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Madagascar: Ambodiroka hydroelectric project

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EXECUTIVE SUMMARY

The Ambodiroka hydroelectric project, located on the Betsiboka River near the town of Maevatanana, has been extensively studied during the past eight years. During March 1959, Electricity de France (DAFECO) completed a preliminary study in which it concluded that the most economic project would be a 40 MW powerplant built along the left bank of the river near the bridge to Mahajanga. Based on this preliminary study the national power and water company of Madagascar (Jiro Sy Rano Malagasy, JIRAMA) funded a more extensive feasibility study which the French consulting firm of Coyne et Bellier completed during 1981.

The Coyne et Bellier report included an economic study that considered three possible demand scenarios, all of which assumed rapid load growth in the Mahajanga area. The study concluded that, under the most likely scenario, the optimal plant capacity would be between 30 to 40 MW. Under the weakest demand scenario, the optimum would be about 20 MW. The study also assumed a high, and gradually increasing price for diesel fuel which brought Coyne et Bellier to the final conclusion that a 40 MW project should be developed. With respect to both demand and fuel prices, recent developments have proved these economic assumptions to be incorrect, giving rise to the present study.

The objectives of this study are to update the demand forecast in the Mahajanga service area, and to do a preliminary design on a scaled-down hydroelectric project at the Ambodiroka site which would allow staged investment to follow more closely the demand growth pattern. The scope of the study was further refined in the field to assess several design alternatives in the range of 12 - 15 MW on both sides of the Betsiboka River.

The study was prepared for JIRAMA under the sponsorship of the Office of Energy of the U.S. Agency for International Development (AID).

REVISED DEMAND ASSESSMENT

Coyne et Bellier's design was based on an original demand estimate that assumed a large expansion in cement production in the Mahajanga area. When it became clear that this would not materialize, in 1984 Coyne et Bellier revised its estimate downwards. The team saw little evidence to support even this estimate. Coyne et Bellier's revised estimate forecasted 67.4 GWh of consumption during 1985, rising to 108.3 GWh in 1995 and 242.1 GWh in 2010. Actual consumption in 1985 was only 55.9 GWh. The discrepancy can be partly explained by severe problems encountered by JIRAMA in maintaining the system at full capacity during 1985, but it is extremely unlikely that demand will meet the 1984 forecast.

Using fairly conservative assumptions provided by the World Bank, the team assumes demand will grow to 72.5 GWh in 1995 and to 103.84 GWh by 2010. Peak demand in 1994, the projected commissioning date for the project, would be approximately 13.7 MW including losses, increasing to 26 MW by the year 2014.

DESIGN OPTIONS FOR THE AMBODIROKA HYDROPOWER PROJECT

Coyne et Bellier in their "Etude de Factibilité" have been guided by four basic design conditions:

1. The most economic power for the project, based on the projected electrical demand in Mahajanga, is 40 MW.

2. The site should be developed to maximize the design head of the power project.

3. The project must be designed to operate during a 1,000 year flood when the flow, Q, equals 33,000 m^3/s.

4. The project must be designed to minimize the transport of solids through the turbine.

In light of the lower demand forecast for Mahajanga, the team revised these design conditions to be:

1. The powerplant should be capable of delivering 13.5 MW to the busbar in Mahajanga, or sized to about 14 MW, assuming the plant would be operational in 1994 and allowing for transmission losses of 4%.

2. The potential to increase the power to 25-30 MW at the site should be preserved.

3. With respect to flood protection, the most economic level should be chosen.

4. The design should incorporate facilities to minimize the transport of sediment to the turbine.

Given the revised design conditions, the team considered three design alternatives. Each design alternative was studied from the standpoint of cost, difficulty of construction and plant operations.

Figs. 1-3 show design Alternative I (see Annex A). The design has a minimum design head of 54 m and the gates, power canal and intake are sized for flows up to 60 m^3/s. Thus, the initial design power of 14.4 MW can be easily expanded to 28.8 MW by adding another penstock and appropriate power equipment.

The design includes embankments capable of controlling a 1,000 year flood (Q = 33,000). However, it is more beneficial to lower the cost by building the embankments to control only a 10 year flood (Q = 7,500 m^3/s). During floods that exceed Q = 7,500 m^3/s the powerplant would not be operable for a few hours once every 10 years. This configuration is referred to as Alternative I, Option 2.

Fig. 4 shows the plan view of Alternative II, which is the mirror image of Alternative I. However, it is not possible to lower the cost of Alternative II by lowering the initial embankments, because a natural channel around the

island on the left bank would have to be closed at great expense in order to protect the powerhouse during a very high flood. Thus, for Alternative II there exists only one viable option.

Fig. 5 shows the plan view of Alternative III. The diversion structure envisioned by Coyne et Bellier would be constructed up to the road bridge. An embankment upstream of the bridge controls floods up to Q = 33,000 m^3/s. The sediment basin, intake and powerhouse structures are like those proposed by DAFECO (March, 1979), but modified to reflect the control provided by the embankment upstream of the bridge. Alternative III has a minimum design power of 14.4 MW and can be expanded to 28.8 MW by adding another penstock and appropriate power equipment.

It should also be noted that the cross channel weirs, needed in all alternatives, use a minimum quantity of concrete. Several millions of dollars can be saved by this type of dike design, but over the life of the powerplant, these dikes will require some maintenance.

A cost estimate was made for each alternative using unit prices that are comparable to unit prices used by Coyne et Bellier in 1980, assuming an annual level of inflation of about 12%. Equipment prices were provided by U.S. and European manufacturers and the local firm CIMELTA-JEUMONT.

A summary of the cost estimates are shown in Table 1. Three points should be noted. First, the costs shown do not reflect any economic optimization of the design. Economic optimization will most likely lower the overall project cost. However, the cost would increase $1-3 million, for example, if the Coyne et Bellier cross channel dike design is used. Second, there is little cost difference among the alternatives. Third, over 50% of the project cost is in local currency. To obtain this high percentage of local currency, local contractors and fabricators should be used to the maximum possible extent.

Table 1. Alternative Project Cost Estimates for the Ambodiroka Project

Alternative	Total cost[a] (m FMG)	Foreign exchange (US$'000)	Local currency (m FMG)
I 1	17,700	12,730	9,890
I 2	16,700	12,670	8,880
II	17,900	12,760	10,040
III	16,840	12,750	9,000

[a]US$1.00 = FMG 615.

The design options were also compared in terms of difficulty of construction and operational aspects. With respect to the former, no major difference exists between construction on the right and left sides of the river.

Building the left side diversion structure up to the bridge would appear somewhat less complicated than building the right side facilities upstream of the bridge. On the other hand, constructing the powerhouse on the left side may be somewhat more complicated than on the right side.

In terms of operational characteristics, each alternative is designed to operate from the minimum flow, Q = 30 m^3/s, up to Q = 33,000 m^3/s, except Alternative I2. For floods over Q = 7,500 m^3/s Alternative I2 would be shutdown for a minimum of 2-6 hours, because no sediment control would exist, during which time diesels would generate power. It is likely that the powerplant would remain closed several more hours while the basin and power canal are flushed clean. However, the benefits lost during these floods would be quite small because they are expected only once every 10 years.

Under normal operations sediment control in Alternative I is achieved by passing the flow slowly through the sediment basin to the power canal. Flow into the power canal is controlled by a control gate at its entrance. A flush gate in the sediment basin is used to clean it.

In Alternative II, Option 2, the possibility exists that flood waters will exit into the channel around the island, which could damage the powerhouse if located near this channel. To avoid this possibility, an additional embankment must be built below the bridge at a cost that would make this design uneconomic. <u>It is therefore concluded that Alternative II, Option 2 is not viable</u>.

Much of the same system used in Alternative II is used for Alternative III except the sediment basin is downsteam of the bridge. Flow into the sediment basin is controlled by gates in the embankment at the end of the diversion canal. Sediment control is somewhat easier in this arrangement, because flow into the sediment basin is totally controlled by the entrance gates.

<u>Thus, the operation of Alternative III is somewhat better than Alternative I or Alternative II, because it can be continuously operated during major floods and because sediment control is somewhat better</u>.

ECONOMIC ANALYSIS

The economic analysis makes a preliminary least-cost calculation to determine the optimal choice of three power supply options, given the revised demand forecast:

- to continue to meet demand with diesel generation, which would require investment in a total of 30 MW of diesel generators during the 20-year period of the study;

- to develop the Ambodiroka site to 26.6 MW according the the design proposed in the Coyne et Bellier study (without adding the 13 MW second stage); or

- to construct a simpler hydroelectric scheme at Ambodiroka in two 14 MW stages, which would require a small amount of diesel generation for peaking during the first stage of the project.

No distinction is made between the various design options which are under consideration for the reduced-scale scheme for purposes of this analysis, since these options are all comparable in cost. The results of the analysis show that the scaled-down hydropower scheme is less than half the cost of the original hydroelectric design, in present worth terms. The analysis further shows that, with a lower cost of fuel and the slower rate of demand growth than were assumed by Coyne et Bellier, it would be more cost-effective to generate with diesel than to construct the Ambodiroka hydropower project as originally designed.

The economic analysis will also help in selecting an optimal design. The preliminary analysis indicates that, in the cases of Alternatives I and II, that the design should not attempt to permit continuous operation of the powerplant under all flood conditions, and that either Alternative I, Option 2 or Alternative III appear to provide the greatest net economic benefit.

FINANCIAL ANALYSIS

The primary objective of the financial analysis is to determine the tariff level needed to cover operating expenses, including loan amortization. JIRAMA's existing average tariff in Mahajanga is FMG 47.3, or about $0.08/kWh. Assuming a loan amount of $32.2 million at 12% interest over 25 years, the income statement indicates that an average tariff of $0.08 would cover 97% of first-year operating expenses, with surplus income carried in each succeeding year. However, this tariff level does not currently meet JIRAMA's operating costs. An increase of 17.8% in the average tariff would be needed to meet operating costs, based on 1985 income and cost data.

RECOMMENDATIONS

The team recommends that a full feasibility study be conducted on the reduced-scale Ambodiroka hydroelectric scheme. The results of the economic and financial analysis indicate that the project would be extremely attractive. Moreover, since significant work has already been invested over the years to study the site, the cost of a feasibility study would be relatively low, approximately $300,000.

In recommending a final design choice, the feasibility study should consider primarily Alternatives I, Option 2 and Alternative III.

The team also recommends increasing the tariff rate to close the present operating deficit. Since the majority of electricity use is by the large industrial users and since they are presently subsidized by residential users, it would seem appropriate to apply future tariff increases to industrial rates.

I. BACKGROUND

INTRODUCTION

The World Bank and the Government of Madagascar are completing preparations for the Energy I project, whose principal component will be an IDA credit of up to approximately $25 million. Several elements of the project under consideration have been developed in collaboration with the U.S. Agency for International Development (AID), one of which addresses small-hydropower plants. In 1985, AID conducted a preliminary assessment that found significant potential for small-hydropower development in Madagascar. The World Bank subsequently determined that this subsector should be given priority consideration in Madagascar's power program. The Bank sought and received AID assistance in fielding a second assessment team as part of the Energy I Project Preparation Mission, whose terms of reference consisted of making recommendations on three project components under consideration:

- a feasibility study of a reduced-scale version of a 40 MW hydroelectric project at Ambodiroka that could be developed incrementally to meet expected demand growth in the Mahajanga area;
- a training program to strengthen JIRAMA's investment programming and project implementation capability with respect to small-hydropower schemes; and
- the construction of a 1 MW demonstration project near Tolagnaro that would be integrated with the training program to give JIRAMA staff hands-on experience in the assessment, design, and construction management of small-hydropower plants.

This report addresses the first of these components. The study was conducted by Granville Smith of STS Energenics, Paul Clark of the National Rural Electric Cooperative Association (NRECA), and Eugene Palmer of STS d'Appolonia in February 1986 under funds made available through a cooperative agreement between AID and NRECA.

GENERAL SITUATION

One of JIRAMA's largest isolated power systems is located in the northern city of Mahajanga, located at the mouth of the Betsiboka River. Mahajanga has a population of approximately 120,000. In the past it has been an important port, but heavy siltation from the river has restricted its use in recent years. There are several industries, including two textile plants and two shrimp fisheries. Electric power is the primary source of energy for these industries. For some time, JIRAMA has been interested in reducing the total dependency on diesel generation for the system, owing to its high fuel costs and, more recently, difficulties in maintaining the system.

The Ambodiroka hydroelectric project, located on the Betsiboka River near the town of Maevatanana, has been extensively studied during the past eight years as the prime candidate for achieving this goal. During March 1959 DAFECO

(Electricity de France) completed a preliminary study in which it concluded that the most economic project would be a 40 MW powerplant built along the left bank of the river near the bridge to Mahajanga. Based on this preliminary study JIRAMA funded a more extensive feasibility study which the French consulting firm of Coyne et Bellier completed during 1981.

The Coyne et Bellier report included an economic study that considered three possible demand scenarios, all of which assumed rapid load growth in the Mahajanga area. The study concluded that, under the most likely scenario, the optimal plant capacity would be between 30 to 40 MW. Under the weakest demand scenario, the optimum would be about 20 MW. The study also assumed a high, and gradually increasing price for diesel fuel which brought Coyne et Bellier to the final conclusion that a 40 MW project should be developed.

With respect to both demand and fuel prices, recent developments have proved these economic assumptions to be incorrect, giving rise to the present study.

JIRAMA'S SYSTEM

JIRAMA provides electricity to a total subscription of 6,361 consumers including 49 consumers receiving the higher voltage service and 6,312 low-voltage consumers. The plant has seven MAN medium-speed diesel generator sets, three of which use bunker oil and four others which burn diesel oil. The distribution system consists of 63.8 km of 5 kV and 20 kV line and 110.7 km of 220-380 V secondary lines. The distribution network is entirely 3-phase construction, using mostly concrete poles which are manufactured in-country, according to the Regional Director.

JIRAMA's service reliability deteriorated in recent years primarily because of the lack of foreign currency to purchase spare parts. JIRAMA's records show that between 1977 and 1984, JIRAMA spent 38% of budgeted amounts on scheduled maintenance. This resulted in FMG 32 million in costs to repair damaged equipment.

The inadequate maintenance has contributed to a serious problem of service interruption and voltage drops, particularly during 1984 and early 1985. Every industrial user interviewed said that JIRAMA's service had caused problems, including forcing the suspension of activity and also damaging electrical equipment. The situation had improved considerably in the latter half of 1985, although the third largest unit was out of service for unscheduled repairs during the team's visit. This outage was curtailing the number of working hours at the largest industry in the city.

JIRAMA'S FINANCIAL POSITION

As shown in Table 2, JIRAMA's electricity operation in Mahajanga does not presently cover costs. In 1985, its net operating loss was FMG 7.15/kWh sold. Including depreciation of equipment and net revenues from related activities, JIRAMA's net loss was FMG 11.18/kWh sold. This was an improvement over 1984, when JIRAMA's net loss was FMG 14.25/kWh sold and its overall loss was FMG 15.24/kWh sold.

The improvement was a result of a 10% increase in the average tariff charged. The average tariff charged to domestic users in 1985 increased by 21% over

1984, from FMG 82.45 to FMG 99.73, while the average industrial tariff rose by only 7%, from FMG 36.20 to FMG 38.77.

JIRAMA's cost of operating in 1985 was FMG 54.44/kWh sold. To break even on its operations, it would have to raise its average revenue by 17.8%.

Table 2. Financial Statement for JIRAMA's Mahajanga Powerplant (FMG)

Revenues and expenses	1985	1984
Electricity sales (kWh)	55,911,554	49,484,003
Revenue		
Electricity sales	2,583,893,140	2,081,363,111
Rents (meters)	58,777,590	48,432,760
Subtotal	2,642,670,730	2,129,795,871
Production expenses		
Diesel fuel and fuel oil	2,265,747,217	2,021,527,171
Salaries	45,458,218	45,768,034
Other	414,080,752	396,836,128
Subtotal	2,725,286,187	2,464,131,333
Other operating expenses		
Salaries	64,931,725	74,647,521
Other	43,860,135	35,269,871
Subtotal	108,791,860	109,917,392
Home-office administration	208,088,850	260,701,000
Total expenses	3,042,166,897	2,834,749,725
Net revenue from electricity	(399,496,167)	(704,953,854)
Net revenue from related services	21,039,695	10,871,915
Cost of goods from inventory	39,575,890	227,786,950
Amortization/depreciation	(284,683,035)	(283,325,490)
Net financial expenses	(1,384,746)	(4,484,179)
Net profit	(624,949,263)	(754,104,658)
Average tariff/kWh sold	46.21	42.06
Average revenue/kWh sold	47.27	43.04
Average cost/kWh sold	54.44	57.29
Net operating revenue/kWh sold	(7.15)	(14.25)
Net total profit/kWh sold	(11.18)	(15.24)

II. DEMAND ASSESSMENT

Coyne et Bellier's design was based on an original demand estimate that assumed a large expansion in cement production in the Mahajanga area. When it became clear in 1984 that this would not materialize, Coyne et Bellier revised its estimate downwards. The team saw little evidence to support even this estimate. Coyne et Bellier's revised estimate forecasted 67.4 GWh of consumption during 1985, rising to 108.3 GWh in 1995 and 242.1 GWh in 2010. Actual consumption in 1985 was only 55.9 GWh. The discrepancy can be partly explained by severe problems encountered by JIRAMA in maintaining the system at full capacity during 1985, but it is extremely unlikely that demand will meet the 1984 forecast.

RESIDENTIAL USE

Electricity consumption by residences accounts for about 10% of JIRAMA's sales. There were 6,237 residential users in 1985. Roughly one-third of all households in Mahajanga receive electricity. Average monthly consumption in 1985 was 75 kWh. JIRAMA's Regional Director said that perhaps 5% of residential consumers use electricity for cooking. Most of the use is for lighting. Both the number of residential users and residential consumption dropped between 1983 and 1984, but 1985 saw a recovery in both categories.

Growth potential in residential electricity consumption is restricted by JIRAMA's present policy of requiring new consumers to pay the full cost of connection up front. In Mahajanga, this averages FMG 66,000 per connection, which does not include housewiring. According to the Regional Director, this is equivalent to 25% of a typical household head's annual income. Moreover, the average cost of electricity is quite high, averaging just over FMG 100/kWh for a consumer using 50 kWh/month. The substitution of lower-cost hydropower for the present diesel generation could have a positive effect on residential demand.

PUBLIC LIGHTING AND GOVERNMENT USE

Public lighting and government use accounts for less than 2% of electricity consumption. Both declined in 1984, particularly public lighting. JIRAMA has had trouble maintaining the public lighting system in recent years, and the cyclone which ravaged the city destroyed many of the lamppost fixtures, which JIRAMA has been slow to replace. In 1985 there were 14 major government consumers, including a hospital, the airport, and the provincial and municipal office headquarters. Government use of electricity is distinguished only by the fact that government consumers owe JIRAMA FMG 158 million, an amount equal to the total value of consumption. Growth in this sector will be dependent on JIRAMA's ability to replace public light fixtures, but even this would not significantly affect overall demand.

INDUSTRIAL USE

Industrial and commercial consumers account for over 80% of JIRAMA's energy sales. In 1985, there were 29 users receiving 5 kV or 20 kV service and 239 smaller users (force motrice basse tension, FMBT) receiving 220-380 V service.

SOTEMA

The Société Textile de Mahajanga (SOTEMA) is by far the largest user of electricity in the area, accounting for about 5.8 MW of JIRAMA's average 6-day load of 8.5 MW. SOTEMA accounted for 70% of JIRAMA's energy sales in 1985, and 65% of its revenues from electricity. SOTEMA produces over 100 textile products, 90% going to the domestic market. It is interested in expanding exports, mainly to increase access to foreign currency. SOTEMA is by far the largest industrial employer, with about 4,000 employees.

SOTEMA is planning a 14.5 billion FMG investment to overhaul the entire plant. Foreign currency problems in recent years have postponed needed equipment repairs and replacements, with the consequence that the textile products are of increasingly poor quality. This primarily affects the export business. Overall efficiency at the plant has also deteriorated, according to SOTEMA management. A costly fire in November 1984 caused by badly deteriorated electrical equipment underscored the problem.

Electricity accounts for most of SOTEMA's energy needs, and in 1985 represented nearly 25% of operating costs. SOTEMA also used 9.2 million liters of diesel and fuel oil in 1985 to provide process heat and steam.

Electricity consumption fell steadily from 44.5 GWh in 1981 to a low in 1984 of 34.8 GWh, reflecting the general deterioration of the economy and the universal problem of shortages of foreign currency needed to buy raw materials and spare parts. However, most of the 2.2 GWh decline from 1983 to 1984 is attributable to several unusual circumstances, including the fire, which affected production for three weeks, a cyclone, which stopped production for over a week, and frequent power blackouts and brownouts, which SOTEMA estimates resulted in the loss of 115,000 machine-hours.

SOTEMA's electricity consumption rebounded strongly in the second half of 1985, after the management called a meeting with JIRAMA in March to discuss problems with electricity service. There was a marked improvement soon thereafter, according to SOTEMA. Consumption for all of 1985 was 39.57 GWh, and SOTEMA estimates that it will increase the plant's demand to nearly 7 MW and its energy consumption to as much as 47.87 GWh.

SOMAPECHE and SOPEBO

Mahajanga's fishing industry, consisting of two fisheries (SOMAPECHE and SOPEBO), is the second largest user of electricity. The two fisheries are located in the same compound and share common Japanese ownership and facilities. The primary catch is shrimp, virtually all of which is exported to Japan. It is the third largest earner of foreign currency, behind vanilla and coffee. The fisheries employ approximately 1,000 people.

Electricity is used primarily for cold storage and ice-making for the boats. The fisheries combined have a fleet of 26 fishing boats, several cold storage rooms, and an ice plant which produces 7,000 tons of ice per day during the fishing season. The fisheries are busiest during the first three months of the shrimping season, which begins in mid-February and ends in mid-December.

Electricity use is dependent on the catch, which can vary from year to year. The 1984 cyclone did considerable damage to the facilities and greatly reduced the yield. Electricity consumption was cut by two-thirds between 1983 and 1984. SOPEBO is adding two new boats to its fleet of 11, and is building a new cold-storage facility. This could add between 0.2 and 0.3 GWh to consumption. Pierre Verdet, Director of SOMAPECHE, estimates that the two fisheries will consume between 2.5 and 3.5 GWh in an average year.

Long-term growth prospects are limited, according to M. Verdet. He cited a restrictive government policy toward privately-owned fisheries in favor of a state-owned fishery in Nosy Be, and fears that the growing number of foreign fleets in the area could eventually deplete the supply of shrimp.

FITIM

The Filature et Tissage de Mahajanga (FITIM) is a state-owned producer of jute textiles and is the third largest industrial user of electricity in the city. Its primary products are agro-industrial containers (e.g. rice bags) and linen. FITIM serves only a domestic market. Most of the jute is now imported, which accounted for a sharp decline in production in recent years. In 1976, FITIM produced 4,535 tons of jute products. This dropped to 1,664 tons in 1982 and 588 tons in 1985. FITIM has had to rely on local supplies of jute since 1982.

Inaccessibility to foreign jute supplies and spare parts for the plant's machinery cut drastically into FITIM's electricity use. Its consumption of 0.35 GWh in 1985 was less one-sixth of the 1980 consumption level. A rehabilitation is planned in 1986 which will be financed by a Swiss-based firm (MARCOTADE). MARCOTADE will take over 49% ownership of the business. A general overhaul of the plant is planned, which will nearly double its capacity to 8,000 tons per year. The investment plan will also include a local jute cultivation project, partly by establishing a jute régie which FITIM will operate, and by contracting with local farmers for the supply of jute. Employment at the plant will increase from the present 300 to between 900 and 950. The plant manager expects electricity consumption could double by the time the plant is operating at full capacity. The rehabilitation is scheduled to be completed by 1987, but the potential for full capacity might not be assured until 1990, by which time FITIM expects its jute supply to be entirely local.

HASYMA

The largest of seven cotton processing facilities operated by the national cotton company (Hasy Malagasy, HASYMA) is located in Mahajanga. It has the capacity to produce up to 26,000 tons per year of cotton fibre and cotton seed. In normal production years, it produces enough cotton fibre to satisfy SOTEMA's requirements, plus supply other textile plants in the country.

Cotton seed is sold to two local seed oil processing plants and to local planters. Production targets are set by the government, which fully owns the company. The plant's production, however, has fallen in recent years. Production of fibre and seed dropped from 20,000 tons in 1981, when the plant opened, to 8,833 tons in 1984. In that year, the plant operated at just 33% according to the plant director. However, contradictory information was received from SOTEMA, who said it has had to import cotton in recent years due to local crop failures. Nevertheless, HASYMA had a surplus in 1985 and expects another surplus in 1986, which it is seeking to export.

Electricity is used for separating the fibre from the seed, and for baling. Electricity use, which is concentrated largely between the harvest season of August through December, fell from 0.8 GWh in 1981 to 0.37 GWh in 1984; peak demand was 448 kW and 300 kW respectively in those years.

The outlook for HASYMA is dependent on local demand for cotton fibre, thus it tends to follow SOTEMA's business pattern. By illustration, its electricity consumption increased by 16% to 0.43 GWh in 1985, compared to a 14% increase for SOTEMA. It could also depend on the company's success in gaining an export market for cotton fibre. No expansion plans are under consideration, although a second plant may be constructed between Mahajanga and Antananarivo at some unspecified date.

SIB

The Société Industrielle du Boina (SIB) is a privately-owned processing plant which produces primarily seed oil and soap for the domestic market. SIB also has a rice mill which also sells animal feed from pulverized hulls, and produces other items for foreign buyers, including beeswax and pillow stuffing material made from locally grown "kapok" pods.

SIB's electricity use is concentrated between August and April, the post-harvest season for both cotton and rice. Consumption between 1975 and 1980 averaged 1.25 GWh, but dropped to an average of 0.70 GWh between 1981-84. According to the owner, consumption dropped sharply in 1983-84 due to the general economic decline and due to problems with JIRAMA's electricity service. Consumption in 1985 was 1.09 GWh. SIB's total load is now about 160 kW. SIB is planning a rehabilitation project in the near future which the owner says could double the capacity of the plant, but he could not give details on the rehabilitation, nor its expected effect on power requirements. However, he does expect both his demand and energy consumption to increase.

OTHER EXISTING AND POTENTIAL DEMAND

The only other existing large user of electricity in Mahajanga is JIRAMA itself, primarily to operate five water-pumping stations in JIRAMA's water supply operations. This consumption was 2.76 GWh in 1985, up from 2.55 GWh in 1984, and could be expected to grow proportionally to the population growth rate.

Potential for new consumption is fairly limited. An aging cement plant about 30 km southwest of Mahajanga now generates its own power from a 1.2 MW diesel unit, consuming 6 GWh per year. JIRAMA has no plans to connect this load.

Two other larger cement plants which were envisioned in the Coyne and Bellier study have not materialized. HASYMA may construct a second cotton-processing plant in the region, but the location and timing of this investment is not known. The President of the Faritany also indicated that iron ore mining may be undertaken in the province, but there are no specific plans for this activity.

JIRAMA operates three diesel plants within 20 km of the proposed transmission route, with a total existing load of 310 kW and energy consumption of 0.59 GWh during 1985. There are also said to be numerous small, diesel auto-generators in the area. These loads are at present quite small, but the population along the river is sizable and there is immediate potential for electricity use in rice cultivation and processing. There are an estimated 80,000 people in the vicinity of Marovoay, for example, which is the site of a KFW-financed rice rehabilitation project involving 1,000 farmers. The project is using 13 diesel low-lift irrigation pumps for fields directly adjacent to the River, and has plans to install a 500 kW steam-driven generator fueled by rice husks to process the rice and provide electricity to the project's workshop and living quarters. There is some discussion about selling 160 kW of expected surplus to the local JIRAMA plant.

Financing for major rural electrification project would be difficult to raise, given other priorities and the present financial and economic environment, and there are no plans to make such an investment. However, the President of the Faritany indicated that rural electrification in the Betsiboka basin is one of his priorities for the province.

REVISED DEMAND FORECAST

Table 3 provides the revised demand forecast. The specific growth assumptions are those determined by JIRAMA and the World Bank from a demand assessment conducted in December 1985, with two modifications:

- SOTEMA's consumption after 1990 is increased by 5 GWh from 40 GWh to 45 GWh; and

- FITIM's consumption after 1990 is increased by 1 GWh, from 2 GWh to 3 GWh.

Consumption levels for all of the categories after 1984 were slightly changed, since the 1985 numbers were updated to reflect actual consumption. The exception was "particuliers," or large industrial users, which showed higher consumption levels in the outyears due to a calculating error in the earlier forecast.

The demand levels are consequently higher than estimated in December, but still well below the levels assumed by Coyne et Bellier in their 1985 report. Instead of 108.3 GWh in 1995, demand is presently estimated to reach 72.5 GWh by that year; in 2010 demand will have reached 103.8 GWh instead of 242.1 GWh, as estimated by Coyne et Bellier.

The team's assessment indicates that the recent decline in electricity consumption has probably been, to a considerable degree, because of to JIRAMA's difficulties in supplying electricity, notably to SOTEMA. Future

Table 3. Revised Demand Forecast for Mahajanga

Year	UD	EP	FMBT	Part's	ADM	Cess. Int	Sotema	Soma	Fitim	Total Sales	Net Gener.	Peak Load
1981	5.28	0.64	1.37	3.39	0.49	1.90	44.53	1.51	1.65	60.76		10.27
1982	5.62	0.30	1.17	3.40	0.49	2.06	38.45	1.53	1.11	54.13		9.63
1983	5.84	0.36	1.21	3.86	0.54	2.55	37.08	1.58	0.31	53.35		8.94
1984	5.29	0.25	1.08	4.10	0.49	2.55	34.77	0.49	0.46	49.48		9.50
1985	5.74	0.35	1.22	4.19	0.57	2.76	39.60	1.05	0.43	55.91		9.58
1986	5.94	0.36	1.28	4.38	0.57	2.79	40.00	1.50	0.60	57.42	60.69	10.68
1987	6.15	0.36	1.35	4.58	0.58	2.82	40.00	1.50	1.00	58.32	61.67	10.91
1988	6.36	0.37	1.41	4.78	0.58	2.84	40.00	1.50	1.50	59.35	62.77	11.17
1989	6.59	0.38	1.48	5.00	0.58	2.87	40.00	1.50	1.50	59.90	63.38	11.32
1990	6.82	0.39	1.56	5.22	0.58	2.90	45.00	1.50	3.00	66.97	70.78	12.51
1991	7.16	0.41	1.70	5.59	0.60	2.99	45.00	1.50	3.00	67.94	71.84	12.77
1992	7.52	0.43	1.85	5.98	0.62	3.08	45.00	1.50	3.00	68.97	72.98	13.05
1993	7.89	0.45	2.02	6.40	0.64	3.17	45.00	1.50	3.00	70.06	74.18	13.34
1994	8.29	0.47	2.20	6.84	0.66	3.26	45.00	1.50	3.00	71.22	75.46	13.66
1995	8.70	0.49	2.40	7.32	0.68	3.36	45.00	1.50	3.00	72.45	76.82	13.99
1996	9.14	0.52	2.61	7.84	0.70	3.46	45.00	1.50	3.00	73.76	78.26	14.34
1997	9.59	0.54	2.85	8.38	0.72	3.57	45.00	1.50	3.00	75.15	79.79	14.72
1998	10.07	0.57	3.10	8.97	0.74	3.67	45.00	1.50	3.00	76.63	81.42	15.11
1999	10.58	0.60	3.38	9.60	0.76	3.78	45.00	1.50	3.00	78.20	83.15	15.53
2000	11.16	0.63	3.69	10.27	0.79	3.90	45.00	1.50	3.00	79.88	84.99	15.98
2001	11.66	0.66	4.02	10.99	0.81	4.02	45.00	1.50	3.00	81.65	86.95	16.46
2002	12.24	0.69	4.38	11.76	0.83	4.14	45.00	1.50	3.00	83.55	89.04	16.96
2003	12.86	0.73	4.77	12.58	0.86	4.26	45.00	1.50	3.00	85.56	91.26	17.50
2004	13.50	0.77	5.20	13.46	0.88	4.39	45.00	1.50	3.00	87.70	93.62	18.07
2005	14.17	0.80	5.67	14.41	0.91	4.52	45.00	1.50	3.00	89.98	96.13	18.68
2006	14.88	0.84	6.18	15.41	0.94	4.65	45.00	1.50	3.00	92.41	98.81	19.33
2007	15.63	0.89	6.74	16.49	0.97	4.79	45.00	1.50	3.00	95.00	101.66	20.01
2008	16.41	0.93	7.34	17.65	0.99	4.94	45.00	1.50	3.00	97.76	104.70	20.75
2009	17.23	0.98	8.01	18.88	1.02	5.09	45.00	1.50	3.00	100.70	107.94	21.52
2010	18.09	1.03	8.73	20.21	1.06	5.24	45.00	1.50	3.00	103.84	111.40	22.35
2011	18.99	1.08	9.51	21.62	1.09	5.40	45.00	1.50	3.00	107.18	115.09	23.24
2012	19.94	1.13	10.37	23.13	1.12	5.56	45.00	1.50	3.00	110.75	119.02	24.18
2013	20.94	1.19	11.30	24.75	1.15	5.72	45.00	1.50	3.00	114.56	123.21	25.18
2014	21.99	1.25	12.32	26.49	1.19	5.90	45.00	1.50	3.00	118.62	127.69	26.25

Assumptions

Growth in:		after 1990			after 199
UD is	0.035	0.05	Distribution losses (HT)	0.04	0.04
EP is	0.02	0.05	Distribution losses (LT)	0.09	0.09
FMBT is	0.05	0.09			
part's is	0.045	0.07			
admin. is	0.005	0.03			
cess int. is	0.01	0.03	SOTEMA demand (MW)	5.8	6.45
			SOTEMA load factor	0.79	0.79
SOTEMA consumption	40.0	45.0	UD load factor	0.36	0.36
SOMAPECHE consumption	1.5	1.5	all other users load factor	0.5	0.5
FITIM consumption		3.0			

consumption levels, therefore, will depend on power supply constraints as well as demand constraints. Insofar as JIRAMA succeeded in improving the reliability of the system during the latter half of 1985, a recovery in demand has already been seen. This improvement in reliability most likely accounted for much of the 3.5 GWh discrepancy between the December 1985 forecasts and the team's revised forecast. It is furthermore not an unreasonable assumption that the Ambodiroka hydroelectric project itself would have a positive impact on demand, and that a similar conclusion could be drawn regarding other load centers currently served by diesel.

The peak load estimate is based on an assumption that SOTEMA will increase its current demand from 5.8 MW to 6.45 MW and by estimating load factors of other user categories. SOTEMA's technical manager provided information necessary to estimate its load growth. Residential use, primarily for lighting, was estimated to have a load factor of 36%; a 50% load factor was assumed for all other users.

Under these assumptions, the peak load on the network is estimated to reach 13.6 MW by 1994, the estimated commissioning date for the Ambodiroka project, doubling to over 26 MW by the 20th year, 2014. These demand levels include distribution losses, but not transmission losses for the powerplant to the busbar in Mahajanga.

III. AMBODIROKA PROJECT DESIGN OPTIONS

DESIGN CRITERIA

Coyne et Bellier evaluated five powerplant configurations (Variations A1, A2, B1, B2, and II; Chapter 3, Vol. 1, Tome 1; Etude de Factibilité) based on the design by DAFECO. Of the variations with design head 56.3 m Coyne et Bellier concluded that the original design by DAFECO was the best (See comments, page 152, Vol. 1, Tome 1; Etude de Factibilité). However, Coyne et Bellier favored those variations with design head 70.0 and showed that, from the point of view of cost and project risk, their Variation II was the best.

Economic studies of Variation II by Coyne et Bellier demonstrated that a 40 MW plant was most economically beneficial assuming their projected demand. For lower demand projections Coyne et Bellier concluded that lower power designs were most economical.

Based on their preliminary technical findings and the associated economic studies Coyne et Bellier completed the "Etude de Factibilité" of Variation II during March 1981. This variation, which is shown in detail in Coyne et Bellier's Cahier de Plans, Plan 82-103/4, will be referred to as the Coyne et Bellier design.

Coyne et Bellier design criteria

Maximum head

The Coyne et Bellier design makes maximum use of the available head at the site. Design variations with lower head were presented to JIRAMA. JIRAMA favored the higher head project, because the firm power is maximized.

Operation during maximum expected flood

It has long been observed that the water elevations in the river can rise dramatically over very short periods of time. Coyne et Bellier analyzed the hydrology in the river basin extensively (Chapter 2, Vol. 1, Tome 1, Etude de Factibilité) including the construction of a physical model of the project. Based on their hydrology analysis they determined that the project should be designed to operate during a flood of 33,000 m^3/s, which would be expected to occur every 1,000 years. Coyne et Bellier's design incorporates a number of necessary design features which would allow the plant to operate under maximum flood conditions.

Sediment control

Coyne et Bellier determined that the sediment load in the river is normally about 2 kg/m^3. This is an extremely high sediment load. Coyne et Bellier analyzed the sediment transport conditions in the river and included special facilities in their design to minimize the sediment transport through the turbines.

Thus, four basic design criteria underlie the Coyne et Bellier design:

1. The most economic power for the project, based on the projected electrical demand in Mahajanga, is 40 MW.

2. The site should be developed to maximize the design head of the power project.

3. The project must be designed to operate during a 1,000 year flood when the flow, Q, equals 33,000 m^3/s.

4. The project must be designed to minimize the transport of solids through the turbine.

Revised design criteria

Based on the revised demand forecast, the design power for the project has been decreased from 40 MW to 14 MW. However, the design should allow an expansion of the 14 MW powerplant to at least 25 MW at some future date.

JIRAMA believes that the potential must exist for a future expansion of the power site from 14 MW to the maximum 40 MW level. Thus, another design condition is that the powerplant should not eliminate the possibility of building a 40 MW project at some future date. One should note that this condition does not imply that the design head of the smaller powerplant must be maximized.

Coyne et Bellier's third design condition noted above requires expensive facilities be added to their design so that it can operate during the 1,000 year flood. The marginal economic benefit of this condition is not large, since the probability that the design flood will occur during the lifetime of the plant is not large. Also, all floods are of very short duration and, thus, would not affect the powerplant for significant periods of time. On the other hand, the powerplant must be designed to withstand the 1,000 year flood even though it may not be operational. Therefore, the design team has required that the powerplant not be damaged during a 1,000 year flood, but has relaxed Coyne et Bellier's third condition in order to study the economic benefits of a powerplant designed for a 10 year and a 100 year flood.

Finally, Coyne et Bellier's fourth design condition has been incorporated into the basic designs discussed in this report.

To summarize the basic design conditions accepted for the analysis and alternative designs discussed in this project are:

1. The powerplant should be capable of delivering 13.5 MW to the busbar in Mahajanga, or sized to about 14 MW.

2. The potential to increase the power to 25-30 MW at the site should be preserved.

3. With respect to flood protection, the most economic level should be chosen.

4. The design should incorporate facilities to minimize the transport of sediment to the turbines.

SITE CONDITIONS

During its mission in Madagascar in February 1986, the hydropower team was given the opportunity to review the study by DAFECO and the Etude de Factibilité by Coyne et Bellier. It also visited the site during a two day period accompanied by JIRAMA engineer, Benjamina Rakotondrafara, and one of the JIRAMA surveyors, M. Rakotondralambo, who had worked on the original survey of the site.

Although both Coyne et Bellier and DAFECO had proposed to build their projects along the left side of the river, the team also studied the right bank in detail during its site visit. A review of the materials describing the project site suggested four reasons for studying the right bank.

First, the total potential of the run-of-the-river project designed by Coyne et Bellier equalled 40 MW. One method of preserving the ultimate power potential of the site would be to build a small plant along the right bank leaving the left bank for future development.

Second, the topography of the right side is much less complex than the left side. It appeared that during major floods that the right side of river would be less complicated to control. JIRAMA had made the point that fluctuations in the water level during floods are less severe along the left bank. However, the left bank appeared to have a higher coefficient of friction than the right bank and, therefore, the depth of water during a major flood should be higher than the right side where the velocity of flow is higher.

On the other hand, along the left side a natural channel exists at a higher elevation than the main channel and Coyne et Bellier have used this natural channel as part of their design. During most of every year the flow to the intake must be limited, because it far exceeds the design flow for the project. Using the left side naturally limits the flow and at the same time most of the debris and sediment in the river would be carried down the main channel. This point favors development along the left bank.

However, the natural diversion channel along the left bank is complicated by a large depression on the left side of river downstream of the bridge. Coyne et Bellier have designed embankments to control the water in this area. Also, beyond this swale one finds an island around which high waters can flow. Coyne et Bellier have had to design another embankment to maintain flow towards their intake. In the hydropower team's review of these multiple embankment features, it appeared that fewer structures would be needed along the right bank and, therefore, the cost of construction would be lower along the right bank.

Also, the hydro team noted that the left side of the river could be difficult as a construction location. Coyne et Bellier suggested in their Etude de

Factibilité the the island on the left side might not be stable. On the right side of the river, most of the project works would be constructed on dry land.

Third, the right bank construction is based primarily on rock excavation while the left bank construction requires a series of concrete dikes. The cost of rock excavation is much less than the cost of concrete ($13/m^3$ vs. $450/m^3$). Therefore, it appeared that the cost of construction on the right side would be much less than the cost of construction on the left side.

After careful consideration of the costs of constructing the left side structure and the costs of constructing the right side structures we have found no significant difference.

Fourth, sediment in the water would be controlled on the right side by a sediment basin upstream of the power canal. During low flows diversions into and out of the sediment are easily controlled by gates. However, during major floods it is necessary to build an embankment around the sediment basin to control the flood waters, or it is necessary to shutdown the powerplant when sediment control is lost. That is, without the protective embankments sediment control is lost due to flooding in the sediment basin and the powerplant must be stopped to protect the turbine from damage. The cost of these embankments can not be avoided if one wishes to operate the powerplant during floods.

A better design would employ a sediment basin at the downstream end of the power canal. No embankments at the entrance to the power canal would be needed for flood protection. On the right side of the river such a design is not possible at a reasonable cost. However, on the left side upstream of the bridge a large area exists where a sediment flood flows by building a high concrete wall with three inlet gates at the entrance to the sediment basin. This wall functions essentially like the embankment around the sediment basin in the right bank design except a major part of the flood flow can drain away from the left bank toward the central main river channel as the water backs up behind the wall on the left bank. Thus, from the point of view of controlling sediment during major floods, the left bank design would appear more advantageous. This conclusion must be qualified by noting that economically it may be more beneficial to build the right bank plan without large embankments and shut down the powerplant during floods. The economic benefits of the various alternatives have been analyzed in detail later in this report.

In summary, from a technical point of view the right side construction plan has the disadvantage of requiring large embankments if operation of the powerplant is demanded during major floods. It has the advantage of being constructed in sound rock. The left side construction has the advantage of a downstream sediment control area, but it has the disadvantage of requiring a long series of concrete dikes up to the sediment basin and embankments around the sediment basin. Coyne et Bellier have suggested that the island is not a stable geologic formation and may have been inundated during its existence. If this is true, construction on the island may pose an additional risk.

It is possible, of course, to build the mirror image of the right bank construction on the left bank. This alternative has also been analyzed and discussed in the next section.

Thus, technical advantages and disadvantages exist on both sides of the river, and the final determination of the best construction plan must be made by considering the costs of each alternative and the associated economic benefits.

DESIGN ALTERNATIVES

Three alternative powerplant layouts have been analyzed. The layout for each is described below. Following the description of each alternative its cost is estimated.

The economic benefits depend not only on the cost of the project but also on its energy production capability. One of the differences among the project alternatives is their ability to operate under flooding conditions. This issue is discussed for each alternative and the results used in determining its economic benefits.

Alternative I

The principal features of Alternative I are shown in Fig. 1-3. Alternative I has two options. The first option provides high embankments which allow the powerplant to operate under major flood conditions. In the second option the embankments are lower and the powerplant is not able to operate during major floods. The exact operational differences are discussed in the section OPERATIONAL FACTORS.

Cross channel diversion dikes

As shown in Fig. 1, a series of concrete dikes extends across the river. The crest elevation of the dikes is El. 137.5. A typical cross section of these dikes, shown in Fig. 3, shows that they are quite small except in the center of the river where one dike has a depth of about 3 m. The dikes are anchored in rock and protected on their up-stream and downstream faces by large boulders.

The overflow weir at the right end of the cross channel dikes has a crest elevation of El. 135. This section of dike and the gate adjacent to it help control flow into the sedimentation basin. The channel in front of the entrance weir to the sediment basin allows excess water to flush sediment from in front of the entrance weir through the main channel gate. One can note also in Fig. 2, cross section F-F, that the channel in front of the intake and through the gate structure area has been excavated to improve flow through the gate.

Sediment control structure

A sedimentation basin will be constructed adjacent to the right bank. The sedimentation basin will be excavated in rock with an invert elevation at its upper end of El. 132 decreasing to El. 129 at the entrance to the power canal. The dimensions of the basin are: 220 m long, 60 m wide.

The entrance to the sedimentation basin is a 110 m intake concrete weir along the left edge of the basin with crest elevation of El. 134. A cross section of the weir is shown in Fig. 3. The crest elevations of the intake weir and

the overflow weir have been carefully chosen to guarantee up to 60 m^3/s of flow into the sedimentation basin.

The upstream end of the basin consists of an earth and stone embankment. The crest elevation of this embankment will depend on the level of flood protection desired for the sediment control structure. For the maximum protection design flood (Q = 33,000 m^3/s) the crest elevation of the upstream embankment is El. 145. The embankment along the left side of the sediment pond extends to the bridge. The cross sections of the embankment are shown in Fig. 3. The embankment across the entrance to the power canal has a crest elevation of El. 143.

Also shown in Fig. 3 is a cross section of the upstream embankment for a 10-year flood (Q = 7,500 m^3/s). The embankment's crest elevation is El. 140.

The operation of the powerplant for the 1,000-year and 10-year flood protection is discussed under OPERATIONAL FACTORS. We will refer to the designs that incorporate the 1,000-year and 10-year flood protection embankments as Alternative I, Option 1, and Alternative I, Option 2, respectively.

Sediment basin flush gate

At the lower end of the sedimentation basin a flush gate will be placed in a conduit through the embankment. This flush gate will be used to clean the sedimentation basin.

Power canal control gate

At the lower end of the sediment control structure a gate will be used to control flows into the power canal. The control gate is placed in a concrete conduit connecting the sediment basin and the power canal and will be automatically controlled by the elevation of the water in the sedimentation basin. The dimension of the gate will be: 3 m high, 10 m wide.

Power canal

A 660 m long power canal will convey the flow from the sedimentation basin into the intake structure. The canal will be excavated in rock in a rectangular shape with a floor width of 10 m.

The slope of the canal varies from 1% for the first 260 m to 1/2% for the final 400 m. The floor elevation of the canal drops from El. 129 to El. 124 at the intake structure.

The embankments previously described run along the left side of the canal. At several points this embankment is capped in concrete as shown in the cross section in Fig. 2.

Near the intake structure a flush gate will be placed in the left side of the canal. The purpose of this gate is to allow the power canal to be flushed clean and also to evacuate any water run-off that may enter the canal from its right side. Also, under emergency shutdown of the powerplant the flush gate will be used to bypass water away from the powerplant.

The power canal is designed to carry flows up to 60 m^3/s.

Intake structure

The intake structure is excavated in rock at the end of the power canal. Its dimensions are 12 m long by 10 m wide, and its invert elevation is El. 121. On the vertical left wall of the intake structure two penstock portals will be constructed.

At the entrance to the intake structure a gate will be used to isolate the intake from the power canal. This 3 m x 10 m gate will close during emergency shutdown of the powerplant and during certain flooding situations which could arise in Option 2.

Penstock

One steel, 3.3 diameter penstock will convey water from the intake to the powerhouse. The penstock will be above ground and will enter the upper level of the powerhouse where it will make a vertical bend downwards towards the turbine room (see cross section A-A, Fig.2).

Powerhouse

The powerhouse shown in Fig 1-3, will be placed in an excavated area below the intake structure. Although the powerhouse is not underground three sides of the powerhouse will be backed by rock. It will be constructed from reinforced concrete and will be waterproofed.

The ground level of the powerhouse will contain the turbine, draft tube and tailrace exit. The tailrace will be excavated in rock from the powerhouse to the river channel.

The first floor of the powerhouse will contain the electrical generator, governor and associated controls. A special area of the first floor will be available for repair and maintenance of major equipment items.

The second floor of the powerhouse will contain the electrical control equipment, cooling and ventilating equipment, auxiliary power supply and miscellaneous equipment. A gantry crane located just below the ceiling of the second floor will be used to lower the turbine and generator into place. The crane will be used to move all equipment items to areas of the powerhouse where they can be maintained or repaired.

A stoplog structure will be used to close the tailrace for repairs or during emergencies.

Equipment

The principal equipment items include:

o Turbine:	Type:	vertical Francis
	Power:	14.4 MW
	Net head:	54 m
	Design flow:	30 m^3/s

- o Generator: Type: Synchronous
  Power: 15 MVa
  Voltage: 4.1 kV
- o Governor and controls
- o Automatic capability electrical controls
- o 2.6 m diameter butterfly valve
- o 15 MVa, oil cooled transformer
- o Air conditions and ventilation equipment
- o Auxiliary power supply
- o Crane
- o Repair tools
- o Elevator

Substation

The transformer and electrical protection equipment will be located in a substation adjacent to the powerhouse.

Access road

An access road across the power canal to the upper level of the powerhouse will be constructed as shown in Fig. 1.

Cost estimate

The estimated cost for Alternative I is shown in Table 4. The cost estimate is based on unit prices provided by the local construction firms, COLAS and GAMBOGI, and equipment prices provided by U.S. and French equipment manufacturer and the local equipment fabricator, CIMELTA. The unit prices were checked against those used by Coyne et Bellier and were found to be higher. The difference between the Coyne et Bellier estimates, which were completed during 1980, and these estimates can be accounted for by assuming a 12% annual inflation in unit prices. A contingency of 10% on equipment and 20% on civil works has been included.

The cost estimate is divided into two options. The first option includes embankments sufficiently high to allow the powerplant to operate during the maximum expected flood (Q = 33,000 m^3/s). The second option allows operation during a 10-year flood (Q = 7,500 m^3/s). For higher flows the powerplant would not be operable.

<u>It should be emphasized that for both options the project is designed to structurally withstand the maximum expected flood</u>.

<u>Alternative II</u>

The principal features of Alternative II are shown in Fig. 4.

Cross channel diversion dikes

The cross channel diversion dikes span the main channel along the line of closure used in Alternative I. Cross sections of the dikes are shown in

Table 4. Alternative I Cost Estimate for the Ambodiroka Project

Item	Cost estimate[a] Option 1 ($ 000)	Option 2 ($ 000)
Cross channel diversion dikes	1,050	1,050
Sedimentation basin	560	560
Embankment	1,330	730
Power canal	2,260	1,560
Intake structure	260	260
Penstock	370	370
Powerhouse and tailrace	2,010	2,010
Access road and bridge	90	90
Equipment	2,200	2,200
Substation	200	200
Transmission line	12,000	12,000
Construction crew housing and supplies	160	160
Shippine and Insurance	50	50
Subtotal	22,540	21,240
Design and construction management	2,700	2,550
Contingency	3,530	3,260
Total construction cost	28,770	27,050

[a]Approximately 9% taxes have been included in the estimate of the civil works costs. No TUT tax or other taxes have been included.

Fig. 3 and, like Alternative I, have a crest elevation 137.5 with the exception of the lower control dike near the sediment pond which has a crest elevation 135.

The topography of the river channel up to the sediment basin entrance is somewhat higher than the similar channel area on the right side and as a result it will be necessary to excavate some additional rock.

Sediment control structure

The sediment control structure is basically the same as the sediment control structure in Alternative I except additional excavation, previously mentioned, is required from the main channel up to the entrance weir and from the main channel gate back to the main channel.

Power canal control gate

The flow from the sedimentation basin would be controlled by a control gate like the one described in Alternative I.

Power canal

The power canal is excavated in rock from the lower end of the sedimentation basin to the center of the island. The topography along the left side of the river suggests that the invert of the canal will be somewhat lower than the canal in Alternative I. Also, a number of sand deposits and fractures along the left side indicate that additional concrete will be needed to waterproof the canal and protect its right side.

Below the bridge an additional embankment is required on the left side of the canal to maintain flow into the intake structure.

The canal gates are the same as those used in Alternative I.

Intake structure

The intake structure would be similar to the intake structure described in Alternative I.

Penstock, powerhouse and tailrace

The penstock, powerhouse and tailrace would be similar to those described in Alternative I.

Cost estimate

The estimated cost for Alternative II is shown in Table 5. The bases for the cost estimate for Alternative II are the same as for Alternative I. The options described in Alternative I are also the same.

<u>Alternative III</u>

Option 1

The layout for Alternative III is similar to Coyne et Bellier's Variation II up to the bridge. That is, a series of dikes across the river at El. 136 and along the right diverts low flows toward the natural channel under the small bridge on the left side of the river. Coyne et Bellier designed dikes with considerably more concrete than the type shown in Fig. 3. It is anticipated that the lower cost option will be used although this option will require more maintenance over the life of the powerplant. Some rock excavation is needed to improve the channel.

Just upstream of the bridge a critical embankment across the channel is constructed. The crest elevation 132 is above the water surface elevation expected during the maximum flood. This embankment controls all flood waters and, therefore, allows the sediment control structure to always operate.

Table 5. Alternative II Cost Estimate for the Ambodiroka Project

Item	Cost estimate[a] Option 1 ($ 000)	Option 2 ($ 000)
Cross channel diversion dikes	1,060	1,060
Sedimentation	240	240
Embankments	1,890	1,040
Power canal	2,000	1,380
Intake structure	320	320
Penstock	440	440
Powerhouse and tailrace	2,030	2,030
Access road	190	190
Equipment	2,450	2,450
Substation	200	200
Transmission line	12,000	12,000
Construction crew housing and supplies	160	160
Shipping and insurance	50	50
Subtotal	22,780	21,310
Design and construction management	2,740	2,590
Contingency	3,560	3,300
Total cost	29,080	27,200

[a]Approximately 9% taxes have been included in the estimate of the civil works costs. No TUT tax or other taxes have been included.

A conduit through the embankment allows up to 100 m^3/s of water to pass into the sedimentation basin just below the bridge. In the first stage of development just 30 m^3/s of flow are needed for power. The additional water would be used to flush the sedimentation basin. Gates in the conduit control the flow into the basin. The dimensions of the conduit are: 15 m wide, 3 m high, 45 m long. Trashracks will cover the entrance to the conduit.

The flush gate on the right side of the diversion channel just upstream of the bridge would be like the one proposed by DAFECO (Coyne et Bellier's Variation A1). It would provide a way to flush gravel and other sediment out of the diversion channel and would help eliminate high waters in front of the embankment during floods. The dimensions of this gate are: 3 m high, 3 m wide.

Just below the bridge a second embankment along the right side of the channel contains the water in the sediment pond. An important difference between Alternative III and DAFECO's design is the size of this embankment. In DAFECO's design this embankment had a crest elevation 133 in order to contain

floods. In Alternative III the crest elevation is El. 130, because the total quantity of water in the sedimentation basin is limited by the first embankment. A gate structure, 3 m x 10 m, in the second embankment provides a way to flush the sedimentation basin.

The intake structure, penstock location and powerhouse location would be similar to the plan by DAFECO.

The intake structure would be sized for 60 m^3/s. However, one penstock, diameter 3.3 m, would be used initially to carry 30 m^3/s from the intake to the powerhouse. The dimensions of the powerhouse and tailrace would be somewhat smaller than proposed by DAFECO.

Cost estimate

The cost estimate for Alternative III is shown in Table 6. The unit costs and equipment costs have been established as described in the cost estimates for Alternative I.

Table 6. Alternative III Cost Estimate for the Ambodiroka Project

Item	Cost estimate[a] Option 1
Main control diversion dikes	2,180
Sedimentation basin and embankments	1,600
Power canal and intake	450
Penstock	460
Powerhouse	2,010
Access road	190
Equipment	2,200
Substation	200
Transmission line	12,000
Construction crew housing and	160
Shipping and insurance supplies	50
Subtotal	21,500
Design and construction management	2,580
Contingency	3,300
Total cost	27,380

[a]Approximately 9% taxes have been included in the estimate of the civil works costs. No TUT tax or other taxes have been included.

OPERATIONAL FACTORS

Each proposed project alternative is designed to operate under a wide range of conditions. Operation during very low flows, major floods and emergency shutdown of the powerplant are of particular importance. Operation during each of these conditions as well as "normal" operation is discussed below.

Low flow operation

With respect to low flows (Q = 30 m^3/s) each project alternative includes a series of cross channel diversion dikes which divert low flows into the sedimentation basin (Alternative I, II) or the power canal (Alternative III). The elevation of these dikes and the elevation of the sediment basin entrances have been carefully chosen to provide at least 30 m^3/s of flow.

One should note also that during low flows water in the main channel will back up behind the main channel diversion dikes and, hence, will drop much of its sediment load. The sediment, which accumulates in front of the entrance to the sediment basin, will be flushed through the gate adjacent to the sediment basin.

Normal flow operation

Over 90% of the time the flows in the Betsiboka will not exceed 450 m^3/s. For flows up to 7,500 m^3/s the operation of Alternative I and II (both Options 1 and 2) are identical. The control gate at the entrance to the power canal will be opened sufficiently to allow 30 m^3/s into the power canal. The flush gate in the sediment basin will be opened at appropriate times to flush sediment from the bottom of the sedimentation basin.

The total flow into the sedimentation basin will be equal to the combined flow at the flush gate and the control gate. The elevation of the embankments is sufficiently high to avoid any overtopping into the sedimentation basin or power canal.

Under normal flow conditions, the flush gate at the end of the power canal will be closed and the control gate at the entrance to the intake will be open.

Flood flow operation

A run-of-river hydroelectric project on the Betsiboka River could be subjected to flow rates of extremely wide variation during its operational life. In order to evaluate the effect of these flow variations on the operation of the power project, and to determine economic tradeoffs associated with the provision of the varying levels of flood protection which would be associated with the flow rate variations, estimates were made of the river flow profiles in relation to the four recurrence intervals shown in Table 7.

The flow profiles were estimated using the U.S. Army Corps of Engineers HEC-2 computer program on a VAX 11/730 computer system. The HEC-2 computer program calculates water surface profiles for gradually varying flow in natural or manmade channels. It has the capability to calculate both sub-critical and super critical flow profiles. The Standard Step Method, the solution of the

Table 7. Peak flows in the Betsiboka River at Ambodiroka at Several Recurrence Intervals

Recurrence interval (years)	Peak flow rate (m^3/s)
1,000	2,3000
100	20,000
10	7,500
2	3,500

one-dimensional energy equation with energy loss due to friction evaluated using Manning's equation, is used to compute the flow profile.

The river channel was evaluated by selecting 12 "representative" river reaches; the locations of the sections representing the river reaches in the vicinity of the power project are shown in Fig. 1. The upstream section satisfies the boundary condition given by the staging curve presented in Coyne and Bellier's Etude de Factibilité. As the topographic map did not provide sufficient information at this location, a trapeziodal section was assumed. Sections 690, 800, and 1025 (Fig. 1) represented the river channel in the vicinity of the cross channel diversion dike and the upstream sedimentation pond. Section 1200 was used to represent the river channel along the power canal. Section 1560 represents the river channel in the vicinity of the bridge. This section includes the effects of the piers and the abutment sections. Sections 1710, 1870, and 1900 were used to evaluate the losses associated with the steep, narrow, channel below the bridge. Sections 1870 and 1900 represent the degree to which channel improvement may be necessary to limit the water surface in the vicinity of the proposed powerhouse. Sections 2010, 2110, 2350, and 2530 represent the river profile downstream of the tailrace. Sections 2350 and 2530 correspond to Points 1 and 2, respectively, of Coyne and Bellier's hydraulic profile.

The hydraulic characteristics used for each of the relevant sections are presented in Table 8 and typical results for the four recurrence interval storms are presented in Table 9. The results of the analyses are shown in conjunction with the river profile presented on Fig. 1. Based on available topography, the river slope is generally mild and sub-critical flow conditions are computed for the channel to Section 1200. In the vicinity of the bridge, the river slope becomes steep, and the flow passes through critical depth and is calculated to be super critical in the downstream reaches. Due to the channel contraction at, and downstream of, the bridge, the steep slopes and super critical flow may create cross waves and turbulent flow conditions. Downstream of the Section 2530, (Point 2 in Coyne et Bellier) the river channel appears to be mild and sub-critical conditions may occur, causing backwater effects at this section. These backwater effects should not propagate to the tailwater section for the proposed powerhouse, and can be further evaluated during the feasibility study.

Table 8. Channel Reach Characteristics on the Betsiboka River at Ambodiroka

Section Number	Reach length (M)	Manning's coefficients Left bank	Channel	Right bank	Slope (M/M)
690	110	0.040	0.040	0.040	0.0014
800	225	0.040	0.040	0.040	0.0018
1025	175	0.040	0.040	0.040	0.0027
1200	360	0.040	0.040	0.040	0.0014
1560	150	0.040	0.040	0.040	0.0039
1750	160	0.040	0.040	0.040	0.0180
1870	30	0.040	0.040	0.040	0.0081
1900	110	0.040	0.040	0.040	0.0100
2010	100	0.040	0.040	0.040	0.0063
2110	240	0.040	0.040	0.040	0.0070
2350	180	0.040	0.040	0.040	0.0063

Table 9. Summary of Flow Profile at Ambodiroka

Section number	Recurrence interval (years) 1000	100	10	2
690	145.4	142.6	139.1	137.6
800	143.7	142.8	140.2	139.1
1025	140.1	137.5	135.0	134.0
1200	137.4	135.5	133.2	131.9
1560	130.7	127.0	120.8	118.1
1710	116.2	110.8	102.4	98.0
1870	94.7	87.8	79.7	76.6
1900	91.6	86.7	78.1	74.6
2010	75.4	71.4	67.1	65.6
2110	76.4	72.7	67.9	65.7
2350	73.4	70.8	67.5	58.0
2530	64.3	60.7	55.1	55.5

Alternative I

Two options have been considered for the construction of Alternative I. Option I has high embankments which never allow water to flow into the sediment basin except across its entrance weir. No flows can enter the power canal except through the control gate. Thus, the powerplant remains operational up to Q = 33,000 m^3/s.

In Option 2 flood flows exceeding Q = 7,500 m^3/s can overtop the embankments. These flows can enter the power canal uncontrolled and sediment control will be lost. Since it would be very bad to pass water with a heavy sediment load through the turbine the powerplant must be shutdown. Shutdown will automatically occur. The control gates at the entrance to the power canal and intake structure will close in unison with the butterfly valve. At the same time, the flush gate in the power canal will open. Closure time on the gates is very short compared to the time it takes water to overtop the embankment so that sediment will never reach the turbine.

We note that the hydrograph for major floods shows that floods last no more than 6-8 hours at Ambodiroka. Therefore, during a major flood the powerplant will be inoperable for 6-8 hours in Option 2. Also, it is possible that some debris in the power canal will need to be cleaned out so that the powerplant could be inoperable another 8-24 hours to allow the power canal to be cleaned.

Finally, it is very important to note that the elevation of the tailwater increase dramatically during a major flood. The tailwater curve shown in Fig. 6 reflects the tailwater assuming that the main channel has been improved as shown in Fig. 2. Even so, the tailwater elevation can increase from El. 65 to El. 83 over a 6-8 hour period.

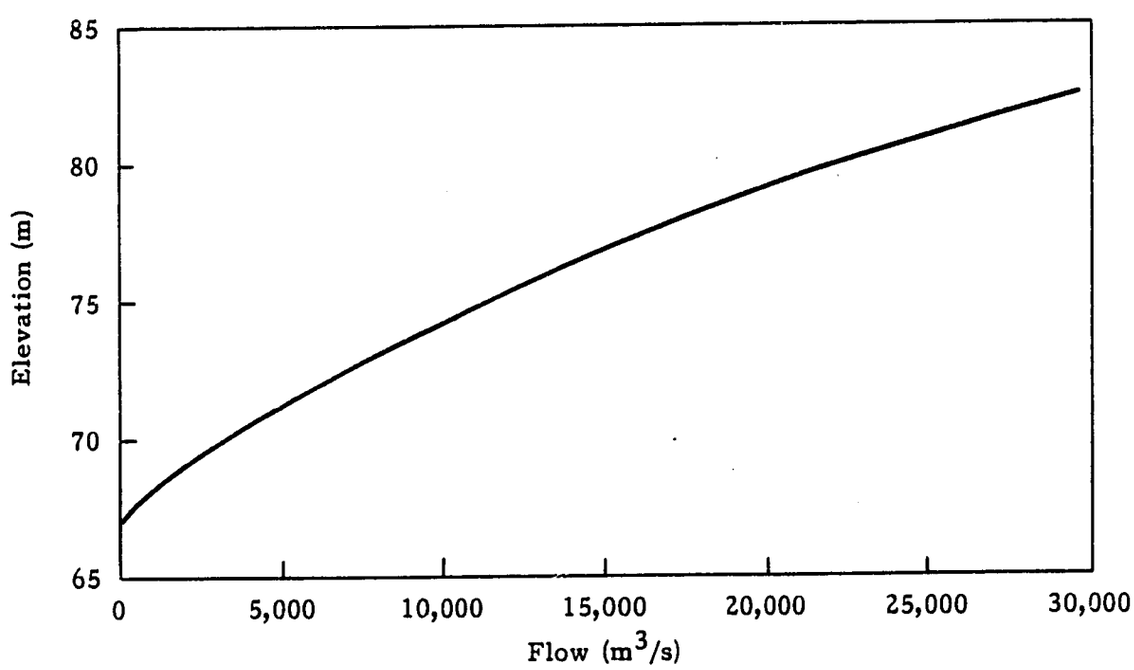


Fig. 6. Tailwater curve at the location of the powerhouse.

During this period the available head to operate the turbine would decrease from 54 m to 36 m. The turbine can operate at this head, but its power output will fall from 14.4 MW to about 8.8 MW.

Alternative II

The operation during major floods for Alternative II, Options 1 and 2, will be identical to the operation of Alternative I, Options 1 and 2 during major floods with one major exception.

In Alternative II, Option 2, the possibility exists that flood waters will exit into the channel around the island. If the final location of the powerhouse is near this channel, then it could be damaged during high floods. To avoid this possibility an additional embankment must be built below the bridge on the left side of tne power canal. However, the cost of this embankment exceeds the cost of building the higher embankments in Option 1. Therefore, it is concluded that Alternative II, Option 2 is not viable.

Moreover, as will be seen below, Alternative II Option 1 has the highest total economic cost and, therefore, no further consideration was given to Alternative II.

Alternative III

Alternative III is designed to operate during flows up to Q = 33,000 m^3/s. As previously mentioned, flows up to Q = 33,000 m^3/s are stopped from entry into the sediment basin by an embankment just upstream of the bridge.

During major floods only the necessary 30 m^3/s would be allowed into the sedimentation basin. The flush gate in the right wall of the diversion channel near the first embankment would be opened to allow sediment to be flushed toward the main channel.

Like Alternative I, it is important to note that the elevation of the tailwater increases dramatically during a major flood. Nevertheless, the head would be sufficient to continuously operate the powerplant.

DIFFICULTY OF CONSTRUCTION

One of the reasons for investigating Alternative I was based on the idea that construction along the right side of the river would be less difficult and, therefore, less expensive than construction along the left bank of the river. Even though the cost estimates for the right and left side did not reveal any substantial difference, a construction sequence was considered for both alternatives in order to help conceptualize the projects in more detail and to investigate further the validity of the project costs estimates.

A brief summary of the construction sequence is given below for Alternatives I and III excluding the construction of the transmission line which is the same for both alternatives.

<u>Construction sequence: Alternative I</u>

1. Initiate construction near the bridge excavating upstream and downstream. Materials from the excavation are used for the dikes.

2. Construct a small cofferdam around the sedimentation basin.

3. Excavate the sedimentation basin, improve channel in front of intake and use materials for the embankments.

4. Build dikes out to the main channel.

5. Build conduits through embankments and install control gates and flush gate.

6. Remove cofferdam upstream and downstream of cross channel dikes.

7. Build cofferdams upstream and downstream of cross channel dikes.

8. Divers water through sediment flush gates.

9. Build dikes across the main channel.

10. During construction of dikes also build access road to powerhouse area.

11. Cofferdam tailrace area and excavate powerhouse and tailrace.

12. Build access road across island so that main channel improvements can be made.

13. Construct powerhouse and tailrace.

14. Install penstock and powerhouse equipment.

<u>Construction sequence: Alternative III</u>

1. Put in cofferdams for the construction of dikes along the right side of the intake channel.

2. Build dikes along the right side of the intake channel and install the sediment control gate structure.

3. Cofferdam the intake channel and excavate.

4. Use excavation materials for cofferdam across the main channel of the river and the primary embankment at the end of the intake channel.

5. Build dikes across the main channel of the river.

6. Excavate sedimentation basin and intake structure and build embankment along its right side.

7. Build access road to powerhouse area.

8. Improve the main channel.

9. Cofferdam tailrace area and excavate powerhouse and tailrace.

10. Construct powerhouse and tailrace.

11. Install penstock and powerhouse equipment.

Reviewing the construction sequence has led the team to conclude that no major difference exists between construction on the right side of the river and construction on the left side of the river. Building the features upstream of the bridge appear to be somewhat less complicated on the left side of the river, while building the features downstream of the bridge appear less complicated on the right side. Thus, there should be no major cost differences between construction for Alternatives I and III based on the construction sequence outlined above.

SUMMARY

In summary, the final selection of the project will depend on several factors, including difficulty of construction, capital cost, and operational aspects.

Table 10 summarizes the total construction cost for the various alternatives. From the table it appears that the best options are Alternative I, Option 2 and Alternative III. However, Alternative III is built to operate during the 1000 year flood and, therefore, will earn greater benefits than Alternative I, Option 2, which will lose benefits during extreme flooding periods. Thus, the design choice should be framed in net economic cost terms, which is discussed briefly in the next section.

Table 10. Comparison of Cost Estimates for the Ambodiroka Project

Alternative	Cost estimate ($000)
Alternative I	
Option 1 (1000 year flood)	28,770
Option 2 (10 year flood)	27,050
Alternative II	
Option 1 (1000 year flood)	29,080
Option 2 (10 year flood)	27,200
Alternative III	27,380

IV. ECONOMIC AND FINANCIAL ANALYSIS

ECONOMIC ANALYSIS

The primary purpose of the economic analysis is to make a preliminary least-cost calculation to determine the optimal choice of three power supply options, given the revised demand forecast:

- to continue to meet demand with diesel generation, which would require investment in a total of 30 MW of diesel generators during the 20-year period of the study;
- to develop the Ambodiroka site to 26.6 MW according the the design proposed in the Coyne et Bellier study (without adding the 13 MW second stage); or
- to construct a simpler hydroelectric scheme at Ambodiroka in two 14 MW stages, which would require a small amount of diesel generation for peaking during the first stage of the project.

For purposes of this analysis, no distinction is made between the various design options which are under consideration for the reduced-scale scheme, since the options are all comparable in cost. A preliminary determination of the relative economic merits of each alternative will be presented below.

The essential assumptions for this prefeasibility economic analysis are given in Table 11.

Table 11. Assumptions Used in the Economic Analysis

Item	All diesel	Hydropower 26 MW	Hydropower 14+14 MW
Capital cost ($\$10^6$)	28.5	68	27 + 3
Useful life (years)	15	30	30
Average fuel cost ($/kWh)	0.055		0.055
O&M cost ($\$10^6$)	.75	0.34	0.14[a]
Discount rate	0.10	0.10	0.10

[a]$0.27 million after 2006.

It is assumed that JIRAMA would have to purchase 30 MW of new diesel capacity during the period 1994-2013, including replacement costs for units that are retired during the period. The capital cost of diesel generators is assumed to be $950/kW installed, which is comparable to costs assumed in the Coyne et Bellier study. O&M costs for the hydropower generating plant are assumed to

be .5% of the capital cost. The diesel O&M costs are derived from JIRAMA's financial data. The capital cost for the 26 MW (first stage) Coyne et Bellier scheme are derived from their feasibility study.

It is further assumed that there would be no need for diesel generation with the Coyne et Bellier scheme, but that diesel would be required to meet peak demand between 1995 and 2007 under the reduced-scale scheme. The second 14 MW unit, with a capital cost of $3 million, would be added in 2007, when the workday base load is projected to reach 13.7 MW.

The chief differences from Coyne et Bellier's assumptions are with respect to demand and fuel cost. The demand assumptions are less optimistic than the "weak" demand growth scenario in the Coyne et Bellier study. Fuel costs are also lower. An average fuel cost of approximately FMG 100 per liter is used in this analysis, based on a base cost of FMG 92 per liter for fuel oil plus transport costs from Toamasina. Furthermore, there is no fuel-cost escalator assumed, whereas Coyne et Bellier assumed 4% real growth in the price of fuel.

The results of the analysis, summarized in Table 12, show that the scaled-down hydropower scheme is less than half the cost of the original hydroelectric design, in present worth terms. The analysis further shows that, with a lower cost of fuel and the slower rate of demand growth than were assumed by Coyne et Bellier, it would be more cost-effective to generate with diesel than to construct the Ambodiroka hydropower project as originally designed.

Table 12. Results of the Economic Analysis

Supply option	PW cost ($000)
All diesel	63,151
26 MW hydropower	67,878
14+14 MW hydropower	29,465

Design optimization

A detailed economic comparison of the various design options should be conducted during the feasibility stage. The key issue in this analysis should be to determine the relative benefits from providing greater protection against flooding, which will otherwise limit the operation of the powerplant.

A first approximation can be made by estimating the total costs--in terms of construction and lost benefits--from adopting different design approaches. Table 13 provides cost estimates of constructing Alternative I, Option 2 to withstand the 2 year, 6 year, 10 year, 100 year, and 1000 year floods. Only

items whose costs change as a function of flood protection level are shown. For each design option, a corresponding loss of benefits is calculated resulting from the design's vulnerability to flooding.

Table 13. Total Cost Estimates for Different Levels of Flood Protection

Option	Cost of flood protection ($000)	Lost revenue over life of project ($000)	Total cost ($000)
1000 year flood	2600	0	2600
100 year flood	1250	5	1255
10 year flood	670	50	720
6 year flood	480	70	550
2 year flood	570	260	830

The "lost revenue over life of the project" was computed by assuming a project life of 40 years and by assuming that the powerplant is inoperable for 30 hours during a flood.

The table indicates that the best option is the "6 year flood" option. The design and cost estimates given in this report are for the 10 year flood in order to add a small increment of security to the project operations.

Similar cost comparisons can be made for the left bank options in Alternatives II and III, but it appears that either Alternative I, Option 2 or Alternative III are the most economic designs, based on this preliminary analysis. Taking into account lost benefits from flooding, Alternative I, Option 2 has a net economic cost of $27.6 million, compared to Alternative III's cost of $27.4 million.

FINANCIAL ANALYSIS

The primary objective of the financial analysis is to determine the tariff level needed to cover operating expenses, including loan amortization. JIRAMA's existing average tariff in Mahajanga is FMG 47.3, or about $0.08/kWh. Based on 1985 operating costs, a 17.8% increase in the average tariff charged would be needed to meet the cost of service.

Table 14 summarizes the assumptions used in the financial analysis of the Ambodiroka project. The income statement (see Annex B) indicates that the existing tariff would cover 97% of first-year expenses with surplus income carried in each succeeding year.

Table 14. Assumptions for Financial Analysis of the Ambodiroka Hydroelectric Project

Assumption	Value
Construction period (years)	2-3
Depreciable life of assets (years)	25
Average tariff ($/kWh)	$0.08
Annual cost of O&M (hydropower; % of project cost)	.005%
Cost of O&M (diesel; $/kWh)	$0.02
Cost of fuel ($/kWh)	$0.075
Annual administrative cost	$500,000
Annual cost of insurance (% of project cost)	0.25%
Annual cost of promotion (% of sales)	0.50%
Unpaid receivables (% of sales)	10.00%
Inflation rate	5.00%
Discount rate	10.00%

V. RECOMMENDATIONS

The team recommends that a full feasibility study be conducted on the reduced-scale Ambodiroka hydroelectric scheme. The results of the economic and financial analysis indicate that the project would be extremely attractive. Moreover, since significant work has already been invested over the years to study the site, the cost of a feasibility study would be relatively low, approximately $300,000.

In recommending a final design choice, the feasibility study should consider first Alternative III. If detailed investigations of Alternative III result in significant cost increases, then serious investigation of Alternative I, Option 2, should also be completed.

The team also recommends increasing the tariff rate to close the present operating deficit. Since the majority of electricity use is by the large industrial users and since they are presently subsidized by residential users, it would seem appropriate to apply future tariff increases to industrial rates. An increase in the average industrial tariff on the order of 20% would be sufficient to cover JIRAMA's operating costs.

ANNEX B. FINANCIAL SPREADSHEET

MAHAJANGA INCOME STATEMENT YEAR	1994	1995	1996	1997	1998	1999
Project Electricity Sales (kWh)	71220000	72450000	73760000	75150000	76630000	78200000
Average Tariff (US$/kWh)	$0.08	$0.08	$0.09	$0.09	$0.10	$0.10
Sales Income	$5,697,600	$6,085,800	$6,505,632	$6,959,641	$7,451,540	$7,984,417
Unpaid Receivables	(569,760)	(608,580)	(650,563)	(695,964)	(745,154)	(798,442)
Total Gross Income	$5,127,840	$5,477,220	$5,855,069	$6,263,677	$6,706,386	$7,185,976
Cost of Sales						
O&M (hydro)	$163,000	$171,150	$179,707	$188,693	$198,128	$208,034
Diesel fuel	0	$30,750	$66,937	$107,494	$154,543	$208,763
O&M (diesel)	0	$8,200	$17,850	$28,665	$41,211	$55,670
Administration and other costs	$500,000	$525,000	$551,250	$578,812	$607,753	$638,141
Sub-Total - Operating Costs	$663,000	$739,567	$786,248	$835,979	$889,447	$946,937
Interest	$4,500,000	$4,500,000	$4,500,000	$4,466,250	$4,428,450	$4,386,114
Insurance	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750
Promotion	$28,488	$30,429	$32,528	$34,798	$37,258	$39,922
Depreciation/Amortization	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000
Sub-Total - Capital/Other Costs	$5,926,238	$5,928,179	$5,930,278	$5,898,798	$5,863,458	$5,823,787
Total Costs	$6,589,238	$6,667,746	$6,716,526	$6,734,777	$6,752,905	$6,770,724
NET INCOME	(1,461,398)	(1,190,527)	(861,457)	(471,100)	(46,519)	415,252

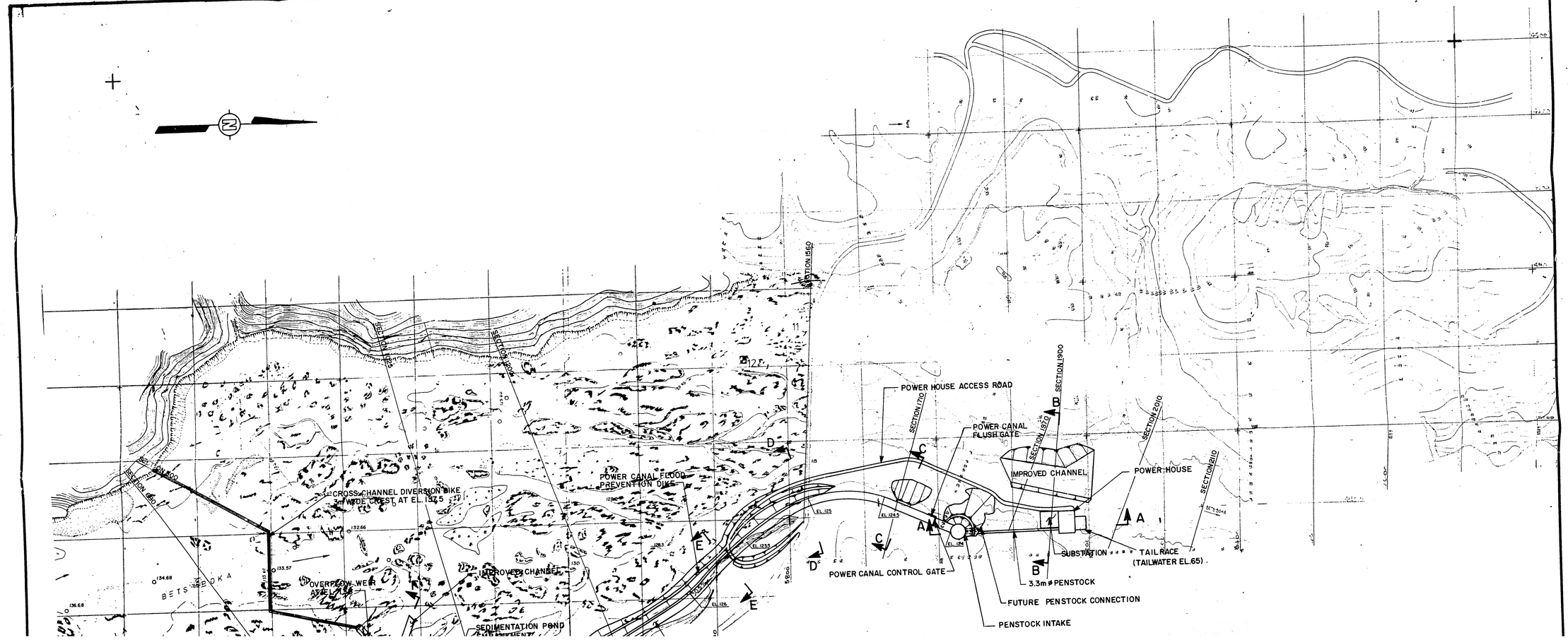
CASH FLOW STATEMENT

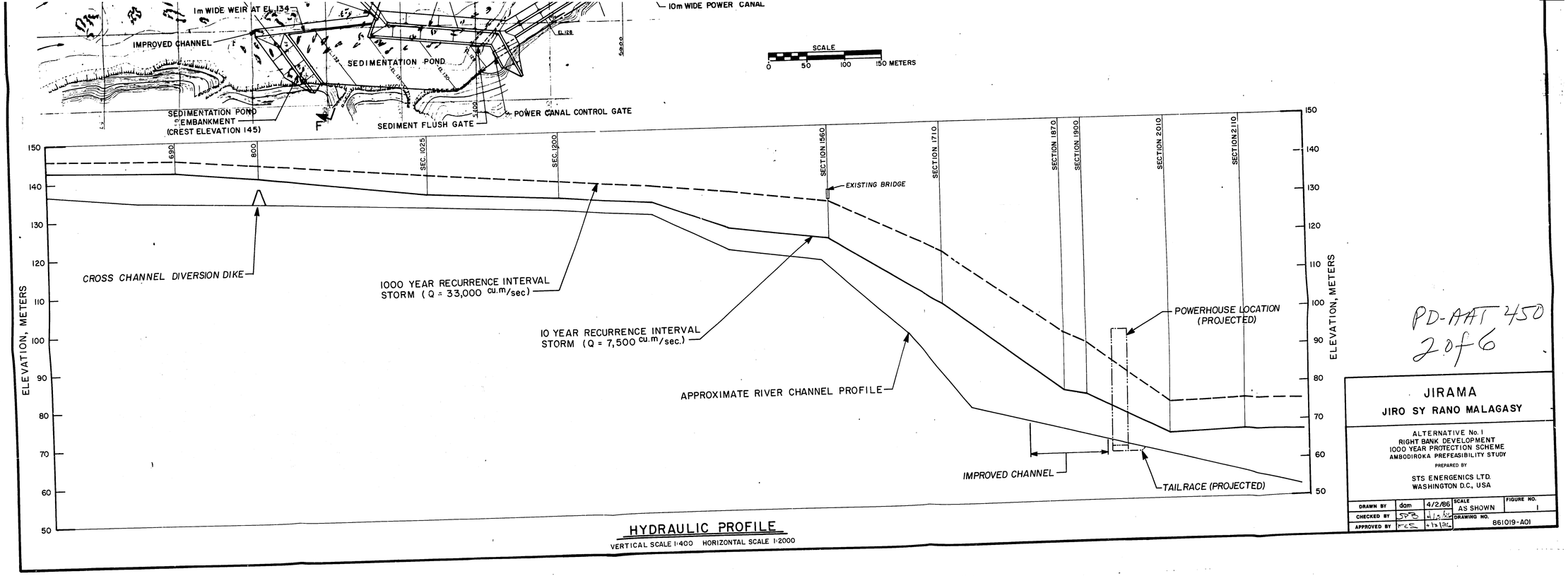
INCOME						
Net Income	(1,461,398)	(1,190,527)	(861,457)	(471,100)	(46,519)	415,252
Depreciation/Amortization	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000
Total Cash Income	(157,398)	113,473	442,543	832,900	1,257,481	1,719,252
OUTGO						
Total Capital Expense	$0	$0	$0	$0	$0	$0
Loan Repayments	$0	$0	$281,249	$314,999	$352,799	$395,134
Sub-Total - Cash Outgo	$0	$0	$281,249	$314,999	$352,799	$395,134
Net Cash Flow	(157,398)	113,473	161,294	517,901	904,682	1,324,118

Capital Cost of Project:	$32,600,000
Total Project Cost (1)	$37,500,000
Discount Rate	10%
NPV (10 years)	$5,416,526
NPV (25 years)	$23,537,078

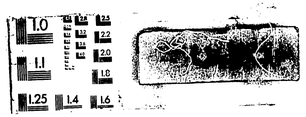
2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019
79880000	81650000	83550000	85560000	8770000	89980000	92410000	95000000	97760000	100700000	103840000	107180000	110750000	114560000	118620000	123030000	127720000	132680000	137940000	143520000
$0.11	$0.11	$0.12	$0.12	$0.13	$0.14	$0.14	$0.15	$0.16	$0.17	$0.17	$0.18	$0.19	$0.20	$0.21	$0.22	$0.23	$0.25	$0.26	$0.27
$8,563,747	$9,191,180	$9,875,312	$10,614,531	$1,142,832	$12,311,707	$13,276,407	$14,330,933	$15,484,649	$16,747,845	$18,133,576	$19,652,682	$21,322,646	$23,158,993	$25,178,734	$27,420,558	$29,889,144	$32,602,382	$35,589,623	$38,880,773
(856,375)	(919,118)	(987,531)	(1,061,853)	(114,283)	(1,231,171)	(1,327,641)	(1,433,093)	(1,548,465)	(1,674,785)	(1,813,358)	(1,965,268)	(2,132,265)	(2,315,899)	(2,517,873)	(2,742,056)	(2,988,914)	(3,260,238)	(3,558,962)	(3,888,077)
$7,707,372	$8,272,062	$8,887,781	$9,556,678	$1,028,549	$11,080,536	$11,948,766	$12,897,840	$13,936,184	$15,073,061	$16,320,218	$17,687,414	$19,190,382	$20,843,094	$22,660,861	$24,678,502	$26,900,230	$29,342,144	$32,030,661	$34,992,696
$218,436	$229,357	$240,825	$252,866	$265,510	$278,785	$292,725	$307,361	$322,729	$338,865	$355,809	$373,599	$392,279	$411,893	$432,488	$454,112	$476,817	$500,658	$525,691	$551,976
$268,976	$338,709	$416,854	$506,398	$684,136	$805,081	$941,542	$1,093,677	$1,264,328	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
$71,727	$90,322	$111,161	$135,039	$182,436	$214,688	$251,076	$291,647	$337,154	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
$670,048	$703,550	$738,728	$775,664	$814,447	$855,170	$897,928	$942,825	$989,966	$1,039,464	$1,091,437	$1,146,009	$1,203,310	$1,263,475	$1,326,649	$1,392,981	$1,462,630	$1,535,762	$1,612,550	$1,693,177
$1,008,221	$1,074,392	$1,145,250	$1,221,749	$1,325,513	$1,416,075	$1,513,817	$1,618,924	$1,732,341	$1,447,246	$1,519,608	$1,595,589	$1,675,368	$1,759,136	$1,847,093	$1,939,448	$2,036,420	$2,138,241	$2,245,153	$2,357,411
$4,338,698	$4,285,592	$4,226,113	$4,159,497	$4,084,887	$4,001,324	$3,907,733	$3,802,911	$3,685,510	$3,554,021	$3,406,754	$3,241,815	$3,057,083	$2,850,183	$2,618,455	$2,358,919	$2,068,240	$1,742,679	$1,378,050	$969,667
$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750	$93,750
$42,819	$45,956	$49,377	$53,093	$5,714	$61,559	$66,382	$71,655	$77,423	$83,739	$90,668	$98,263	$106,613	$115,795	$125,894	$137,103	$149,446	$163,012	$177,948	$194,404
$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000
$5,779,267	$5,729,298	$5,673,240	$5,610,340	$5,488,351	$5,460,632	$5,371,865	$5,272,315	$5,160,683	$5,035,511	$4,895,172	$4,737,828	$4,561,446	$4,363,728	$4,142,098	$3,893,772	$3,615,436	$3,303,441	$2,953,748	$2,561,820
$6,787,488	$6,803,690	$6,818,490	$6,832,088	$6,813,864	$6,876,708	$6,885,682	$6,891,239	$6,893,024	$6,482,756	$6,414,780	$6,333,417	$6,236,814	$6,122,864	$5,989,192	$5,833,220	$5,651,856	$5,441,682	$5,198,902	$4,919,231
919,884	1,468,372	2,069,291	2,724,590	(5,785,315)	4,203,829	5,063,084	6,006,601	7,043,160	8,590,304	9,905,438	11,353,997	12,953,568	14,720,230	16,671,669	18,845,282	21,248,374	23,900,462	26,831,759	30,073,464

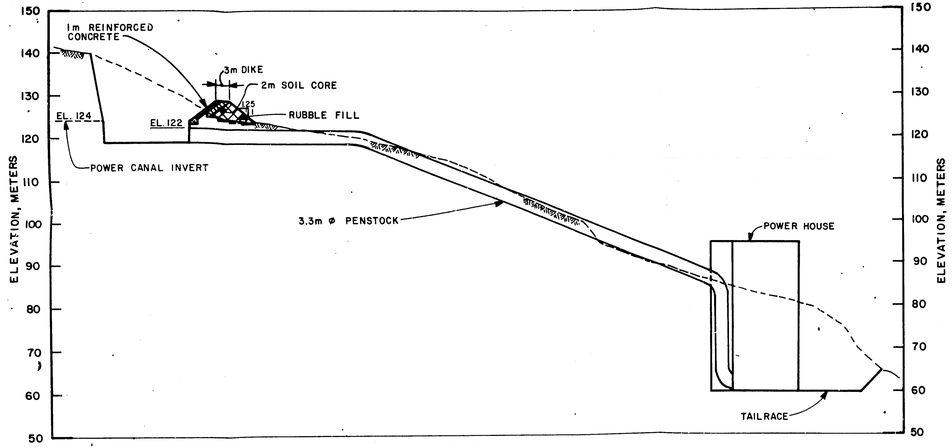
919,884	1,468,372	2,069,291	2,724,590	(5,785,315)	4,203,829	5,063,084	6,006,601	7,043,160	8,590,304	9,905,438	11,353,997	12,953,568	14,720,230	16,671,669	18,845,282	21,248,374	23,900,462	26,831,759	30,073,464
$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000	$1,304,000
2,223,884	2,772,372	3,373,291	4,028,590	(4,481,315)	5,507,829	6,367,084	7,310,601	8,347,160	9,894,304	11,209,438	12,657,997	14,257,568	16,024,230	17,975,669	20,149,282	22,552,374	25,204,462	28,135,759	31,377,464
$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
$442,551	$495,657	$555,135	$621,752	$696,362	$779,925	$873,516	$978,338	$1,095,739	$1,227,227	$1,374,495	$1,539,434	$1,724,166	$1,931,066	$2,162,794	$2,422,329	$2,713,009	$3,038,570	$3,403,199	$3,811,582
$442,551	$495,657	$555,135	$621,752	$696,362	$779,925	$873,516	$978,338	$1,095,739	$1,227,227	$1,374,495	$1,539,434	$1,724,166	$1,931,066	$2,162,794	$2,422,329	$2,713,009	$3,038,570	$3,403,199	$3,811,582
1,781,334	2,276,716	2,818,156	3,406,838	(5,177,677)	4,727,903	5,493,568	6,332,263	7,251,421	8,667,077	9,834,943	11,118,563	12,533,402	14,093,164	15,812,875	17,726,953	19,839,365	22,165,892	24,732,560	27,565,882


N
CROSS CHANNEL DIVERSION DIKE
3m WIDE CREST AT EL. 137.5
OVERFLOW WEIR
AT EL. 136
IMPROVED CHANNEL
SEDIMENTATION POND
BETSUBOKA
POWER CANAL FLOOD
PREVENTION DIKE
POWER HOUSE ACCESS ROAD
POWER CANAL
FLUSH GATE
IMPROVED CHANNEL
POWER HOUSE
SUBSTATION
TAIL RACE
(TAILWATER EL.65)
POWER CANAL CONTROL GATE
3.3m ϕ PENSTOCK
FUTURE PENSTOCK CONNECTION
PENSTOCK INTAKE
SECTION 1550
SECTION 1700
SECTION 1800
SECTION 1900
SECTION 2010
SECTION 2110
SECTION 2200
SECTION 2300
SECTION 2400
SECTION 2500
A
B
C
D
E
EL. 125
EL. 124.5
EL. 124
EL. 126
EL. 128.3
EL. 130
EL. 132.66
EL. 133.57
EL. 134.68
EL. 136.68

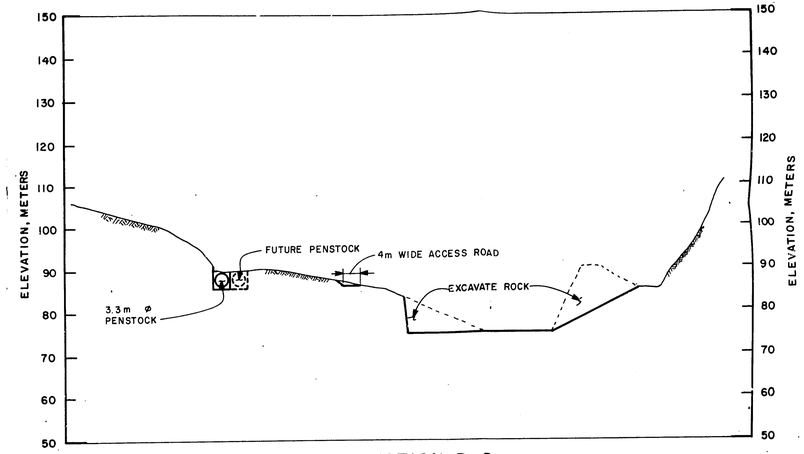


PD-AAT 450
2 of 6





SECTION A-A



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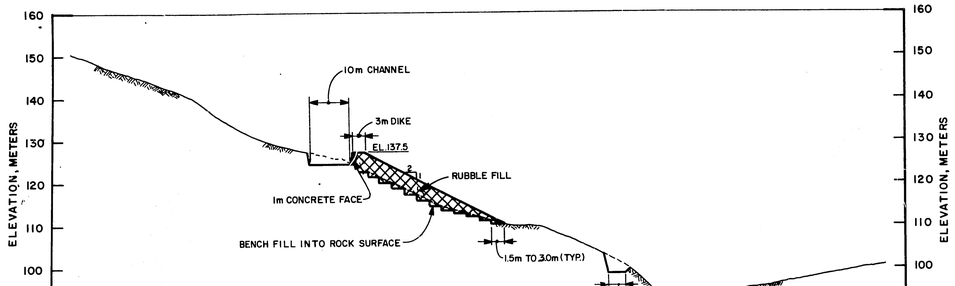
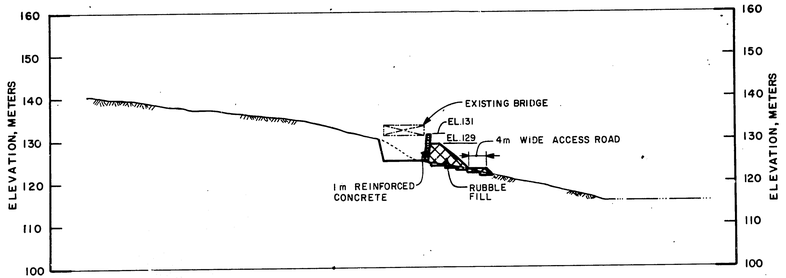
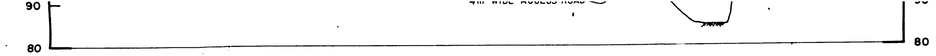
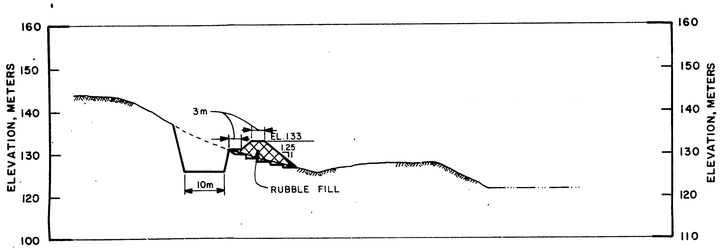




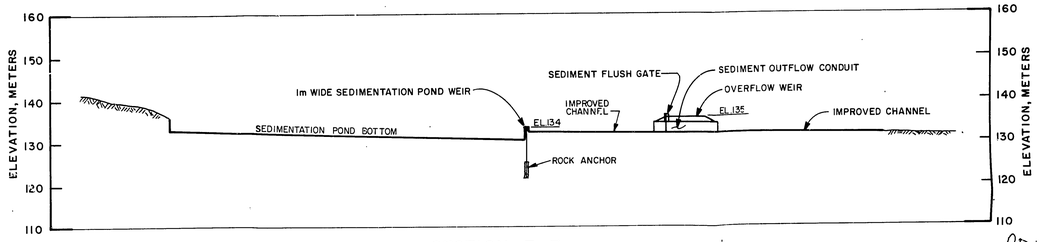
SECTION D-D



SECTION C-C



SECTION E-E



SECTION F-F

PDAAT450
3 of 6

JIRAMA

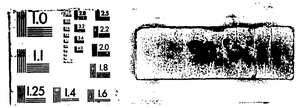
JIRO SY RANO MALAGASY

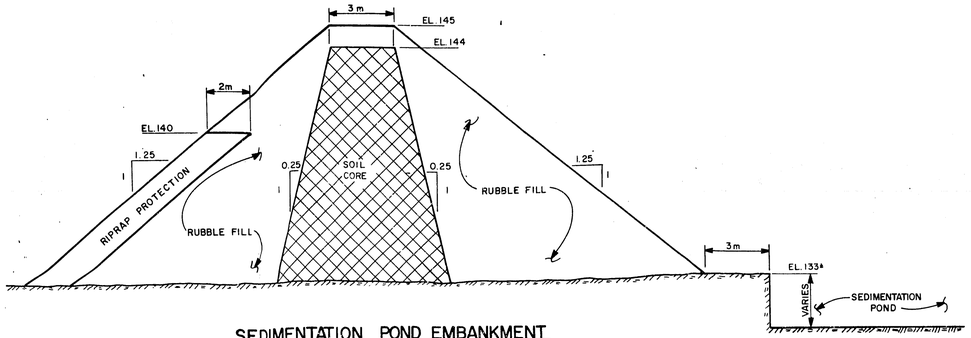
SECTIONS
AMBODIROKA PREFEASIBILITY STUDY

PREPARED BY

STS ENERGENICS LTD.
WASHINGTON D.C., USA

DRAWN BY	dam	4/1/86	SCALE 1:50	FIGURE NO. 2
CHECKED BY			DRAWING NO. 861019-A04	
APPROVED BY				



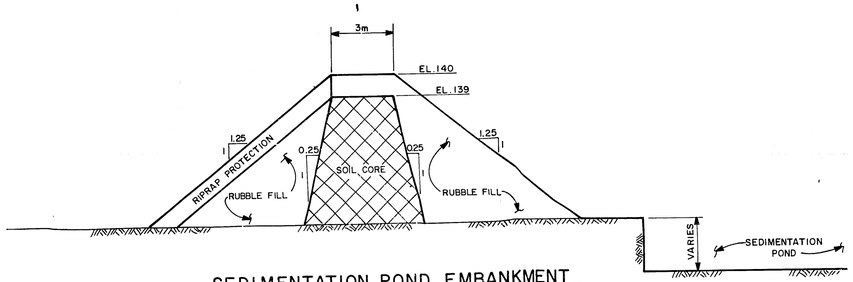


SEDIMENTATION POND EMBANKMENT

1000 YEAR RECURRENCE INTERVAL

STORM PROTECTION SCHEME

SCALE 1:100

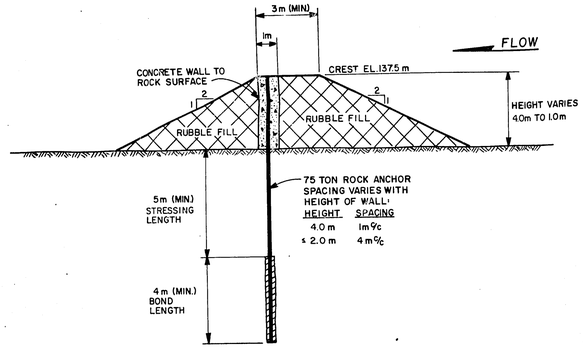


SEDIMENTATION POND EMBANKMENT

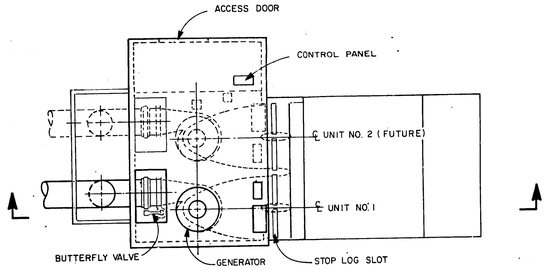
10 YEAR RECURRENCE INTERVAL

STORM PROTECTION SCHEME

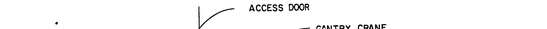
SCALE 1:100

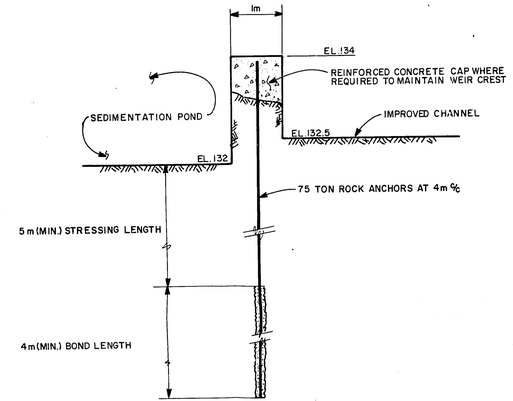


TYPICAL SECTION DIVERSION EMBANKMENT



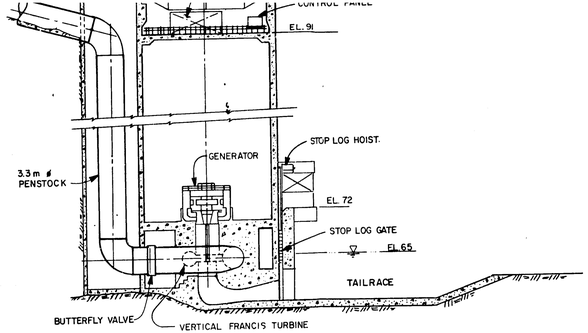
PLAN





TYPICAL SEDIMENTATION POND WEIR SECTION

SCALE 1:40



ELEVATION

POWER HOUSE DETAIL

SCALE 1:20

PDAA-450
407 6

JIRAMA

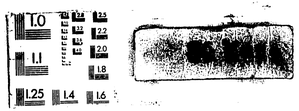
JIRO SY RANO MALAGASY

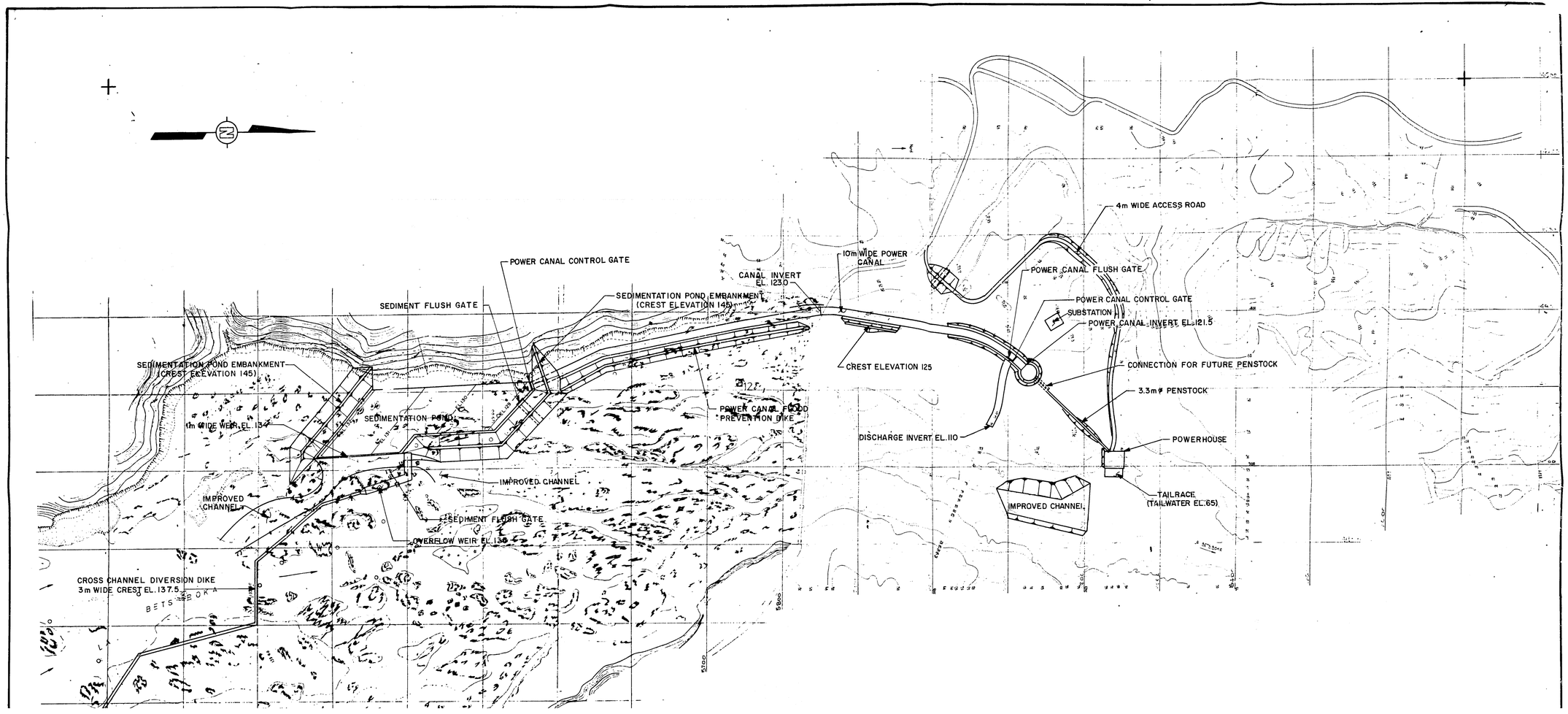
DETAILS
AMBODIROKA PREFEASIBILITY STUDY

PREPARED BY

STS ENERGENICS LTD.
WASHINGTON D.C., USA

DRAWN BY	rjc	4/2/86	SCALE AS SHOWN	FIGURE NO. 3
CHECKED BY	SDB	4/14/86	DRAWING NO. 861019-A05	
APPROVED BY	RJS	4/16/86		




POWER CANAL CONTROL GATE
SEDIMENT FLUSH GATE
SEDIMENTATION POND EMBANKMENT (CREST ELEVATION 145)
CANAL INVERT EL. 123.0
10m WIDE POWER CANAL
4m WIDE ACCESS ROAD
POWER CANAL FLUSH GATE
POWER CANAL CONTROL GATE
SUBSTATION
POWER CANAL INVERT EL. 121.5
SEDIMENTATION POND EMBANKMENT (CREST ELEVATION 145)
1m WIDE WEIR EL. 134
SEDIMENTATION POND
CREST ELEVATION 125
CONNECTION FOR FUTURE PENSTOCK
3.3m ø PENSTOCK
POWER CANAL FLOOD PREVENTION DIKE
DISCHARGE INVERT EL. 110
POWERHOUSE
IMPROVED CHANNEL
IMPROVED CHANNEL
SEDIMENT FLUSH GATE
OVERFLOW WEIR EL. 136
IMPROVED CHANNEL
TAILRACE (TAILWATER EL. 65)
CROSS CHANNEL DIVERSION DIKE
3m WIDE CREST EL. 137.5
BETS'EBOKA
5900
5700

SCALE

0 50 100 150 METERS

PD AAT 450

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JIRAMA

JIRO SY RANO MALAGASY

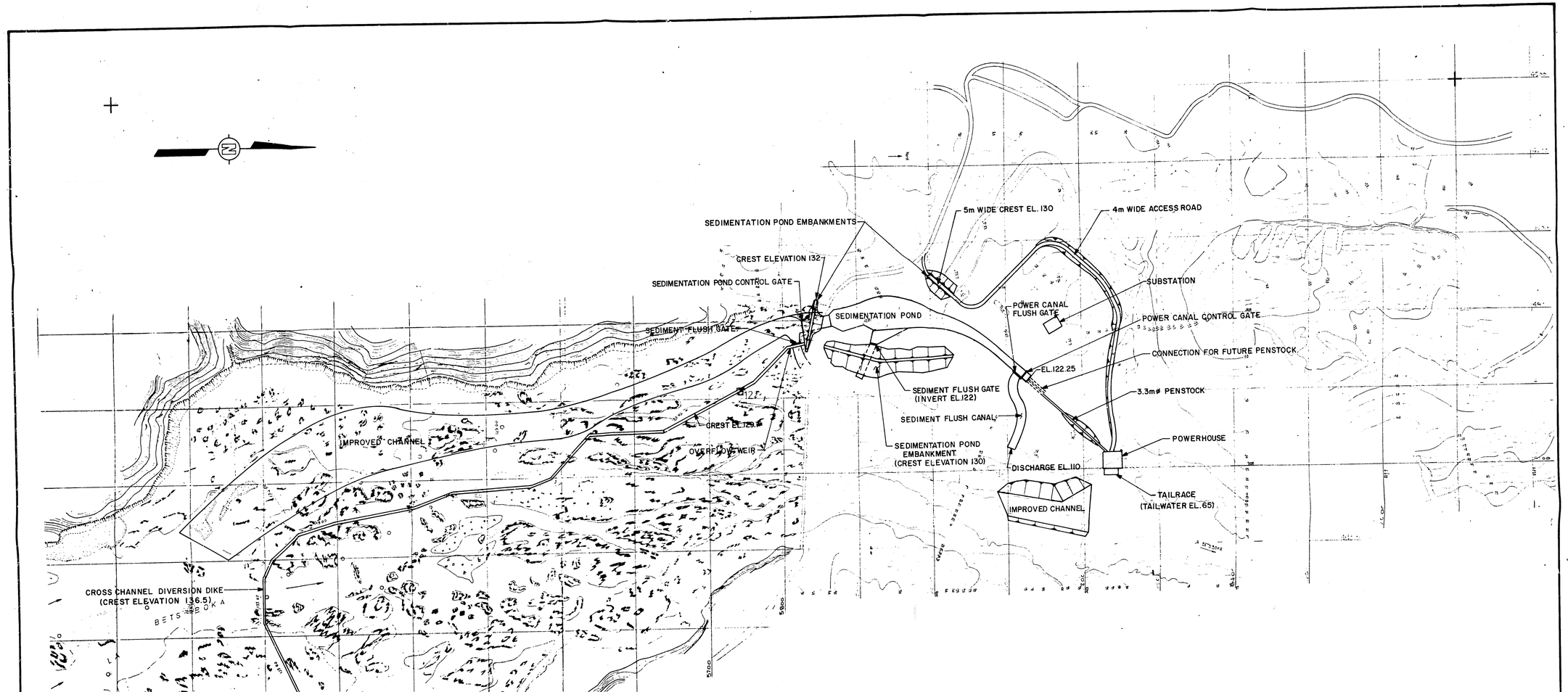
ALTERNATIVE No.2
LEFT BANK DEVELOPMENT
1000 YEAR PROTECTION SCHEME
AMBODIROKA PREFEASIBILITY STUDY

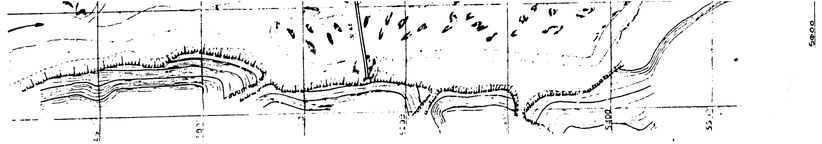
PREPARED BY

STS ENERGENICS LTD.
WASHINGTON D.C., USA

DRAWN BY	dam	4/2/86	SCALE 1:2000	FIGURE NO. 4
CHECKED BY			DRAWING NO. 861019-A02	
APPROVED BY				

1.0 1.1 1.25 1.4 1.6 2.2 2.0 1.8


SEDIMENTATION POND EMBANKMENTS
CREST ELEVATION 132
SEDIMENTATION POND CONTROL GATE
SEDIMENT FLUSH GATE
SEDIMENTATION POND
5m WIDE CREST EL. 130
4m WIDE ACCESS ROAD
SUBSTATION
POWER CANAL FLUSH GATE
POWER CANAL CONTROL GATE
CONNECTION FOR FUTURE PENSTOCK
EL.122.25
3.3mø PENSTOCK
SEDIMENT FLUSH GATE (INVERT EL.122)
SEDIMENT FLUSH CANAL
SEDIMENTATION POND EMBANKMENT (CREST ELEVATION 130)
POWERHOUSE
DISCHARGE EL.110
TAILRACE (TAILWATER EL.65)
IMPROVED CHANNEL
CREST EL.129.5
OVERFLOW WEIR
IMPROVED CHANNEL
CROSS CHANNEL DIVERSION DIKE (CREST ELEVATION 136.5)
BETSIBOKA
5800
5700



SCALE

0 50 100 150 METERS

6 of 6

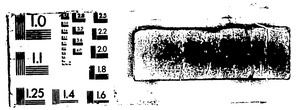
PDAAT-450

JIRAMA

JIRO SY RANO MALAGASY

ALTERNATIVE No. 3
LEFT BANK DEVELOPMENT
SEDIMENTATION CONTROL DOWNSTREAM OF BRIDGE
AMBODIROKA PREFEASIBILITY STUDY

PREPARED BY

STS ENERGENICS LTD.
WASHINGTON DC., USA

DRAWN BY	dom	4/2/86	SCALE 1:2000	FIGURE NO. 5
CHECKED BY	JPB	4/3/86	DRAWING NO.	
APPROVED BY	RJS	4/3/86	861019-A03