copper	mg/l	Cu	1
Total Coliforms	NMP/100 ml	Coli/100ml	1.000
Phenolic Compounds	mg/l	Fenoles	0,5
Hexavalent Chromium	mg/l	Cr^{6+}	0,05
Total Chromium	mg/l	Cr_t	5
Biochemical Oxygen Demand (BOD)	mg O_2/l	BOD_5	35
Chemical Oxygen Demand	mg/l	COD	100
Detergents	mg/l		1
Detergent foam	Mm	PE	7
Fluoride	Mg/l	F-	1,5
Total Phosphorous	mg/l	P	5
Total Hydrocarbons	mg/l		5
Total Iron	mg/l	Fe	5
Manganese	mg/l	Mn	0,3
Mercapatans	mg/l		0,02
Mercury	mg/l	Hg	0,001

Table 2.1-1 (continued)
Maximum Effluent Discharge Limits to Receiving Water Bodies
Reglamento Técnico DGNTI-COPANIT 35-2000

Parameter	Unit	Symbol	Limit
Molybdenum	mg/l	Mo	2,5
Nickel	mg/l	Ni	0,2
Nitrates	mg/l	NO_3	6
Total Organic Nitrogen	mg/l	N	10
Ammonium Nitrogen	mg/l	NH_3-N	3
Odor			No perceptible
Organochlorides	mg/l		1,5
Pentaclorophenol	mg/l	C_6OHCl_5	0,009
pH	Unidad	pH	5,5 - 9,0
Lead	mg/l	Pb	0,050
Selenium	mg/l	Se	0,01
Sodium	%	% Na	35
Settleable Solids	mg/l	S.SED.	15
Total Suspended Solids	mg/l	TSS	35
Total Dissolved Solids	mg/l	TDS	500
Sulfates	mg/l	SO_4^{-2}	1.000
Sulfur	mg/l	S^{-2}	1
Temperature	ºC		± 3°C De la T. N
Toluene	mg/l	$C_6H_5CH_3$	0,7
Tricloroethane	mg/l	HC_2Cl_3	0,04
Triclorомethane	mg/l	$CHCl_3$	0,02
Turbidity	NTU	NTU	30
Xylene	mg/l	$C_6H_4C_2H_6$	0,05
Zinc	mg/l	Zn	3

NOTE:
Color: The discharged effluent should not add color to the receiving water body
All concentrations refer to total values.
T.N: Normal site temperature.

2.2 Wastewater Flows and Characteristics

Design flow for treatment works located at the Rio Juan Diaz site projected to the year 2020 are tabulated below.

Table 2.2-1
Design Wastewater Flows

Parameter	Peaking Factor	Flowrate m^3/s	Flowrate mgd
Annual Average Daily Flow (AADF)	1.00	4.5	102
Sustained Peak	1.40	6.3	143
Peak Hour	2.00	9.0	204
Extreme Wet Weather Peak	2.35	10.5	240

The annual average daily flow is the average flow to the treatment plant over 365 days or one year. Flow to the treatment plant will vary hourly following diurnal flow patterns. During this diurnal flow pattern, peak hour flows occur both during dry and wet weather conditions. For the large collection system serving the population of Panamá City, both sustained hydraulic peaks and peak hour flows will occur. For the purposes of this report, a sustained peak is defined as a peak flow expected daily for a period of three hours or more. A peak flow lasting three hours exceeds the hydraulic retention time of an activated sludge aeration basin and is sufficient to stress biological systems and impact effluent quality. Therefore, the biological process tankage must be designed to accommodate such hydraulic peaks without upsets.

A maximum hour peak flowrate is a peak flowrate experienced by the plant for a duration of no more than one hour at any given time. While such a hydraulic surge will place stress on the biological systems of an activated sludge treatment plant, the short duration of the peak flow will be dampened by properly sized tankage.

Due to the magnitude of seasonal rainfall in Panamá City, an extreme wet weather peak is also anticipated. While such peak events will be experienced by the plant on a hydraulic basis, such events will not carry waste loadings of equal magnitude. Therefore, it would not be economical to hydraulically size biological treatment units for such an event. During extreme wet weather peak events most flows above that of the peak hour will bypass aeration units. Flows will be recombined in the secondary clarifiers for partial treatment of any bypassed flow prior to chlorination. The Panamanian Government over the next 15 years will either implement an extensive Inflow / Infiltration program to reduce wet weather flows or construct additional treatment works.

The major unit processes of the proposed treatment works will be sized to treat the peak hour flow rate with the largest unit out of service.

Wastewater characteristics for the entire sewage flow generated in Panamá City are summarized below. The wastewater strengths are typical of residential communities with medium to high collection system inflow and infiltration. Recent sampling and analytical test program have confirmed these values.

Table 2.2-2
Raw Wastewater Concentrations (mg/liter)

Wastewater Constituent	Annual Average	Maximum Month	Maximum Week	Maximum Day
$CBOD_5$	155	170	187	204
COD	400	440	490	530
TSS	155	170	187	204
NH3-N	17	18	20	22
TKN	22	24	26	29
TP	9	10	11	12

Wastewater strength will vary by the hour, day and season. Unit processes must be sized to handle both high organic loadings as well as hydraulic peaks through the individual treatment units.

2.3 Treatment Goals

Conventional parameters used for the design of a wastewater treatment plant are biochemical oxygen demand (BOD_5), total suspended solids (TSS), total phosphorus (TP), nitrogen and grease and oil

(AyG). Three (3) nitrogen based parameters listed in Table 2.1-1 are: Nitrate, Ammonia Nitrogen, and Total Organic Nitrogen. Total Organic Nitrogen consists of natural materials such as proteins and peptides, nucleic acids, urea, and numerous synthetic organic materials (NO_X-N). For the purposes of this report, the Total Organic Nitrogen limit of 10 mg/l will be applied as Total Nitrogen (TN). The use of TN (NO_X-N+TKN) is a more traditional parameter and insures that the Total Organic Nitrogen limit will be met at all times.

The chemical, physical and biological constituents listed in Table 2.1-1 must be treated and respective concentrations reduced below maximum limits prior to discharge into receiving waters. Effluent disposal to the Bay of Panamá requires the level of treatment necessary to meet the DGNTI-COPANIT 35-2000 Requisitos Generales De Las Descargas De Efluentes Liquidos A Cuerpos Receptores limits. Maximum values of effluent constituents routinely monitored at treatment works are presented in Table 2.3-1.

Table 2.3-1
Selected Effluent Limits

Parameter	Effluent Limit	
BOD_5	35	mg/l
Total Nitrogen, TN	10	mg/l
Total Phosphorus, TP	5	mg/l
Total Suspended Solids, TSS	35	mg/l
Total Residual Chlorine, TRC	1,5	mg/l
Grease and oils, AyG	20	mg/l
Total Coliforms	1.000	NMP/100ml

Marine outfall and diffuser systems provide an additional level of pollutant dilution and concentration reduction when a wastewater effluent (freshwater) is discharged from a submerged marine outfall diffuser and is mixed with seawater. Initial dilution and mixing with seawater occurs in the marine environment from both the rising freshwater plume and local sea currents. In many countries, regulatory agencies may define a mixing zone in the vicinity of the marine outfall as the surface area where wastewater effluent and seawater mix in order to reduce pollutant levels to acceptable concentrations. Typically, constituent concentrations are measured at the edge of the mixing zone where the receiving water quality standard or maximum pollutant concentration level must be achieved. Within the mixing zone, higher concentrations of wastewater pollutants exceeding water quality standards are allowed. In many countries, water quality standards are more stringent than specific pollutant limitations. Hence, the dilution associated with a mixing zone reduces constituent concentrations to acceptable levels for marine life. Panamá is in the process of developing water quality criteria.

The dilution of wastewater constituents within a marine mixing zone can be viewed as treatment process. There is a trade off between the environmental degradation allowed in the mixing zone and the economic savings associated with inexpensive treatment (dilution in seawater).

Under Reglamento Técnico DGNTI-COPANIT 35-2000, wastewater constituents must be reduced to maximum allowable limits prior to discharge into receiving waters. According to existing Panamanian law, no treatment credit is allowed for effluent dilution from a marine diffuser system.

2.4 Simplified BNR Activated Sludge Process

2.4.1 Process Description

The simplified BNR activated sludge process was recommended in the June 6, 2003 “Final Technical Assistance Report, Panamá Bay Sanitation Project”. The “Best Nutrient Removal (BNR)” modification was step-feed nitrification / dentrification of the activated sludge process. The aeration tank configuration included an anoxic zone in the front of each pass. Raw wastewater and return sludge are fed into the anoxic zone and allowed to mix in the absence of oxygen. The anoxic zone chamber is fully enclosed to prevent atmospheric oxygen to be dissolved in the tank content and to prevent the release of nuisance hydrogen sulfide gas. This is a simple modification to the activated sludge process and is easy to operate. The same anoxic zone was presented in the earlier July 2002 CMP supplement entitled “Conceptual Design and Implementation Report”. The process flow schematic is shown on Figure 2-1. The plant layout at the Juan Diaz site is shown on Figure 2-2. Expected performance is summarized below.

Table 2.4-1
Simplified BNR Activated Sludge process Performance

		Effluent Concentration (mg/l)			
Parameter	**Removal Percent**	**Annual Average**	**Maximum Month**	**Maximum Day**	**Discharge Limits [(1)]**
BOD_5	85%	20	<25	30	35
TSS	85%	20	<25	30	35
TN	77%	<7	<8	<8	10
TP	46%	<5	<5	5	5
AyG	75%	<10	<12	12	20

Note: (1) Maximum effluent discharge limits to receiving waters as prescribed in Regmento Técnico DGNTI-COPANTI 35-2000.

The simplified BNR activated sludge process meets Panamanian effluent discharge limits.

2.4.2 Primary Clarification Issue

The recommended activated sludge treatment process scheme does not have primary clarifiers. Primary treatment via gravity settling has been utilized for almost 100 years in major cities throughout the world. In most locations, primary sedimentation tanks were initially installed as the only means of waste treatment to minimize the nuisance discharge of pollutants to receiving waters, to protect public health and to improve water quality. As regulatory requirements became more stringent the original gravity settling primary treatment units were upgraded either by chemical addition to improve sedimentation performance or by installing secondary treatment works. In the United States, secondary treatment was mandated on a federal level in 1968. Over the next decades treatment works were upgraded or expanded in all communities across the USA. These construction programs were subsidized by federal grants up to 75 to 85 percent of the capital cost depending upon the technologies installed. Coastal cities such as Boston and San Diego postponed installation of costly secondary treatment works for almost 30 years due to a variety of fiscal, institutional and water quality reasons. The traditional wastewater treatment process sequence is primary sedimentation tanks followed by the activated sludge process.

In cold or temperate climates, gravity settling is an effective unit process. In large systems located in tropical and semi-tropical climates, primary treatment has not always been installed due to the following reasons:

a. At warmer liquid temperatures, the sedimentation process is not as effective.

b. In large collection systems, wastewater will become septic and some solids components will begin to breakdown into soluble BOD. This reduces the BOD level in the settled solids and the overall BOD removal by the sedimentation process.

c. In large collection systems with long detention times, hydrogen sulfide will be generated causing odor problems in treatment plant headworks and primary tanks. As a result of odor problems, primary sedimentation tanks are usually covered and all gases are collected and processed in chemical scrubbers. This is an additional capital and O&M expense.

In South Florida, regional wastewater treatment plants were constructed or expanded in the 1970's in response to the federal requirements mandating secondary treatment. The status of these secondary plants with respect to primary treatment works is as follows:

Table 2.4-2
South Florida Regional Secondary Treatment Works

Location	Design Capacity MGD	M^3/Sec.	Primary Treatment Facilities Yes	No
Miami-Dade County				
-Homestead	8	0.35	--	√
-South District	100	4.39	--	√
-Virginia Key	143	6.27	--	√
-North District	120	5.26	√	--
Broward County				
-Hollywood	58	2.54	--	√
-Fort Lauderdale	52	2.28	--	√
-Broward County OES	80	3.50	--	√
-Miramar	8	0.35	--	√
Palm Beach County				
-Boca Raton	20	0.88	√	--
-Boynton/Delray	26	1.14	--	√
-South Central	30	1.32	--	√
-ENCON	8	0.35	--	√
TOTAL	653	28.64		

These plants serve a population over 4 million South Florida residents. Two treatment plants with primary sedimentation tanks are the North District Plant located in Miami-Dade County and the Boca Raton facility. The primary treatment tanks have aluminum covers. All gases are captured and treated in chemical scrubbers.

In general primary sedimentation tanks are not utilized in semi-tropical south Florida for the following reasons:

a. Fair to poor BOD_5 removal anticipated. Performance has been confirmed in smaller primary treatment plants phased out of operation in the 1980's.

b. Elimination of odor problems associated with primary tanks and avoidance of odor scrubbing capital and O&M cost. Most regional plants are located in the close proximity of residential communities and odor emissions are not tolerated.

c. Cost effectiveness not demonstrated. Once the federal grant program was eliminated, communities could not fiscally justify the additional cost of primary sedimentation works.

d. Less land is required.

The construction cost of high rate primary clarifiers with pile supported foundations, covers to control odors and associated works is estimated at $11,000,000.

Table 2.4-3
Primary Clarification Process Present Worth Cost Analysis

Option		Present Worth Cost (1)
(A)	Construction of primary sedimentation tanks	$11,00,000
(B)	No primary tanks. Power cost associated with additional BOD_5 removal in aeration basin	$4,595,000

Note: (1) Present worth cost calculated on a Capital Recovery Factor of 0.1241 based upon a 12 percent interest rate over a 30 year period.

The "No Primary Tank" option is 50 percent more cost effective. This analysis does not take into account the additional O&M cost needed for the primary clarifiers or the incremental cost of increasing the solids digestion facilities associated with higher solids load from primary tanks.

The primary tanks associated with conventional activated sludge process removals more TSS and generates more (raw) solids to be processed in the anaerobic digesters.

BNR Activated Sludge Process	Solids to Anaerobic Digestion
No Primary Tanks	51
With Primary Tanks	73

Larger and more costly anaerobic digestion tanks are needed with the conventional BNR activated sludge process. There exists no cost justification for primary tanks. A process advantage of primary clarification that with chemical addition (CEPT), higher flows can be treated and provide good TSS removal and adequate BOD removals. This allows more flexibility in phasing future works. The final clarifiers for the biological activated sludge plant can be provided with more "rugged" mechanisms and operate in the primary mode when required. Hazen and Sawyer has designed secondary clarifiers to operate in the dual mode (primary or secondary) in large industrial activated sludge plants. We recommend that this modification be made to about 25 percent of the secondary tanks. This will allow the biological treatment units to operate within their hydraulic limits and provide a method to add chemicals and treat excess wet weather flows.

2.4.3 Construction Cost

Construction cost have been developed for simplified BNR activated sludge plants in modules of 25, 51, 76 and 102 MGD as a planning tool.

Table No. 2.4-4
Simplified BNR Activated Sludge Treatment Construction Cost
Rio Juan Diaz Wastewater Plant

		Construction Cost versus Plant Capacity			
Item	**Description**	**102 MGD**	**76 MGD**	**51 MGD**	**25 MGD**
1.	Sitework	$4,500,000	$3,500,000	$2,500,000	$1,500,000
2.	Paving	1,150,000	900,000	700,000	500,000
3.	Pretreatment Works	8,870,000	8,200,000	7,700,000	7,000,000
4.	Aeration Basins	25,780,000	19,400,000	13,000,000	6,500,000
5.	Blower Building	4,900,000	4,000,000	2,500,000	1,500,000
6.	Secondary Clarifiers	23,910,000	18,000,000	12,000,000	6,500,000
7.	RAS-WAS Pump Stations	5,410,000	4,000,000	2,800,000	1,400,000
8.	Chlorine Facility	2,040,000	1,700,000	1,300,000	1,000,000
9.	Aeration Basin Dist. Box	810,000	700,000	500,000	300,000
10.	Clarifier Dist. Box / Piping	2,860,000	2,200,000	1,500,000	800,000
11.	Yard Piping	9,500,000	7,800,000	5,000,000	2,470,000
12.	Anaerobic Digesters	18,610,000	15,000,000	11,000,000	4,800,000
13.	Solids Handling	5,530,000	5,000,000	4,500,000	3,000,000
14.	Generators	4,330,000	3,800,000	3,500,000	1,500,000
15.	Maintenance Building	800,000	800,000	800,000	400,000
16.	Sludge Landfill	6,000,000	5,000,000	4,000,000	3,000,000
	Sub-Total	125,000,000	100,000,000	73,330,000	42,170,000
17.	20% Overhead and Profit	25,000,000	20,000,000	14,670,000	7,830,000
	Total Construction Cost	**$150,000,000**	**$120,000,000**	**$88,000,000**	**$50,000,000**

The construction cost of the smaller 25 MGD activated sludge plant is $50,000,000. The unit cost is $2 per gallon. This cost level is typical of primary treatment works in Latin America. The unit cost decreases with the larger plant size.

2.4.4 Simplified BNR Activated Sludge Operating Cost

Annual operational and maintenance cost have been developed for various size plants.

Table 2.4-5
Annual Operational and Maintenance Cost
Simplified BNR Activated Sludge Plant
Juan Diaz WWTP, Republic of Panamá

Flow – MGD	25	51	76	102
Manpower	$300,000	$600,000	$860,000	$1,000,000
Electrical Power	1,100,000	2,200,000	3,300,000	4,300,000
Chemical	200,000	380,000	540,000	750,000
Solids Disposal	550,000	1,000,000	1,500,000	2,000,000
Maintenance	125,000	250,000	375,000	500,000
Total Annual O&M Cost	**$2,275,000**	**$4,430,000**	**$6,575,000**	**$8,550,000**
Unit O&M Cost in $ / 1,000 gallon	$0.249	$0.238	$0.234	$0.230

2.5 Conventional BNR Activated Sludge Process

2.5.1 Process Description

The unit process is similar to the activated sludge treatment plant presented in the July 2002 CMP supplement. The plant includes primary treatment gravity settling tanks. The process schematic is show on Figure 2-3. In order to meet nutrient effluent limits, the step-feed nitrification / dentrification BNR modification of the activated sludge process is proposed. Expected performance is summarized below.

Table 2.5-1
Simplified BNR Activated Sludge process Performance

		Effluent Concentration (mg/l)			
Parameter	**Removal Percent**	**Annual Average**	**Maximum Month**	**Maximum Day**	**Discharge Limits (1)**
BOD$_5$	85%	20	<25	30	35
TSS	85%	20	<25	30	35
TN	77%	<7	<8	<8	10
TP	46%	<5	<5	5	5
AyG	75%	<10	<12	12	20

Note: (1) Maximum effluent discharge limits to receiving waters as prescribed in Regmento Técnico DGNTI-COPANTI 35-2000.

2.5.2 Primary Clarification

High rate gravity clarifiers are proposed in order to minimize capital cost. This is common design practice in large scale primary treatment works followed by biological treatment units in metropolitan areas such as New York City, Washington, D.C., and Miami, Florida.

The plant area requirements will increase for siting the primary clarifiers and their associated works. Due to the high hydrogen sulfide levels present in the raw wastewater any dedicated primary clarification tanks would have to be covered to contain the odorous emissions. All contained gases would be treated in chemical scrubbers.

Number of Units	8
Tank Dimensions	
- Diameter	125 foot
- Side Water Depth	15 foot
Surface Loading Rate	
Annual Average	1040 gpd/s.f.
Peak Flow	2080 gpd/s.f.
Wet Weather Peak Flow	2440 gpd/s.f.

Removals through the high rate primary treatment units is estimated as follows:

	Primary Removals Percent
BOD_5	25%
TSS	50%

Primary tanks will add reliability to the overall process. Solids and organic loadings are dampened out when passing through the primary units. Chemicals can be added to reduce organic loading to the aeration tanks. Chemical addition to primary tanks to remove colloidal solids and the associated BOD_5 is more costly than the equivalent electrical power cost to remove the incremental BOD_5. Chemical addition has been practiced at several plants in order to defer capital expenditures to enlarge the biological treatment works. These cost principles have been well documented at the 400 MGD Hyperion WWTP located in Los Angeles and other locations. Primary treatment units also generate more total solids than an equivalent sized biological plant without front end primary units as noted below.

Activated Sludge Process	Solids Loading to Anaerobic Digesters
Conventional with Primary Clarification	73 tons / day
Without Primary Clarification	51 tons / day

The additional solids load increases anaerobic digester capital and O&M costs.

2.5.3 Construction and Operation Cost

Construction cost for various sized conventional activated sludge treatment plants are tabulated below.

Table No. 2.5-2
Conventional Activated Sludge Treatment Construction Cost
Rio Juan Diaz Wastewater Plant

Item	Description	Construction Cost versus Plant Capacity			
		102 MGD	**76 MGD**	**51 MGD**	**25 MGD**
1.	Sitework	$4,500,000	$3,500,000	$2,500,000	$1,500,000
2.	Paving	1,150,000	900,000	700,000	500,000
3.	Pretreatment Works	8,870,000	8,200,000	7,700,000	7,000,000
4.	Primary Tanks with Odor Control	11,000,000	7,800,000	4,700,000	2,500,000
5.	Aeration Basins	25,780,000	19,400,000	13,000,000	6,500,000
6.	Blower Building	4,900,000	4,000,000	2,500,000	1,500,000
7.	Secondary Clarifiers	23,910,000	18,000,000	12,000,000	6,500,000
8.	RAS-WAS Pump Stations	5,410,000	4,000,000	2,800,000	1,400,000
9.	Chlorine Facility	2,040,000	1,700,000	1,300,000	1,000,000
10.	Aeration Basin Dist. Box	810,000	700,000	500,000	300,000
11.	Clarifier Dist. Box / Piping	2,860,000	2,200,000	1,500,000	800,000
12.	Yard Piping	9,500,000	7,800,000	5,000,000	2,470,000
13.	Anaerobic Digesters	21,000,000	17,000,000	11,000,000	4,800,000
14.	Solids Handling	5,530,000	5,000,000	4,500,000	3,000,000
15.	Generators	4,330,000	3,800,000	3,500,000	1,500,000
16.	Maintenance Building	800,000	800,000	800,000	400,000
17.	Sludge Landfill	6,000,000	5,000,000	4,000,000	3,000,000
	Sub-Total	$138,390,000	$109,800,000	$80,000,000	$46,370,000
18.	20% Overhead and Profit	27,610,000	22,200,000	16,000,000	9,630,000
Total Construction Cost		**$166,000,000**	**$132,000,000**	**$96,000,000**	**$56,000,000**

Note: Scope of work includes an activated sludge treatment plant with primary tanks and BNR step feed configuration for TN removal, interim biosolids landfill and a short diffuser pipeline into Panamá Bay.

Annual O&M cost are tabulated below for the conventional BNR activated sludge treatment plants rated at different flows. Unit O&M Cost ranged from $0.223 per 2,000 gallon to $0.215 per 1,000 gallons.

Table 2.5-3
Annual Operational and Maintenance Cost
Conventional Secondary Treatment with Primary Clarifiers
Juan Diaz WWTP, Republic of Panamá

Annual O&M Cost				
Flow – MGD	25	51	76	102
Manpower	$300,000	$600,000	$860,000	$1,000,000
Electrical Power	950,000	2,000,000	2,900,000	3,750,000
Chemical	200,000	400,000	560,000	750,000
Solids Disposal	550,000	1,000,000	1,500,000	2,000,000
Maintenance	125,000	380,000	400,000	500,000
Total Annual O&M Cost	**$2,125,000**	**$4,250,000**	**$6,200,000**	**$8,000,000**
Unit O&M Cost in $ / 1,000 gallon	$0.233	$0.228	$0.224	$0.215

2.5.3 Evaluation for Conventional and Simplified BNR Activated Sludge Plants

A cost comparison of both BNR activated sludge plants is summarized below.

Table 2.5-4
Cost Comparison of 102 MGD BNR Activated Sludge Plants

	Simplified without Primary Clarifiers	**Conventional with Primary Clarifiers**
Construction Cost	$150,000,000	$166,000,000
30% Project Management Cost	45,000,000	49,800,000
Capital Cost	**$195,000,000**	**$215,800,000**
Annual Operation and Maintenance Cost	$8,550,000	$8,000,000
Present Worth Analysis		
- Capital Cost	$195,000,000	$215,800,000
- P.W. of O&M Cost	67,000,000	62,700,000
Total Present Worth Cost	**$262,000,000**	**$278,500,000**

Our observations are as follows:

1. The simplified BNR activated sludge plant has a lower capital cost ($20.8 million) and a lower total present worth cost (5 to 6%).

2. As noted in Section 2.3.2, Primary Clarifiers are not cost effective in terms of BOD removal by fine bubble aeration in the biological process.

3. An additional 5 to 10 percent BOD removal across the primary tanks could be achieved by spending an additional $8 to 10 million capital expenditure to construct additional gravity tanks. This incremental BOD removal is not cost effective.

4. Primary tanks will require covers to control odors and additional O&M cost for chemical scrubbers.

5. Primary tanks will need additional plant area.

The simplified BNR activated sludge plant is less capital intensive and has the process capability to meet Panamanian effluent discharge limits under all design conditions. Due to capital constraints imposed on the Panamá Bay Sanitation Project, the simplified BNR activated sludge plant is the preferred process over the conventional BNR process.

2.6 Enhanced Primary Treatment

2.6.1 CEPT Process Description

Enhanced coagulation is a chemical treatment process which was popular in the United States from 1930 through the 1950s. It was initially developed in England in the 1890s. The decline of the chemical treatment process in the United States was due to the development of biological processes that produced better effluent results and the national policy of defining secondary treatment (which did not include enhanced coagulation).

Several large-scale wastewater treatments utilize enhanced coagulation with primary treatment as a cost-effective method to remove incremental BOD rather than by biological treatment. The cost savings is a trade-off between chemicals versus power in terms of dollars per pound BOD removed. Ferric chloride and ferrous chloride are the most popular chemicals. The insoluble precipitate, ferric hydroxide is compatible with the anaerobic digestion process (biological and chemical).

The process for an operating facility is commonly referred to as “Chemically Enhanced Primary Treatment (CEPT)”. Chemicals (metal salts and polymers) cause the suspended particles to clump together via the process of coagulation and flocculation. The particle aggregates, or flocs settle faster thereby enhancing treatment efficiency, measured as removal of solids, organic matter and nutrient from the wastewater.

In typical domestic wastewater the relative allocation of BOD_5 in different physical factions is summarized below.

Raw Wastewater BOD_5 Demand Physical Source	Percent of Total BOD_5 Demand
Soluble Faction	30%
Colloidal Solids	35%
Settleable Solids	35%

With conventional primary tanks located in temperate to cool climates, approximately 35% total suspended solids removal is achievable by gravity settling. In large collection systems located in semi-tropical or tropical climates where septic conditions exist, sewage solids will start to chemically breakdown (hydrolyze) into components. BOD_5 demand will transfer from the settleable solids faction to the soluble or colloidal solids faction. This phenomenon has been observed in operating primary treatment works located in Miami, Fort Lauderdale and Los Angeles. BOD_5 removals across primary gravity settling tanks is less effective in facilities located in warm semi-tropical climates. With conventional gravity settling tanks in Panamá, the expected BOD_5 removal would be about 25 percent.

Enhanced primary treatment will increase BOD_5 removal by removing colloidal solids. When large quantities of industrial process waste discharge into a municipal collection system, then overall solids and BOD_5 removals at the treatment works will be impacted by the industrial wastewater characteristics. An example is boxboard recycling plants which discharge process waste with high solid loads (tissue particles) with associated BOD_5 contribution. Joint municipal-industrial treatment works with papermill process waste have demonstrated high TSS and BOD_5 removals with primary and enhanced primary treatment. Certain municipalities such as Toronto, Canada have reported high BOD_5 removals through enhanced primary treatment facilities. Care must be taken when interpreting this data and a full understanding of industrial waste volumes, loadings and characteristics discharged into the respective municipal collection system and transported to treatment works is necessary when assessing plant performance criteria.

Table 2.6-1 illustrated the average removal of total suspended solids (TSS) and BOD_5 at domestic wastewater treatment works. The data is based upon a survey of 100 wastewater treatment plants in the United States.

Table 2.6-1
Comparison of Removal Efficiencies in Primary and Enhanced Treatment Plants (1)

	TSS (%)	BOD_5 (%)	TP (%)	TN (%)	FOG(2) (%)
Conventional Primary Treatment	55	35	20	15	51
Chemically Enhanced Primary Treatment	85	57	85	37	71

(1) Source: National Research Council, 1992.

(2) FOG is fats, oils and grease (AyG).

Theoretically, high chemical dosages and efficient coagulation and flocculation processes capable of removing all colloidal and settable solids, up to 70 percent BOD5 removal can be achieved with domestic raw sewage in a CEPT facility. Soluble BOD5 can not be removed by the CEPT process. From a practical and economic viewpoint, BOD5 removals in the 50 to 60 percent range are more realistic for large scale CEPT works. Typical CEPT operating experience reported at two large scale U.S. treatment plants are summarized below. Los Angeles' raw waste is similar to San Diego's and reflects the high influent BOD5 and TSS concentrations typical of separate sewer systems with low infiltration.

Table 2.6-2
Treatment Performance at Two Large California CEPT Facilities

	Point Loma WWTP San Diego, California	Hyperion WWTP Los Angeles, California
Flow (mg/l)	191	370
TSS		
- In (mg/l)	305	270
- Out (mg/l)	60	45
- Removal	80%	83%
BOD_5		
- In (mg/l)	276	300
- Out (mg/l)	119	145
- Removal	57%	52%
Ferric Chloride (mg/l)	35	20
Anionic Polymer (mg/l)	0.26	0.25

Hazen and Sawyer conducted bench scale enhanced coagulation tests at the Miami-Dade, Florida, 140 MGD Central District Wastewater Treatment Plant to assess the feasibility of enhanced settling of raw wastewater. The results are shown on Exhibits 2-1 and 2-2.

Exhibit 2-1

Performance of
Enhanced Coagulation
on Raw Influent BOD_5 Reduction
Total BOD_5 Removal (%)
versus
Ferric Chloride Dosage

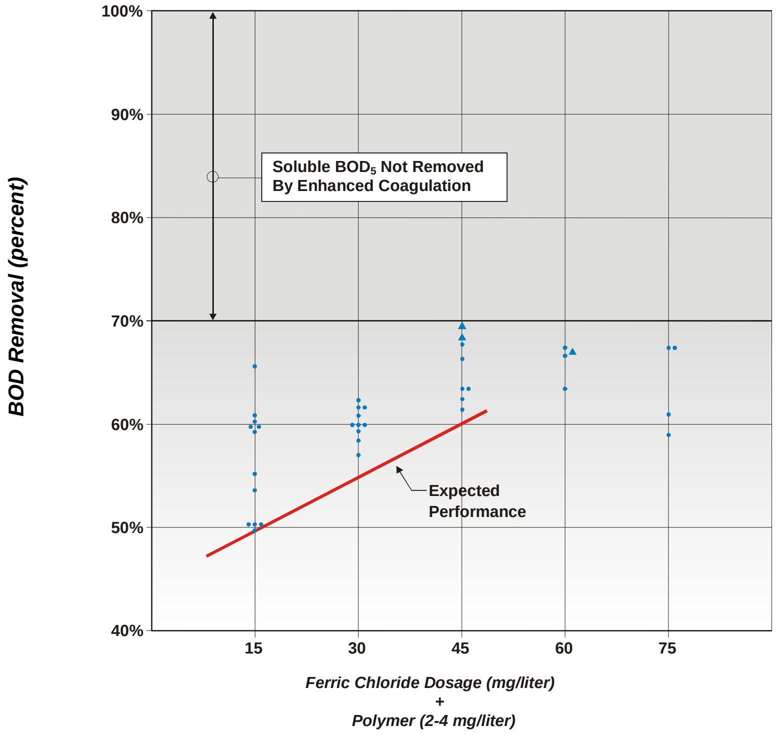
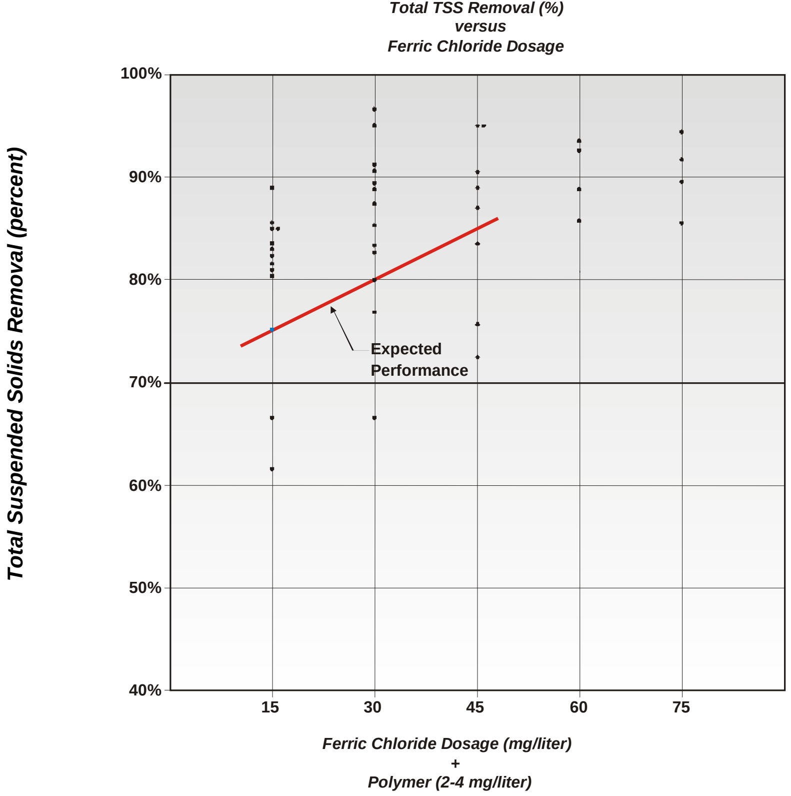


Exhibit 2-2

Performance of Enhanced Coagulation on Raw Influent Total Suspended Solids Reduction



Projected CEPT removals at the Miami-Dade, Florida 140 MGD wastewater plant with ferric chloride dosages of 15 to 45 mg/l and the addition of polymer are as follows:

	Removals
BOD_5	50 - 60%
TSS	75 - 85%

These removals are consistent with full-scale operating results reported from Los Angeles and San Diego enhanced coagulation primary treatment large scale operations.

The USEPA has issued regulations and time tables for municipalities to limit and treat raw wastewater spills, bypasses and overflows under all wet weather conditions. As a result, a number of proprietary unit treatment processes have been developed by equipment suppliers using high level chemical dosages (ferric chloride and polymer), settling aids and high rate gravity clarification to treat bypass raw wastewater flows over a relatively short period of time. High chemical dosages would not be economical if the process operated continuously. However, when operating 10 to 20 days a year to treat bypass flows and meet federal discharge requirements, these proprietary process are a viable solution. The goal of these treatment processes is to remove TSS and the BOD_5 associated with settleable and colloidal solids. These processes are a modification of the CEPT process. High rate clarification coupled with high chemical dosages are used to minimize capital cost and optimize performance of these commercial systems. In each process, a large number of small particles (sand or sludge) is added to provide a nuclei for rapid floc formation. A description of the two most successful proprietary processes is outlined in Table 2.6-3.

Table 2.6-3
Summary of High Rate Chemically Enhanced Clarification Systems

Commercial Process	Description	Features
Actiflo	Microsand ballasted flocculation and lamella clarification.	Microsand provides nuclei for floc formation. Floc is dense and settles rapidly. Lamella clarification provides high rate settling in a small tank volume.
Densadeg	Two-stage flocculation with chemically-conditioned recycled sludge followed by lamella clarification.	Settled sludge solids are recycled to accelerate floc formation. Dense floc is formed that settles rapidly. Lamella clarification provides high rate settling in a small tank volume.

Performance data of full scale pilot units is tabulated in Table 2.6-4.

Table 2.6-4
High Rate Clarification Chemically Enhanced Process Performance

	TSS Removal (%)	BOD Removal (%)	Ferric Coagulant Dosage (mg/l)	Polymer Dosage (mg/l)
Actiflo				
Galveston, Texas	91	73	100	1.0
Mexico City, Mexico	91	-	80	0.8
Cincinnati, Ohio	79	45	45 - 100	1.0 – 1.3
San Francisco, California	70 – 80	50 – 60	60 – 80	1.0
Fort Worth, Texas	88	63	125	1.3
New York, New York(1)	84	54	60 – 100	0.5 – 1.0
Densadeg				
Birmingham, Alabama	80 – 95	-	45	1.5
Little Rock, Arkansas	65 – 90	50 – 80	60 – 65	1.5 – 2.0
Bremerton, Washington	85	61	60	2.0
San Francisco, California	75 – 90	60 – 75	70 – 90	2.0
Fort Worth, Texas	92	51	150	1.8
New York, New York(1)	69	54	50 – 70	1.4 – 1.8

Note: (1) Results from full scale test programs conducted by Hazen and Sawyer staff in New York City.

Even with ferric coagulant dosages in the 60 to 90 mg/l range, typical BOD_5 removals range between 50 to 60 percent with these high rate chemically enhanced clarification unit processes. This is consistent with large CEPT performance of two South California plants treating over 500 MGD of wastewater.

2.6.2 CEPT Design Criteria

It is anticipated that the chemically enhanced primary treatment (CEPT) works located at the Juan Diaz site will perform as follows.

Table 2.6-5
CEPT Plant Performance

		Effluent Concentration (mg/l)			
Parameter	**Removal Percent**	**Annual Average**	**Maximum Month**	**Maximum Week**	**Discharge Limits** (1)
BOD_5	55%	70	76	84	35
TSS	80%	31	34	37	35
TN	37%	14	15	16	10
TP	85%	1.4	1.5	1.6	5
AyG	71%	12	13	14	20

Note: (1) Maximum effluent discharge limits to receiving waters as prescribed in Reglamento Técnico DGNTI-COPANIT 35-2000

The BOD_5 and TSS removals are consistent with performance documented by the National Research Council (1992) survey and full scale operation at two large CEPT operations in California treating over 500 MGD of raw wastewater. Iron salt coagulant dosages of 30 to 45 mg/liter coupled with anoxic polymers (0.2 to 0.5 mg/l) will be needed to maintain consistent BOD_5 removals. Flexibility would be provided to adjust chemical dosages to maximize performance and economics at plant start-up.

In October 2001, the Consolidated Master Plan evaluated CEPT. Removal rates of 80 percent of TSS and 40 percent BOD_5 across the primary clarifiers were used in the conceptual design. The engineers considered these removal rates as "reasonable for the environmental conditions in Panamá, for instance, the warm climate and the pumping of wastewater in the collection system would contribute to the hydrolysis of particulate BOD_5." Hazen and Sawyer is using 55% BOD removal for the CEPT process versus the 40% BOD removal used in the previous October 2001 report prepared by others.

BOD_5 and Total Nitrogen (TN) concentrations in the treated CEPT effluent would exceed and contravene the Panamanian discharge limits outlined in Reglamento Técnico DGNTI-COPANIT 35-2000. Other conventional pollutant constituents (TSS, TP and AyG) would be at acceptable or marginal effluent levels and comply with the discharge limits.

Under the scope for work of this assignment, the engineers are requested to review the feasibility of a wastewater system consisting of the following two elements:

- Chemical Enhanced Primary Treatment (CEPT) Plant
- Marine Outfall and Diffuser System

Both BOD_5 and TN effluent levels from a CEPT plant in the marine outfall pipe and at the diffuser discharge location would contravene discharge limits. Once the effluent was discharged from the marine outfall diffuser, the effluent (freshwater) would rise and mix with seawater. When the rising mixture of effluent and seawater reached the surface waters, the diluted mixture would be at pollutant concentrations less than the discharge limits. **As noted before, Panamanian regulations define the point of discharge at the "end of pipe", not at the edge of a seawater mixing zone.**

2.6.3 CEPT Unit Sizing

The CEPT treatment plant will have many of the same elements as the original recommended “simplified” BNR activated sludge plant. The headworks which consist of the following facilities will be common for all alternatives.

- An enclosed building with mechanical screens and degritting classifiers. All interior air will be collected and treated in chemical scrubbers prior to release to the atmosphere.
- Covered vortex grit works will be located adjacent to the screen building. All air in the covered tankage will be collected and treated in chemical scrubbers.
- Administration building.
- Site work for the headworks and administrative building will be the same magnitude of cost.

Shown on Figure 2-4 is the process flow schematic and material balance for the CEPT plant. After the pretreatment (screening and grit removal) works, the following unit processes are needed.

- Chemical storage and feed systems for iron salts (ferric or ferrous chloride and an anoxic polymer).
- Flash mix system to insure proper mixing and energy to dissolve the chemicals into solution. These works would consist of inline mixers or dedicated tankage.
- Flocculation works to optimize the coagulation and flocculation of chemicals and solids. Successful European flocculation works have 10 to 20 minutes detention time. Due to the high hydrogen sulfide levels present in the raw wastewater any dedicated flocculation tanks would have to be covered to contain the odorous emissions. All contained gases would be treated in chemical scrubbers. It is more cost effective to install the flocculation chamber in the gravity clarifier mechanism. This avoids duplicate odor collection works and the construction of independent reinforced concrete tankage for flocculators and gravity settlers.
- Gravity settling tanks with an internal flocculation chamber (clari-flocculator units). The gravity settling tanks will have aluminum covers. Due to the warm weather and long detention time in the collection system, the raw wastewater will be septic. Hydrogen sulfide will be generated. The primary tanks will be covered and all gases are collected and processed in chemical scrubbers. This is common practice in treatment works located in warm climates in the USA for the last 25 years. This practice minimizes odor nuisance to adjacent residences.

  The gravity settling tanks are sized as follows:

Number of Units	16

Tank Dimensions	
- Diameter	125 foot
- Side Water Depth	15 foot
Surface Loading Rate	
Peak Flow	1040 gpd/s.f.
Wet Weather Peak Flow	1220 gpd/s.f.

Surface loading rates are calculated based on the nominal tank diameter. However, the incorporation of the flocculation chamber within the feed area will reduce the effective tankage surface area and volume. Several competing clarifier mechanism configurations can meet the design criteria for a flocculation chamber in a circular gravity settling clarifier.

- Settled raw sludge from the clarifier at about 1.5 percent solids consistency will be pumped to belt press solids thickening equipment. Thickened solids at 4.0 percent consistency are fed into the anaerobic digestion system. The CEPT solids loading consist of raw settleable and colloidal TSS and chemical precipitate from the iron salts ($FeCl_3$, etc.). Loading to the anaerobic digestion system compared to the "simplified" BNR activated sludge process is as follows:

Process	**Solids Loading Tons /Day**
CEPT	71
"Simplified" BNR Activated Sludge	51

Waste activated solids generated from the BNR activated sludge process have been partially broken down (equivalent to aerobic digestion) in the biological process. The CEPT process generates about 40 percent more waste solids which must be processed, dewatered and disposed of to the interim landfill or other acceptable land application sites.

- Digested solids will be dewatered to a 25 percent dry "sludge cake" and hauled to acceptable disposal sites. An interim landfill will be located adjacent to the treatment works.

Site area requirements for the CEPT plant will be the same as the "simplified" BNR activated sludge plant. Area will be reserved for future biological treatment units. The existing primary clarifiers can be converted to secondary clarifiers. Hence, the CEPT can be converted to a "simplified" BNR activated sludge plant in the future in the event regulatory or environmental concerns dictate an upgrade of facilities.

The major operating cost with CEPT plants will be the purchase and handling of ferric salts. Chemical suppliers from the USA and Mexico have been contacted for pricing. Ferric salts are ferric sulfate, ferrous chloride and ferrous sulfate which are shipped as concentrated liquid. These liquid compounds are corrosive and need special containers. One chemical supplier is confirming if dry ferric sulfate in

1,000 lb. sacks can be economically shipped from Japan. F.O.B prices at USA / Mexico ports range from $330.00 to $350.00 per ton (dry weight). We have assumed $400.00 per ton delivered to Panamá and then transported and unloaded at the Juan Diaz plant site. This is the same price quoted for ferric chloride at the 91 MGD CEPT plant in Bogotá, Colombia. The range of annual cost for FeCl3 delivered to the Juan Diaz site for a 102 MGD CEPT plant depending upon chemical dosage is estimated as follows.

Ferric Chloride		
mg/l	**lb. / Day**	**Annual Chemical Cost**
45	38,300	$2,796,000.00
40	34,000	$2,484,000.00
35	29,800	$2,175,000.00

Annual chemical coagulant cost range between $2 to $3 million dollars to insure TSS compliance with existing discharge limits and obtain relatively high removals of BOD (non-compliance).

In the event no coagulant chemicals were added at the CEPT plant (budget limitation or delayed chemical shipment) then we would expect about 25 percent BOD removal and 50 percent TSS removal across the gravity tanks. The final effluent quality would not meet Panamanian discharge limits for most conventional parameters (BOD5, TSS, TN).

2.6.4 CEPT Construction Cost

Construction cost estimates have been prepared for 25 MGD, 51 MGD, 76 MGD and the ultimate plant size of 102 MGD CEPT plant at the Juan Diaz site. These estimates will provide input for the planners to phase works to match flows and available funds.

Table No. 2.6-6
Enhanced Primary Treatment Construction Cost
Rio Juan Diaz Wastewater Treatment Plant

		Construction Cost versus Plant Capacity			
Item	**Description**	**102 MGD**	**76 MGD**	**51 MGD**	**25 MGD**
1.	Site Work	$3,000,000	$2,500,000	$2,000,000	$1,000,000
2.	Paving	800,000	700,000	500,000	300,000
3.	Pretreatment Works	8,900,000	8,200,000	7,600,000	7,000,000
4.	Primary Clarifiers	19,000,000	13,000,000	8,000,000	4,000,000
5.	Chemical / Odor Control Works	3,000,000	2,300,000	1,500,000	800,000
6.	Primary Solids Pump Station	2,400,000	1,800,000	1,200,000	600,000
7.	Chlorine Facility	2,000,000	1,700,000	1,400,000	1,000,000
8.	Clarifier Distribution Box / Piping	2,900,000	2,200,000	1,500,000	800,000
9.	Yard Piping	5,000,000	4,000,000	2,600,000	1,400,000
10.	Anaerobic Digesters	20,000,000	16,000,000	12,000,000	6,000,000
11.	Solids Handling	6,000,000	5,400,000	4,700,000	3,000,000
12.	Generators	2,000,000	1,700,000	1,500,000	1,000,000
13.	Maintenance Building	400,000	400,000	400,000	400,000
14.	Sludge Landfill	4,000,000	3,500,000	3,000,000	2,000,000
	Subtotal	79,400,000	63,400,000	46,900,000	29,300,000
15.	20% Overhead and Profit	15,600,000	12,600,000	9,100,000	5,700,000
	Total Construction Cost	**$95,000,000**	**$76,000,000**	**$56,000,000**	**$35,000,000**

Note: Scope of work includes an enhanced primary treatment plant with chemical feed / mixing works, covered primary clarifiers and an interim biosolids disposal landfill. Cost of an effluent pump station and marine outfall are estimated separately.

To confirm the "magnitude of construction cost" of $95,000,000 for a 102 MGD CEPT plant, we compared the cost of the $4m^3/s$ (91.2 MGD) Salitre Primary Treatment Plant recently installed in Bogotá, Colombia. This plant must contractually meet the following performance criteria.

	Removal Percent
BOD_5	40%
TSS	60%

CEPT works were added to insure performance compliance. The 91 MGD plant was completed in the year 2000. The Inter-American Development Bank (IDB) participated in the funding package for the DBO project. A French Consortium was awarded a 30-year concession contract. Hazen and Sawyer was retained by the IDB to review the project technical, economic benefits and environmental issues. Financing was finalized and work commenced on September 17, 1997. In 1999, Hazen and Sawyer was retained by the French Consortium to evaluate the project progress and cost as part of the requirements for internal financing. In 1999, construction cost of the 91 MGD Primary Treatment Plant (CEPT capabilities) was $79,000,000. The Salitre Plant has all the same elements and treatment units as proposed for the 102 MGD CEPT plant at the Juan Diaz site. Salitre Plant escalated construction costs from year 1999 to 2003 are tabulated below.

Table 2.6-6
91 MGD Salitre CEPT Construction Cost Adjustment
Bogotá, Colombia[1]

CEPT Plant Capacity MGD	Year	Adjusted Construction Cost	Unit Construction Cost $/ MGD [2]
91	1999	$79,000,000	$868,000
91	2000	80,185,000	881,000
91	2001	81,388,000	894,000
91	2002	82,610,000	908,000
91	2003	83,850,000	921,000

Notes: (1) Salitre CEPT Plant start-up was in the year 2000.
(2) Construction cost escalated 1.5 percent / year from 1999 to 2003.

Using the year 2003 adjusted $921,000 / MGD unit construction cost, the 102 MGD Juan Diaz CEPT is calculated to cost $93,000,000. This cost compares with the Hazen and Sawyer construction cost estimate as follows.

Source of Cost Estimate	Juan Diaz 102 MGD CEPT Construction Cost
Estimated by Hazen and Sawyer	$95,000,000
Calculated equivalent cost using 91 MGD Salitre CEPT, Bogotá, Colombia adjusted actual construction cost.	$93,900,000

The cost comparisons confirms the "order of magnitude" of the 102 MGD CEPT plant construction cost for planning purposes.

2.6.5 CEPT Plant Operating Cost

Annual operational and maintenance cost have been developed for various sized CEPT plants. During the planning phase, plant size was identified in different time frames. Unit O&M costs range between $0.164 to $0.179 per 1,000 gallon wastewater treated. A key variable to control cost is the ability to maintain staff at levels required to efficiently operate and maintain the works. Coagulant chemical dosages were estimated in the 35 mg/liter range.

Table 2.6-7
Annual Operational and Maintenance Cost
Enhanced Primary Treatment
Juan Diaz WWTP, Republic of Panamá

Flow – MGD	25	51	76	102
Manpower	$250,000	$400,000	$500,000	$600,000
Electrical Power	400,000	775,000	1,200,000	1,500,000
Chemical	650,000	1,250,000	1,900,000	2,500,000
Solids Disposal	550,000	1,000,000	1,500,000	2,000,000
Maintenance	150,000	200,000	250,000	300,000
Total Annual O&M Cost	**$2,000,000**	**$3,625,000**	**$5,350,000**	**$6,900,000**
Unit O&M Cost in $ / 1,000 gallon	$0.219	$0.195	$0.193	$0.183

2.7 Marine Outfall

2.7.1 Design Criteria

As part of the scope of work of this assignment, the sizing and cost of a marine outfall system is to be determined for CEPT works located at the Juan Diaz site. The consultants preparing the 1998 CESOC Master Plan funded by the Inter-American Development Bank (IDB) conducted extensive field testing in Panamá Bay to obtain data on currents, salinity, temperature and other pertinent oceanographic data. Along proposed diffuser routes, site specific data was obtained on area wide currents as well as along the vertical water column. The Princeton Ocean Hydrodynamic Model was used to simulate the current dynamics in Panamá Bay. At site specific marine outfall diffuser locations, the impact of effluent discharges into the marine water for both nearfield and farfield regimes were calculated to assess the physical dilution capability and impact on water quality and marine ecology.

The basic design assumptions presented in the 1998 CESOC Master Plan were used to select the marine outfall route. Alternative 5A Emisario Submarino, was the example used to locate the diffuser.

Diffuser Length	750 meters
Average Diffuser Depth at Mean Sea Level	9.3 meters
Diffuser Spacing	5 meters
Average Current Speed	12 cm/s
Initial Dilution Goal	100 : 1

In Panamá Bay, normal tide variations are as follows.

	Elevation Meters
Mean Sea Level	+ 0.307
High Tide (median)	+ 2.140
Low Tide (median)	(-) 1.695

The daily tide variation is about 3.835 meters (12.6 feet). In order to have a 9.3 meter depth to the top of the diffuser pipe at mean sea level conditions, the top of pipe must be set at about El. (-) 9.0. At low tide, the water depth over the diffuser is about 7.3 meters. On Figure 2-5 the route of the marine outfall pipe from the Juan Diaz Plant is shown. The diffuser ends at about the 10 meter depth low tide water contour. The average depth of the 750 foot long diffuser is 9.3 meters measured at mean sea level. The total length of the marine outfall is 7.5 km (24,600 L.F.).

Drawing PD-ES-01 entitled "Perfil Del Emisario Submarino y Detalles", dated May 2001, prepared by CESOC, shows a 1400 mm diameter marine outfall. The 750 meter diffuser section is placed on the sea bottom and is not buried. Diffusers are located on the sides of the High Density Polyethylene Pipe (PEAD) at a water depth of 9.3 meters measured at mean sea level. In the CESOC text, this marine outfall is ALTERNATIVE 5A sized for a 4.05 m^3/s peak flow (92 MGD).

Initial dilutions of the raw wastewater were calculated independently using the NRFIELD (previously known as RSB near field model).[1] This diffuser model was used on outfall projects located in Boston, Massachusetts; Mamala Bay, Hawaii; Singapore; and Cartagena, Colombia. Dr. J. Phillip Roberts, from Georgia Tech participated in the development of the recent version of the near field model, is a consultant on many World Bank marine outfall projects.

The CESOC Initial Dilution Goal is 100 to 1 at mean sea tide level and an average current speed of 12 cm/second. The NRFIELD model was run for the following conditions:

Flow Rate	90 MGD
Diffuser Length	750 m
Diffuser Submergence at Mean Sea Level	9.6 m
Diffuser Port Diameter	4 inches

Initial dilutions were calculated varying current speed as well as tide levels.

Table 2.7-1
Near Field Initial Dilution Ratios
Calculated with NRFIELD Models

Current Speed cm/s	Low Tide	Mean Sea Level	High Tide
0	53	65	78
5	53	65	78
10	86	104	125
12	106	**134**	164
14	120	154	190
16	133	172	216

At mean sea tide level and a 12 cm/s current, the calculated initial dilution is 134 versus the goal of 100. The CESOC approach to calculate initial dilution ratios is valid.

It should be noted that at slack tide (low current speeds between 0 and 5 cm/s) conditions, the magnitude of the initial dilution ratios range from 50 to 1 to 80 to 1. Nearfield surface water BOD_5 levels in the vicinity of a marine outfall diffuser with CEPT treatment would be about 1 mg/liter concentration under slack tide conditions.

Beyond the near field, the plume drifts with the bay currents and is diffused by available turbulence in the bay waters. The rate of mixing in this region, known as the far field, is much slower than in the near field. Another important process in the far field is bacterial reduction due to mortality and decay.

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[1] *Baumgartner, D. J., Frick, W. E., and Roberts, P. J. W. (1994). "Dilution Models for Effluent Discharges (Third Edition)>" EPA/600/R-94/086, U.S. Environmental Protection Agency, Office of Research and Development, Washington, D.C. Brooks, N. H. (1960) "Diffusion of Sewage Effluent in an Ocean Current." University of California, 246-267, 1959.*

In Panamá Bay, within three hours bacteria levels are reduced about 1 log (1,000 MPN/100 ml to 100 MPN/100 ml).

2.7.2 Marine Outfall Hydraulics

The hydraulic design of the Juan Diaz WWTP is as follows.

Parameter	Peaking Factor
Annual Average Daily Flow (AADF)	1.00
Sustained Peak	1.40
Peak Flow	2.00
Extreme Wet Weather Peak	2.35

The marine outfall pipe is sized to convey the peak hour flow at about 8.0 feet / second (2.44 m/s) velocity. Maximum velocity rates are 10.0 feet / second (3.0 m/s) in order to maintain pipe integrity. Pipeline friction losses increase almost 50 percent as velocity increases from 8 feet / second to 10 feet / second. In order to utilize the additional 20 percent pipeline hydraulic capacity, effluent pump, motor, and electrical equipment must be sized for 50 percent higher load. The ability to handle extreme wet weather peak flows is a policy issue. These extreme flows can be pumped (extra horsepower) or allowed to bypass after treatment into adjacent receiving waters.

Marine outfall pipe sizing is tabulated below based upon transporting the peak hour flow at a velocity in the pipe at 8 feet / second.

Table 2.7-2
Marine Outfall Hydraulics

Flows				
- AADF (MGD)	25	51	76	102
- Peak Hour (MGD)	50	102	152	204
Pipe Data				
- Interior Diameter (inches)	42	60	72	84
- Fluid Velocity (feet/second)	8.04	8.00	8.3	8.3
- Friction Loss (feet/1,000 feet @ C=130)	4.05	3.07	2.24	1.91
- Total Friction Loss for 24,600 L.F. Outfall Pipe (feet)	100	76	55	47

Effluent pumps are required to lift and transport peak hour flows through the outfall. Typically, at low and average flow rates, the marine outfall operates in the gravity mode. As wastewater flows increase (wet weather at sustained peak flow rates) the pumps must be activated to convey treated effluent through the marine outfall. This is normal operating procedure at South Florida marine outfall installations.

2.7.3 Marine Outfall Cost Estimate

Pipe materials used in marine outfalls handling treated wastewater effluents are as follows.

1. Reinforced Concrete Pipe (RCP) – This pipe has been installed in marine outfalls for the past 100 years. Pipe companies could bring the pipe molds to Panamá and fabricate the pipe

locally. Modern practice is to cast a plastic T lock liner on the top 180° of the pipe interior. This protects the pipe against any potential corrosion due to hydrogen sulfide. Large diameter RCP marine outfalls have recently been installed in San Diego, California and Miami, Florida. Service life of an RCP marine outfall is 100 years or more.

2. Ductile Iron Pipe (DIP) – DIP and cast iron pipe have been used extensively for marine outfalls and is a common pipe material in the waterworks field. In some locations the DIP may require cathodic protection against corrosion. The French fabricate up to a 72-inch diameter DIP.

3. Glass Reinforced Fiberglass Pipe (GRP) – GRP manufactured in Colombia (Flowtite), Argentina and the USA (HOBAS) would be competitive for the Panamá outfall. The fiberglass pipe is corrosion resistant and is suitable for transporting wastewater.

4. Polyethylene Pipe (PEAD) – PEAD is fabricated from high density polyethylene (HDPE) and has been recently used in marine outfalls in Brazil, Chile, and many European countries. Large diameter PEAD is fabricated in Norway, Canada, and Chile. Current practice at many locations is to install the PEAD on the sea bottom. Concrete weights are attached to resist flotation and to secure the pipe against a design "wave". PEAD pipe is fabricated based upon a minimum 50-year service life. Often the same philosophy is followed in PEAD marine outfall systems (50-year service life). The owner of a PEAD marine outfall system must fully understand all potential risks and benefits when allowing this pipe material. PEAD systems can be rapidly installed, reducing installation cost.

5. Concrete Coated Steel Pipe (CEST) – These concrete / steel pipelines used extensively in the oil industry require a cathodic protection system to resist corrosion and extend service life.

Marine outfall pipe diameters (42, 60, 72, and 84-inch I.D.) were selected for initially sized wastewater plants. On Table 2.6-3 are listed commercially available pipe materials for each diameter outfall pipe and installation cost. The installation costs were developed from marine outfall cost presented in the CESOC study, the recent independent cost estimates prepared by a Chilean consultant for the Cartagena, Colombia, 72-inch marine outfall to be bid this year (H&S design) and recent outfall costs in other international locations. The outfall installation costs are consistent with the CESOC Master Plan marine engineering cost estimates funded by IDB. These installation costs are adequate for planning purposes.

Table 2.7-3
Marine Outfall Pipe Material and Installation Cost
Panamá, Republic of Panamá

Pipe Diameter			Pipe Material(1) and Fabrication(2) Location	Installation Cost $/meter(3)
O.D. mm	I.D. mm	I.D. inch		
1220	1067	42.0	RCP – Pan, USA, Co, Mex	$4,600
1130	1105	43.5	DIP – USA, Fr, Br, Co	
1200	1130	44.5	PEAD - No, Ch, USA, Ca	
1067	1097	43.2	GRP – USA, Co	
1143	1041	41.0	CEST – USA, Ja, C	
1727	1524	60.0	RCP – Pan, USA, Co, Mex	$6,600
1600	1500	59.0	PEAD - No, Ch, Ca, USA	
1565	1534	60.4	DIP – USA, Fr	
1598	1542	60.7	GRP – USA	
1637	1600	63.0	GRP – Co, USA	
1651	1500	59.0	CEST – USA, Ja, C	
2083	1830	72.0	RCP – Pan, USA, Co	$8,200
2000	1846	72.7	PEAD - No	
1870	1830	72.0	DIP – Fr	
1841	1800	70.9	GRP – Co, USA	
1915	1852	72.9	GRP – USA	
1980	1803	71.0	CEST – USA, Ja, C	
2400	2134	84.0	RCP – Pan, USA, Co	$10,200
2335	2108	83.0	CEST – USA, Ja, C	

(1) Pipe Materials

RCP: Reinforced Concrete Pipe
DIP: Ductile Iron Pipe
PEAD: High Density Polyethylene Pipe (HDPE)
GRP: Glass Reinforced Fiberglass Pipe
CEST: Concrete Encased Steel Pipe

(2) Fabrication Location

Pan – Panamá	*Ca – Canada*
USA – United States	*Ch – Chile*
Co – Colombia	*No – Norway*
Fr – France	*Mex – Mexico*
Ja – Japan	*C - China*

(3) Total Estimating Cost is $3,200,000 Mobilization Cost plus $ per meter pipe installation cost.

Due to funding constraints, elements of the wastewater may be phased and placed in operation over a 5 to 10-year period. Phasing projections indicated that a 50 MGD WWTP plant could be initially constructed and the remainder of the system could be implemented and constructed within 8 years. Under this scenario, two alternatives for the marine outfall were developed for evaluation.

Alternative A – Construct the 84-inch diameter marine outfall in the initial phase sized. The outfall is sized for the ultimate flow (102 MGD).

Alternative B – Construct a 60-inch diameter marine outfall for the 51 MGD treatment works. Install effluent pumps capable of handling up to 125 MGD (10 feet/second maximum pipeline velocity) through the marine outfall. Within 4 to 5 years after installation of the first 60-inch diameter outfall, commence construction of a second parallel 60-inch diameter marine outfall.

The cost estimate of both systems is presented on Table 2.6-4 and summarized below.

	Alternative A **1-84 inch Diameter Outfall**	**Alternative B** **2-60 inch Diameter Outfalls**
Capital Cost		
Year 1	$123,400,000	$88,300,000
Year 8	7,800,000	75,000,000
Total Capital Cost	$131,200,000	$163,300,000
Total Present Worth	$127,400,000	$126,800,000

The single 84-inch diameter marine outfall is $32,000,000 less costly to install than two 60-inch diameter outfalls installed over an 8-year period.

From an economic present worth analysis viewpoint, both investments are equivalent using a 12 percent discount rate and a 2 percent per year cost escalation factor.

Table 2.7-4
Marine Outfall Capital Cost Comparison of Phasing Construction

	Alternative A One (1) 84-inch Diameter Outfall	**Alternative B Two (2) 60-inch Diameter Outfalls**
Stage I Year	1	1
Construction Cost		
- Outfall Mobilization	$ 3,200,000	$ 3,200,000
- Outfall Installation	76,500,000	49,500,000
- Pump Station	15,200,000	15,200,000
	$ 94,900,000	$ 67,900,000
30% allowance for P.M., Engr, Legal and Contingencies	28,500,000	20,400,000
Capital Cost at Year 1	**$123,400,000**	**$88,300,000**
Stage II Year	8	8
Construction Cost		
- Outfall Mobilization	--	$ 3,200,000
- Outfall Installation	--	49,500,000
- Pump Station Expansion	5,200,000	5,200,000
	$5,200,000	$ 57,900,000
30% allowance for P.M., Engr, Legal and Contingencies	1,600,000	17,100,000
Capital Cost at Year 8	7,800,000	75,000,000
Total Program Capital Expenditure	**$131,200,000**	**$163,300,000**
Present Worth Analysis		
Phase I Present Worth	$123,400,000	$88,300,000
Phase II Present Worth(1)	4,000,000	38,500,000
Total Present Worth Cost	**$127,400,000**	**$126,800,000**

Notes: (1) Total Present Worth Analysis calculated on a 12 percent discount rate and a 2 percent annual construction cost escalation rate.

Reasons for a single marine outfall project sized for the ultimate flow considered for planning purposes are outlined below.

1. Technical – A single marine outfall is hydraulically more efficient and will require 500 less installed motor horsepower in the effluent pump system.

2. Local Impacts – Installation of an outfall will take 24 to 30 months. Construction equipment in Panamá Bay will restrict marine traffic in the vicinity of the construction activities. Water turbidity levels will increase in the vicinity of the outfall construction. Installation of two outfalls will double the construction time from 2 to 2 ½ years to 4 to 5 years over an 10-year period that marine construction activity is ongoing off-shore from the Juan Diaz site.

3. Environmental – Construction of a marine outfall will have temporary impacts on Panamá Bay water quality and ecology. In the event the project is phased, then the impacts will be doubled

and prolonged over two construction periods. The regulatory and environmental community may require additional construction permit conditions and controls after observing the first marine pipeline installation. Project cost may increase substantially for a second marine outfall.

4. Public Health – Construction workers on the second marine outfall will be working adjacent to the operating outfall in seawater mixed with treated effluent. To lower risks, higher dosages of chlorine will be required to disinfect effluent. In the event of a treatment plant upset, marine contractor must be warned and activities in which workers are exposed to contaminated waters must cease. This will result in higher project risks and costs both to the operators of the plant and the marine contractor.

5. Public Acceptance – Marine outfall projects are subject to close review by the environmental community and other concerned public groups. A phased outfall approach extends the project exposure over an 8-year period. Any public frustration over any element of the entire program can be focused on the phase two marine outfall. If the second outfall project is delayed, then the entire program is in trouble. Increased flows at the treatment plant while the second marine outfall project is delayed may result in effluent bypassing when capacity of the first marine outfall is exceeded, causing additional public anxiety.

6. Implementation – Marine outfalls are traditionally installed to handle flows projected over a 25 to 50-year planning horizon. They require specialized equipment and workers in the marine construction field. Permitting is complex due to the environmentally sensitive nature of the project. Extensive project resources are required to obtain public and agency approval to implement a marine outfall project. Contemplating the construction of two sequential marine outfall projects over a 10-year period is not practical or realistic from a program management viewpoint.

7. Economic - The economic analysis shows both projects have the same present worth cost, based upon a 12 percent discount rate and 2 percent cost escalation over the 8 years. In capital dollars, the single outfall is $32,000,000 less costly. In all probability the second phase 60-inch marine outfall capital cost will be higher than currently estimated due to working conditions adjacent to the Phase I operating outfall and additional conditions imposed by regulatory and funding agencies due to the extended construction period in the bay.

Due to the short implementation horizon (10 years) and practical project implementation considerations, one 84-inch diameter marine outfall from the Juan Diaz WWTP site is the preferred option. However, funding constraints may dictate phasing of two (2) marine outfalls.