APPENDIX XIII

Preliminary Sewage Treatment System Design and Cost Estimate

XIII PRELIMINARY SEWAGE TREATMENT SYSTEM DESIGN AND COST ESTIMATE

XIII.1 GENERAL

The Soapberry site is located between the Jamaica Railway on the north, Hunt's Bay on the south and it is between the Duhaney River and Sandy Gully, on the east and the dike of the Rio Cobre River flood protection system on the west. The land area is sufficient for facultative lagoon systems to serve Stages 1 and 2 of the proposed KMA sewerage development, including approximately 1/3 of the 287,000 projected Portmore population. There are, however, problematic site features to be dealt with:

1. The land is near sea level, with most of the area lying between 1.0 and 4.0 feet above mean sea level, and the highest elevation on the site, in the northwest, only about 12.0 feet above mean sea level.
2. The soils and groundwater conditions over much of the site will require special construction methods.
3. The central portion of the site is underlain with a layer of peat that is expected to vary from shallow lenses to 75 feet thick, which will give rise to settlement that must be anticipated by design.
4. The prevailing winds are southeasterly, with 60% of the higher wind velocities approaching from the exposed southeast, side of the site.

Because of the low site elevations and the high water levels that occur during severe storms, the tops of the lowest dikes need to be set above elevation 9.0 feet above mean sea level. System layouts have been developed using preliminary soils reports that were prepared in 1975 and 1976 for then proposed residential and industrial use of the site. The reports are only a guide, since they were prepared for a different engineering purpose, and were largely based on interpretation of soils information gathered from a number of sources. Detailed site borings over the entire site are needed before detailed lagoon design is started.

Layouts have been prepared using limited available site topography. It is apparent that some drainage and dredging/fill work has been done on the site, since the topographic maps were prepared. Thus detailed topographic mapping is also required for the site along with the geotechnical information.

Based on the preliminary geotechnical reports, it has been assumed that the portion of Soapberry to the north and west consists of more consolidated clays, sands and silt, with a relatively shallow thickness of peat underlying the zone (Zone A in the soils report). That portion of Soapberry to the east of Zone A has thick layers of peat underneath, and relatively low bearing capacity (Zone B of the soils report). Zones A and B are shown on Figure XIII.1. The central area of the Soapberry site, between the dashed lines shown on Figure XIII.1, may contain the greatest thickness of peat underlying surface materials. On the basis of the limited site information, there is uncertainty as to the actual soils conditions, however, it is apparent that Zone A, closest to the Rio Cobre, offers the best conditions for construction of facultative lagoon cells. There will be considerable settlement in much of Zone B and somewhat less in Zone A. A request was made to NWC to initiate a detailed geotechnical program to guide this phase of work, but this has not been provided; consequently, quantities and cost estimates have been developed to reflect these uncertainties to the best practical extent without more complete data.

The quantities of material used in construction of the lagoons have been adjusted in proportion to the height of dikes and the estimated depth of peat underlying the dikes. A factor, which varies from a minimum of 20% for compaction and settlement to a maximum of 100% in the worst areas, has been applied to all of the calculated earth quantities.

Prior to further preliminary design, or any detail design being undertaken, additional field information must be obtained, as follows:

1. General site investigation to establish access, existing drainage provisions, existing fill on-site and other general information.
2. Field survey in sufficient detail to provide reliable contours, to locate ditches and to locate test holes.
3. Geotechnical work including site test holes to determine depth of peat, other subsurface materials, permeability of clays, depth to water, etc.

4. Geotechnical report to provide complete soil profile, to recommend design and construction procedures, and to confirm the proposed elevations of the bottoms of lagoon ponds.

The investigative work may include provision of temporary surface drainage, additional geotechnical work to determine thickness of underlying peat, possible rate of settlement of dikes, suitability of the upper layers of material for dikes, and estimates of quantities required for subsequent stages.

Also, as there will be a shortage of suitable material to construct dikes, investigations will be needed to determine sources of borrow and suitability of dredged material for berm construction.

The site is low and much of it is barely above seal level, which may require general site filling in some areas. During the period of investigation, it will be determined how best to pre-load future dikes in advance of dike construction, in order that some of the settlement can take place before the subsequent stages are built.

Another consideration in the lagoon layout is the fact that 60% of the higher wind velocities are from the southeast, and the entire Soapberry site is open to the southeast. There is very little choice in orientation of lagoon cells since the highest and best soils conditions are along the west side, and the land slopes from the northwest toward the Bay. The wind direction can, to an extent, be used to facilitate the treatment process, by maintaining an aerobic surface zone over the deeper anaerobic section to minimize the release of odours. However, wave action due to winds needs to be addressed in the design.

Flow through the lagoons will be from the northwest to the southeast, following the general slope. The first stage is planned for the west side adjacent to the Rio Cobre flood dike, with subsequent construction to the east of the first stage, as shown in Section XIII.4 Site Limits.

There is existing flood protection along the Rio Cobre River which is expected to prevent future flooding of the lagoon site from that source, and the lagoon dikes will be high enough for flood protection from the Duhaney River. It was estimated in the Soapberry evaluation reports that maximum hurricane flood height along the Hunts Bay shore would be in the order of 6.8 feet above mean sea level. The proposed minimum lagoon dike top elevation is well above this elevation.

The bottom of the lagoon cells is set as low as practical to provide adequate lagoon liquid depth, and to provide material for construction of dikes. Preliminary pond bottom elevations were established considering:

1. Spring tides are up to elevation +0.8 feet which is the highest tide level.
2. Basic maximum still water level, which is the combination of spring tides and storm surges, are reported to be 3.0 feet above mean sea level.

Detailed geotechnical evaluations will be needed to finally determine the most practical elevation for pond excavation; however, to develop preliminary costs, several reasonable trial elevations were assumed. One trial assumed that lagoon bottoms could be excavated to +1.5 feet above mean sea level, which may not be possible over the entire site. Another trial was made using a bottom elevation of +3.0 feet which is at the basic maximum still water depth. This is considered to be more buildable, however there would be a shortage of earth for construction of Stage 2 with this option. A third trial was made using a bottom elevation of +2.0 feet above mean sea level.

XIII.2 LAGOON SEWAGE FLOWS

The sewage treatment systems were developed to provide effective treatment of the projected sewage flows for Stage 1 and Stage 2. Lagoon construction can be staged for these principal stages based on the sewage flows summarized in Table XIII.1 entitled "Sewage Flows to Soapberry Lagoon". The projected average daily flow from Stage 1 is 34.35 mgd and for Stage 2 it is 60.46 mgd. The lagoons are shown in 4 series of ponds, with Series 1 and 2 planned for Stage 1 and Series 3 and 4 planned for Stage 2. Each series is comprised of 4 ponds through which sewage flows in series. The relative sizes are:

Stage 1 - 34.35 mgd average flow
Series 1 - 17.2 mgd
Series 2 - 17.2 mgd

Stage 2 - add 26.11 mgd average flow
Series 3 - 13.1 mgd
Series 4 - 13.1 mgd

TABLE XIII.1

SEWAGE FLOWS TO SOAPBERRY LAGOON

Stage Source	Average Daily Flow			Peak Hourly Flow		
	MIGD	IGPM	CFS	MIGD	IGPM	CFS
Stage 1						
Nanse Pen	10.93	759	20.29	19.90	13,820	36.94
Greenwich	17.00	11,810	31.57	28.71	19,940	53.32
Hunt's Bay	1.26	880	2.34	3.34	2,320	6.19
Portmore	5.16	3,580	9.59	10.89	7,560	20.23
Total	**34.35**	**23,850**	**63.78**	**56.18**	**39,010**	**104.32**
Stage 2						
Nanse Pen	26.02	18,070	48.34	41.07	28,520	76.31
Greenwich	27.06	18,800	50.28	42.46	29,490	78.90
Hunt's Bay	0.68	470	1.27	1.99	1,380	3.70
Riverton	1.54	1,070	2.86	3.96	2,750	7.36
Portmore	5.16	3,580	9.59	10.89	7,560	20.23
Total	**60.46**	**41,990**	**11,234**	**100.37**	**69,700**	**186.50**

NOTE: Hunt's Bay and Riverton sewage flows join Greenwich flows in Hunt's Bay area, near Sandy Gully.

Nanse Pen pumps directly to the Soapberry Lagoons.

Portmore sewage is pumped directly to the lagoons from Independence City and Caymanas Gardens Pump Stations.

The projected average daily flow from Portmore is 4.80 MIGD as reported in Appendix X. These flows for Portmore are increased 10% for an added infiltration provision, and higher peak factors have been used here for design.

XIII.3 SEWAGE TREATMENT

The sewage treatment process proposed is based upon the requirement that each "Series" of ponds provide the necessary number of days retention, and that 8 primary cells within the first pond be sized for 1¼ days retention with 4 cells in the second pond sized for ½ day retention. Each series of 4 ponds provides:

Pond 1 - 10 days retention
Pond 2 - 5 days retention
Pond 3 - 3 days retention
Pond 4 - 2 days retention

Individual pond depths may vary slightly, however it is desirable to have at least 12 feet of liquid depth in the first pond and a minimum of 10 feet of liquid depth in other following ponds. The tops of submerged dikes around the primary cells contained within the initial ponds are 3' below the operating depth of the pond.

XIII.4 SITE LIMITS

The sewage flows to the site from the northwest corner of Soapberry from Portmore, and to the northeast corner of Soapberry from Kingston. The slope of the Soapberry site is from northwest to southeast. The discharge of treated effluent is proposed to be to the Rio Cobre River near the mouth of the River entering Hunt's Bay. The most suitable soils for lagoon construction are in the northwest part of the site and along the Rio Cobre River. All lagoon arrangements were based on presumption that:

1. Flows through the lagoons would be generally north to south.
2. Construction of the first stages would commence from the west side with final stages of construction on the east side.

Because flow of sewage from Greenwich is planned to be by gravity, the maximum elevation of the water level in the first lagoon pond was set at 15.0 feet above mean sea level. The site is low and the lowest bottom elevation considered for any pond was 1.5 feet above mean sea level.

XIII.5 LAGOON REQUIREMENTS

Several trial layouts were examined, all of which conformed to the site limits and to the following requirements.

1. Staging of sewage flows and peak sewage flows in accordance with Table XII.1 "Sewage Flows to Soapberry Lagoon".
2. Total retention of 20 days made up of: Pond 1 - 10 days, Pond 2 - 5 days, Pond 3 - 3 days and Pond 4 - 2 days.
3. Eight primary cells are within Pond 1 - 1¼ days retention in each.
4. Four primary cells are within Pond 2 - ½ days retention in each.
5. Minimum liquid depth 10 feet in any pond.
6. Maximum liquid elevation 15.0 feet above mean sea level.
7. Effluent discharge to Rio Cobre River near Hunt's Bay. Effluent to be discharged below river level.
8. Minimum dike tops are 12 feet wide, finished with marl surface for protection and access.
9. Dike top widths of 16 feet are proposed where necessary to provide for possible future piping. Wide dike tops are also surfaced with marl.
10. Maximum side slopes are 3:1 on pond dikes and 2:1 on divider dikes in primary cells.
11. Submerged primary cell dikes have top width 6 feet and top elevation 3 feet below pond operating level.
12. The first pond in each series has freeboard of 4.0 feet and all other ponds, which are smaller, have 3.0 feet freeboard.
13. Concrete slope protection is provided on the inside slopes of all ponds from elevation 2.5 feet below high liquid level to the top of dike.
14. Heavy rip-rap storm protection is provided on exterior slopes adjacent to Hunt's Bay from elevation 12.0 feet to bottom of dike slopes.
15. Lagoon hydraulic structures are sized to carry peak sewage flows to Pond #1, and to carry average sewage flows following Pond #1 in each series.
16. Lagoon dike clearance from the Rio Cobre dike is 150 feet from centreline of flood dike to centreline of lagoon dike.
17. Railway clearance from the Jamaica Railway is 200 ft. from track to centreline of lagoon dike.

18. Septage Haul - In all cases there will be provision in Stage 1 for dumping truck hauled septage in the lagoon.

Each series of ponds is as follows:

Pond 1 - 10 days retention
Pond 2 - 5 days retention
Pond 3 - 3 days retention
Pond 4 - 2 days retention

There are 8 submerged primary cells within Pond 1 of each series. Each is sized to hold 1/8 of the sewage flow in that series, for 1.25 days. Within Pond 1 of Series 1 there is an additional pair of cells for receiving truck hauled septage. There are 4 further submerged primary cells within the second pond in each series, which are each sized to retain ¼ of the sewage flow for ½ day. (Figures XIII.2 to XIII.6 show the layouts and profiles of the lagoon arrangements described.)

XIII.6 TRIAL LAYOUTS

The trial layouts were each made, meeting the above requirements, with the variable items being pond top and bottom elevations, with resulting different liquid depths and pond dimensions. Layout drawings were prepared and a schematic profile of each showed the relative elevation, depths and drop in elevations through a series of ponds.

Trial #1 - Refer to Figure XIII.2 Trial #1 Lagoon Layout and Figure XIII.3 Trial #1 Profile

This trial was based on a lagoon bottom elevation of 1.5 feet above mean sea level which may be difficult, and perhaps impractical, to construct. Maximum pond liquid depths are 12.5 feet and minimum pond depths are 10.0 feet. This trial gives relatively low dike top elevations and results in the least fill quantities.

There is sufficient excavation material on site to excavate from Stage 1 ponds, construct Stage 1 dikes and pre-load some of the area for Stage 2 dikes. Also there is material available for Stage 2 construction, which reduces the need for off-site borrow for Stage 2 to approximately 60,000 cubic yards.

Trial #2 - Refer to Figure XIII.4 Trial #2 Lagoon Layout and Figure XIII.5 Trial #2 Profile

This trial assumes that lagoon bottoms are excavated to elevation 3.0 feet above mean sea level. The dike top elevations are relatively high as a result, to give pond depths of maximum 12.0 feet and minimum 10.0 feet. The earth balance however is not good over the two stages. There is sufficient material to excavate Stage 1 ponds and to construct Stage 1 dikes; however, because of the higher dikes and reduced excavation quantities available, it would be necessary to borrow some 670,000 cubic yards of material off-site for Stage 2. Although some may be obtainable from dredging in Hunt's Bay, not all can come from that source.

Trial #3- Refer to Figure XIII.2 Trial Lagoon Layout #2 and Figure XIII.6 Trial #2 Profile

This trial was based on the same layout as Trial #1, assuming excavation of ponds to 2.0 feet above mean sea level and constructing dikes 0.5 feet higher than those proposed in Trial #1. Pond depths vary from 10.0 feet minimum to 12.5 feet maximum.

The earth balance is such that excavation for Stage 1 ponds provides sufficient material to construct Stage 1 dikes plus pre-load parts of Stage 2. When Stage 2 is constructed it would be necessary to borrow approximately 400,000 cubic yards of material off-site. Again, some of the material needed could be dredged from Hunts Bay.

XIII.7 LAGOON STRUCTURES AND PIPING

Sewage flows come to the site from the following sources:

Stage 1	Portmore	1 - 30" forcemain 1 - 8" forcemain
	Nanse Pen	1 - 30" forcemain
	Greenwich	1 - 48" siphon 1 - 30" siphon
Stage 2	Greenwich Nanse Pen	1 - 48" siphon 1 - 42" forcemain

The costs of lagoon piping include extension of the Portmore forcemains from the NW corner of Soapberry to the Distribution Structure and extension of the Nanse Pen and Greenwich pipelines to the distribution structures from grid line 600, where the limits were arbitrarily set between the Kingston Collection and Transmission System piping and Soapberry Lagoon piping. (Refer to Figure XIII.7 for a schematic drawing of the lagoon piping.)

Series 1 and Series 2 ponds are built in Stage 1, and sewage flows to the ponds from:

Portmore - 30" and 8" forcemains
Nanse Pen - 30" forcemain
Greenwich - 36" and 48" siphons (Refer to Figure XIII.8 for a siphon discharging from the Greenwich Transfer Station.)

When flows increase and Series 3 and 4 of Stage 2 are built, the distribution of flows is planned as follows:

Stage 1 (Pond Series 1 and 2)

- Portmore - 30" and 8" forcemains
- Nanse Pen - 30" and 42" forcemains (to full capacity)
- Greenwich - part of Greenwich flow

Stage 2 (Pond Series 3 and 4)

- Greenwich siphons with some flow to Pond Series 1 and Series 2.

The Stage 1 Lagoon Structure Plan in Figure XIII.9 shows the inlet structures and piping for Stage 1 (Series 1). The inlet piping for all 4 series of lagoons will be basically the same. Sewage flows initially to the Distribution Structures and then is divided to Splitter Boxes in which flows are split into 8 pipes feeding the primary cells in each pond. A series of sketches has been prepared to show the various lagoon structures. Elevations have been shown based on only the Trial #2 assumptions.

Because of the very large flows and large pipe sizes, multiple structures have been shown to keep pipe sizes down. Also because of possible settlement problems, multiple pipes and structures where chosen for easier maintenance. Where concrete structures are shown, the costs of cast-in-place concrete piles have been included; however, geotechnical studies may determine that different support is required.

Preliminary estimates have been made assuming use of heat fused polyethylene pipe materials within the lagoon system for flexibility and for corrosion resistance.

Several lagoon sections have been marked on the Lagoon Structure Plan. These lagoon sections indicate typical lagoon dikes which are shown further in Figure XIII.10, Figure XIII.11, and Figure XIII.12. While the sections are based on Trial #2 elevations, the sections are basically the same for all three trials.

a) <u>Distribution Structure</u> - see Stage 1 Distribution Structure Plan (Figure XIII.13)

Peak sewage flows enter this structure and are split in ½ to each of two splitter boxes. This reinforced concrete structure is on piles and has a grated cover with handrails around it. Motorized sluice gates are provided on the outgoing lines to the splitter boxes and also on the 48" Greenwich Sewer to permit control of Stage 2 flows.

The gravity flow lines to the splitter boxes are pile supported concrete pipelines.

b) <u>Splitter Boxes and Lagoon Inlets</u> - Refer to Figure XIII.14 Typical Flow Splitter Box Plan and Figure XIII.15 Typical Lagoon Inlet Pipe Section

The splitter box splits the flow into two sections - one for short inlet pipes and one for long inlet pipes, which are a larger size. If necessary, flows can be balanced by manipulation of sluice gates. The pipes discharge 2.5 ft. above the bottom of primary lagoon cells within the first pond.

c) <u>Septage Inlet</u>

The septage inlet will be a 12 in. diameter pipe with connection for dumping truck hauled septage waste. (Septage facilities at Greenwich will not be used once the lagoon is in operation.)

d) Pond Overflows (Pond 1 to 2)

The inlet pipes from the splitter box discharge near the bottoms of primary cells and sewage flows over primary cell dikes to the dike between Pond 1 and Pond 2. Refer to the Typical Plan of Pond Overflows in Figure XIII.16. Liquid then overflows from just below the high level of Pond 1 to near the bottom of the primary cells in Pond 2, through two overflow structures from Pond 1 to Pond 2. These are shown in Plan on the sketches of "Typical Overflow Structures Pond 1 to Pond 2 (Figure XIII.17) and Typical Overflow Cross-Section Pond 1 to Pond 2" (Figure XIII.18). These are reinforced concrete structures on piles, with metal weir plate to overflow ½ of the design average flow rate at no more than 6 in. of head. The piping, which is 24 in. ID, then discharges near the bottom of the Pond 2 primary cells.

e) Pond Overflows - Pond 2 to 3 and 3 to 4 (Refer to Figure XIII.19)

Following Pond 2 the liquid from just below the high level of Pond 2 overflows into Pond 3 near the pond bottom.

With reference to the typical plan of pond overflows (Figure XIII.20), it is proposed that four structures be used to distribute flows evenly and for maintenance purposes. These are reinforced concrete structures on piles with overflow weirs, as shown on Typical Overflow Structures - Pond 2-3 and Pond 3-4.

f) Lagoon Overflow Structure

Sewage flows into the fourth pond from Pond 3. The pond layouts show in all trials that the fourth pond for Series 1 and Series 2 is a single pond for both series. Likewise in Stage 2 a single fourth pond applies to both Series 3 and Series 4.

The treated effluent then overflows from the upper levels of Pond 4 and is piped to the effluent diffuser during overflow conditions. Under normal conditions, effluent is pumped to the Wetlands site(s). (Refer to Figure XIII.21 Lagoon Overflow Structure - Plan and Figure XIII.22 Lagoon Overflow Structure Section.) Figure XIII.23 shows a typical section through the dike alongside Hunts Bay.

Effluent is piped by three pipes from just below the high water level to an overflow weir sized and set to overflow the average flow from Stage 1 (Series 1 and Series 2) with 12 in. of head loss to a 48 in. I.D. effluent outfall pipeline. The structure is a reinforced concrete structure on cast-in-place concrete piles with metal weir. The effluent outfall pipe extends into the Rio Cobre River near its outlet to Hunts Bay.

g) Effluent Diffuser

With reference to **Figure XIII.24** and **Figure XIII.25** of the "Effluent Diffuser in Rio Cobre River" the proposed diffuser consists of a buried 42 in. I.D. pipeline in the deepest portion of the River with a series of 8-16" diameter risers at 5 foot spacing. The Rio Cobre River has been dredged in this area and river data will need to be obtained for detailed design. It is the intention that the risers discharge effluent below the normal river water level, in the main channel of the River. The sketch is based upon assumptions that a suitable location can be found and that river depths are such that a buried diffuser can be used. The riser pipes will be sized so that the effluent flow can be discharged with a velocity that will provide a good degree of initial diffusion of effluent in the River, before the flow enters Hunts Bay.

Stage 2 will have a similar overflow structure and pipeline, with a separate diffuser structure designed for dispersion of Stage 2 flows in another part of the River.

XIII.8 SUMMARY OF CAPITAL COSTS SOAPBERRY LAGOONS (US DOLLARS)

		Trial #1 (cell bottom 1.5 ft above MSL)	Trial #2 (cell bottom 3.0 ft above MSL)	Trial #3 (cell bottom 2.0 ft above MSL)
STAGE 1				
1.	Clearing	157,000	157,000	157,000
2.	Stripping	554,000	349,000	554,000
3.	Excavation & Compaction	2,534,000	2,221,000	2,941,000
4.	Excavation & Stockpiling	1,210,000	*817,000	574,000
5.	Excavation & Placing from Stockpiles to Dikes	690,000	587,000	516,000
6.	Geotextile	973,000	867,000	973,000
7.	Marl Dike Top	52,000	57,000	52,000

STAGE 1 (continued)

8.	Slope Protection	1,973,000	2,025,000	1,973,000
9.	Rip-rap	19,000	21,000	19,000
10.	Fence	154,000	154,000	154,000
11.	Access Road	13,000	13,000	13,000
12.	Headworks, Earthwork	49,000	49,000	49,000
13.	Outlet; Earthwork	6,000	6,000	6,000
		Trial #1	**Trial #2**	**Trial #3**
14.	Pipelines to Distribution Structure			
	- Portmore	205,000	205,000	205,000
	- Nanse Pen	1,218,000	1,218,000	1,218,000
	- Greenwich	1,593,300	1,593,300	1,593,300
15.	Distribution Structure	195,000	195,000	195,000
16.	Pipeline to Splitter Boxes	300,000	300,000	300,000
17.	Splitter Boxes	69,000	69,000	69,000
18.	Inlet Pipes to Pond 1	990,000	990,000	990,000
19.	Septage Piping	23,000	23,000	23,000
20.	Overflow Pond 1 to 2	184,000	184,000	184,000
21.	Overflow Pond 2 to 3	175,000	175,000	175,000
22.	Overflow Pond 3 to 4	175,000	175,000	175,000
23.	Lagoon Effluent Overflow	366,000	366,000	366,000
24.	Effluent Pipeline and River Diffuser	792,000	792,000	792,000
	TOTAL	**14,667,300**	**13,618,300**	**14,266,300**

* Item 4 in Trial #2 is for imported fill material.

	Trial #1	**Trial #2**	**Trial #3**

STAGE 2

1.	Clearing	-	-	-
2.	Stripping	205,000	205,000	205,000
3.	Excavation & Compaction	1,483,000	216,000	908,000
4.	Excavation from Stockpile & Placing	1,477,000	345,000	319,000
5.	Imported Fill Material	212,000	2,556,000	1,426,000
6.	Geotextile	460,000	710,000	460,000
7.	Marl Dike Top	34,000	34,000	34,000
8.	Slope Protection	1,828,000	1,895,000	1,828,000
9.	Rip-rap	18,000	19,000	18,000
10.	Fence	74,000	74,000	74,000
11.	Access Road	13,000	13,000	13,000
12.	Headworks, Earthwork	49,000	49,000	49,000
13.	Outlet; Earthwork	6,000	6,000	6,000
14.	Pipelines (Greenwich)	480,000	480,000	480,000
15.	Distribution Structure	190,000	190,000	190,000
16.	Pipeline to Splitter Boxes	260,000	260,000	260,000
17.	Splitter Boxes	65,000	65,000	65,000
18.	Inlet Pipes - Pond 1	930,000	930,000	930,000
19.	Overflow Pond 1 to 2	170,000	170,000	170,000
20.	Overflow Pond 2 to 3	170,000	170,000	170,000
21.	Overflow Pond 3 to 4	170,000	170,000	170,000
22.	Lagoon Effluent Overflow	350,000	350,000	350,000
23	Effluent Pipeline and River Diffuser	1,174,000	1,174,000	1,174,000
24	Pre-load Allowance	-	735,000	-
	TOTAL	**9,818,000**	**10,816,000**	**9,298,000**

Total Stage 1	**Trial #1**	**Trial #2**	**Trial #3**
Total Items 1) to 24)	14,667,300	13,618,300	14,266,300
Contingency Allowance (20%)	2,933,500	2,723,700	2,853,300
Total Estimate	17,600,300	16,342,000	17,119,600
Engineering Allowance (15%)	2,640,100	2,451,300	2,568,000
Total Stage 1	**20,240,900**	**18,793,300**	**19,687,600**

Note: Trial 1 and Trial 2 include some pre-loading for Stage 2 dikes.

Total Stage 2	Trial #1	Trial #2	Trial #3
Total Items 1) to 24)	9,818,000	10,816,000	9,298,000
Contingency Allowance (20%)	1,964,000	2,163,000	1,860,000
Total Estimate	11,782,000	12,979,000	11,158,000
Engineering Allowance (15%)	1,767,000	1,947,000	1,674,000
Total Stage 2	**13,549,000**	**14,926,000**	**12,832,000**
TOTAL STAGES 1 & 2	**33,789,900**	**33,719,300**	**32,519,600**

XIII.9 REVIEW AND CONCLUSIONS

The total estimated costs of each of the 3 trials are of a similar magnitude. Trial #1 has the advantage of nearly balanced earthwork, with excess excavation from Stage 1 being available to pre-load dikes in Stage 2. This alternative, however, may not be practical due to the depth of excavation proposed, which can only be finally established through further site exploration and drilling.

Trial #2, while rated as being more constructible in Stage 1, has the disadvantage of making Stage 2 more difficult to construct. There is no excess excavated material available to pre-load Stage 2 dikes, and Stage 2 construction is dependent upon imported fill materials (670,000 cubic yards).

The third trial, with bottom elevation at 2.0 feet above MSL, is a compromise. It is slightly more practical to build than Trial 1, and eases the construction of Stage 2. There is some material available for pre-loading the dikes of Stage 2, but there is still a need for importing 400,000 cubic yards of material for Stage 2. The estimates of capital cost of Soapberry lagoon carried forward in this Phase II report are based on the Trial #3 results. The lagoon structures have been shown on figures, based upon the elevations used in Trial #3. If geotechnical studies indicate that the lagoons can be constructed with lower bottom elevations, detailed design can be prepared on the adjusted basis determined.

XIII.10 OPERATION AND MAINTENANCE

In normal lagoon operation, fulltime operation is not usually necessary, however, because of the size of the proposed facility, the area involved, and the very large flows to be handled, it has been estimated that normal daytime operation will require 2 persons, with one person at the site continuously the remainder of the time. In addition, because of the large size of the Soapberry Lagoon Site, although it is to be fenced all around, security will be a concern. It has therefore been assumed that there will be a full time security person and vehicle assigned to the Soapberry site.

The operation will require daily inspection of all structures and slope protection. Because of the large site area. This will involve assignment of a vehicle for the operator(s) for 4 to 5 hours daily.

Normal operation procedure will involve the following inspections on a regular basis:

1. Inspection at hydraulic structures to observe if flows are normal and to note problems at:
   - Distribution Structures
   - Splitter Boxes
   - Lagoon overflow structures between Ponds 1, 2.
   - Lagoon overflow structures between Ponds 2, 3.
   - Lagoon overflow structures between Ponds 3, 4.
   - Effluent overflow structures.
2. Inspections of dikes and protection including:
   - Marl dike tops.
   - Concrete slope protection.
   - Rip-rap.
3. Inspection of pipes where possible.

Emergency repairs may be necessary from time to time. These could involve repairs to pipes and structures, repairs to slope protection, roads, rip-rap, and fences. There will need to be power to operate motorized sluice gates, and to provide a minimum of lighting.

After the lagoons have been in operation for some time, dependent on grit loads, it will be necessary to clean the primary cells of accumulated sludge and grit.

Sludge may be pumped from the bottom of lagoon primary cells by using a floating lagoon pumping system such as a commercially manufactured lagoon pumping package which has a chopper pump mounted on a floating platform which has cable guides and floating discharge pipes. Prices were obtained for an electric motor driven unit with a portable engine driven generator. A very rough estimate shows that such a 20 hp unit used for ½ month per year on a regular basis could provide suitable sludge removal. Practical disposal of sludge is expected to be possible by spreading on the unused portion of the Soapberry site.

Estimate of Operating & Maintenance Costs (US $): Stage 1 (in 1993 $)

	Item	1993 $
1.	Security and Operation Personnel	
	- Security Person with vehicle	19,000/year
	- Operation personnel plus vehicle	22,000/year
2.	Power	2,000/year
3.	Pipeline Maintenance	9,000/year
4.	Road Maintenance	1,000/year
5.	Structure Repairs/Maintenance	10,000/year
6.	Dike and Slope Protection Allowance	18,000/year
7.	Vegetation Control	2,000/year
8.	Sludge Pumping and distributon cost for ½ month per year, Personnel, Fuel and Truck	5,000/year
	Annual O&M Cost	**88,000/year**

Purchase of Sludge Pumping Unit and Generator (capital item) 90,000

Estimate of O&M Costs (US $): Stage 2		**1993 $**
1.	Security and Operation Personnel	
	- Security Person with Vehicle	19,000/year
	- Operation Personnel plus Vehicle	28,000/year
2.	Power	4,000/year

3.	Pipeline Maintenance	10,800/year
4.	Road Maintenance	2,000/year
5.	Structure Repairs/Maintenance	20,000/year
6.	Dike and Slope Protection Allowance	30,000/year
7.	Vegetation Control	3,000/year
8.	Sludge Pumping Cost for 1 month per year incl. Personnel, Fuel and Truck	10,000/year
	Annual O&M Cost	**126,800**