FEASIBILITY STUDY

ON

SMALL-SCALE POWER PLANTS

REHABILITATION PROJECT

IN

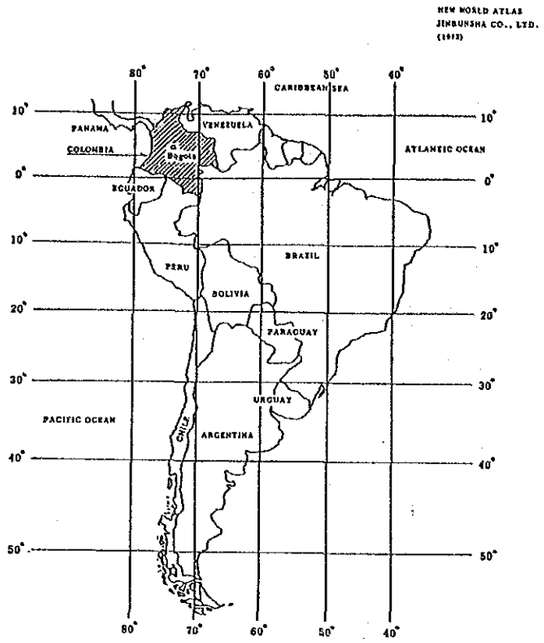
THE REPUBLIC OF COLOMBIA

LA VUELTA HYDROELECTRIC POWER PLANT

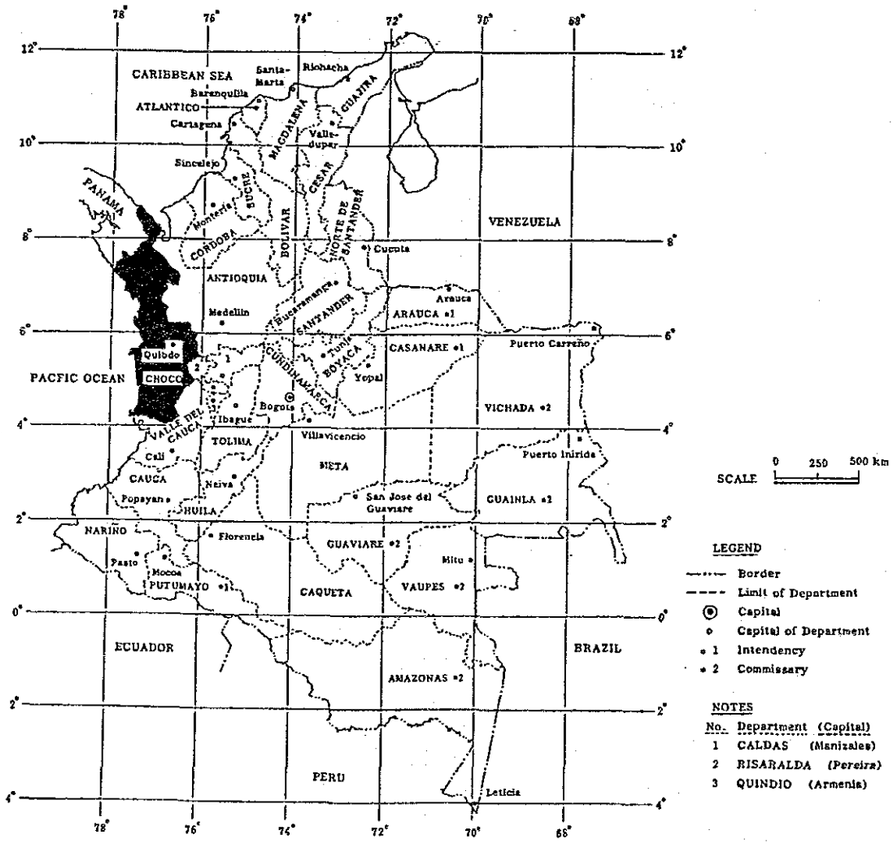
MARCH 1990

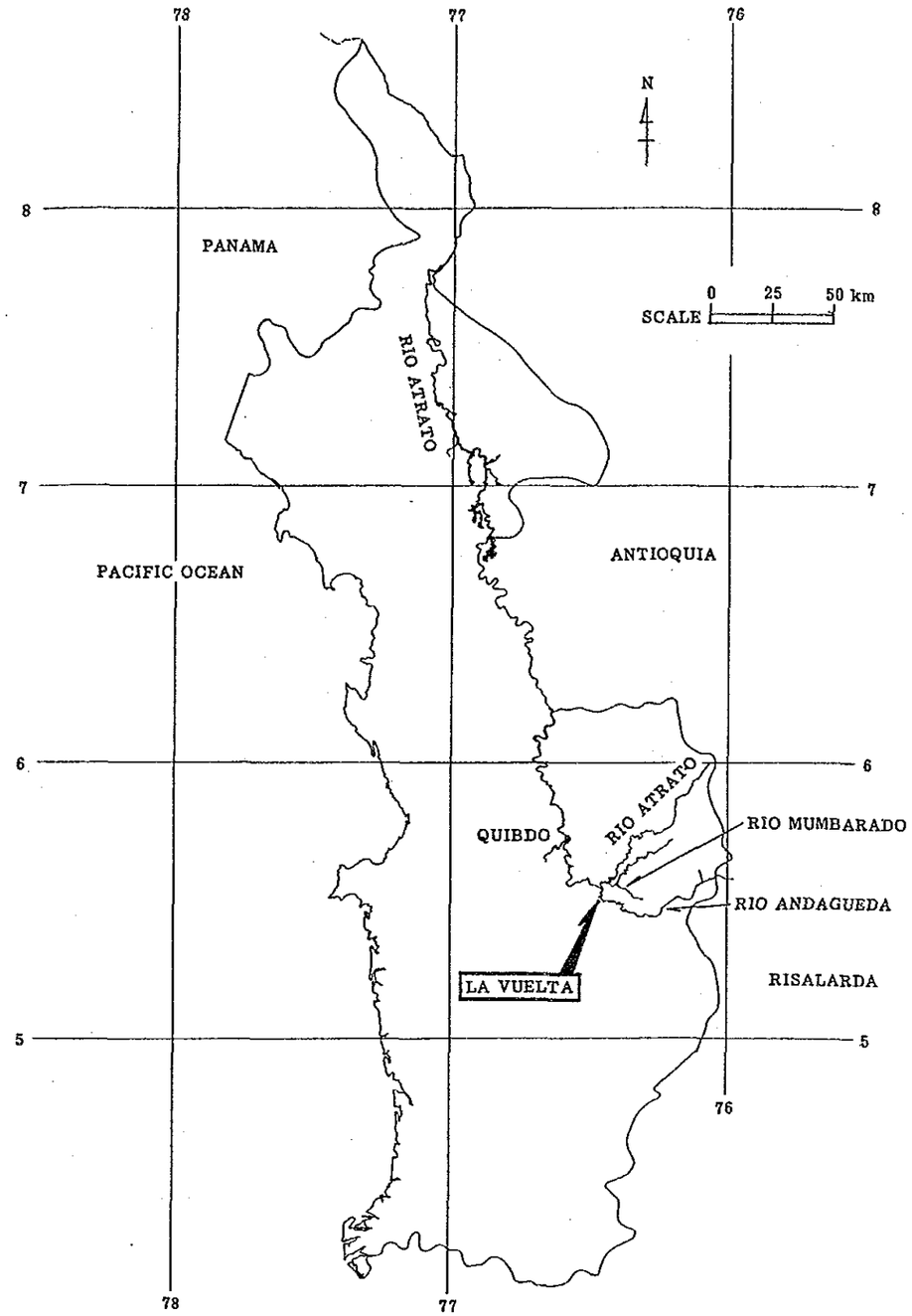
Japan International Cooperation Agency

MAP OF SOUTH AMERICA

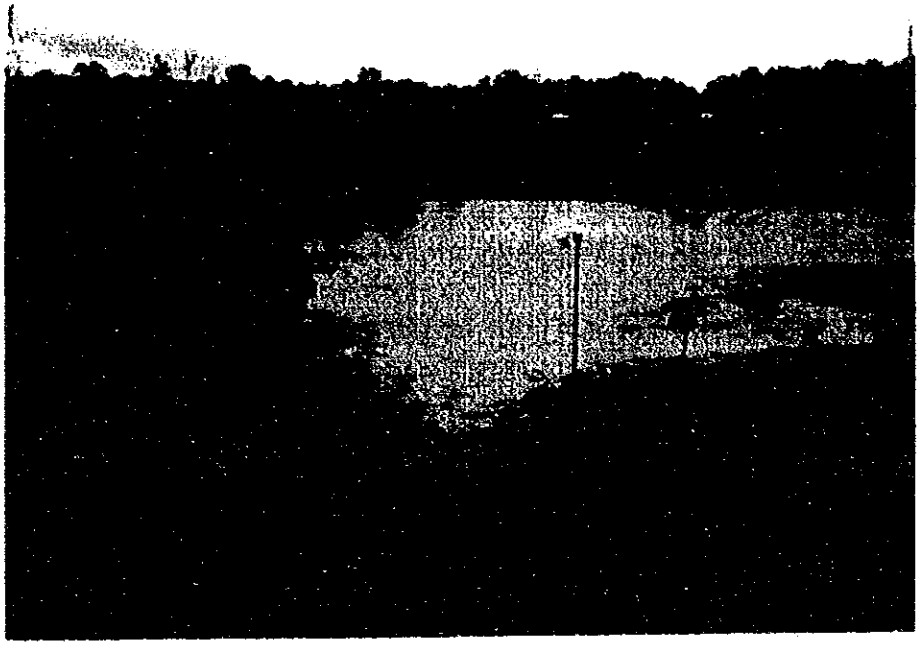


POLITICAL DIVISION IN THE REPUBLIC OF COLOMBIA

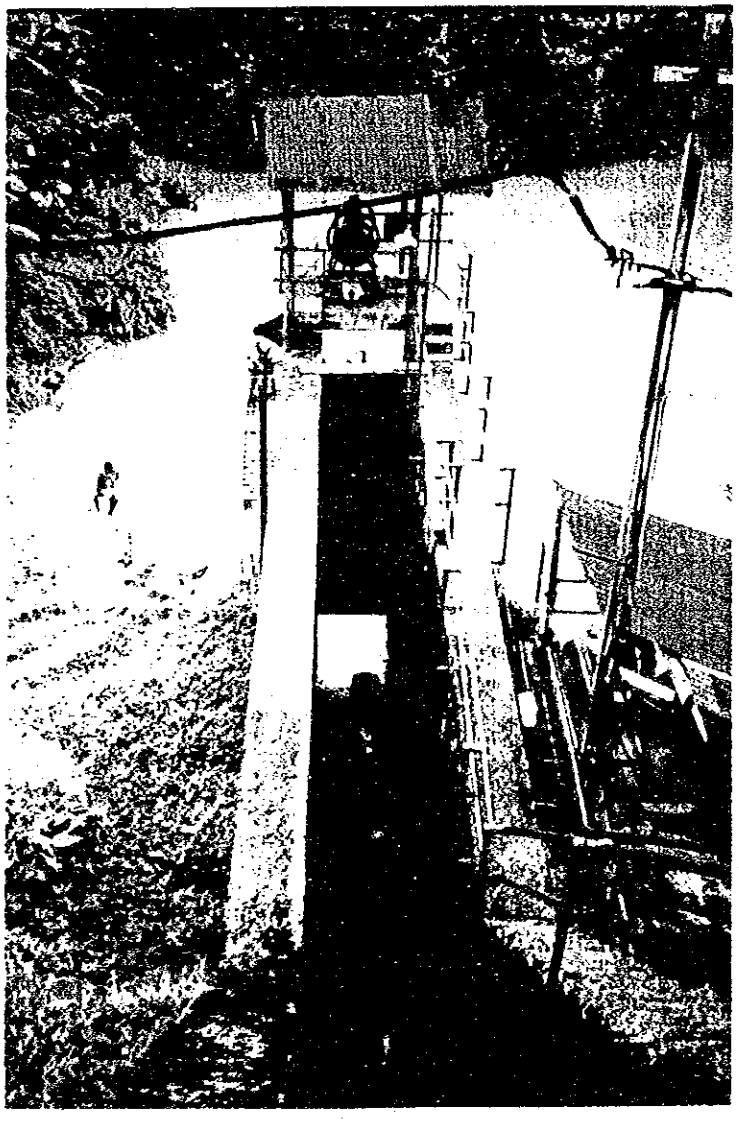




Location Map of Study Area

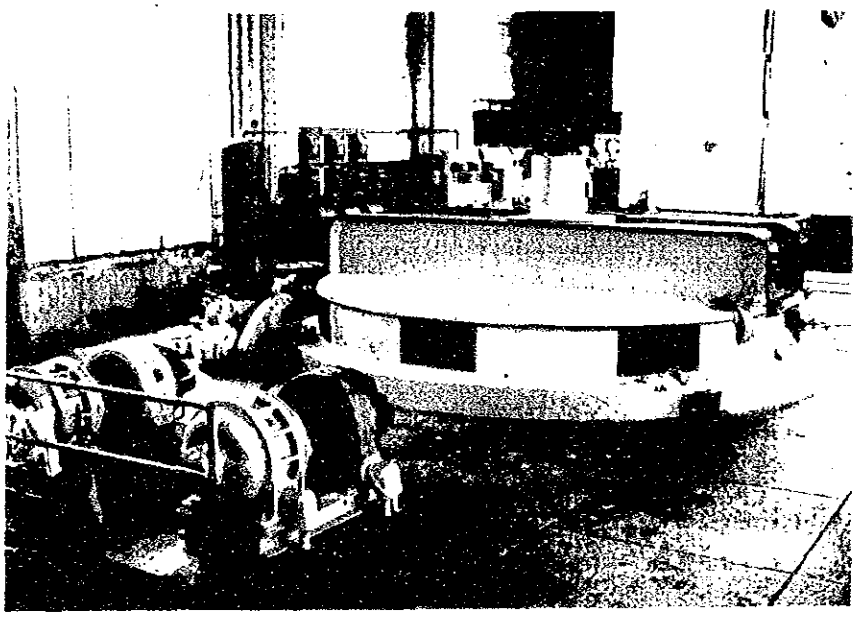


Rio Andagueda and Intake

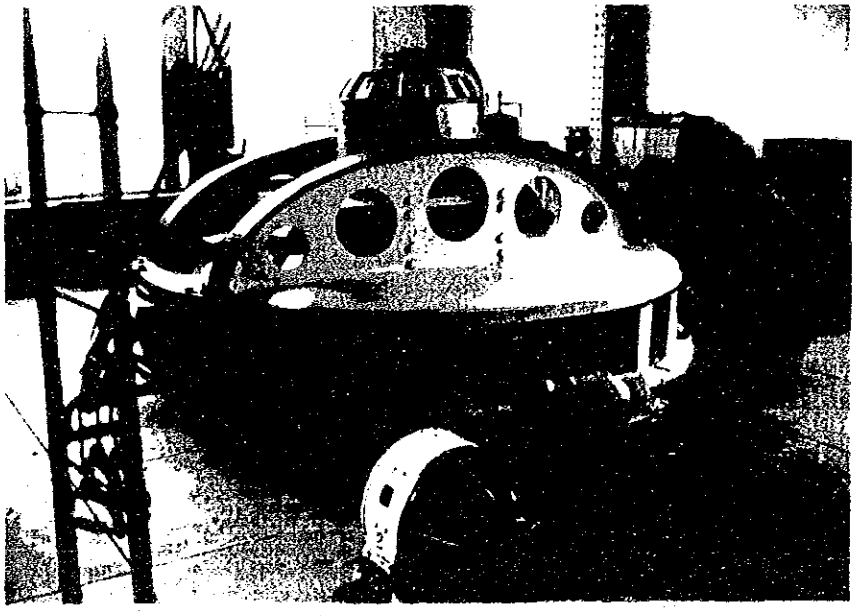


Navigation lock

Powerhouse



Vertical axis
Francis turbine



Vertical axis
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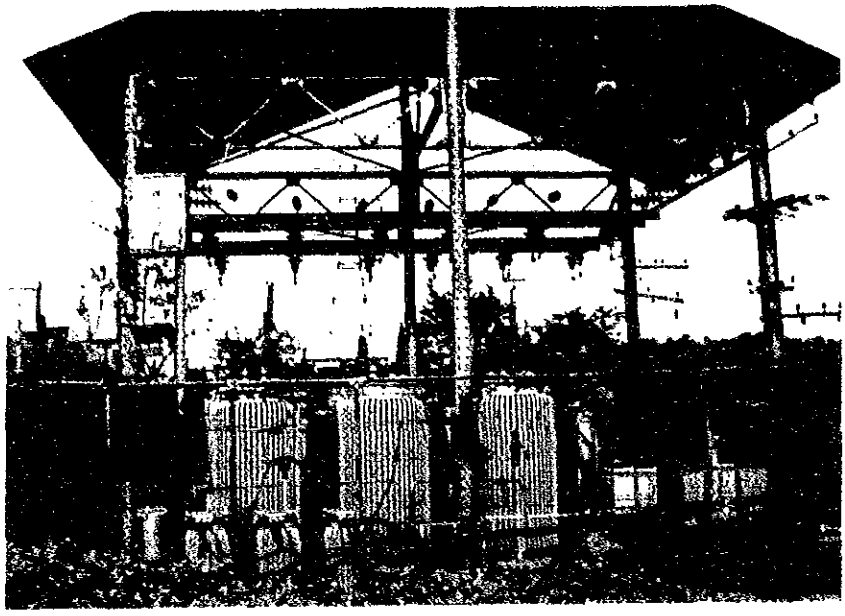

Substation

Location Map of Study Area
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CHAPTER 1 INTRODUCTION

The feasibility study (hereinafter referred to as the FS) for the rehabilitation plan of La Vuelta run-of-river type hydroelectric power plant (rated output of 2 mW) was conducted following the pre-FS that was carried out for eight months from November, 1987 to June, 1988. This report is prepared to summarize the results of the FS.

This FS was performed in accordance with the Scope of Work (S/W) agreed and signed in July 1988 between Japan International Cooperation Agency (JICA) and Instituto Colombiano de Energia-Electrica (ICEL). The study period was 17 months from November, 1988 to March, 1990.

From among 62 small-scale hydroelectric power plants operated by ICEL that were nominated for the study of the rehabilitation plan, La Vuelta hydroelectric power plant (hereinafter referred to as La Vuelta P/P) was selected as a candidate for the FS for the following reasons:

1) Discharge observation data is comparatively well prepared.

2) Complete drawings of the existing facilities and buildings exist.

3) Demand for electricity is expected to grow from the promotion of the regional development plan.

4) Metares Presiosos del Choco S.A., owner of the power plant, and has agreed to cooperate ain the expansion of the generating capacity of the plant.

From this FS, post-rehabilitation generating capacity for which JICA Study Team proposes as an optimum rehabilitation plan is as follows:

- Maximum output : 7.7 MW
- Annual probable generated power : 65.7 GWh
- Facility utilization factor : 96 %

CHAPTER 2 SUMMARY OF STUDY RESULTS

The power plant owned by the mining firm Choco, is the run-of-river type hydroelectric power plant with a maximum rated output of 2000 kW, built along the Andagueda river in Choco Department. Since its construction in 1916, it has been in operation for 74 years. However, the maximum output of the plant has decreased to 500 kW. The registered annual generated power for 1988 was 2,364 MWh.

(1) Present condition of generating facilities and their problems

This hydroelectric power plant harnesses the Andagueda river, which winds through the geography, generating power from a low head devised by shortcutting the bends in the river.

The diversion weir built at the time of construction of the power plant had been washed away and the facilities in the forebay were also destroyed. The present diversion weir was built by laying wire over the river 130 m downstream of the intake site and hanging the lumber fence, "trincho", which is made of special wood of a greater specific gravity than the wire. The intake, built square to the river stream, is an unlined open channel 15 - 35 m wide, 78 m long and 4 m deep in the water. In front of the power generating building, there is a regulating gate and a screen for the intake. The highest water lever ever registered at the time of flood was 75 feet at the hydrological gauge before the intake.

Although the design of the powerhouse is bulky, and the capacity of the power generation equipment obsolete, the structure itself is well maintained. Upstream from the powerhouse, there is a navigation lock for canoes which is still in operation. The difference in water level of the navigation lock indicates the gross head available for the power plant. The mean gross head registered from January through September, 1921 was 14 ft (4.3 m).

Two vertical shaft-type Francis turbines, manufactured in 1915 and 1930, provided a rated output of 1000 kW each at installation. However, after 60 years of use, output has decreased to the current level of 25% (500 kW combined) of the rated output.

(2) Alternative rehabilitation plans

From the flow duration curves at the intake point shown in Fig. 2.1, the current available discharge (Q = 54 m^3/s) can be increased to 100 m^3/s in the generation plan for this power plant. Therefore, two alternative plans utilizing the maximum available discharge of 50 m^3/s and 100 m^3/s are considered.

Since the existing turbines are of an obsolete Francis-type no longer manufactured, they will have to be replaced with new generating equipment. However, since generating equipment is not standardized, it is impossible to install replacements within the existing building. Even if installation were possible, it would be difficult to make estimates for the installation work. Therefore, this study will be confined to the conception of a new layout plan for a site adjacent to existing facilities. Although the rehabilitation cost for the washed-away diversion weir accounts for a relatively high percentage of the total cost of the rehabilitation plan, the reconstruction of the diversion weir in reinforced concrete to increase the head must be taken into account.

There are two alternatives to the existing power plant rehabilitation plan, as shown in Table 2.1. The first calls for the restoration of TRINCHO at its existing site; the second recommends the rebuilding of the reinforced concrete diversion weir.

Fig-2.1 TYPICAL FLØW DURATIØN CURVE AT INTAKE SITE

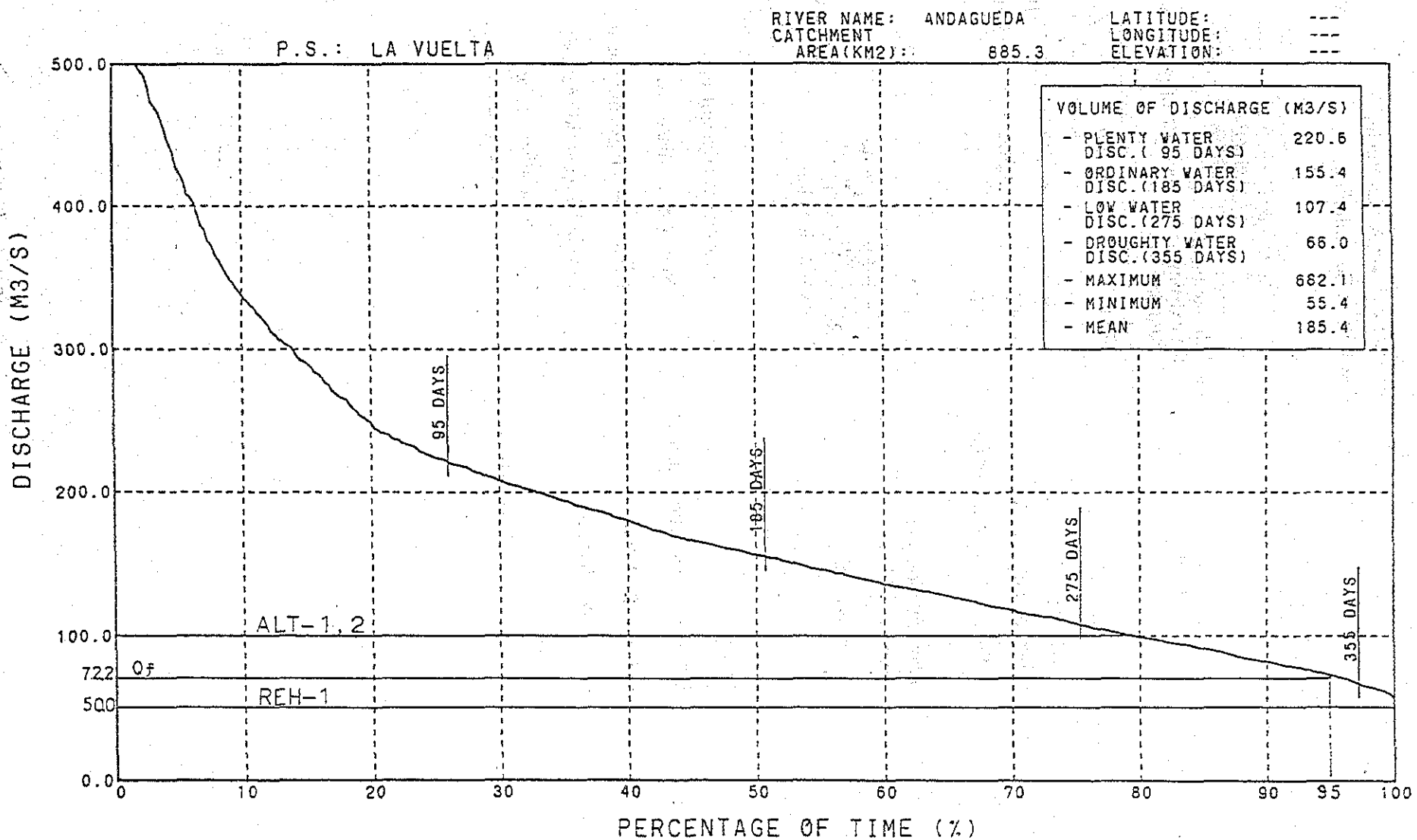


Table 2.1 Alternative Plans for La Vuelta Power Plant Rehabilitation

Item	Alternative		
	Rehabilitation of the existing facilities	Increase of power output	
	REH-1	ALT-1	ALT-2
Discharge, Q (m^3/s)	50	100	100
Max. output, P (kW)	1,700	3,500	7,700
Facility utilization factor (%)	100	96	96
Rehabilitation and improvement plan:			
Diversion weir	Restore TRINCHO at existing location		Renovate the TRINCHO with reinforced concrete
Forebay	New one at adjacent site		
Intake and conduction channel	New one at adjacent site		
Generating equipment	Replace with new equipment		
Powerhouse building	New building at adjacent site		
Tailrace	New one at adjacent site		

(3) Selection of optimum plan

Comparative study results of alternative plans are summarized in Table 2.2. ALT-2, designed to double the available discharge from 54 m^3/s to 100 m^3/s and increase the head by renovating the diversion weir into the reinforced concrete dam, is relatively advantageous. However, the following items need to be surveyed to assure the feasibility of ALT-2.

1) A geological survey of the bedrock at the site of the concrete intake dam and of the soil conditions of the terrace on the left bank of the river have not been made.

2) The scope of impact of the backwater created by the dam have not been confirmed.

3) A survey of the compensation for housing, agricultural fields and forestry to be submerged under water has not been made.

Therefore, the basic design for the feasibility study at this stage is based on ALT-1, and is explained in Chapter 11.

The feasibility of the rehabilitation of the La Vuelta power plant can not be considered solely from the standpoint of profitability for the following reasons.

(1) Since this power station utilizes a small head, a relatively expensive tubular turbine generator is required.

(2) Most of the facilities in the rehabilitation plan call for new construction.

(3) Since this site is remote, transportation and construction costs will be relatively high.

Table2.2 Comparison of Rehabilitation Plan for the La Vuelta Power Plant

Alternative Plan	① Specifications for Existing Generating Facilities					② Rehabilitation Plan							③ Recovered or Increased Energy	
	⑩	⑪	⑫	⑬ Present facility capacity		⑳	㉑	㉒	㉓	㉔	㉕	㉖	㉚	㉛
	Max. available discharge Q_o (m^3/s)	Net head H_o (m)	Rated output P_o (kW)	⑭ Output P_e (kW)	⑮ Generated energy E_e (GWh)	Max. available discharge Q_1 (m^3/s)	Standard net head H_1 (m)	Theoretical output =9.8x⑳x㉑ (kW)	Resultant efficiency η	Output =㉒x㉒ P_1 (kW)	Annual probable generated energy E_1 (GWh)	Facility utilization factor ε (%)	Output =㉔-⑭ ΔP (kW)	Annual probable generated energy ㉓-⑮ ΔE (GWh)
REH-1	54.0	4.8	2,000	500	6.25	50.0	4.4	2,156	0.815	1,700	15.4	100	1,200	9.1
ALT-1						100.0	4.4	4,312	0.823	3,500	29.9	96	3,000	23.6
ALT-2						100.0	9.65	9,457	0.823	7,700	65.7	96	7,200	59.4

Alternative Plan	④ Rehabilitation Work Cost (US$1000)					⑤ Construction Cost per kW (US$/kW)		⑥ Total of Annual Cost at Generating Terminal (US$1000)					⑦ Average Generating Cost per kWh (mills/kWh)		⑧ Cost/Benefit	⑨
	㊵ Generating Equipment Cost			㊹	㊸	㊿	(51)	(60)	(61) Principal repayment amount for construction cost (25-year average)			(65)	(70)	(71)		
	(41) Foreign currency portion C_1f	(42) Local currency portion $C_1\ell$	(43) (41)+(42) C_1	Civil work cost C_2	(43)+(44) C	Cost per ΔP =(43)/(30) $C/\Delta P$	Cost per P_1 =(43)/(24) C/P_1	Operation and maintenance costs AOM	(62) Foreign currency portion 2.610 x (41) ÷ 25	(63) Local currency portion 2.016 x [(42)+(44)] ÷ 25	(64) (62)+(63)	(60)+(64)	per E_1 =(65)/(25) ÷0.95	per ΔE =(65)/(31) ÷0.95	C/B	Priority order
REH-1	3,950	1,600	5,550	2,410	7,960	6,600	4,700	6.8	414	323	737	744	51	86	4.24	3
ALT-1	5,400	2,150	7,550	3,320	10,870	3,600	3,100	14.0	561	441	1,002	1,016	36	45	2.71	2
ALT-2	7,400	2,950	10,350	9,700	20,120	2,800	2,600	30.8	772	1,026	1,798	1,829	29	32	2.29	1

(Notes)

① : For the existing generating equipment specifications, refer to the facility register record attached to the pre-FS report.

⑦ : Generating cost = $\frac{\text{Total of annual average cost at generating terminal}}{\text{Annual average supplied electric power}}$

③ : C/B is the value of cost and benefit ratio calculated according to the financial analysis.

⑬ : Ee is computed according to the average annual operation record for 5 years from 1984 to 1988.

㉒ : η is the resultant efficiency of turbine and generator.

㉕ : E_1(Energia Media)

㉖ : $\varepsilon = \frac{\text{Annual water amount for turbine } (m^3/s \cdot hr)}{Q_1 \times 365 \times 24} \times 100(\%)$

㊿ : The annual AOM is the amount which is equivalent to US$4 per kW.

(61) : Interest is calculated by a repayment of principal in equal annual amounts under the following conditions.

Foreign currency portion: Annual interest rate of 10%, unredeemable for 4 years, repayment over 25 years

Local currency portion : Annual interest rate of 21%, unredeemable for 1 year, repayment over 8 years

CHAPTER 3 STUDY PLAN

3.1 Organization of Study Team

3.1.1 JICA FS Study Team

JICA FS Study Team, listed below, includes the team leader and two members who participated in the pre-FS, engineers, geologists, a hydrologist and an economist.

Name	Position	Assignment
Masami Ono	Team leader	Total coordinator (civil engineer)
Murao Toyama	Team member	Power generation planner (civil engineer)
Susumu Nonaka	"	Hydrologist
Yoshio Kawasaki	"	Generating equipment planner (civil engineer)
Akira Takahashi	"	Generating equipment planner (mechanical engineer)
Masayuki Tamai	"	Generating equipment planner (electrical engineer)
Nobuhiko Uchiseto	"	Geologist
Takashi Inoue	"	Geologist
Masaaki Ueda	"	Economist

3.1.2 Counterpart Engineers from ICEL

Engineers who were engaged in this study as counterparts of the JICA FS Study Team are as follows:

Name	Field	Position
Juvenal Peñaloza Rosas	Civil Engineering	Head of Central Eng. Div.
Jairo E. Gonzalez Morales	Civil Engineering	Central Eng. Div.
Mario Gutierrez Ospina	Civil Engineering	Central Eng. Div.
Rafael Torres Mariño	Civil Engineering	Central Eng. Div.
Rafael Gomez Florez	Civil Engineering	Central Eng. Div.
Jorge E. Hurtado Muños	Civil Engineering	Central Eng. Div.

3.1.3 Supporting Technical Staff from E. CHOCO

JICA FS Study Team obtained cooperation and support from the following technical staff in conducting the site reconnaissance, collecting data and performing engineering consultation necessary for this study.

Supporting Staff	Position
Juan B. Hinestroza C.	President
Jose Wilson Guerrero	Chief of Planning Office
Jose Antonio Correa M.	Electrical Engineer
Luz Elba Gonzalez	Electrical Engineer
Carlos Osorio Molina	Manager of "Metales Preciosos del CHOCO"
Juan Ramon Gilabert	Chief of La Vuelta Power Plant "Metales Preciosos del Choco"

3.2 Study Items and Study Schedule

This FS was conducted for 17 months from November, 1988 to March, 1990 in accordance with S/W agreed and signed in July, 1988 between JICA and ICEL.

3.2.1 Study Items

Study items for the FS as defined by the S/W include the following:

(1) Review of existing data
(2) Site reconnaissance
(3) Field work
    1) Topographic survey
    2) Photogrammetric mapping
    3) Geological investigation
    4) Data collection
(4) Power survey
(5) Optimum plan
(6) Feasibility design
(7) Stability and safety analyses
(8) Construction method
(9) Cost estimation
(10) Economic and financial analyses
(11) Maintenance manual

3.2.2 Study Schedule

The overall study schedule as indicated in the S/W is shown in Table 3.1.

Table 3.2 Field Survey Schedule

The first site reconnaissance

Date	Schedule	Detail of Study Item	Member	
			ICEL	JICA
Feb. 15	Bogota→Quibdo →La Vuelta	Field survey at La Vuelta P/P	J. Morales	Masami Ono Yoshio Kawasaki
Feb. 16		Discussion at E. CHOCO and data collection		
Feb. 17	Quibdo→Bogota			

The second field survey

Date	Schedule	Detail of Study Item	Member	
			ICEL	JICA
Jun. 27	Bogota→Quibdo →La Vuelta	Field survey at La Vuelta P/P	J. Morales	Murao Toyama Nobuhiko Uchiseto
Jun. 28				
Jun. 29	La Vuelta→Quibdo	Discussion at E. CHOCO		

3.3 Detail of Field Survey Work

The field survey work planned upon consultation between JICA Study Team and ICEL counterpart staff according to the results of the site reconnaissance includes topographic surveying and boring survey as described below, but does not include photogrammetric mapping.

3.3.1 Scope of Topographic Surveying

The scope of topographic surveying is shown in Figure 3.1. The scale For topographic maps is as follows:

(1) Diversion weir and river bed

The current conditions are drawn on a scale of 1/200 (longitudinal and cross sectional view).

(2) Bench mark

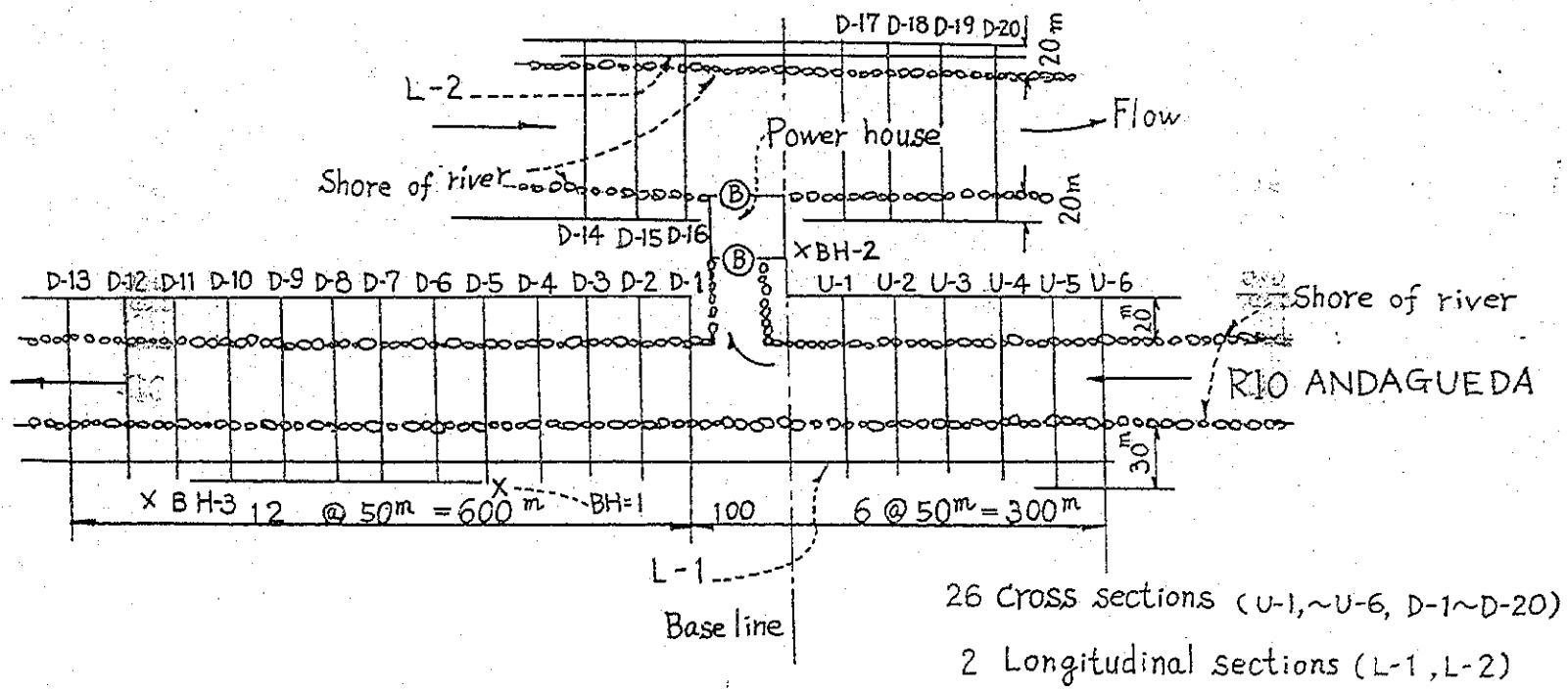
The bench marks will be set up at two locations.

3.3.2 Boring Survey Work Plan

The boring surveys were to be conducted as follows:

No.	Location	Depth	Note
BH-1	Right-hand shore of River	10 m	The location of boring holes is shown in Fig. 3.1.
BH-2	Powerhouse building	10 - 20 m	
BH-3	Left-hand shore of River	10 m	

However, the boring surveys were not executed.



(Legend)

(B): Bench Mark

BH: Boring Hole

Fig. 3.1 Scope of Topographic Survey

CHAPTER 4 PRESENT CONDITION OF THE STUDY AREA

4.1 Power Conditions in the Power Sector

Power conditions in the power plant owned by the public electric power company which is studied for rehabilitation are described below.

4.1.1 Balance of Power Supply and Demand

Table 4.1 shows the figures for supply and demand in the past five years (from 1983 to 1987). In 1987 peak demand was 18 MW, requiring the purchase of electricity (63.2 GWh) to meet the demand of 56.8 GWh.

The breakdown of power demand in 1987 indicates that power demand for residential use was highest at 69%, while lowest for industrial use at 6%.

The annual average rate of increase in power demand from 1983 to 1987 was 26.6%, and the rate of buying electricity increased accordingly.

Table 4.1 Transition of Power Supply and Demand
(1983 - 1987)

Item	1983	1984	1985	1986	1987	Annual Average Increase Rate(%) *
DEMAND						
1. Peak Demand (MW)	7.1	10.4	10.5	8.6	18.0	26.2
2. Electric Energy (GWh)						
1) Residential	15.5	19.9	25.2	32.0	39.4	26.3
2) Commercial	3.2	3.9	4.4	5.2	5.6	15.0
3) Industrial	0.8	0.8	2.2	1.9	3.5	44.6
4) Miscellaneous	2.6	3.1	5.2	6.2	8.3	33.7
Total	22.1	27.7	37.0	45.3	56.8	26.6
SUPPLY						
1. Installed Capacity (MW)	0	0	0	0	0	0
2. Generated Energy (GWh)	0	0	0	0	0	0
3. Power Loss (GWh)	10.7	13.2	10.7	8.2	6.4	-12.1

(Source: INFORME ESTADISTICO: RESUMEN 1983-1987)

* Annual average increase rate is calculated as follows:

Example: When peak demand is 26.2%, $7.1 \times (1 + x)^4 = 18.0$

$x = 0.262$ (26.2%)

Electricity is supplied to these areas by La Vuelta Power Plant and with imports from E. CHOCO which are distributed by Metales Preciosos del Choco.

Table 4.2 Current Electricity Demand in the Vicinity of La Vuelta Power Plant

City	Number of Consumers	Demand (KVA)
Lloro	400	262.5
Istmina	337	3,189.0
Novita	75	50.0
Total	812	3,501.5

(Source: E. CHOCO)

4.1.2 Present Conditions of Generating Facilities

(1) Generating facilities

There is no power plant owned by the public power company.

However, the present conditions of the power plants of F/S are as shown in Table 4.3.

Table 4.3 Conditions of La Vuelta Power Plant (1985)

Item	1985
1) Installed capacity (kW)	2,000.0
2) Generated energy (MWh)	4,602.0
3) Utilization factor (%)	26.0
4) Operating time (%)	74.0

(Source: E. CHOCO)

(2) Transmission facilities

115 kV transmission lines are provided for E. CHOCO. 33 kV transmission lines are provided for the La Vuelta P/P.

4.1.3 Generating Cost and Electric Charges

Table 4.4 lists the changes of generating cost and electric charges from 1983 to 1987.

Table 4.4 Generating Cost and Electric Charges

Item	1983	1984	1985	1986	1987	Annual Average Increase Rate(%)
Generating Cost (COL$/kWh)	3.92	4.40	4.17	5.04	5.80	10.3
Electric Charge (Average): (COL$/kWh)						
1. Residential	3.03	3.04	3.32	4.30	5.25	14.7
2. Commercial	4.04	4.20	6.38	8.17	10.31	26.4
3. Industrial	3.62	3.67	6.28	7.78	11.44	33.3
4. Public use	3.66	3.74	5.02	6.54	8.53	23.6
5. Average	3.21	3.25	3.96	5.03	6.78	20.6
Breakdown of Power Demand by Customers						
1. Residential	6,637	8,167	9,713	10,994	12,319	16.7
2. Commercial	784	926	998	1,110	1,201	11.3
3. Industrial	52	68	90	105	159	32.2
4. Others	177	216	232	266	284	12.5
5. Total	7,650	9,377	11,033	12,475	13,963	16.2
Diffusion of Electricity						
1. Overall (1,000 households)	235	239	243	247	250	1.6
2. Power demand (1,000 households)	30	37	44	49	55	16.4
3. Electrification rate (%)	13	15	18	20	22	14.1

(Source: INFORME ESTADISTICO: RESUMEN 1983-1987)

4.1.4 Forecast of Power Demand

E. CHOCO forecasts power demand for the vicinity of La Vuelta until 2000, as shown in the following table.

Table 4.5 Forecast of Power Demand in the Vicinity of La Vuelta Power Plant

Cities	1990		1995		2000	
	Number of Consumers	Demand (KVA)	Number of Consumers	Demand (KVA)	Number of Consumers	Demand (KVA)
Lloro	1,928	1,120.0	2,256	1,635.5	2,623	2,333.5
Istmina	634	3,406.5	714	3,529.0	831	3,809.0
Total	2,562	4,526.5	2,970	5,164.5	3,454	6,142.5

4.2 Operation Record of the Existing Power Plant

4.2.1 Generated Energy

Record of generated energy and operation time are shown in Table 4.6.

Although the utilization factor in 1955 was 79%, it dropped to approximately 30% in the 1980's because of the extended use of the turbines.

Table 4.6 Record of Generated Energy

Year	Output inscribed on the name plate (MW)	Running Period (Hr)	Generated Energy (MWh)	Facility Utilization Factor (%)*	Notes
1955	1MW x 2 units	8,003.6	13,835	79	No record in Jan.
1980	"	7,344.0	5,233	30	No record in Feb. and May
1981	"	5,080.0	4,959	28	No record in Jan., Apr., Jul., Nov. and Dec.
1982	"	6,672.0	4,971	28	No record in Feb., Apr. and Nov.
1985	"	6,504.0	4,602	26	No record in Aug., Sept. and Nov.
1986	"	2,928.0	2,364	13	No record from Feb. through Aug. and Dec.

* Note:

$$\text{Facility utilization (\%)} = \frac{\text{Generated energy (MWh)}}{\text{8760(hr) x output inscribed on the name plate (MW)}} \times 100$$

4.3 General Condition of Generating Equipment and Civil Structures

4.3.1 General Condition of Generating Equipment

The present condition of the generating equipment is summarized below:

(1) Generating equipment

Name plates indicate that No. 1 turbine was manufactured in 1915 and No. 2 in 1920. Defects found in the water turbines and generator from a survey conducted by E. CHOCO are shown in Table 4.7. Although there are no critical defects, E. CHOCO requests new replacements for the equipment because of the significant decrease in turbine output from deterioration over time, as explained in 4.2.1.

Table 4.7 Major Defects in Water Turbine and Generator

Equipment	No. 1 Unit	No. 2 Unit
Water turbine	1) Guide vanes do not close	1) Same as No. 1 unit 2) Casing vibrates
Generator	1) Insulation reinsurance value of coil is lower than STD	2) Same as No. 1 unit
Control panel	1) Inaccurate instruments and protective relay	1) Same as No. 1 unit

(2) Substation

One outdoor 33 kV transformer has been installed. Though there is no significant defect in the transformer, E. CHOCO wants a new replacement since it is obsolete and unreliable.

(3) Transmission line

There are two 51.5 km transmission lines, voltage 33 kV, from the power plant to Andagoya substation. Since no defects were found in the transmission system, rehabilitation is not required.

4.3.2 General Conditions of Civil Structures

(1) Intake facilities

TRINCHO, consisting of wood dams suspended from cables, is located on sandy debris 130 m downstream from the forebay entrance. On the TRINCHO extension, there is a 200 m dam upstream and a 240 m dam downstream, both attached to concrete blocks.

The TRINCHO dam has been constructed with several layers of high-specific-gravity wood, bound by wires and suspended from cables. The upstream of the wood dam has been backfilled with river gravel. The crest elevation of the upstream dam is 78 m, its height, 1.0 m. Excess river discharge flows through or over and damaging the dam, thus repair work is needed every year. Since there is no sand trap, sand and gravel deposits upstream of the TRINCHO dam are usually dredged by the dragline (capacity: 2 m^3/10 minutes) from the left bank of the river.

(2) Forebay

The intake of the forebay is perpendicular to the river flow of the right bank of the river. It has a 13-35 m by 78 m water channel. At the intake there is a 1.5 m high by 35 m long submerged dam to prevent the inflow of sand and gravel. The standing water level of the forebay is 78.90 m. Rocks are exposed at the base of the forebay, as the floor is 75.00 m deep and the standing water depth is 4 m. On the left bank of the forebay, there are a navigation lock (1.79 m wide), pier and waterway.

(3) Intake

Total length of the intake is 26 m and each of the two screened entrances is 11 m wide and 5.8 m tall. The elevation of the river bed is 74.70 m. Three

gates (3 x 6 m each) are installed at each entrance of the intake. At the center of the intake, there is a 2.40 m wide a sand trap channel and another 5.50 m wide by 1.5 m deep channel to prevent inflow of sand and gravels is provided in front of the sand trap.

(4) Power plant and tailrace

The powerhouse building, 32 m by 11 m with a floor elevation of 86.60 m, houses two 1,000 kW Francis turbine units. The layout of the powerhouse building will requires more space since the existing generating equipment and layout design are obsolete. In spite of 70 years since construction, the structures are in a comparatively good condition. There are 4.90 m wide by 2.6 m high trailraces are provided at each turbine. Elevation height of the tailrace floor is 70.10 m and firm tailwater level is 74.70 m.

CHAPTER 5 BASIC DATA COLLECTION

Pre-FS was conducted from November, 1987 to July, 1988. In succession, FS was carried out in November, 1988 to collect topographical, geological, hydrometeorological and other related data as detailed below:

5.1 Topographic Maps

The Rio Andegueda has its fountainhead at Cerro Caramanta flows through Bagado and merges with the Rio Atrato at Lloro. La Vuelta P/P is located on the right bank of the river 10 km upstream from the merging point with the river Atrato where the river winds greatly.

The collected data on the topography in the site are maps with scales of 1/25,000 and 1/500,000 issued by IGAC, and topographic maps measured by E. CHOCO, and the as-built drawings owned by Metales Preciosos del Choco.

(1) Topographic maps published by IGAC

Scale	Drawing No.	Description
1/500,000	-	the whole area of Choco Department
1/100,000	184	} Vicinity of P/P
1/ 25,000	186	} Vicinity of P/P
	184-IV-B	Power plant
	185-III-A&C	Upstream of P/P
	185-I-C	Downstream of P/P

(2) Topographic maps actually measured by E. CHOCO

Topographic survey maps actually measured by E. CHOCO from March to June, 1989 for the study of this power plant are as follows:

Scale	Drawing No.	Description
1:2000	1 de 18	Plan representing the whole area
"	2 de 18	"
1:200	3 de 18 ~ 18 de 18	Plan (16 sheets)
1:200	19 ~ 25	Section (7 sheets)

(3) Some of the representative as-built drawings of Metales Preciosos del Choco which were obtained by the survey team are as follows:

Scale	Drawing No.	Title	Description
1:600	-	La Vuelta Power plant & camp	Topographical map for the site of P/P and vicinity
1:360	-	Planta General de la Central Hidroelectrica La Vuelta	Structure of powerhouse building

5.2 Geologic Survey Data

Geologic survey data that was collected for this survey is as follows:

- Aerial photographs of this power plant and vicinity
- Mapa Geologico de Colombia, 1988: 1:1,500,000 INGEOMINAS

5.3 Hydrometeorological Data

Since La Vuelta P/P does not have the facilities for monitoring discharge, JICA Study Team gathered HIMAT's hydrometeorological data in conducting this survey.

Precipitation at the existing HIMAT, precipitation stations and the duration of monitoring record are listed below.

Table 5.3 List of Data Collected Relating to Hydrometeorology

(1) Precipitation observation record

Meteorological station Observation No.	Name	Controller	Location Latitude	Longitude	Altitude (m)	period
1102-002	QUADUAS	HIMAT	05467	7611	1,500	1977-89
1102-005	PINON EL	"	0544	7622	715	1958-87
1103-501	LLORO	"	0530	7634	90	1983-86
1104-501	APTO EL CARANO	"	0543	7637	53	1970-87
2619-009	BETANIA-LAS GUACAS	"	0545	7559	1,580	1957-87
2619-010	STA BARBARA	"	0534	7554	1,600	1970-87
2619-502	ITA ANDES	"	0540	7553	1,250	1970-86
5401-003	STA CECILIA	"	0520	7608	370	1964-87
5401-009	SAN ANTONIO & CHAM	"	0528	7559	1,170	1963-87

(2) Discharge observation record

Hydrological Observa-gauging station No.	Name	River	Controller	Establish-ment	Location Latitude	Longitude	Altitude (m)	Catchment area (km^2)	tion period
1101-701	Aguasal	Andagueda	HIMAT	1976-05	0530	7632	75	1,030	1977-86
1102-705	Gindrama	Atrato	"	1979-12	0532	7632	75	1,800	1982-87

(3) Water quality data

The observation of water quality was made at the time of the field survey in November, 1988. However, during that time, JICA Study Team was unable to obtain the observation record.

(4) Sediment data

The observation of sediment was made at the time of the field survey in November, 1988. However, during that time, JICA Study Team could not obtain the observation record of sediment.

5.4 Other Related Data

5.4.1 Construction Prices Data

Construction prices for civil works in Colombia are based on "Catalogo de Precios de Materiales de Construccion (Catalog of Construction Material Prices)" published by CAMACOL (Camera Colombiano de la Construction) However, the above publication is not published in all departments of Colombia. To coordinate the data of the power plant sites where the FS was conducted, construction prices used for this study are based on price data used within E. CHOCO (refer to Table 5.2).

Table-5.2 UNIT PRICE LIST

表-5.2 建設工事単価表

	UNIT	EADE	CHEC	CEDELCA		E.CHOCO	CEDENAR	ESSA	ELECTROLIMA
				SILVIA	OVEJAS				
		NOV./88	FEB./89	JUN./89	JUN./89	MAR./89	JUN./89	APR./89	MAY/89
1. EARTH WORK (EARTH)	p/m^3	2,400	2,925	700	800	2,950	990	2,500	1,100
2. EARTH WORK (ROCK)	p/m^3		3,965				1,900		2,800
3. CONCRETE WORK (MASS CON.)	p/m^3	-	-	-	-	24,000	-	-	-
4. CONCRETE WORK (STRUCTURAL)	p/m^3	26,300	27,625	34,000	40,000	26,800	20,500	15,600	17,900
5. REINFORCING BAR	p/t	354,000	454,000	350,000	360,000	447,500	300,000	320,000	215,000
6. GATE	p/t	1,682,000	500,000	1,310,000	1,420,000	1,100,000	1,100,000	1,100,000	480,000
7. SCREEN	p/t	1,682,000	5,00,000	804,195	874,125	1,000,000	1,000,000	1,000,000	650,000
8. PENSTOCK	p/t	1,000,000	1,000,000	1,250,000	1,250,000	-	815,000	1,260,000	420,000
9. POWER HOUSE (REPAIR)	p/m^2	-	10,000	-	-	-	-	-	-
10. POWER HOUSE (NEW CONST.)	p/m^2	-	40,000	47,000	55,000	50,000	50,000	50,000	50,000
11. CYCLOPEAN CONCRETE	p/m^3	-	14,000	17,000	20,000	-	-	8,000	9,000
12. DEMOLITION CONCRETE	p/m^3	13,000	14,000	17,000	20,000	-	-	8,000	9,000
13. STEEL PIPE	p/t	-	-	-	1,250,000	-	-	-	-
14. GABION	p/m^3	-	-	8,800	-	-	-	-	-
15. TUNNEL EXCAVATION	p/m^3	-	-	-	-	-	-	-	19,600
16. TUNNEL CONCRETE	p/m^3	-	-	-	-	-	-	-	25,000

5.4.2 Power Condition Data

(1) JICA Study Team collected the following data for the purpose of examining E. CHOCO's power conditions.

1) Present power demand for the surrounding area of La Vuelta P/P and demand forecast until 2000

2) Operation and maintenance costs during the recent five years

3) E. CHOCO schematic power diagram

(2) JICA Study Team gathered the following data relating to La Vuelta.

1) Single line diagram

2) Equipment layout plan

CHAPTER 6 PRESENT CONDITION OF TOPOGRAPHY AND GEOLOGY

6.1 Topography and Geology in the Area

6.1.1 Topography

The fountainhead of the Rio Andagueda is around the western slope with the elevation of approximately 3,000 m on the western mountains which run in the direction of north-south along with the west coast of Colombia from which the Rio Andagueda flows generally down to the west and joins the Rio Atrato which runs from south to north at Llolo.

After the confluence the Rio Atrato changes its direction to the north and meanders sharply the swamp in low land through Quibdo, the capita of Choco Department, and continues winding down to the north and flows into the Caribbean Sea. The project site is located about 4 km upstream of the confluence of the Rio Andagueda and the Rio Atrato, and the location where the bends of the river approach closer each other.

The topography in the vicinity forms a gentle slope hill with the elevation of 70 - 100 m and a several steps of river terraces have been developed in the both shores of the river. Current river bed width is approximately 100 m. If aeral photograph is interpreted, remarkable lineaments are not observed in the vicinity of the project site.

6.1.2 Geology

In Choco Department, the syncline consisting of tertiary sedimentary rocks having syncline axis in the direction of north-south exists between coastal area (Pacific ocean) and the western mountains. The project site is located generally within the east wing of this syncline. Therefore, the geological structure in the project site has gentle slope towards the west where sedimentary rocks strikes mostly in the north-south direction.

The bedrock in the project site consists of an alternate layers of fine sand stones and mudstones of the tertiary plicocene - Oligocene. The terrace deposit consists of unsolidified sand gravels of the Quarternary period which contains alluvial gold. The alluvial gold is mined in small scale at various places. The talus eposits of layer depth

of 1 - 3 m are distributed at the foot of the hill. In addition, the sand gravel layer of estimated depth of about 5 m exists in the current river bed. The stratigraphy in the vicinity of the project site is shown in Table 6.1.

Table 6.1 Stratigraphy in the Vicinity of Project Site

Age	Schematic column	Strata	Remarks
Quaternary		River bed deposit	
		Talus deposit	
		Terrace deposit	← Gold ore
Tertiary		Fine-grained Sandstone Mudstone	

6.1.3 Geological Structure

Strike and dip of the alternate layers of fine grained sandstone and mudstone the bedrocks are within N20°W~N5°E, 5°~15°W. This means that the strike is generally in N-S direction, namely square to direction of river stream, and that the dip is gentle 5°~15° towards the downsteam of the river.

Model of the geological structure in the project site is as shown in Fig. 6.1.

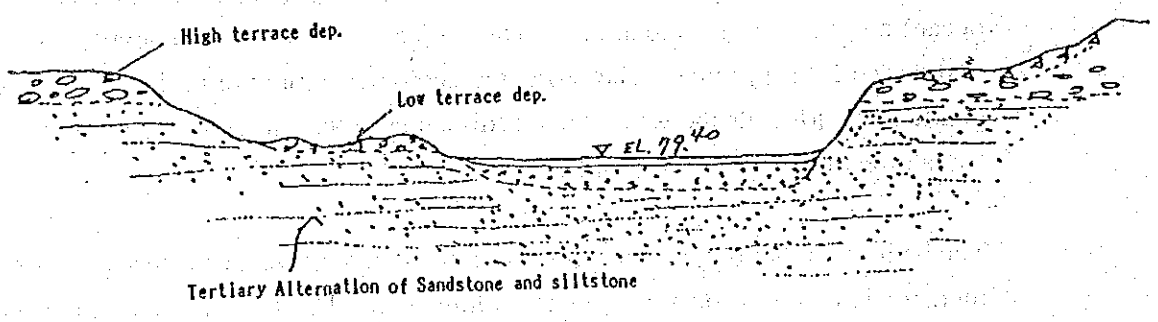


Fig. 6.1 Model Geological Structure in the vicinity of the Project Site

6.2 Geology in the Project Site

The bedrock of the diversion weir, forebay, power plant and tailrace are alternate layers of fine grained sandstone and mudstone of the tertiary period as mentioned above. On the low terrace at the lefthand side shore of the river, low terrace deposits (sand gravel layer) are distributed while on the uppermost of both the right and left hand sides of the river the high terrace deposits (sand and gravel layer containing alluvial gold) are distributed.

6.2.1 Geological evaluation

1) The bedrock in the project site consists of fine grained sandstone and mudstone of the tertiary period, and they belong to so called soft rocks, but as they are fresh when seen from the rock strength point of view, they are evaluated to be sufficiently available for the foundations for concrete dam of about 10 m high and other various structures, and as they show excellent non-permeability, there is no problem for utilizing the bedrocks for the foundation of the dam.

2) As the terrace deposits (sand and gravel layers) and the talus deposits are unconsolidified soft rocks and deform greatly, they are not suitable for the foundations for important structures.

3) There is no slide area in the vicinity of the project site. However, there is a possibility of minor scale of land slide in the new slope when the large scale of excavation which is projected in future is executed, it will be necessary to consider countermeasures for the landslide when detailed design of the project is made.

6.2.2 Geological problems

As the survey made at this time was only on the surface soil of the area in the vicinity of the project site the detailed data on the geological conditions of the subsoil under the project site could not be obtained. The problems to be addressed in future with regard to the project site are as follows:

1) The current layer depth of the sand and gravel of the river bed should be confirmed. To confirm the elevation and the geological conditions of the

bedrocks in the vicinity of the project site, several borings should be executed at the center of the river bed when the detailed design is worked out.

2) The geological conditions under the low terrace deposits at the left hand side bank of the river and the high terrace deposits at both the right and left hand side banks also should be investigated. For the same reasons mentioned int he item 1), boring should also be carried out.

6.3 Distribution of Concrete Aggregates

The candidate materials for the concrete aggregates distributed in the vicinity of the project site are only sand and gravels of the present river bed and sandstone and mudstones of the tertiary period widely found in the alternate layers of the sandstone and mudstone in the vicinity of the project site are not suitable for concrete aggregates. Although the most of the gravels (conglomerates) found in the present river bed are granites, sandstones of the paleozoic, and claystones, the mudstone gravels of the tertiary period are partially included. As these stones seem to contribute to the deterioration of the concrete strength, it is desirable to carry out the aggregate tests for determining the concrete design strength.

There is no problem relating to the aggregate quantity, because the sandstones of this type are widely distributed in the current river bed (Approx. 100 m wide.)

CHAPTER 7 HYDROLOGICAL ANALYSIS

Fig. 7.1 shows the location of the existing gauging stations for monitoring precipitation and discharge in the watershed of the project site.

7.1 General Meteorology in the Planned Area

CHOCO Department located in the northwest part of Colombia lies at 4°00' to 8°40' north latitude and is situated near the equator.

Generally, the lowland areas enjoy a tropical climate and have a hot and very humid rainy season. With the altitude increased, the climate transits into the temperature zone climate. The lowland areas have a temperature of about 28°C, while the areas with the elevation of 1,800 to 2,800 m range from 12° - 18°C.

Quibdo, the capital of Department, lying in the altitude of approximately 40 m has a temperature ranging from about 30°C to 22°C. This temperature level remains constant year to year.

Annual precipitation in the west slope of West Andes Mountain Range is 6,000 mm while the highest precipitation area near Quibdo often exceeds 12,000 mm/year. Precipitation in the highland is as low as 2,000 - 3,000 mm/year.

The Rio Andagueda which has its fountainhead in the West Andes Mountain Range flows on its west slope towards southwest through the project site and is a river of total length of about 76 km between the fountainhead and the merging point with the Rio Atrate.

The project site with the elevation of about 90 m above the sea level is situated southeast to Quibdo the capital of Department and as it lies in the tropical climate area, it has a high temperature of 28°C and a high precipitation of 9,000 mm/year. But as the altitude increases, precipitation decreases to about 2,000 to 3,000 mm annually.

Precipitation fluctuates year to year, but there is a relative clearcut distinction between the dry and rainy seasons. (Refer to Fig. 7.2.)

Observation Item	Gauging Station No	Gauging Station Name	Latitude	Longitude
Discharge	1101-701	Aguasal	0530	7632
	1101-705	Gindrama	0532	7632
Preciptation	1102-002	Guaduas	0546	7611
	1102-005	Pinon El	0544	7622
	1103-501	Lloro	0530	7634
	1104-501	Apto El Carano	0543	7637
	2619-009	Betania Las Guacas	0545	7559
	2619-502	Ita Andes	0540	7553
	2619-010	Sta Borbara	0534	7554
	5401-003	Sta Cecilia	0520	7608
	5401-009	San Antonio De Cham	0528	7559

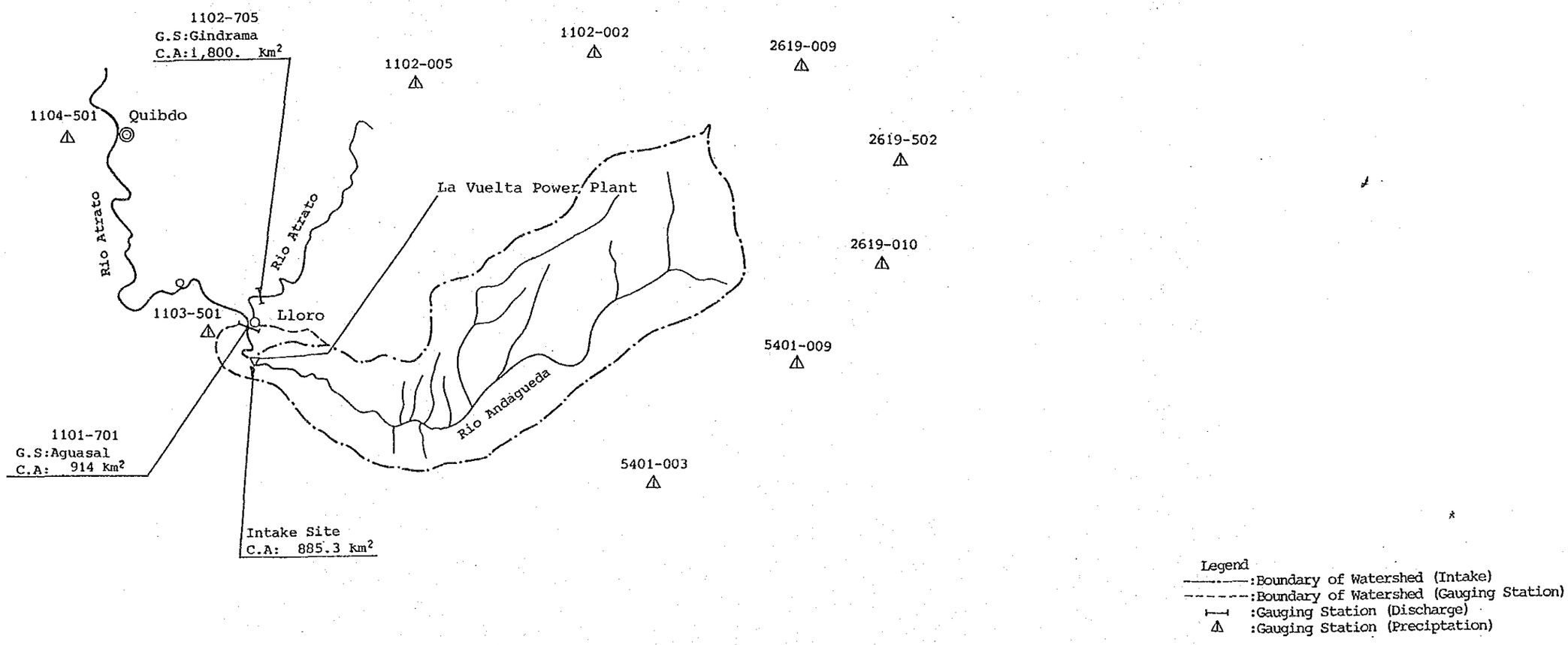


Fig-7.1 Location Map of Gauging Stations in The Watershed of The Study Area.

Meteorological station No.1104-501 Apto El Carano
North latitude: 5°43'
West longitude: 76°37'
Elevation: 53 m
Annual average precipitation: 8,461.4 mm

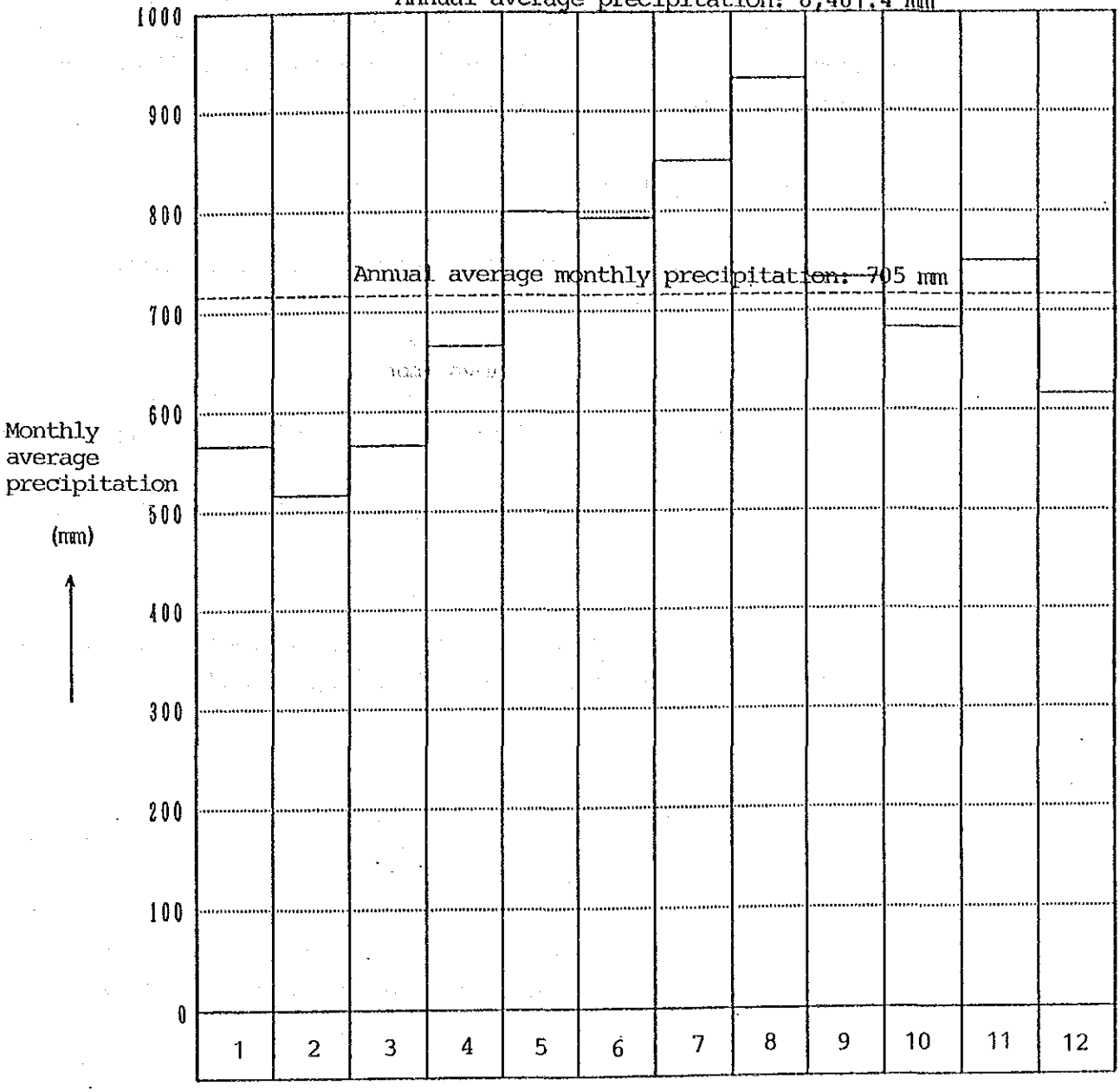


Fig.7.2 Monthly Average Precipitation in the Project Site (1970-87)

7.2 Discharge Analysis

The Study Team gathered the observation data in the past 10 years (from 1977 to 1986) recorded at the Aguasal gauging station operated by HIMAT located about 3 km downstream the La Vuelta P/P and prepared the discharge and the flow-duration curves by converting the river basin according to the reliable 6-year data out of the above record. (Refer to (4) of DWG. No. JV-H-01.)

7.2.1 Comparing Discharge Observation Record

Observation period of the discharge data the team obtained during the survey period is as follows:

Gindrama Gauging Station	1872~1987	6 years	(Established: Feb. 1972)
Aguasal Gauging Station	1977~1986	10 years	(Established: May 1976)

However, some fo the observation year contain non-observing dates and the years in which the complete observation dates are included are as shown below:

Aguasal Gauging Station	1977~1979 1984~1986	3 years 3 years

The data obtained from the Gindrama Gauging Station contain records during six years, but as they include long period of non-observation every year, they are not suitable for analysis.

(1) Confirmation of catchment

In order to confirm the present location of the existing gauging station, longitude and latitude indicated on HIMAT's gauging register were plotted on the topographic maps (1/500,000) published by IGAC, and confirmation of the catchment areas in the gauging station area was made. As a result, it was found

that there was a great deviation in the catchment area of the Auasal gauging station as shown in Table 7 .1.

Table 7.1 Results of Identification of Catchment Areas

Gauging station	Item	HIMAT register	Value confirmed	Difference
Aguasal	Catchment area (km^2)	1,030	914	116
Gindrama	Catchment area (km^2)	1,800	1,800	0

Therefore, in order that the consistency can be maintained between the catchment area of Aguasal and that in the diversion weir of La Vuelta, the value (914.0 km^2) measured by the study team will be employed.

(2) Identification of secular change of flow characteristics

If the flow-duration curves for three years (from 1977 to 1979) and that for three years (1984~1986) observed by Aguasal Gauging Station are compared with the mean flow-duration curve per 100 km^2, they could be presented as shown in Fig. 7.3.

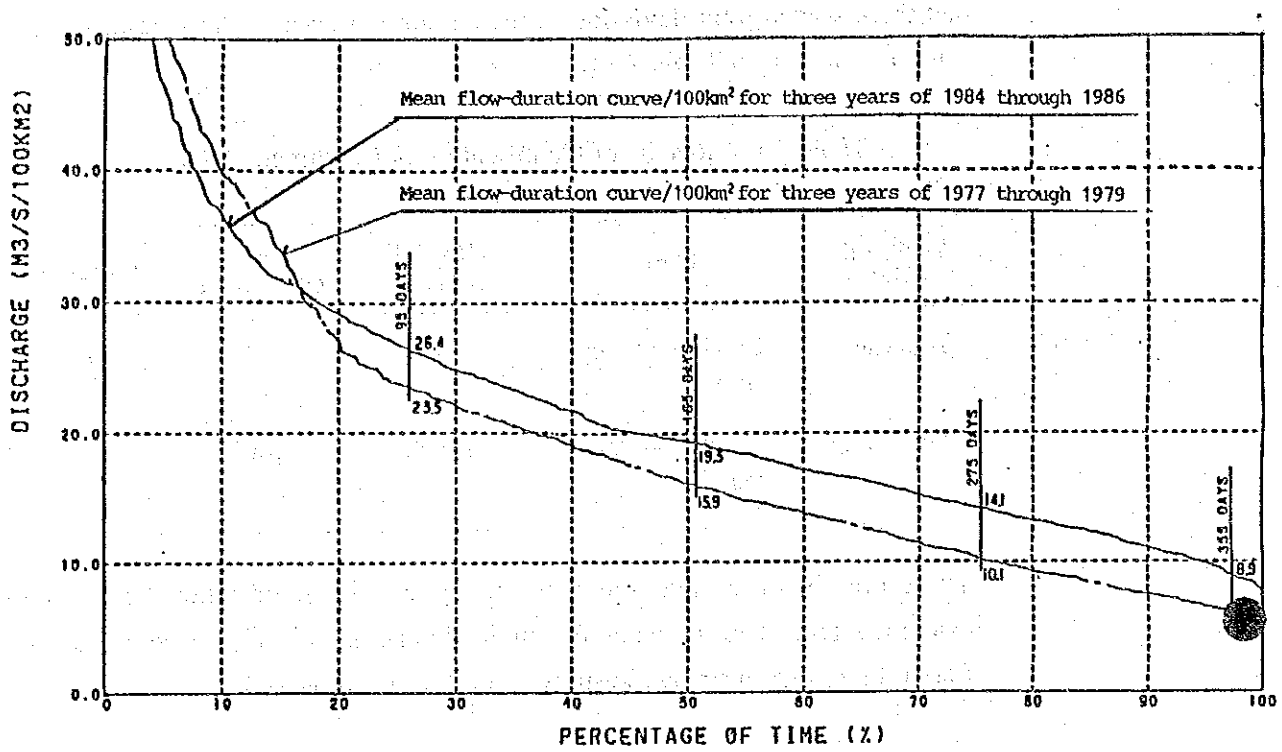


Fig. 7.3 Comparison of Mean Flow-duration Curves per 100 km^2

7.2.2 Typical Flow-duration Curve Form

Year-to-year fluctuations of the river-duration curve occur at this site. In drawing a normal flow-duration curve, the following methods were considered:

a) Parallel method

The daily average discharge for 365 days is arranged in descending order and the flow-duration curve for each year is drawn and averaged.

b) Standard year method

Flow-duration curves for each year are drawn. The median curve is then selected and set as the flow-duration curve for a standard year.

c) Series method

Daily average discharge for 15 years is arranged in descending order with only the Y-axis adjusted for the one-year curve.

d) Curve insertion method

Average values from 355-day flow, 9-month flow, ordinary water discharge and three-month flow observed for a minimum of 10 years are calculated and plotted from a discharge handbook for the flow-duration curve.

Normal flow-duration curves are drawn based on the parallel-method. Non-observed years are not included. The X and Y axes are expressed as daily average discharge (m^3/s) and number of days (%), respectively.

7.2.3 Discharge and Flow-duration Curves at Aguasal Gauging Station

Discharge gate at the Aguasal Gauging Station located about 3 km downstream from the intake site for La Vuelta P/P are arranged as shown in Table 7.2 using 6-year data excluding non-observed dates.

In calculating monthly average discharge data in Table 7.2, the month in which the observing date is less than 10 days is excluded form the calculation. As can be seen from (1) of Drawing No. LV-H-01, the three-month flow period can relatively clearly distinguished from the drought period, and it seems that months April through November correspond to three-month flow period and those from December to March drought period.

Typical flow-duration curves calculated from the 6-year flow-duration curves (covering from 1977 to 1979 and from 1984 to 1986) using the parallel method are indicated in (3) of Drawing No. LV0-H-01. Three-month flow, ordinary water discharge and nine-month flow in flow-duration curves are indicated in numerical values as shown in Table 7.3.

Table 7.4 shows the maximum discharge recorded at Aguasal Gauging Station for 10 years from 1977 to 1986.

Table-7.2 MØNTHLY FLØW TABLE ØF DAILY AVERAGE FLØW AT G.S. SITE

GAUGING ST.: 1101-701 AGUASAL
RIVER NAME: ANDAGUEDA

(UNIT: M3/S)

GAUGING YEAR	TYPE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ØCT	NØV	DEC	ANNUAL TØTAL
1977	MAX.	520.8	235.0	341.4	360.3	607.0	404.0	478.0	487.0	595.0	514.0	473.5	625.2	625.2
	MEAN	60.5	91.2	85.2	109.3	226.2	168.6	154.2	197.5	185.0	289.4	230.7	163.0	165.1
	MIN.	33.6	52.5	39.7	45.0	74.5	87.0	58.0	80.3	54.5	126.5	79.2	58.0	33.6
1978	MAX.	246.8	185.7	473.5	590.8	487.0	581.5	301.5	729.0	627.8	799.0	682.1	576.1	799.0
	MEAN	96.9	92.7	132.0	276.1	193.9	308.5	138.7	150.2	174.0	277.6	236.4	203.5	190.0
	MIN.	49.0	49.5	45.5	91.5	78.0	138.7	57.5	49.5	66.5	134.5	115.5	95.4	45.5
1979	MAX.	266.0	185.6	368.3	544.6	500.5	380.2	380.5	668.4	473.5	877.4	610.0	410.9	877.4
	MEAN	104.7	99.5	123.8	227.5	207.5	205.6	142.4	200.1	214.0	271.3	279.7	200.7	189.7
	MIN.	63.0	63.0	63.0	66.0	66.0	103.2	75.0	67.8	88.4	95.2	134.5	107.8	63.0
1980	MAX.	434.0	360.5	195.6	534.5	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	534.5
	MEAN	148.7	174.9	94.1	179.1	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	149.2
	MIN.	75.0	83.2	51.0	51.5	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	51.0
1981	MAX.	(1)	(1)	(1)	(1)	460.8	584.0	345.5	306.1	(1)	(1)	(1)	263.7	584.0
	MEAN	(1)	(1)	(1)	(1)	220.2	214.3	171.2	184.7	(1)	(1)	(1)	63.8	170.8
	MIN.	(1)	(1)	(1)	(1)	140.8	108.7	78.1	77.8	(1)	(1)	(1)	18.6	18.6
1982	MAX.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	274.0	578.0	302.2	(1)	578.0
	MEAN	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	164.0	232.2	200.0	(1)	198.7
	MIN.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	86.9	123.8	118.6	(1)	86.9
1983	MAX.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	294.0	821.0	349.4	408.0	821.0
	MEAN	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	180.5	217.3	201.5	209.2	202.1
	MIN.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	67.8	101.4	86.4	111.9	67.8
1984	MAX.	330.7	319.6	273.9	395.6	416.4	417.6	444.0	498.5	355.7	573.4	674.0	447.5	674.0
	MEAN	191.1	187.9	153.7	206.9	222.3	240.5	193.7	216.3	176.9	265.4	267.5	207.4	210.8
	MIN.	110.4	107.0	79.5	110.2	131.6	138.5	86.7	97.9	87.7	97.9	103.6	81.1	79.5
1985	MAX.	637.9	315.4	424.3	329.7	330.1	587.4	432.7	474.5	401.0	487.5	622.6	396.6	637.9
	MEAN	205.0	134.1	167.0	179.8	204.0	202.5	183.8	183.9	197.4	240.4	178.0	221.5	191.5
	MIN.	96.3	72.0	91.7	100.2	106.1	108.7	64.7	93.0	117.6	101.6	80.4	95.3	64.7
1986	MAX.	611.8	312.5	503.9	611.8	376.0	414.6	611.8	611.8	606.5	608.8	422.6	306.0	611.8
	MEAN	185.0	157.3	157.3	234.2	190.3	134.9	246.9	283.5	224.0	256.9	217.6	105.6	199.4
	MIN.	79.2	93.1	63.0	90.1	85.3	60.1	72.0	68.4	58.6	101.6	107.0	55.0	55.0
TØTAL	MAX.	637.9	360.5	503.9	611.8	607.0	587.4	611.8	729.0	627.8	877.4	682.1	625.2	877.4
	MEAN	144.6	134.0	130.4	201.8	209.2	210.7	175.8	202.3	189.5	256.3	226.4	171.8	187.7
	MIN.	33.6	49.5	39.7	45.0	66.0	60.1	57.5	49.5	54.5	95.2	79.2	18.6	18.6

NØTE) (1) ALL DATA MISSING

Table-7.3 FLØW DURATIØN TABLE AT GAUGING STATIØN SITE

GAUGING ST.: 1101-701 AGUASAL
RIVER NAME: ANDAGUEDA (UNIT: M3/S)

GAUGING YEAR	MAX. (1ST DAY)	PLENTY (95 DAY)	ØRDINARY (185 DAY)	LØW (275 DAY)	DRØUGHTY (355 DAY)	MIN. (LAST DAY)	MEAN
1977	625.2	198.1	126.5	79.2	42.5	33.6	165.6
1978	799.0	220.2	153.0	91.5	54.5	45.5	190.2
1979	877.4	225.2	155.4	107.2	68.4	63.0	190.0
1984	674.0	251.8	196.3	148.0	93.3	79.5	210.8
1985	637.9	230.7	167.2	128.1	84.3	64.7	192.0
1986	611.8	240.3	164.4	111.2	66.0	55.0	199.7
MEAN	704.2	227.7	160.5	110.9	68.2	57.1	191.4

Table-7.4 MONTHLY ABSOLUTE MAXIMUM FLOW TABLE AT G.S. SITE

GAUGING ST.: 1101701 AGUASAL
RIVER NAME: ANDAGUEDA

(UNIT: M3/S)

GAUGING YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL TOTAL
1977	581.5	400.0	542.0	421.0	869.0	610.0	729.0	729.0	1009.0	785.0	729.8	729.0	1009.0
1978	312.5	299.0	729.0	1009.0	785.0	841.0	460.0	1009.0	953.0	1065.0	911.0	740.2	1065.0
1979	284.0	217.2	620.0	735.0	735.0	602.5	394.0	466.0	752.0	1009.0	751.4	584.2	1009.0
1980	581.5	460.0	217.2	729.0	635.0	620.0	537.0	518.0	553.0	776.0	609.0	647.0	776.0
1981	530.0	588.0	642.0	595.0	701.0	610.0	487.0	662.8	556.0	576.0	822.0	620.0	822.0
1982	717.8	537.0	582.0	479.0	617.0	582.0	620.0	566.0	970.0	1082.0	589.6	527.0	1082.0
1983	230.0	163.0	364.0	480.0	595.0	620.0	582.0	553.0	434.0	846.0	408.0	855.0	855.0
1984	382.8	434.0	536.4	780.0	745.1	795.6	953.0	747.0	630.4	1043.0	1322.0	752.0	1322.0
1985	860.0	733.6	821.6	646.0	620.0	906.8	668.4	640.8	684.8	701.0	1167.0	674.0	1167.0
1986	843.2	470.8	630.4	865.2	428.8	648.8	873.0	880.8	846.0	886.0	584.2	423.6	886.0
TOTAL	860.0	733.6	821.6	1009.0	869.0	906.8	953.0	1009.0	1009.0	1082.0	1322.0	855.0	1322.0

7.2.4 Discharge and Flow-duration Curves at Intake Site

Discharge and flow-duration curves at the intake site of La Vuelta P/P are calculated by multiplying the records observed at the existing Aguasal Gauging Station located about 3 km downstream of the intake site by respective catchment area ratio.

Since officially approved numerical values of the catchment area at the intake site were not available, the value 885.3 km^2 recorded by the survey team was adopted. Therefore, the ratio of catchment area between La Vuelta P/P's intake site and HIMAT's Aguasal gauging station has been set at 885.8/914 = 0.97.

Discharge and flow-duration curves at the intake site converted according to the catchment area ratio are shown in (4) of Drawing No. LV-H-01 and the representative values of monthly and daily average discharge and of three-month flow, ordinary water discharge, nine-month flow and 355-day flow are indicated as follows.

Table 7.5 Representative Discharge at the Intake Site

1) Monthly average discharge

Item	Month												
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
Max. average discharge (m^3/s)	196.0	180.0	162.0	268.0	219.0	299.0	240.0	273.0	217.0	280.0	272.0	216.0	204.0
Daily average discharge (m^3/s)	140.0	130.0	127.0	195.0	203.0	204.0	170.0	195.0	184.0	248.0	220.0	168.0	182.0
Min. average discharg (m^3/s)	80.0	90.0	84.0	84.0	185.0	130.0	134.0	146.0	159.0	710.0	173.0	62.0	145.0

2) Typical discharge of flow-duration curve

Three-month flow (95-day flow)	Ordinary water discharge (185-day flow)	Nine-month flow (275-day flow)	355-day flow
220.6 m^3/s	155.4 m^3/s	107.4 m^3/s	66.0 m^3/s

River utilization factor of a certain available discharge to typical flow-duration curves at the intake site (a ratio of total available discharge and total river

discharge flowing into the intake site) and facility utilization factor (a ratio of total discharge for which water can be taken in to the available discharge throughout the year and total water amount in the event that available discharge is secured throughout the year) are represented graphically in (5) of Drawing LV-H-01.

7.3 Flood Runoff Analysis

The flood discharge is important to maintain the safety of existing facilities and the repaired sections. The design flood discharge is obtained by that the observation record of the discharge at gauging station is statistically processed and this is then converted by the catchment area ratio.

7.3.1 Frequency of Flood

In order to obtain potential flood discharge, annual maximum discharge is summarized according to the discharge data, and shown in Table 7.6

Table 7.6 Annual Flood Discharge

Year Observed	Maximum Discharge (m^3/sec)
1976	1,009
1977	1,065
1978	1,009
1979	776
1980	822
1981	1,082
1982	855
1983	1,322
1984	1,167
1985	886

The observation data is for 10 years, and is comparatively short example. There is several methods to obtain potential flood, but the following three methods are studied.

1. Logarithm normal distribution method (slade method)
2. Order probability method
3. Gumbel method

For the order probability method and Gumbel method, two ways of Thomas plot and Hazen plot are studied.

Figs. 7.4 and 7.5 show that maximum yearly discharge is plotted on X-axis of abscissa and that percentage of excess probability calculated is plotted on Y-axis of ordinate by using the extreme probability paper. Table 7.7 shows the potential flood discharge for major years of return period from the probability curve shown in the figure.

Table 7.7 Potential Flood Discharge

Method	Return Period in Years							
	5	10	20	50	100	200	500	1000
Logarithm normal distribution method (m^3/s)	1,128	1,210	1,282	1,368	1,429	1,486	1,559	1,613
Order probability method:								
Thomas plot (m^3/s)	1,171	1,297	1,417	1,572	1,688	1,805	1,961	2,081
Hazen plot (m^3/s)	1,136	1,236	1,329	1,446	1,533	1,618	1,730	1,815
Gumbel method:								
Thomas plot (m^3/s)	1,169	1,297	1,418	1,576	1,694	1,812	1,968	2,085
Hazen plot (m^3/s)	1,131	1,236	1,336	1,465	1,562	1,659	1,786	1,882

7.3.2 Design Flood Discharge

The design flood discharge is applied to the structures in the project referring to "Generalized design criteria for water-control structures"*, and the 100-year probability discharge is employed from 50 to 100 years of the return period*.

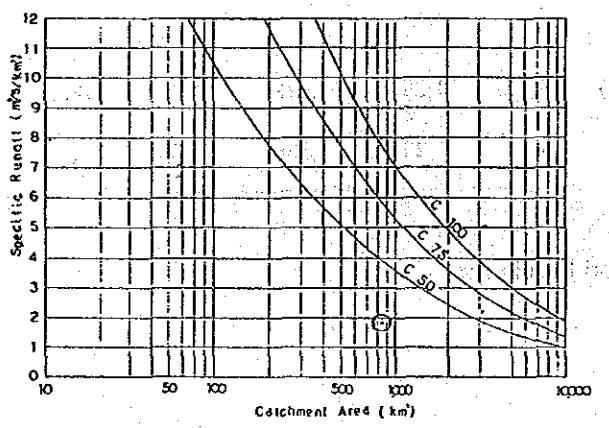
The design flood discharge (Q) in the intake site is obtained by converting with the catchment area ratio.

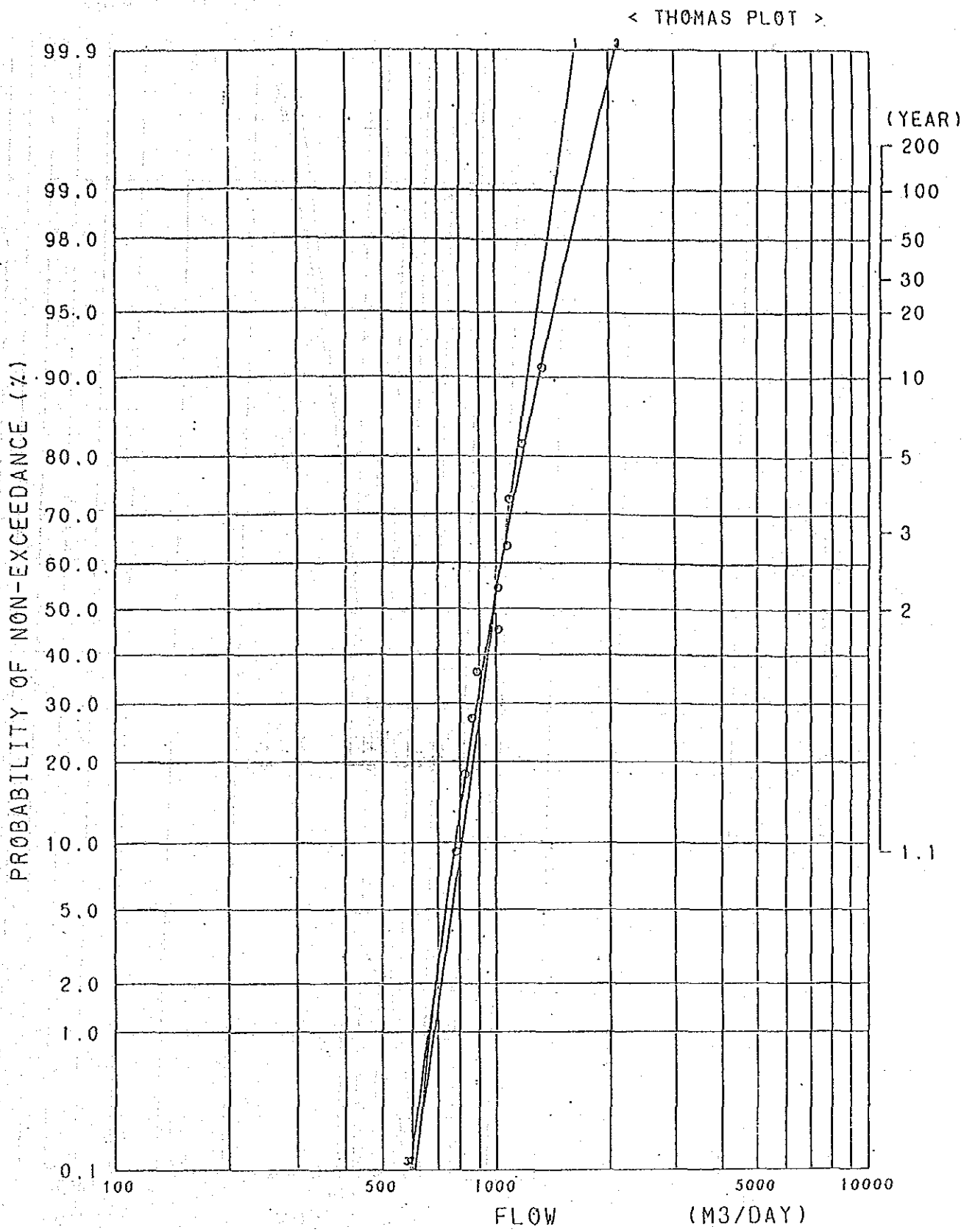
$$Q = 169.4\ m^3/s \times 885.3\ km^2/914\ km^2 = 164.1\ m^3/s\ .....\ 1{,}700\ m^3/s$$

The specific discharge per catchment area (km^2) will be $q = 1.92\ m^3/s$ from the design flood discharge. The corresponding value obtained from the Creager curve (Fig. 7.5), indicating the relationship between specific discharge and catchment area is $C \doteq 3.8$.

* Applied Hydrology Editor Ven Te Chow
David R. Maidment
Larry W. Mays

Fig. 7.6 Design Flood Discharge and Creager Curve





1 : BY SLADE METHOD
2 : BY ORDER PROBABILITY METHOD
3 : BY GUMBEL METHOD

Fig.7.4 Probability Curve of Rio Andagueda
(Thomas Plot)

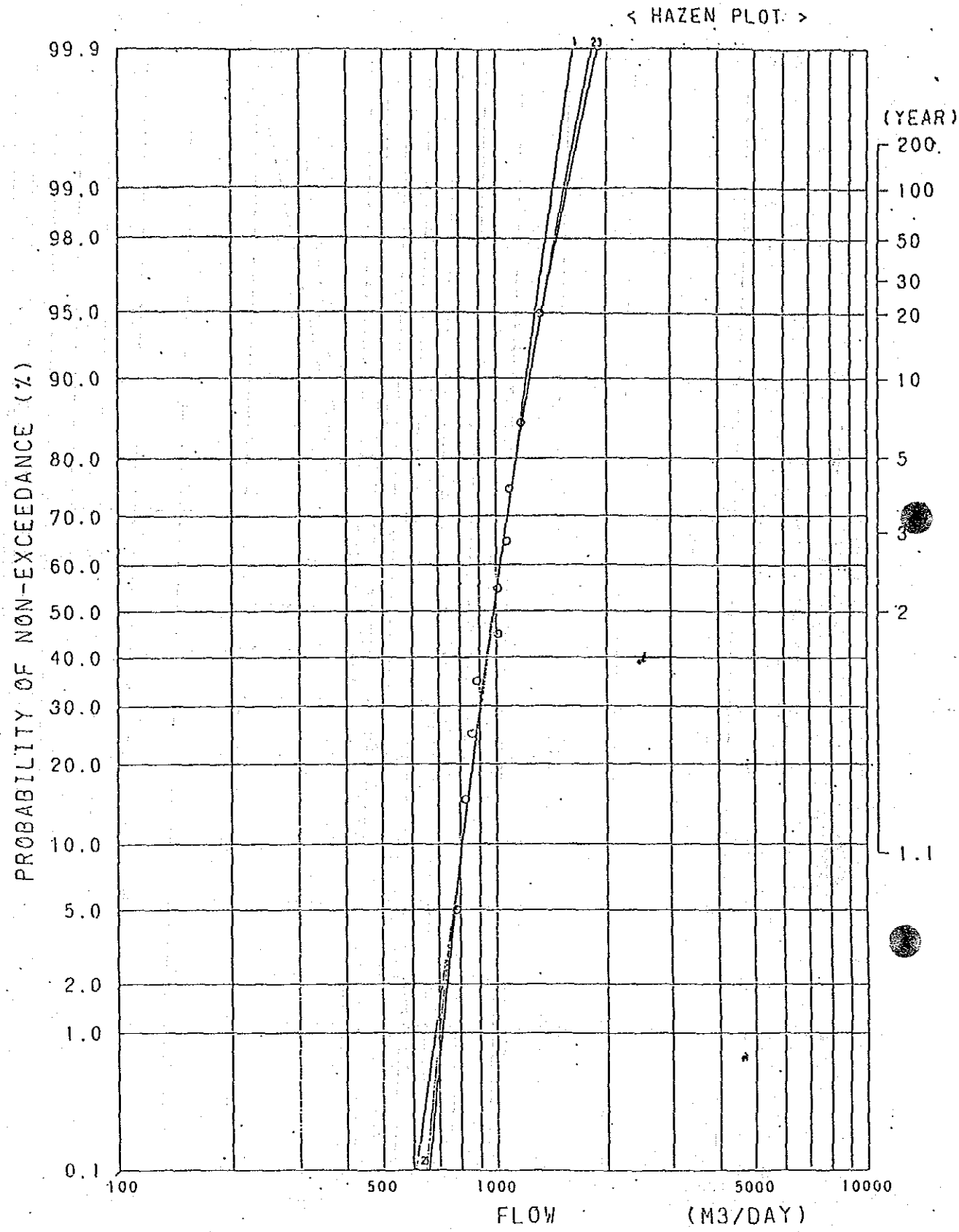


Fig. 7.5 Probability Curve of Rio Andagueda

7.4 Sediment Analysis

The debris produced at the catchment mountain flows down up to the intake point, and further flows to downstream via channel and river. The flow process of debris is shown in Fig. 7.87, and the flow debris volume is studied according to this process.

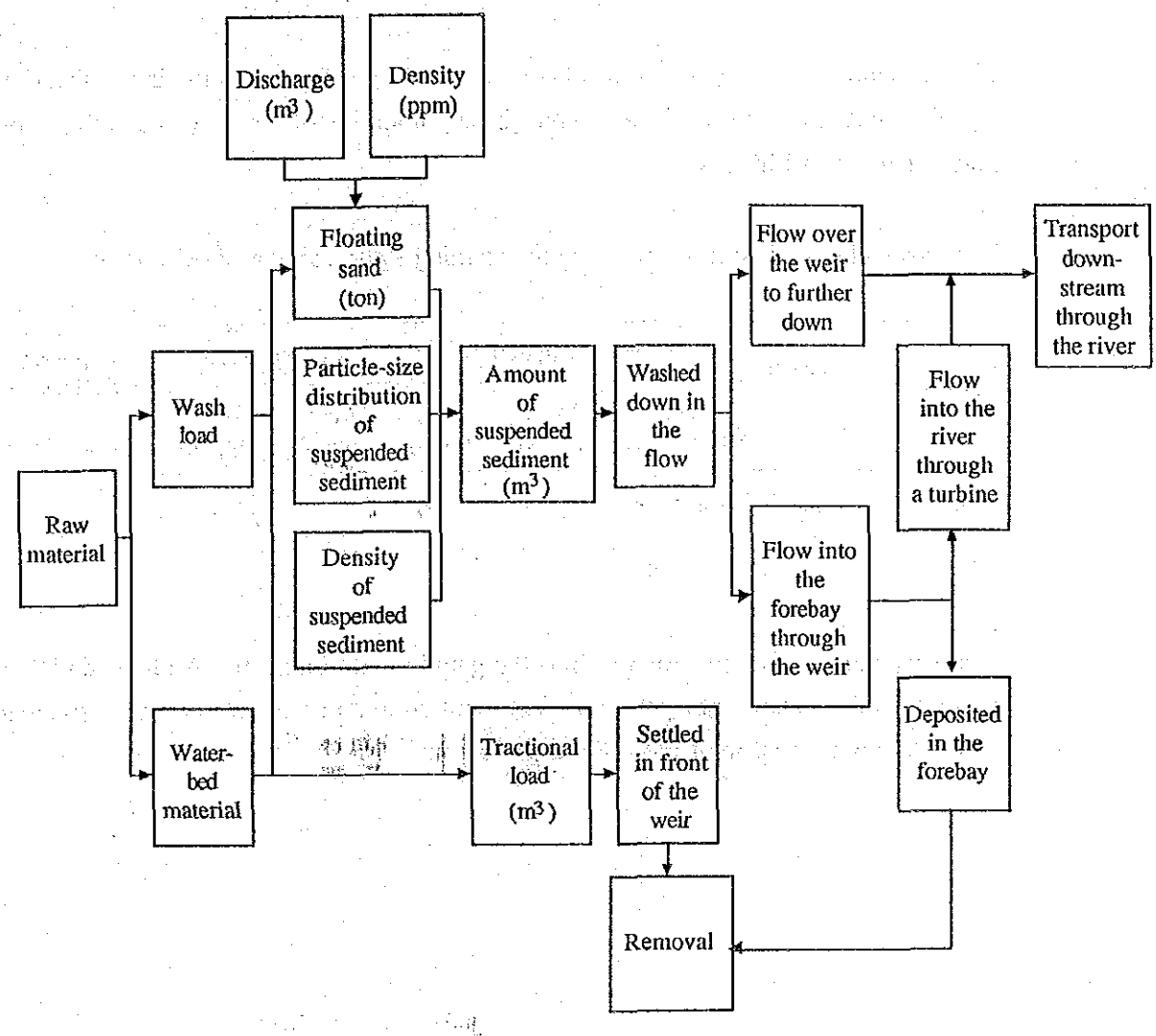


Fig. 7.7 Mechanism of Debris Flow and Calculation Flow of Volume

7.4.1 Debris Flow Status

The catchment at the Rio Andagueda forms a feather like configuration with the main stream at the center into which a lot of relatively short tributaries are draining. the debris flowing from the catchment is mainly debris generated by erosion of river bed of the ravine and bank, and by the cultivation of farm land, and gully erosion by terrace collapse etc. The vegetation of the catchment is good.

The suspended sediment curve has been prepared by referring to the basic shape of the sediment rating curve of the Rio Nus basing upon the observed value at this spot, and is shown in Fig. 7.8.

The suspended sediment (ton/year) at the gauging station spot is shown below.

River	Catchment Area (km^2)	River Discharge Rate			Concentration		Suspended Sediment Rate 10^3 tons/year
		Total 103 m^3/year	Max. (m^3/s)	Min. (m^3/s)	Max. (ppm)	Min. (ppm)	
Andagueda	914	5,437,000	1,322	18.6	-	-	280

The suspended sediment flowing into the gauging station on the Andagueda River reaches 300 tons/km^2 per year per catchment area, and annual average suspended sediment concentration of Pasto River is 50 ppm.

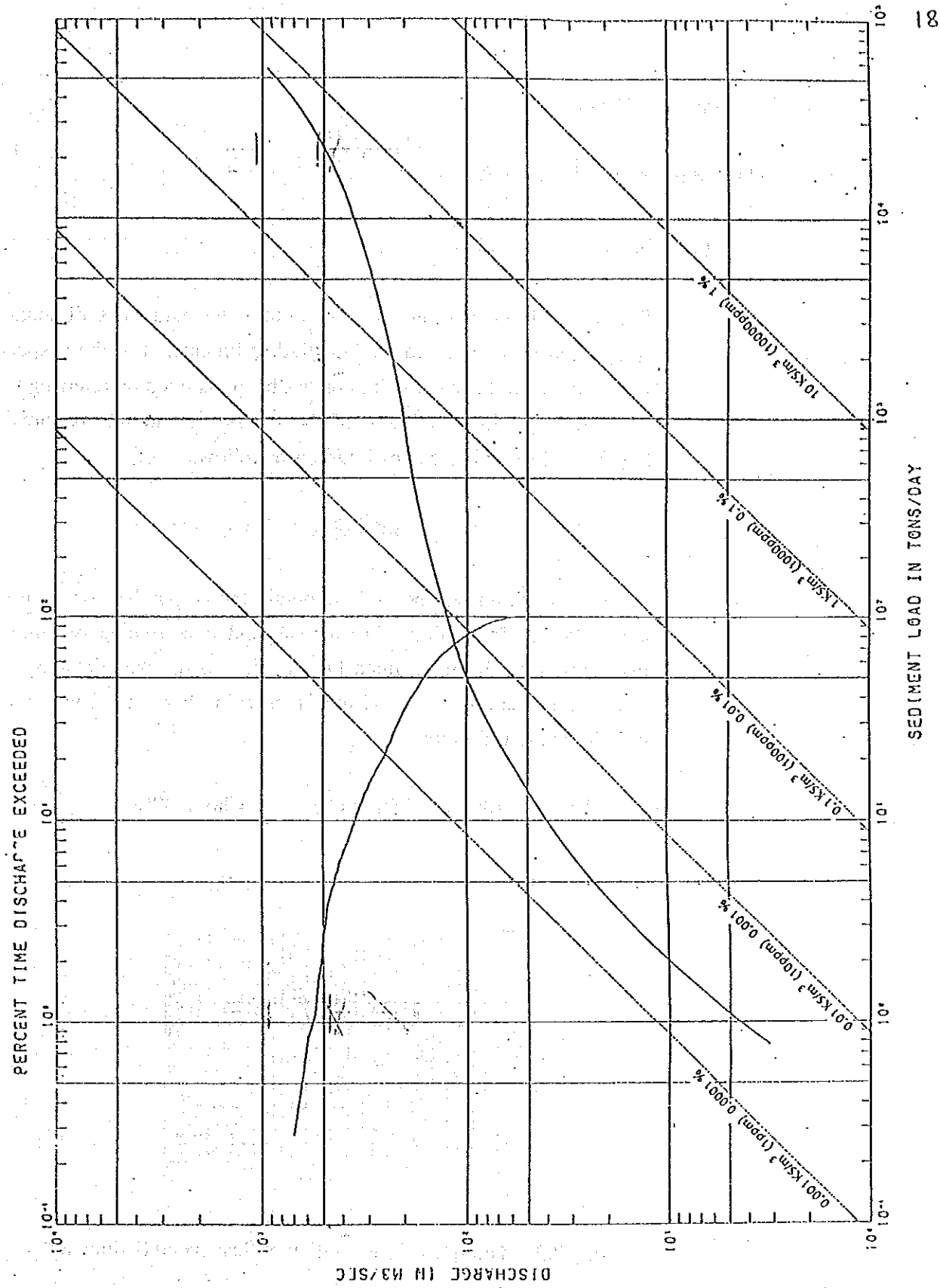


Fig.7.8 Sediment Rating Curve

7.4.2 Assumption of Sediment Rate

(1) Major physical properties

(a) Grain size distribution

The Study Team was not able to obtain the data on sedimentation (suspended sediment, bed-load, settled sediment). For the suspended sediment, the grain size distribution has been assumed by referring to the past data regarding sediment of the reservoir, and this is shown in Fig. 7.9. The grain size constitution is as follows:

Sand = 60% Silt = 60% Clay = 30%

JICA Study Team was not able to obtain the suspended sediment data, and settled sediment data. For the suspended sediment, the grain size distribution has been assumed by referring to the past data regarding sediment of the reservoir, and this is shown in Fig. 7.9. The grain size constitution is as follows:

Sand = 10% Silt = 60% Clay = 30%

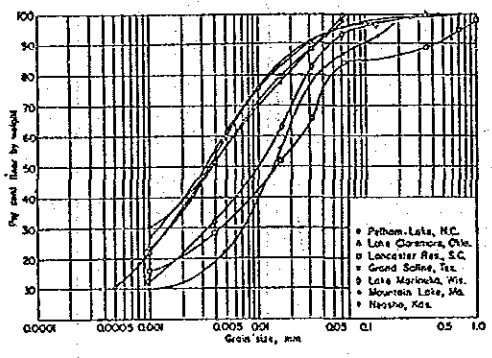


Fig. 7.9 Grain Size Constitution of Suspended Sediment *

* Handbook of Applied Hydrology

(b) Unit volume weight

Since JICA Study Team could not obtain the data for the unit volume weight of sediment, this shall be determined by referring to literature.

The unit volume weight of sand and gravel is also affected to the consolidation load, but the consolidation is comparatively completed in short time. However, fine particle of clay, colloid, etc. will take long time for this. The unit volume weight will become range shown in Table 7.8 from the grain size constitution of sediment at reservoir from the past case example and the active conditions (under or above water) of the load at that time.

Table 7.8 Range of Unit Volume Weight

(units: ton/m^3)*

Grain	Almost under water	Above water
Clay	0.64 - 0.96	0.96 - 1.28
Silt	0.88 - 1.20	1.20 - 1.36
Mix of clay and silt (equal volume)	0.64 - 1.04	1.04 - 1.36
Mix of sand and silt (equal volume)	1.20 - 1.52	1.52 - 1.76
Mix of clay, silt and sand (equal volume)	0.80 - 1.28	1.28 - 1.60
Sand	1.36 - 1.60	1.36 - 1.60
Gravel	1.36 - 2.00	1.36 - 2.00
Sand and gravel	1.52 - 2.08	1.52 - 2.08

* Handbook of Applied Hydrology

(2) Discharge rate of sediment

When the discharge rate of sediment at the intake spot is examined, the suspended sediment and the bed-load are considered. The suspended sediment can be assumed from the sediment record (concentration measurement) and the discharge record. The quantitative record for the flown sand has not been obtained. It is generally said that the flown sand is 10 to 50% of total sediment rate, and the flown sand of the Colorado River is 12 to 50% of total sediment rate. The study team of the World Bank is estimating that the flown sand of the

Indus River at the Tarubera dam (Pakistan) spot will be 5% of the suspended sediment.

(3) Yearly flowing sediment rate

The yearly flowing sediment rate at the intake spot is obtained by converting values at the gauging station into catchment area ratio.

Catchment Area (km^2)	River Discharge Rate (10^6 m^3)	Suspended Sediment Rate (10^3 ton)	Flown Sand Rate (10^3 ton)	Sediment Rate (10^3 ton)
885.3	5,268	272	28	300

Average grain size of the flowing sediment is obtained from the unit weight by average grain size constitution and each grain diameter as follows.

	Flown Sand			
	Gravel	Sand	Silt	Total
Grain size constitution(%)	30	60	10	100
Unit volume weight (ton/m^3)	1.68	1.48	1.04	
Unit weight per grain size (ton/m^3)	0.504	0.888	0.104	1.496... 1.50

	Suspended Sediment			
	Sand	Silt	Clay	Total
Grain size constitution(%)	10	60	30	100
Unit volume weight (ton/m^3)	1.48	1.04	0.80	
Unit weight per grain size (ton/m^3)	0.148	0.624	0.240	1.01

All the flown sands shall deposit at the diversion weir and in front of the intake, and shall not flow into the channel.

The suspended sediment is contained in the discharge within the range of design discharge, and flows down the channel from the intake. Partial rough particles

in the suspended sediment flown into the channel are settled at the desilting basin, and the remaining suspended sediment is discharged into the river through water wheel together with discharge. The river discharge more than design discharge flows down the river by overflowing the weir together with the suspended sediment contained in this discharge.

Catchment area (km^2)	River discharge (10^6 (m^3))	Bed-load 10^3 (ton)	Suspended sediment 10^3 (ton)
885.3	5,437	28	272
		↓	↓
		x 10^3 m^3	x 10^3 m^3
		19	269

Design discharge Q m^3/s	Sediment in forebay x 10^3 m^3	Returned to river flow x 10^3 m^3
50	94	175
100	161	108

Design discharge Q m^3/s	Settled in the forebay 10^3 (m^3)	Flow down the channel 10^3 (m^3)
50	2	159
100	2	159

It is assumed from the results of the above analysis that annual average sediment in front of the diversion weir will be about 50 m^2/day and sediment settled in the forebay will be about 50 m^3/day. A counterplan for removing these sediment shall be fully considered.

7.5 Water Quality Analysis

The results of the water quality test were not available. Judging from the appearance of the catchment environment and the river water, it seemed that there was no problem with regard to the water quality.

CHAPTER 8 GENERATION PLAN

Since the maximum available discharge at the existing power plants was planned as 54 m^3/s, the generation plan is made based on this rate.

The maximum available discharge is changed within a range if the facility utilization factor does not exceed 50% in the typical flow-duration curves at the intake site, and generating output and annual generating energy are calculated. The generation plan is conceived from technological and economical aspects.

8.1 Study of the Alternative Plans

As an alternative plan for the La Vuelta P/P, an idea in which the current TRINCHO type diversion weir is renovated into the concrete dam type and at the same time the increase in the level of intake water that is an increase in the effective head is achieved might be considered. However, as a lot of uncertainties as mentioned below exist at this stage of the study, it was decided that such idea should be excluded from the study, and the outline of generating plan based on such idea is presented in Appendix.

① Geological survey as a fundamental requisite for concrete dam has not been conducted on the riverbed and the terrace at the left bank side of the river.

② Topographic map (1/50,00 or over and an aerial photographic survey of 1/10,000) which is required to study the affect of backwater due to an increase in the height of the dam has not been prepared.

③ No survey on the compensation for the housing, agricultural fields, forestories etc. which are to be submerged under the aster has not been made.

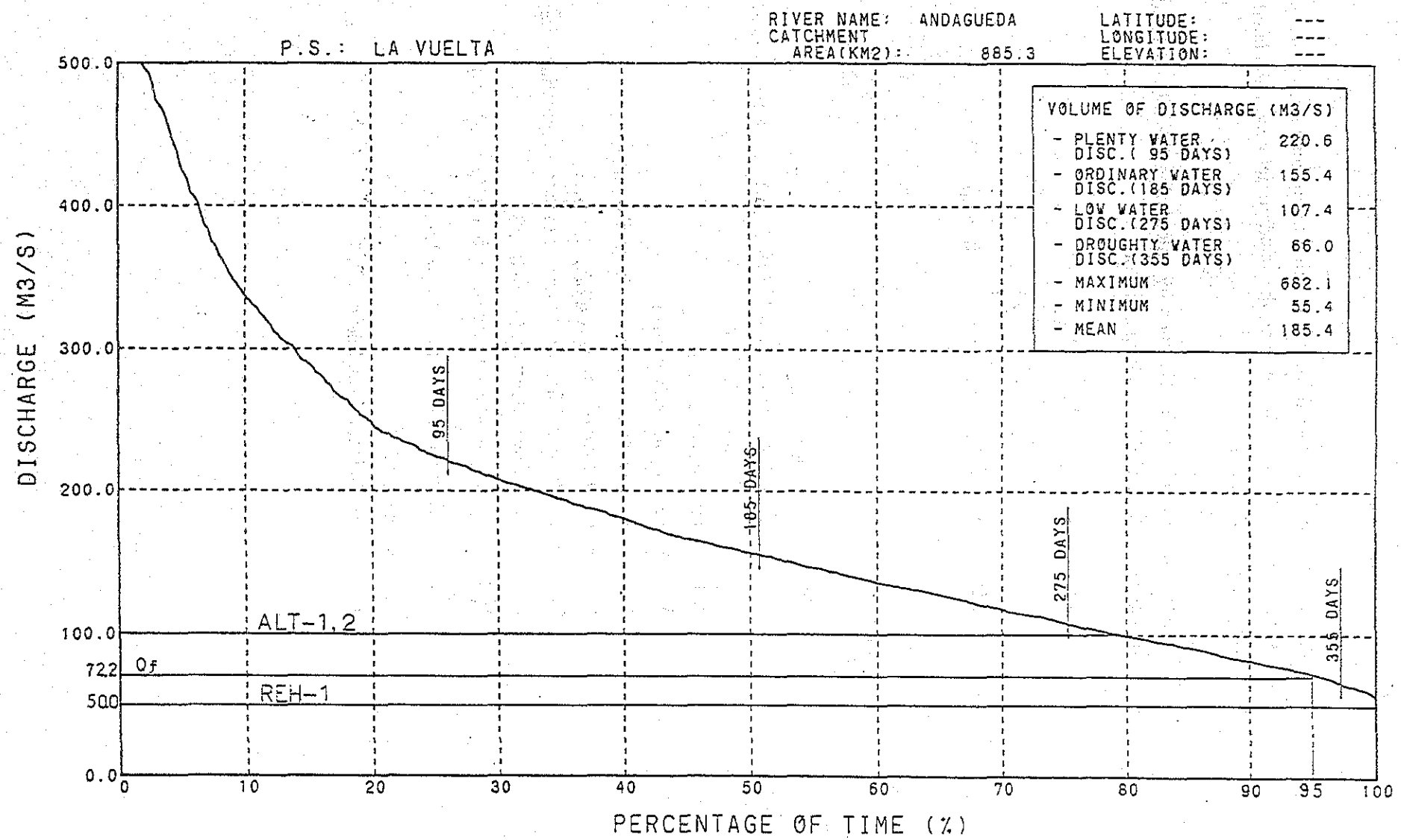
④ For the other items, refer to the minutes of meeting (interim).

(1) Maximum available discharge

The maximum available discharge at the existing power plant is designed to be Q=54 m^3/s. On the other hand, the discharge which can be assured for 95% of a year is Q=72.2 m^2/s as shown in Fig. 8.1. Therefore, as for the maximum

available discharge, the two cases of Q=50.0 m^3/s and Q=100.0 m^3/s are compared to calculate respective generated output and annual generated electric power. If the maximum available discharge is set at Q=150.0 m^3/s or 200 m^3/s, utilization factor of river water would become 86.0% and 75.0% respectively. And the plan could not be accepted as an appropriate one for the run-of-river power plant.

Fig-8.1 TYPICAL FLØW DURATIØN CURVE AT INTAKE SITE



(2) Standard net head

Assuming that the net head for determining the turbine output and calculating annual generated energy is constant, the standard net head calculated under the following standard is used.

Effective head (He) can be obtained by calculating the loss of head between forebay and tailrace in the following equation.

$$He = Hg - \Delta H + \frac{Vg^2}{2g} - \frac{Vf^2}{2g}$$

$$\Delta H = V_2^2 / 2g \left(f_e + \frac{V_2^2 - V_1^2}{V_2^2} + f_p + f_n\right) + \Delta h$$

where:

Hg	=	gross head
		Forebay water level (79.70 m) - tailrace water level (75.20 mm)=4.50 m
ΔH	=	total loss of head (m)
V_1	=	velocity at forebay (m/s)
V_2	=	velocity at the entrance of the intake (m/s)
f_e	=	coefficient of inflow loss; 0.1
f_p	=	coefficient of frictional loss due to pier at regulating gate; 0.095
f_n	=	loss of head due to silt grid; 0.353
$V_a^2 2g$	=	velocity head at entrance of turbine (Vg = 1.0 m/s)
$V_f^2 2g$	=	velocity head at tailrace (Vf = 1.5 m/s)
Δh	=	margin (m)
n	=	coefficient of roughness, 0.015

Table 8.1 Calculated Result of Standard Net Head

Q (m^3/s)	Hg (m)	$V^2/2g\Sigma f$ (m)	Δh (m)	ΔH (m)	$V_a^2/2g$ (m)	$V_f^2/2g$ (m)	He (m)
50	4.50	0.022	0.015	0.037	0.047	0.110	4.40
100	4.50	0.039	0.015	0.054	0.047	0.113	4.38

Accordingly, the standard net head is calculated to be as 4.40 m.

8.2 Generated Output

Theoretical output obtained from available discharge (Q) and the standard net head (H) is multiplied by resultant efficiency of the equipment, and the generated output is calculated by the following formula.

$$P = 9.8 \times Q \times He \times \eta$$

where:

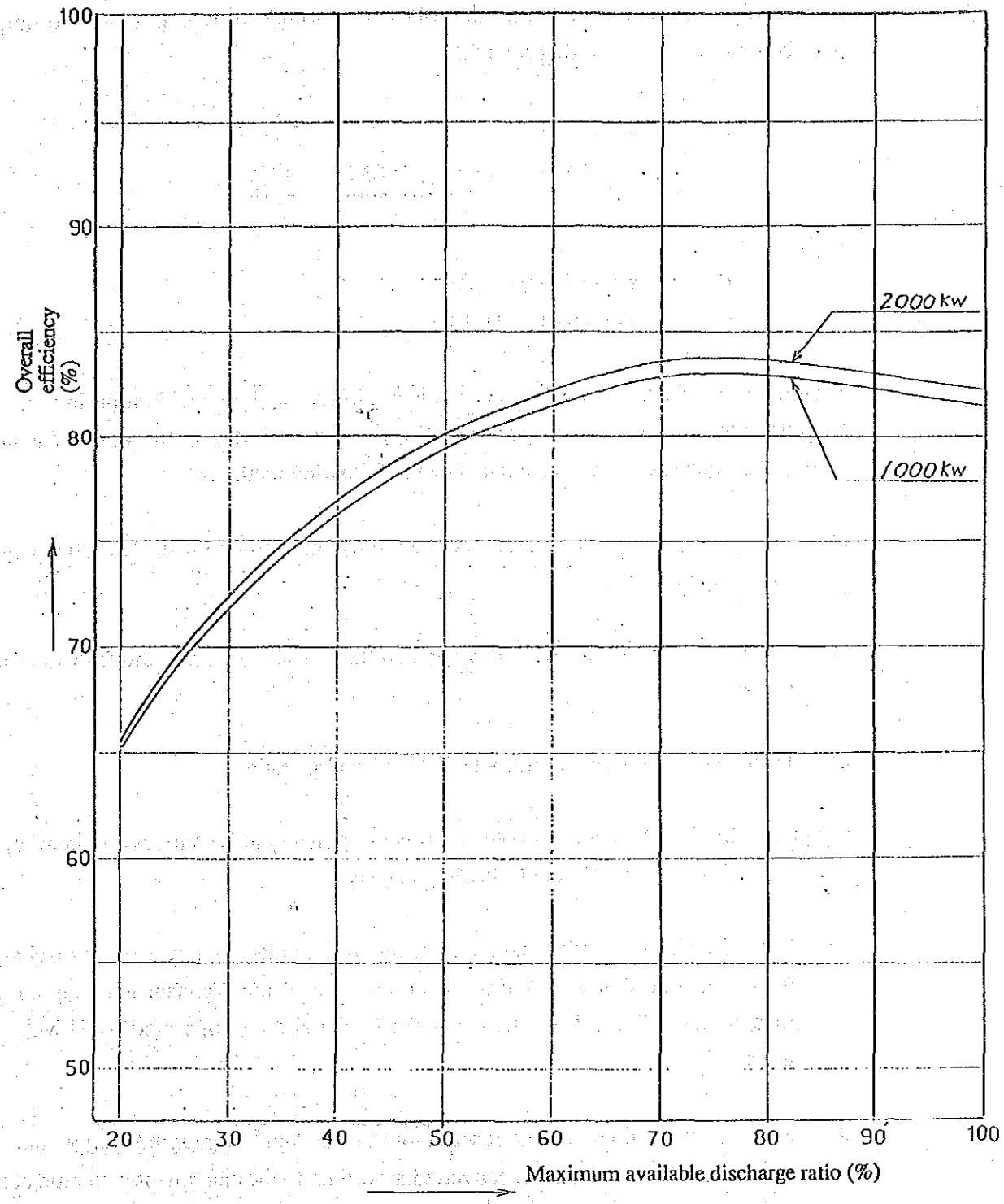
- P = generated output (kW)
- Q = arbitrary available discharge (m^3/s)
- H_e = standard net head (m)
- η = resultant efficiency of turbine and generator (resultant efficiency of the single unit capacity)
- 9.8 = constant (acceleration of gravity, m/s^2)

Resultant efficiency (η) is the value representing total efficiency obtained by multiplying the turbine efficiency (ηt) by the generator efficiency (ηg), and corresponds to the value of the maximum available discharge ratio 100% in the resultant efficiency curb as shown in Fig. 8.2 Table 8.2 shows the calculation result of the generated output for the alternative plan.

Table 8.2 Calculation of Generated Output

Item / Alternative plan	① Available discharge Q (m^3/s)	② Standard net head H (m)	③ 9.8 x ① x ② / Theoretical output (kW)	④ Resultant efficiency η	⑤ ③ x ④ / Generated output p (kW)
Alternative plan (REH-1)	50.0	4.4	2,156	0.815	1,757
Alternative plan (ALT-I)	100.0	4.4	4,312	0.823	3,548

Fig. 8.2 Resultant Efficiency Curve of Conduit Type Bulb Turbine and Generator



(Source: The above curves are drawn according to the study standard for formulation of mini-hydro power generating facilities plan prepared by the Structural Improvement Bureau of the Ministry of Agriculture, Forestry and Fishery)

8.3 Annual Potential Generated Energy

Generated energy is calculated by the following formula in which generated output (kw) is multiplied by operating time (hr).

$$E = P \times t \text{ (kWh)}$$
$$= 9.8 \times Q \times He \times \eta \times t$$

where:

P = generated output (kW)
t = operation time (hour)

Assuming that the power plant operation is not interrupted by accident during the nor suspended for maintenance, inspection and repair purposes during the year, the annual potential generated energy is calculated by the following methods.

(1) Using daily discharge in discharge data plus net head and resultant efficiency at that daily discharge

(2) Combining hydrological regime and resultant efficiency from the flow-duration curve

(3) Using the generating output-to-available discharge ratio

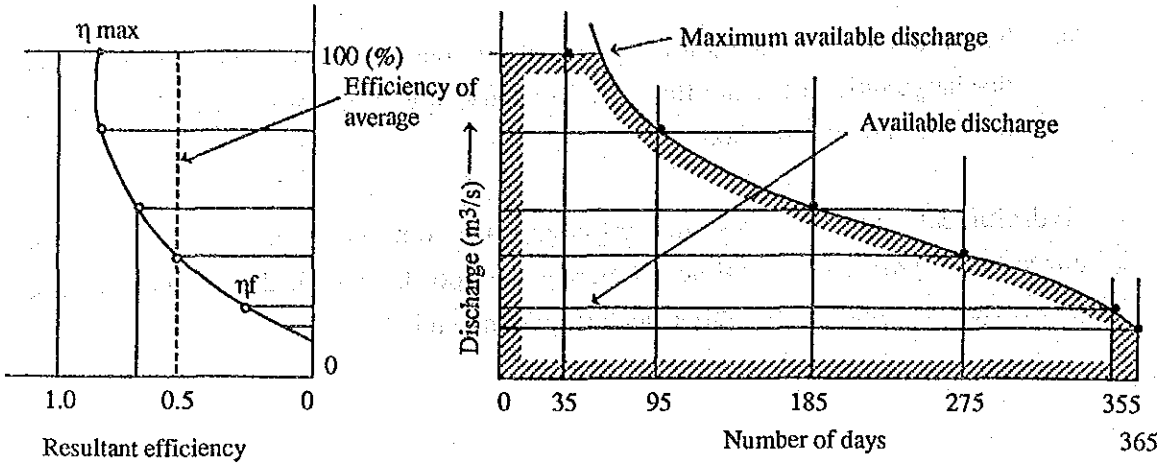
For the calculation of the annual potential generated energy at La Vuelta P/P, item (2) as mentioned above is used for the following reasons.

① Record observed at the intake site of this power plant is not used as discharge data. The one that is converted from the data of the Aguasal gauging station located about 3 km downstream of the intake site and operated by HIMAT is used.

② Since there are no recorded observations at the Aguasal gauging station and the intake site, discharge data is converted according to the catchment area ratio at the above gauging station and intake site.

③ The method for calculation using the average generating output-to-available discharge ratio of (3) and flow-duration curve are used. However, this method is not as accurate as method (2).

Hydrological regime and resultant efficiency are combined from the flow-duration curve, and hydrological regime-efficiency method, by which the annual potential generated energy can be roughly calculated, as shown below.



Max. available discharge = m^3/s Net head, He = m

① Day	② Number of days	③ Available discharge (m^3/s)	④ Burden ratio $\frac{\text{Available discharge}}{\text{Max. available discharge}}$	⑤ Resultant efficiency η	⑥ Generating power (kW)	⑦ Average power (kW)	⑧ Generated energy (kWh)
Max.							
95	95-						
185	185-95 = 90						
275	275-185 = 90						
355	355-275 =80						
365	365-355 = 10						
Total	365					( )	

① Possible intake water days of maximum available discharge are inserted for the day order ①.
② Represents the difference of the day order of calculation stage and right above stage. This example employed hydrological regime representative days as a matter of convenience.
③ The discharge of the day order topped out by maximum available discharge shall be an available discharge.
④ Available discharge divided by maximum available discharge shall be input load factor, and the resultant efficiency (5) shall be read and entered.
⑥ 9.8 x Q x He x η
⑦ Mean value of generated output of calculation stage and right above stage.
⑧ ⑦ x ② x 24 is the generated energy for calculated days, and the total value becomes yearly possible generated energy.

Fig. 8.3 Calculation of Annual Potential Generated Energy by the Hydrological Regime-efficiency Method

8.3.1 Calculation of Annual Potential Generated Energy

The annual potential generated energy for respective alternative plans is calculated according to the hydrological regime and efficiency method, with the following results:

(1) The annual potential generated energy in case of the alternative plan 1 (REH-1) in which new facilities layout (max. available discharge of 25 m^3/s x 2) is worked out::

15.4 GWh (100%)

(2) The annual potential generated energy in the case of the alternative plan 2 (ALT-1) (max. available discharge = 50 m^3/s x 2 = 100 m^3/s):

29.9 GWh (96%)

Table 8.3 Calculation of Annual Potential Generated Energy

(1) Alternative plan 1 (REH-1)

Max. available discharge Q = 25 m^3/s x 2 units
Standard net head He = 4.4 m
Turbine type: Condit type bulb turbine

① Day	② Number of days	③ Available discharge (m^3/s)	④ Burden ratio $\frac{\text{Available discharge}}{\text{Max. available discharge}}$	⑤ Resultant efficiency η	⑥ Generating power (kW)	⑦ Average power (kW)	⑧ Generated energy (MWh)
Max.	365	50	1.0	0.815	1,757	1,757	15,391

(2) Alternative plan 2 (ALT-1)

Max. available discharge: Q = 50 m^3/s x 2 units
Standard net head He: 4.4 m
Turbine type: Conduit type bulb turbine

Day	Number of days	Available discharge (m^3/s)	Burden ratio $\frac{\text{Available discharge}}{\text{Max. available discharge}}$	Resultant efficiency η	Generating power (kW)	Average power (kW)	Generated energy (MWh)
Max.	289	100	1.000	0.823	3,548	3,548	24,608
295	6	97.8	0.978	0.825	3,479	3,513	505
300	5	95.5	0.955	0.826	3,401	3,440	412
305	5	93.9	0.939	0.827	3,348	3,374	404
310	5	91.1	0.911	0.830	3,260	3,304	396
315	5	88.9	0.889	0.830	3,181	3,220	386
320	5	85.3	0.853	0.833	3,063	3,122	374
325	5	83.2	0.832	0.835	2,995	3,029	363
330	5	81.1	0.811	0.837	2,927	2,961	355
335	5	78.7	0.787	0.837	2,840	2,883	345
340	5	76.1	0.761	0.837	2,746	2,793	335
345	5	73.5	0.735	0.837	2,652	2,699	323
350	5	70.2	0.702	0.837	2,533	2,592	311
355	5	66.0	0.660	0.830	2,362	2,447	293
360	5	62.3	0.623	0.827	2,221	2,291	274
365	5	55.4	0.554	0.812	1,937	2,079	249
Total	365					(2,955)	29,933

CHAPTER 9 REHABILITATION PLAN

Since the present facilities-rehabilitating and output increase plans are not based on scrap and build methods, the power-generating capacity will be recovered or improved by making maximum use of existing facilities. The rehabilitation plan was formulated according to standards established by ISA (Interconexion Electrica SA) in June, 1987.

9.1 Formulation of Rehabilitation Plans

As stated in 4.3, all the headrace structures, in the power plant with the exception of renovation of 200-meter-long diversion weir, need to be improved or newly constructed. The generating equipment and transformer requires new procurement or replacement with new equipment. To compare the maximum available discharge, the following two rehabilitation plans are shown in Table 9.1.

$Q = 50.0\ m^3/s$
$Q = 100.0\ m^3/s$

For each rehabilitation plan the total costs, including construction costs per kW output and generating costs are calculated and compared. The optimum rehabilitation plan is then chosen.

Table 9.1 Comparison of Alternative Rehabilitation Plans

Item	Alternative		
	Rehabilitation of the existing facilities	Increase of power output	
	REH-1	ALT-1	ALT-2 (tentative)
Discharge, Q (m^3/s)	50	100	100
Max. output, P (kW)	1,700	3,500	7,700
Facility utilization factor (%)	100	96	96
Rehabilitation and improvement plan:			
Diversion weir	Restore TRINCHO at existing location		Renovate the TRINCHO with reinforced concrete
Forebay	New one at adjacent site		
Intake	New one at adjacent site		
Generating equipment	Replace with new equipment		
Powerhouse building	New building at adjacent site		
Tailrace	New one at adjacent site		

Although the rehabilitation plan of the existing facilities was listed as a candidate for consideration, it was not taken up for the reasons that the existing power house structure is not suitable for the new type of turbine generator to be installed and that operation of the canoes should be maintained during the construction period which is considered to be impossible.

9.2 Estimated Rehabilitation Construction Costs

The estimated construction costs can be calculated from the estimated costs for generating equipment and civil construction. This can then be divided into the foreign currency portion and the local currency portions and calculated at the current exchange rates (September 1989), based on the U.S. dollar.

9.2.1 Estimated Generating Equipment Costs

According to the ISA valuation standard, CIF cost of generating equipment are calculated based on the FOB from Japan. The generating equipment specifications and FOB costs are shown in Table 9.2.

The CIF/FOB ratio for the CIF costs is 1.12, as shown in Table 9.3.

Table 9.2 Generating Equipment Specifications and FOB Costs

Item	Alternative	
	REH-1	ALT-2
1. Specifications		
Design discharge (m^3/s)	25	50
Net head (m)	4.4	4.4
Theoretical output (kW)	1,078	2,156
Turbine type	Conduit bulb	Conduit bulb
Turbine output (kW)	920	1,860
Generator power factor	0.9	0.9
Generator output (kVA)	1,000	2,000
Main transformer capacity (kVA)	2,000	4,000
2. FOB costs (US$1,000)		
Generating Equipment		
(1) Turbine and ancillary equipment	789.3	1,117.15
(2) Generator and ancillary equipment	395	540.7
(3) = (1) + (2) Subtotal:	1,184.3	1,657.85
(4) Number of units	2	2
(5) = (3) x (4) Subtotal:	2,368.6	3,315.7
(6) 4.16 kV switchgear etc.	146.4	146.4
(7) Substation	244.3	275.7
(8) = (5) + (6) + (7) Total:	2,759.3	3,737.8

Table 9.3 Generating Equipment CIF Costs

(units: US$10^3)

Item		Alternative			
		REH-1		ALT-1	
		A	B	A	B
1) FOB cost		2,759.3	-	3,737.9	-
2) Transportation costs, insurance					
	1) x 0.12	331.1	-	448.5	-
3) Tax	1) x 0.223	-	615.3	-	833.6
4) Value-added tax	1) x 0.134	-	369.7	-	500.9
5) Others	1) x 0.22	-	607.0	-	822.3
6) Subtotal		3,090.4	1,592.0	4,186.4	2,156.8
7) Contingency	1) x 0.17	469.0	-	635.4	-
8) Eng. fee	1) x 0.149	411.1	-	556.9	-
9) Total	6) + 7) + 8)	3,970.5	1,592.0	5,378.7	2,156.8
10) Total		5,562.5		7,535.5	

Note: A = foreign currency portion
B = local currency portion

9.2.2 Estimation of Civil Construction Cost

The work volume for the rehabilitation or improvement of the main structures is multiplied by the unit costs (as shown in Table 5.2) as decided by E. CHOCO. The civil construction cost estimates are in the local currency base.

The total civil construction costs for each rehabilitation plan are calculated and compared as shown in Table 9.4.

Table 9.4 Estimation of Civil Construction Cost

(unit: 10^6 pesos)

Item	Alternative	
	REH-1	ALT-1
Diversion weir construction	18.2	18.2
Forebay construction	100.2	130.4
Intake construction	93.0	168.2
Foundation of equipment construction	296.0	363.9
Powerhouse building construction	31.9	36.4
Tailrace construction	46.7	135.1
Temporary facility construction	35.1	35.1
Other construction	52.0	52.0
① Subtotal	673.1	939.3
② Contingency (① x 0.15)	101.0	140.9
③ Engineering fees ((① + ②) x 0.10)	77.4	108.0
④ Total (① + ② + ③)	851.5	1,188.2
⑤ Output loss	37.9	37.9
⑥ Grand Total (④ + ⑤)	889.4	1,226.1

9.3 Comparison of Economic Indices

To compare the economic indices of the construction cost per kW and the generating cost per kW, the basic conditions for all the alternative plans are as follows.

(1) Exchange rate based on September 1989, is as follows.

US$ 1 = ¥140
US$ 1 = 369.4 pesos
1 peso = ¥0.379

(2) The life of new generating equipment, as well as repaired and reconstructed structures is 25 years.

(3) The interest rate is divided into the foreign currency and local currency portions under the following conditions.

- The foreign currency portion is based on an annual interest rate of 10% (unredeemable for 4 years), with repayment of the principal in equal annual amounts over 25 years.

- The local currency portion is based on an annual interest rate of 21% (unredeemable for 1 year), with repayment of the principal in equal annual amounts over 8 years.

(4) The operation, maintenance and management costs of hydroelectric power plants per year is US$4 per installed-capacity (kW).

9.3.1 Comparison of Construction Cost per kW

A comparison of the construction cost per kW is shown in Table 9.5. The ALT-1 plan calls for US$3,600/kW per increase in power output, which is the lowest cost.

Table 9.5 Comparison of Construction Costs per kW

Item		Alternative	
		REH-1	ALT-1
Existing equipment output (kW)			
Rated output	Po	2,000	2,000
Available output	Pe	500	500
Post-rehabilitation output	P_1 (kW)	1,700	3,500
Recovered/increased output $\Delta P = P_1 - Pe$ (kW)		1,200	3,000
Rehabilitation work cost	(US$1,000)		
Foreign currency portion Cf		3,950	5,400
Local currency portion C		4,010	5,470
Total C = Cf + C		7,960	10,870
Construction cost per kW	(US$/kW)		
C/P_1		4,700	3,100
$C/\Delta P$		6,600	3,600

9.3.2 Comparison of Generating Cost per kWh

The generating cost per kWh is calculated from the following equation:

$$\text{Generating cost} = \frac{\text{Total cost at generating terminal}}{\text{Supplied output per year}}$$

where:

the supplied output per year = annual potential generated energy (E) x utilization factor

= 0.95 E

The annual total cost at generating terminal is shown in Figure 9.1. Since the estimated service life of the hydroelectric power plant is 25 years, the operation, maintenance and management costs (AOM per year = US$4 per kW) plus interest payments for the construction are totaled and divided by 25 years.

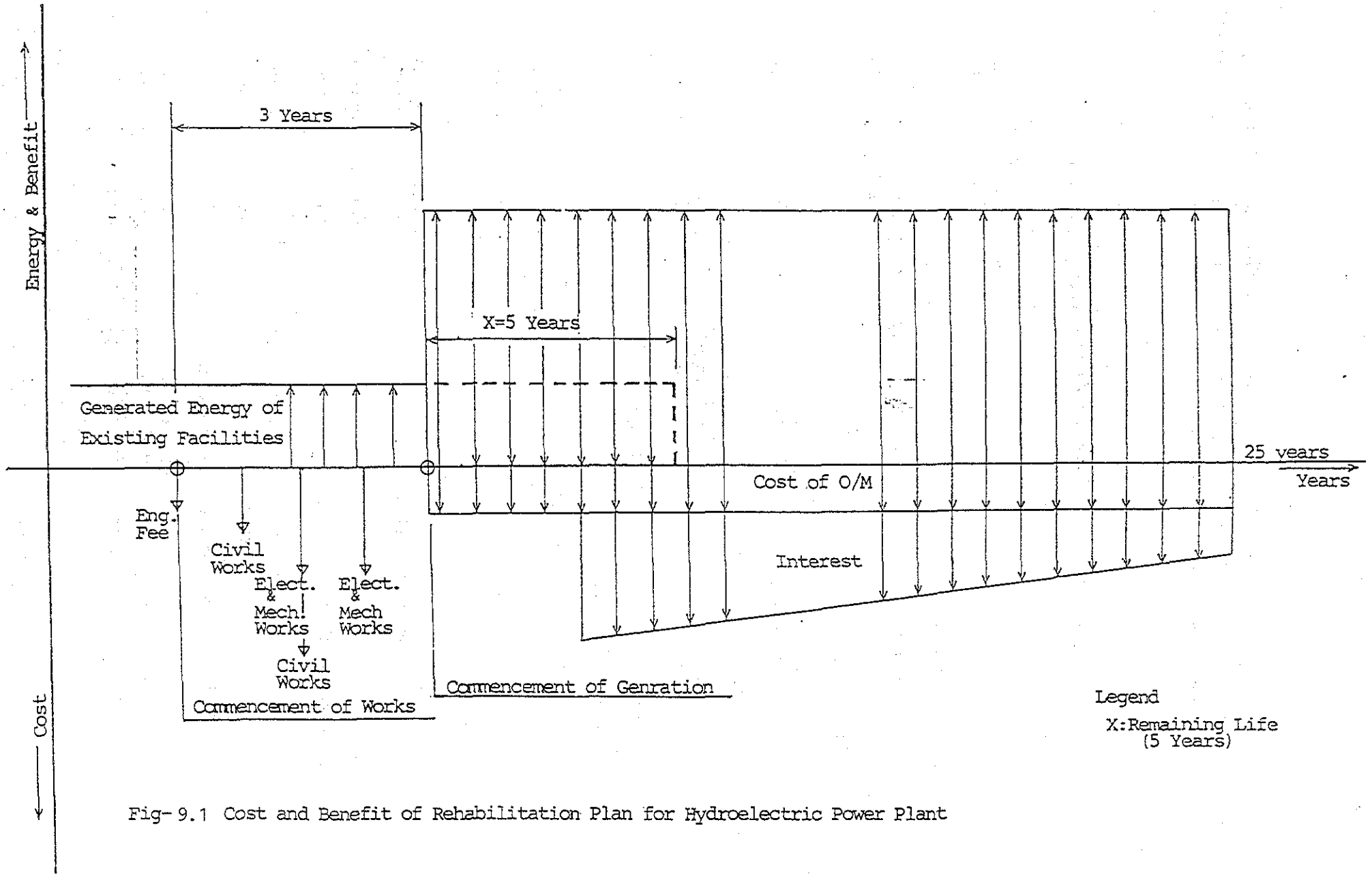


Fig- 9.1 Cost and Benefit of Rehabilitation Plan for Hydroelectric Power Plant

The results of calculation of generating costs per kWh are shown in Table 9.6. The generating cost per annually supplied power is 36 mills/kWh, according to ALT-1, showing the lowest costs respectively.

Table 9.6 Comparison of Generating Cost per kWh

Item		Alternative	
		REH -1	ALT-1
Existing equipment capacity:			
Power output	Pe (kW)	500	500
Energy	Ee (GWh)	6.25	6.25
Rehabilitation plan:			
Power output	P_1 (kW)	1,700	3,500
Generated energy	E_1 (GWh)	15.4	29.9
Recovered/increased power			
Output	$\Delta P = P_1 - Pe$ (kW)	1,200	3,000
Energy	$\Delta E = E_1 - Ee$ (GWh)	9.1	23.6
Total of expenses at generating terminal: (US$1,000)			
Construction work cost			
Foreign currency portion Cf_1		3,950	5,400
Local currency portion $C\ell_1$		4,010	5,470
Construction cost total $C_1 = Cf + C\ell_1$		7,960	10,870
Interest payment C_2			
Foreign currency portion Cf_2		6,392.5	8,659.7
Local currency portion $C\ell_2$		4,066	5,564.4
Total $C_2 = Cf_2 + C\ell_2$		10,458.5	14,224.1
AOM C_3 = US$4 x P_1 x 25 years		170	350
Total $\Sigma Ci = C_1 + C_2 + C_3$		18,601	25,429.6
Average annual cost $C = \Sigma Ci/25$		744	1,017
Generating cost per annually supplied energy (mills/kWh)			
Per E_1	$C/(E_1 \times 0.95)$	51	36
Per ΔE	$C/(\Delta E \times 0.95)$	86	45

9.3.3 Overall Evaluation

Taking the construction costs per kW and generating costs per kWh for each alternative plan into account in the cost-benefit analysis, ALT-1 plan is thus selected as the optimum plan.