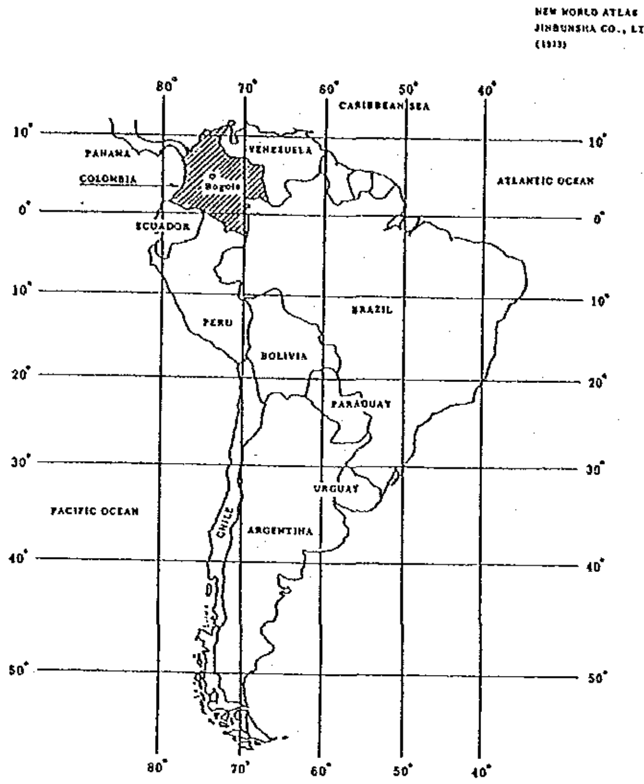
FEASIBILITY STUDY
ON
SMALL-SCALE POWER PLANTS
REHABILITATION PROJECT
IN
THE REPUBLIC OF COLOMBIA

JULIO BRAVO HYDROELECTRIC POWER PLANT

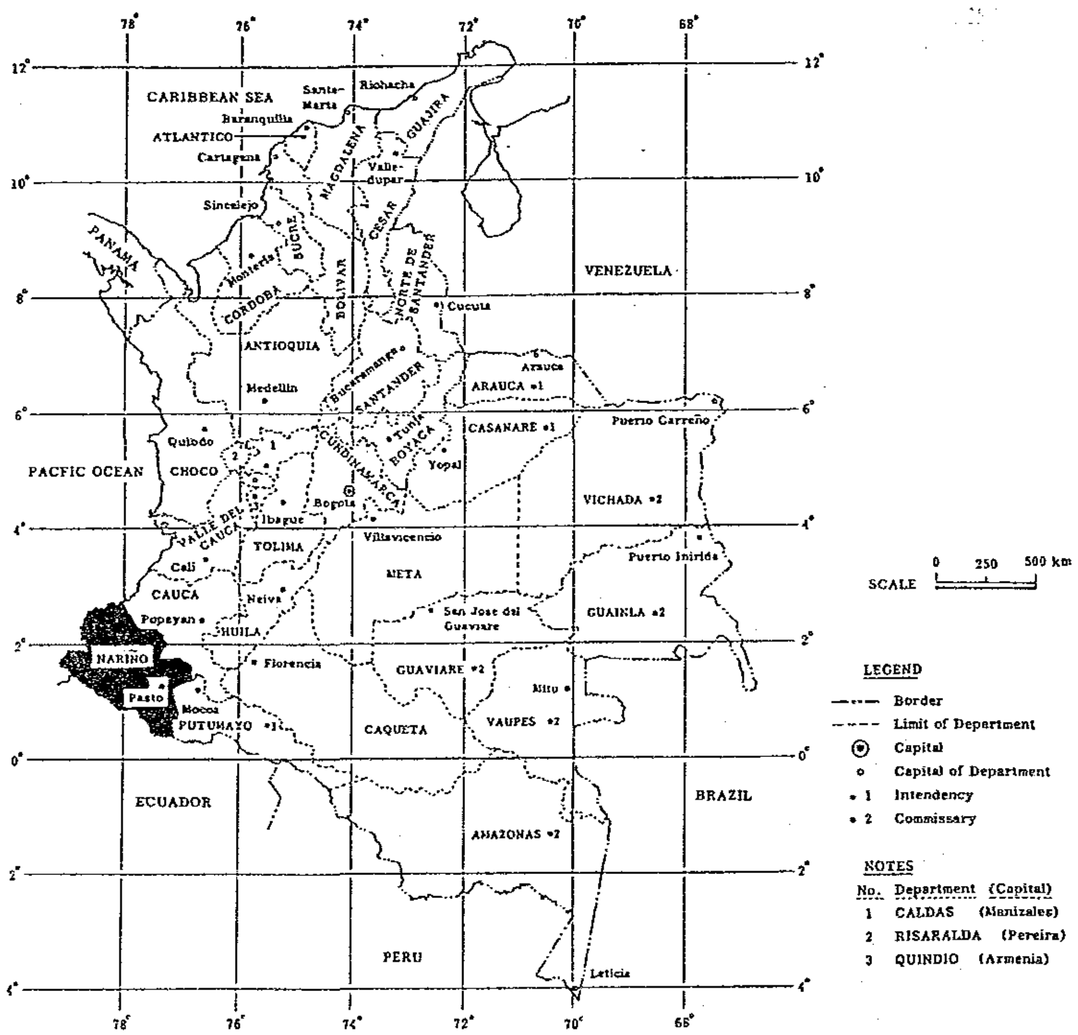
MARCH 1990

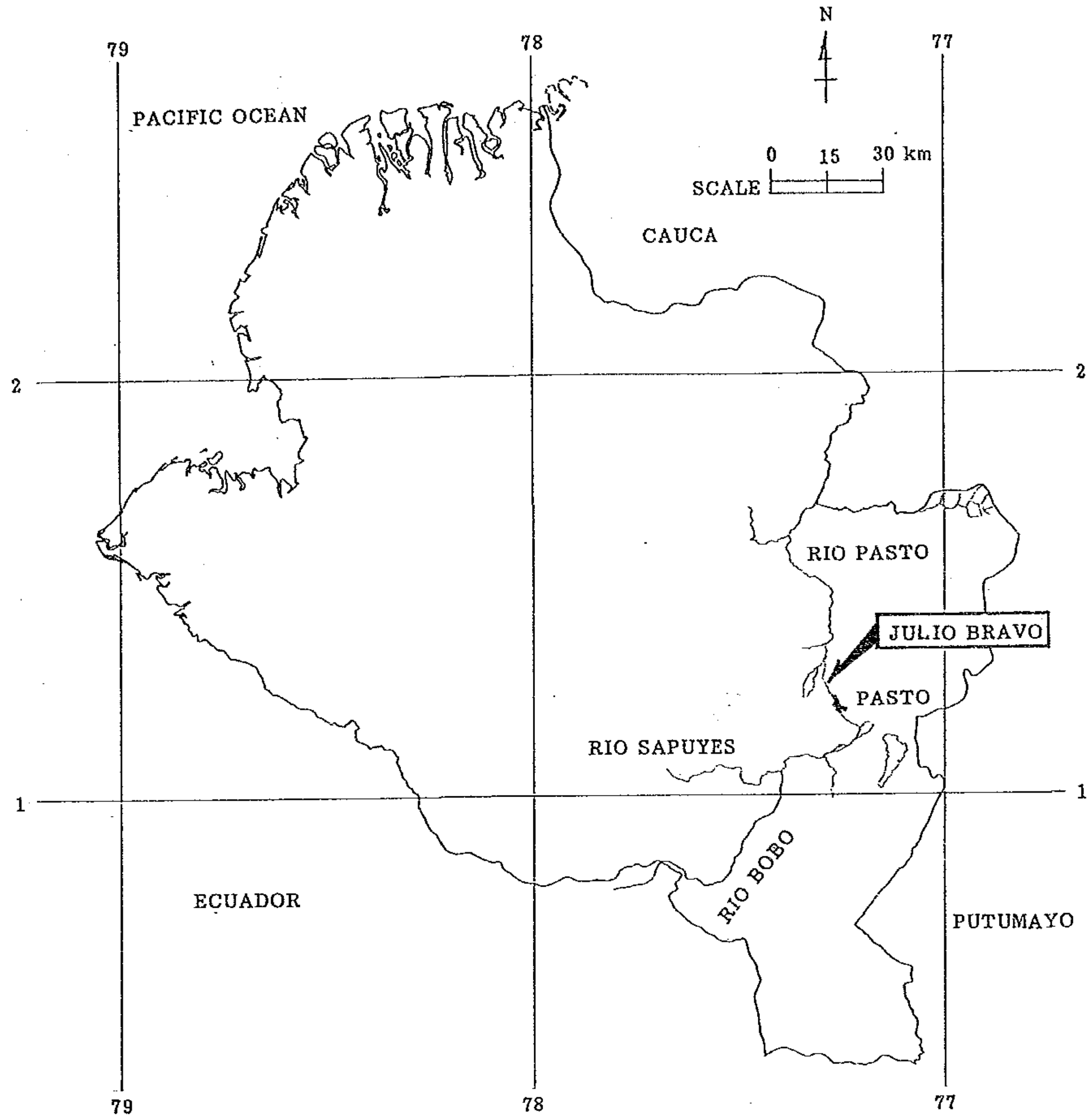
Japan International Cooperation Agency

MAP OF SOUTH AMERICA

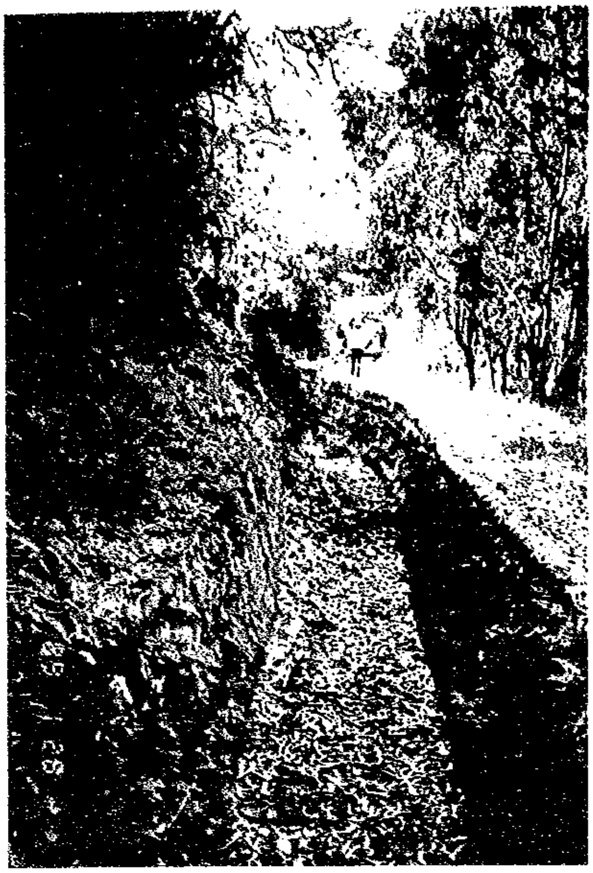


POLITICAL DIVISION IN THE REPUBLIC OF COLOMBIA



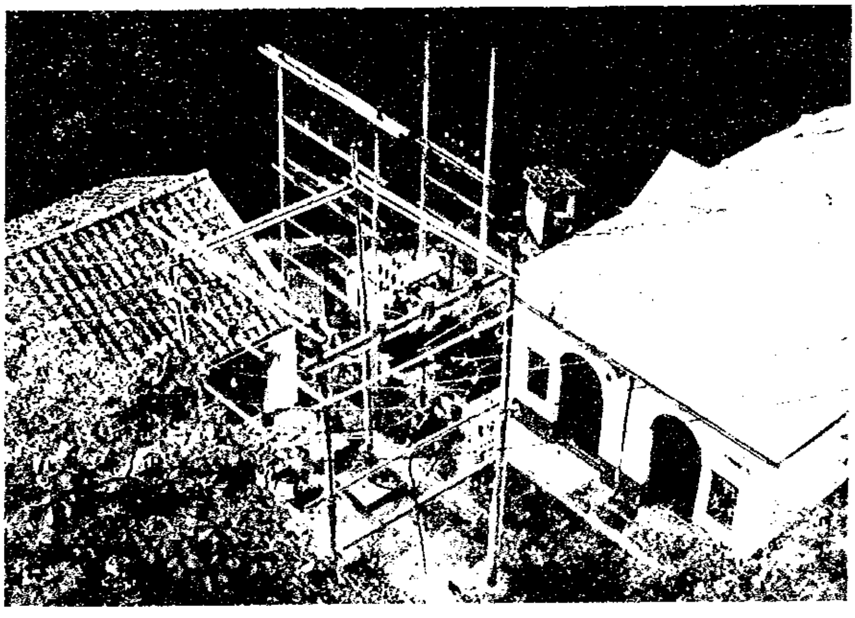


Location Map of Study Area

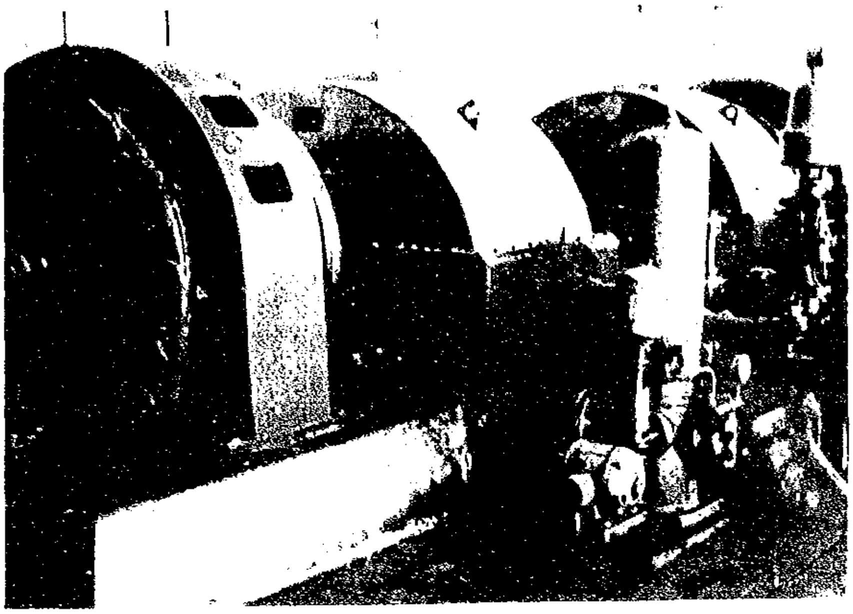


Rio Pasto and conduction channel at intake

Conduction channel



Substation and powerhouse



Pelton turbine and generator

Generator control panel

CONTENTS

CHAPTER 1 INTRODUCTION

The feasibility study (hereinafter referred to as the FS) for the rehabilitation plan of Julio Bravo run-of-river type hydroelectric power plant (rated output of 1.5 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 was conducted for 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, Julio Bravo hydroelectric power plant (hereinafter referred to as Julio Bravo P/P) was selected as a candidate for the FS for the following reasons:

1) Basic data relating to topography, river discharge and so on was comparatively well organized.
2) There is no possibility of environmental destruction, and water right for power generation has already been acquired.
3) About 2,500-meter-long headrace is maintained in a rigid state.
4) CEDENAR nominated Julio Bravo P/P as the highest priority for the rehabilitation.

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

- Maximum output : 3.5 MW
- Annual potential generated power : 29.4 GWh
- Facility utilization factor : 97 %

CHAPTER 2 SUMMARY OF STUDY RESULTS

The power plant owned by CEDENAR, is the run-of-river type (the rated output: 1,500 kW), and is located along the Pasto River in Nariño Department. This power plant has not been operating since 1984 because of damage to penstocks and generating equipment.

(1) Present condition of generating facilities and their problems

Three Pelton turbines (#1, #2 and #3), each with a rated output of 500 kW, were installed in 1942. #3 unit broke down and was removed. In 1948, new penstocks were installed because there was a burst in the old ones. #2 unit was transferred to a new location as #4 unit, and #5 unit was installed as a substitute for #3.

The present generating equipment consists of three units of #1, #4 and #5. The newly installed penstocks (in the second row) could not be used because they were worn out and perforated. Since then, the plant has not been operating. Since the existing #1, #4 and #5 units have been used for 42 to 48 years since their installation, they are severely worn out, and have been left unattended without any inspection or maintenance. Outdoor transformers were removed and transferred to another power plant site. The 2,500-meter-long headrace is an open channel constructed of masonry, and is maintained in fairly good condition, except for several damage to the diversion weir and the intake. Though the present desilting basin is maintained in good shape, it does not function well because of its obsolete design. Sewage from Pasto City located upstream flows into river water near the intake, polluting the river water. (Refer to Table 2.1.)

Table 2.1 Results of the Pasto River Water Quality Analysis

Year	pH	Specific Resistance (Ω·cm)
1985	6.3 - 4.0	345 - 166
1986	6.8 - 4.4	346 - 162
1987	6.8 - 4.2	302 - 182
1988	5.2 - 4.6	460 - 315

(2) Alternative rehabilitation plans

Except for a 2,500-meter segment, the headrace structure needs to be improved or re-constructed, because of damage. New generating equipment and transformer also need to be procured for the reasons stated previously.

The results of hydrological analysis indicate that the existing open channel is capable of safely discharging at a rate of 4.0 m^3/s. Therefore, comparative studies are made not only for the existing facilities-rehabilitating plan, but also for the power generation-optimizing plan, and comparative study plans are formulated.

The designed available discharge is set up to three cases of 2 m^3/s (max. available discharge in the existing power plant), 3 m^3/s and 4 m^3/s within the range that the facility utilization factor does not exceed 50%, as shown in typical flow-duration curve (refer to Fig. 2.1).

Comparative studies are made for respective generation plan. The concept of alternative rehabilitation plan selected is summarized in Table 2.2.

Since the JICA Study Team has an allowance of approximately 20 m in the standard net head for the rehabilitation of the existing facilities (designed available discharge: 2.0 m^3/s), and there is a difference between the theoretically calculated generated output and the installed capacity, the installed capacity of the existing generating facilities (1,500 kW) will inevitably increase.

Fig-2.1 TYPICAL FLOW DURATION CURVE AT INTAKE SITE

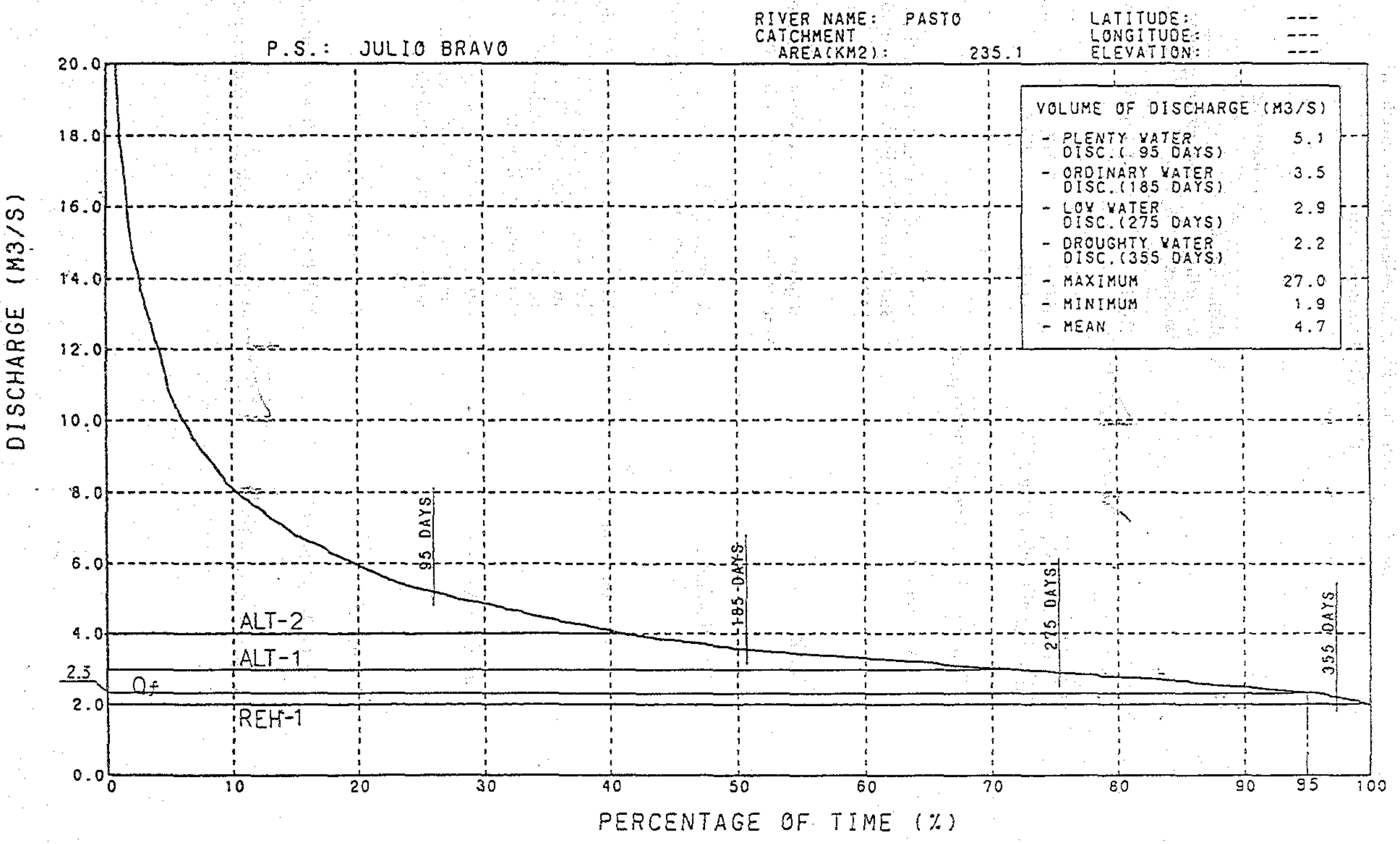


Table 2.2 Alternative Plans for Julio Bravo Hydroelectric Power Plant Rehabilitation

Item	Alternative		
	Rehabilitation of the existing facilities	Increase of power output	
	REH-1	ALT-1	ALT-2
Discharge, Q (m^3/s)	2.0	3.0	4.0
Max. output, P (kW)	2,300.0	3,500.0	4,600.0
Facility utilization factor (%)	100.0	97.0	85.0

Rehabilitation and improvement plan:

Deversion weir	Because of damage to this weir, a new standtrap will be built (common to all plans)
Intake	Improvement to allow for stable intake of maximum available discharge.
Desilting basin	A new, suitable-sized one will be constructed (common to all plans).
Conduction channel	The cross section at certain sections will be enlarged to allow for stable intake of maximum available discharge. Placement of covers for full length of channel (common to all plans).
Head tank	Will be expanded at its present location to increase regulating capacity and a new spillway will be installed.
Penstocks	New construction
Generating equipment	Will be replaced with new equipment.
Powerhouse building	Will be expanded at its present location.

(3) Selection of optimum plan

Comparative study results of alternative plans are summarized in Table 2.3. ALT-1 is selected as an optimum plan of the Julio Bravo Plant rehabilitation because of its economic advantage and many benefits.

The concept of the basic design for ALT-1 that will be carried out during the feasibility study is described in Chapter 11.

Table 2.3 Comparison of Rehabilitation Plan for the Julio Bravo 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
				⑭	⑮									
	Max. available discharge Qo (m^3/s)	Net head Ho (m)	Rated output Po (kW)	Output Pe (kW)	Generated energy Ee (GWh)	Max. available discharge Ql (m^3/s)	Standard net head Hl (m)	Theoretical output =9.8x⑳x㉑ (kW)	Resultant efficiency η	Output =㉒x㉓ Pl (kW)	Annual probable generated energy El (GWh)	Facility utilization factor ε (%)	Output =㉔-⑭ ΔP (kW)	Annual probable generated energy ㉕-⑮ ΔE (GWh)
REH-1	2.0	120.0	1,500	0	0	2.0	143,0	2,802	0.830	2,300	20.4	100	2,300	20.4
ALT-1						3.0	143.0	4,204	0.835	3,500	29.4	97	3,500	29.4
ALT-2						4.0	143.0	5,605	0.835	4,600	34.6	85	4,600	34.6

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	㊺	㊿	51	60	61 Principal repayment amount for construction cost (25-year average)			65	70	71		
	㊶	㊷	㊸	Civil work cost	㊸+㊹	Cost per ΔP =㊺/㉚	Cost per Pl =㊺/㉔	Operation and maintenance costs	62 Foreign currency portion	63 Local currency portion	64	60+64	per El =65/㉕ ÷0.95	per ΔE =65/㉛ ÷0.95	C/B	Priority order
	Foreign currency portion Clf	Local currency portion Clℓ	㊶+㊷ Cl	C2	C	C/ΔP	C/Pl	AOM	2.610 x ㊶ ÷ 25	2.016 x (㊷+㊹) ÷ 25	62+63					
REH-1	1,900	750	2,650	950	3,600	1,570	1,570	9.2	200	136	336	345	18	18	1.16	3
ALT-1	2,300	950	3,250	1,050	4,300	1,220	1,220	14.0	242	158	400	414	15	15	0.96	1
ALT-2	2,650	1,050	3,700	1,200	4,900	1,070	1,070	18.4	275	183	458	476	15	15	0.94	1

(Notes)

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

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

⑧ : C/B is the value of cost and benefit ratio calculated according to the financial analysis.

⑮ : Ee is computed according to the average annual operation record for 5 years from 1984 to 1988 .

㉓ : η is the resultant efficiency of turbine and generator.

㉕ : El(Energia Media)

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

60 : 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	Ing. Civil	Jefe Div. de Centrales
Jairo E. Gonzalez Morales	Ing. Civil	Ing. Div. de Centrales
Mario Gutierrez Ospina	Ing. Civil	Ing. Div. de Position
Rafael Torres Mariño	Ing. Civil	Ing. Div. de Centrales
Rafael Gomez Florez	Ing. Civil	Ing. Div. de Centrales
Jorge E. Hurtado Muños	Ing. Civil	Ing. Div. de Centrales

3.1.3 Supporting Technical Staff from CEDENAR

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

Supporting Staff	Position
Hernando Carreño Pilonieta	President
Enrique Moreno B.	Vice President
Diego Delgado Ruiz	Director of Power Generation/Transmission Program
Juan Carios Salazar	Civil Engineer
Alvaro E. Martinez	Civil Engineer

3.2 Study Items and Study Schedule

The 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 described in the S/W are as follows:

(1) Review of the 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

Table 3.1 shows the overall study schedule as indicated in the S/W.

Table 3.1 Time Schedule of FS

		Year	1988		1989												1990			
		Month	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4
		Project month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	
Working item		1. Review of existing data																		
		2. Site reconnaissance																		
	Field works	(1) Programming																		
		(2) Procurement procedure																		
		(3) Ground survey																		
		(4) Photogrammetric mapping																		
		(5) Geological investigation																		
		(6) Data collection																		
		4. Power survey																		
		5. Optimum plan																		
		6. Feasibility design																		
		7. Stability & safety analyses																		
		8. Construction method																		
		9. Cost estimation																		
		10. Economic and financial analyses																		
		11. Maintenance manual																		
Report		1. Inception report	Δ																	
		2. Progress report				① Δ										② Δ				
		3. Interim report											Δ Δ							
		4. Draft final report															Δ Δ			
		5. Final report																		Δ

Legend: JICA field operation; ICEL field operation; JICA operation in Japan; Δ Δ Report submission

Two field surveys were conducted at Julio Bravo P/P, as shown in Table 3.2.

In the first site reconnaissance, two civil engineers conducted the present-condition study of the existing facilities (mainly civil structures) and collected necessary data.

In the second field survey, a geologist and hydroelectric power generation planner gathered data relating to the geological survey.

Table 3.2 Field Survey Schedule

The first site reconnaissance

Date	Schedule	Detail of Study Item	Member	
			ICEL	JICA
Jan. 27		Discussion at CEDENAR, and data collection	J. Gonzalez	Murao Toyama Yoshio Kawasaki
Jan. 28		Field survey at J. Bravo P/P		
Jan. 29		Data analysis		
Jan. 30		Discussion at CEDENAR		

The second field survey

Date	Schedule	Detail of Study Item	Member	
			ICEL	JICA
July 12	Bogota → Pasto	Discussion at CEDENAR, and data collection	J. Gonzalez	Murao Toyama Nobuhiko Uchiseto
July 13		Field survey at J. Bravo P/P.		
July 14		Same as above		
July 15	Pasto → Bogota	Travel		

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 included topographic surveying and boring survey as described below, but did not include photogrammetric mapping.

3.3.1 Scope of Topographic Surveying

The scope of topographic surveying is shown in Figure 3.1. The scales for the topographic maps are as follows:

(1) The existing reservoir, diversion weir, intake, conduction channel, desilting basin, head tank and powerhouse building are drawn on a scale of 1/200 with a contour line of 2 m. Main structures for the existing facilities and position of bench marks and boring are indicated in the above drawings.

(2) Penstock

Longitudinal section of the existing penstock is drawn on a scale of 1/500 (plan) and 1/200 (section). This section is also drawn on a scale of 1/100, and with 20 m width and 25 m pitch.

(3) Bench mark

The bench marks are set up at the three locations.

3.3.2 Boring Survey Work Plan

The boring survey was conducted as follows:

No.	Location	Depth	Note
BH-1	The right side of the diversion weir	10 m	The location of boring hole is shown in Fig. 3.1.
BH-2	Adjacent to the desilting basin	10 m	
BH-3	Head tank	10 m	
BH-4	Powerhouse building	10 m	

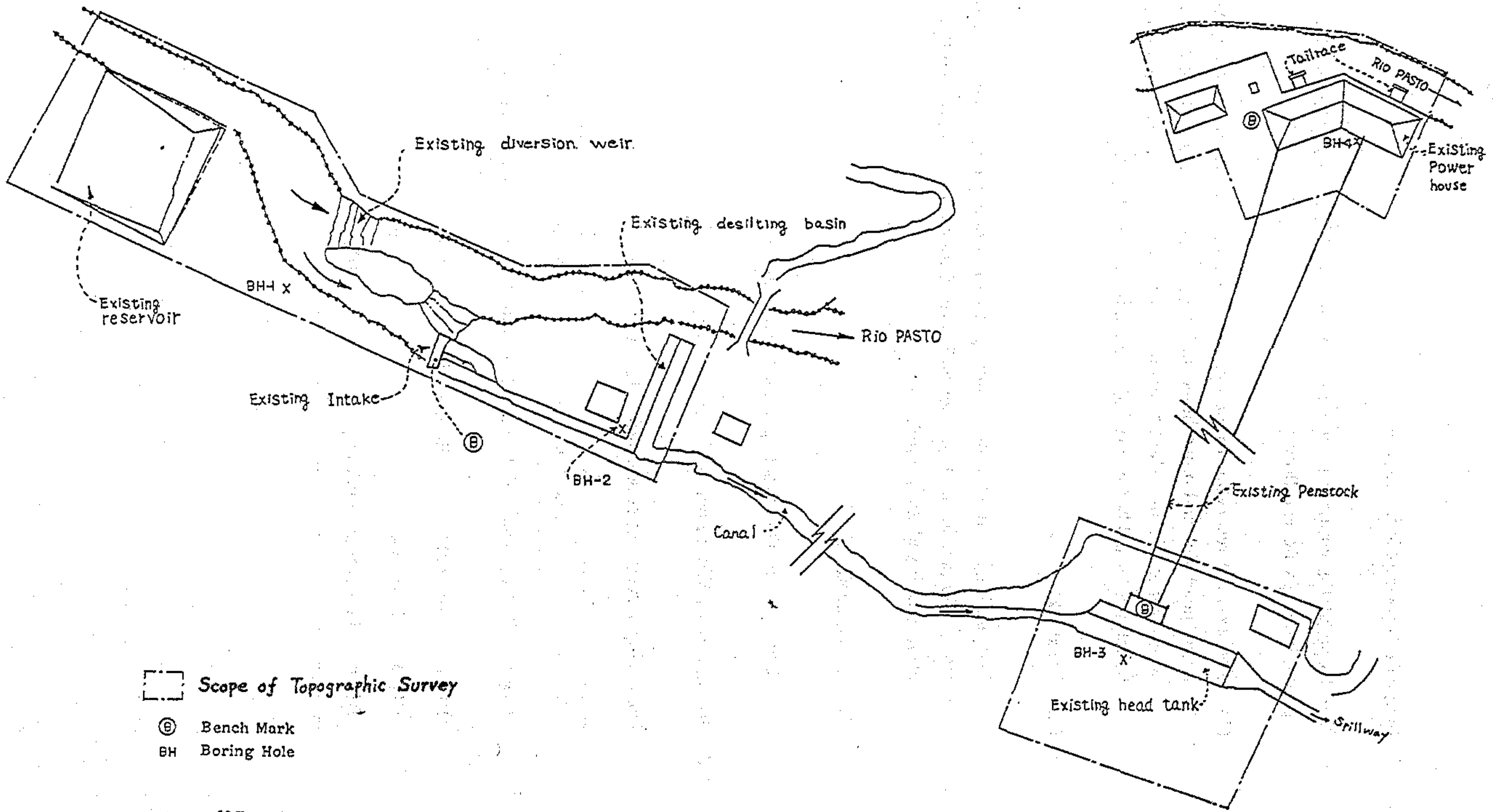


Fig. 3.1 Scope of Topographic Survey and Location of Boring Holes

CHAPTER 4 PRESENT CONDITION OF THE STUDY AREA

4.1 Power Conditions in the Power Sector

Power conditions in the plant owned by the public electric power company being evaluated for rehabilitation (hereinafter called public electric power company) are described below.

4.1.1 Balance of Power Supply and Demand

Table 4.1 shows figures for power supply and demand during the five years from 1983 to 1987. In 1987, peak demand was 79 MW, while installed capacity was 39 MW (49%). In 1987, electric power was 276 GWh, while supplied power was 176 GWh, representing about 64% of total electric power. The public electric power company buys electricity to cover the remaining 294 GWh from an other electric power company.

The breakdown of power demand in 1987 indicates that power demand for residential, commercial, industrial, and miscellaneous uses was 74%, 7%, 5% and 14% respectively. Power demand for residential was high, while that for commercial was low.

Annual average rate of increase in power demand from 1983 to 1987 was 3.9%, while that of generated energy, 1.0%, showing no marked fluctuations. Because annual average rate of increase in power loss was 19.9%, the rate of buying electricity was high.

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)	68	70	74	76	79	3.8
2. Electric Energy (GWh)						
1) Residential	179	196	192	191	205	3.4
2) Commercial	15	15	16	17	19	6.1
3) Industrial	19	18	23	20	15	-5.7
4) Miscellaneous	24	26	27	32	37	11.4
Total	237	255	258	260	276	3.9
SUPPLY						
1. Installed Capacity (MW)	35	35	39	39	39	2.7
2. Generated Energy (GWh)	169	168	176	182	176	1.0
3. Power Loss (GWh)	94	105	139	174	194	19.9

(Source: INFORME ESTADISTICO: RESUMEN 1983-1987)

* Annual average increase rate is calculated as follows:

Example: When peak demand is 3.8%, $68 \times (1 + x)^4 = 79$

$x = 0.038$ (3.8%)

4.1.2 Present Conditions of Generating Facilities

(1) Generating facilities

Table 4.2 shows total installed capacity of the public electric power company. Its generating system facilities include hydroelectric power and diesel power.

Table 4.2 Total Installed Capacity of the Public Electric Power Company

Item	1983	1984	1985	1986	1987	Annual Average Increase Rate (%)
Total Installed Capacity (MW)						
1. Diesel	6	6	10	10	10	13.6
2. Hydroelectric	29	29	29	29	29	0
3. Others	0	0	0	0	0	0
Total	35	35	39	39	39	2.7

(Source: INFORME ESTADISTICO: RESUMEN 1983-87)

Table 4.3 shows condition of power plants for which the FS was conducted.

Table 4.3 Conditions of Julio Bravo Power Plant (1984-1988)

Item	1984	1985	1986	1987	1988
1) Installed capacity (kW)	1,500	1,500	1,500	1,500	1,500
2) Generated energy (MWh)	0	0	0	0	0
3) Utilization factor (%)	0	0	0	0	0
4) Operating time (%)	0	0	0	0	0

(2) Transmission facilities

The public electric power company provides 115 kV transmission lines to its transmission and substation facilities at Julio Bravo P/P. Voltage to be transmitted to Julio Bravo P/P are 6.6 kV and 13.2 kV.

4.1.3 Generating Cost and Electric Charges

Table 4.4 indicates the transition of generating cost and electric charges in the past five years 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$/kWh)	3.84	4.8	5.85	8.17	9.84	26.5
Electric Charge (Average): (COL$/kWh)						
1. Residential	2.32	2.76	3.47	4.61	6.19	27.8
2. Commercial	4.31	5.76	6.69	8.97	11.51	27.8
3. Industrial	3.49	4.51	4.98	8.07	12.80	38.4
4. Public use	3.45	4.31	5.55	7.52	9.84	30.0
5. Average	2.61	3.16	4.00	5.48	7.34	29.5
Breakdown of Power Demand by customer						
1. Residential	82,445	88,931	96,464	102,838	110,438	7.6
2. Commercial	2,526	2,513	2,698	2,844	3,617	9.4
3. Industrial	503	514	547	597	634	6.0
4. Others	739	764	827	871	931	5.9
5. Total	86,213	92,722	100,536	107,150	115,620	7.6
Diffusion of Electricity						
1. Overall (1000 households)	980	999	1,019	1,039	1,059	2.0
2. Power demand (1000 households)	395	426	462	493	529	7.6
3. Electrification rate (%)	40	43	45	47	50	5.7

(Source: INFORME ESTADISTICO: RESUMEN 1983-87)

4.1.4 Forecast of Power Demand

CEDENAR forecast for the power demand from 1990 to 2000 is shown in the following table.

Year	Power Demand	
	Electric Power (GWh)	Peak Demand (MW)
1990	546.406	92.1
1991	581.035	97.2
1992	610.833	102.2
1993	651.228	109.0
1994	693.400	116.0
1995	733.527	122.7
1996	776.189	129.9
2000	979.921	164.0

4.2 Operation Record of the Existing Power Plant

4.2.1 Generated Energy

Record of generated energy during six years from 1979 to 1984 is shown in Table 4.5. Cracks in the penstock were discovered and two units have not been operating since March, 1983.

The value of the utilization factor in 1980 was 49% which is the maximum in the past years, and therefore this power plant was not in operation in good condition.

Table 4.5 Record of Generated Energy

Year	Unit Number	Output inscribed on the name plate (MW)	Running Period	Generated Energy (MWh)	Facility Utilization Factor (%)*
1979	1	0.5	Feb. to Mar.	5,717	33
	2	"	Jan. to Apr.		
	3	"	Jul. to Dec.		
	4	"	May to Sept.		
1980	1	"	0	8,508	49
	2	"	0		
	3	"	Jan. to Jun.		
	4	"	Jul. to Dec.		
1981	1	"	0	7,815	45
	2	"	Dec.		
	3	"	Apr. to Sept.		
	4	"	Jan. to Nov.		
1982	1	"	0	6,005	34
	2	"	Jan. to Apr.		
	3	"	Jun.		
	4	"	May to Dec.		
1983	1	"	0	2,769	16
	2	"	Apr. to Dec.		
	3	"	0		
	4	"	Jan. to Mar.		
1984	1	"	0	472	3
	2	"	Jan. to Feb.		
	3	"	0		
	4	"	0		

(Note)

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

4.2.2 Operation and Maintenance Costs

Record of Julio Bravo P/P's operation and maintenance costs for seven months from October, 1981 to April, 1982 is shown in Table 4.5.

Operation and maintenance costs per generated energy fluctuate, but average figure was 1,217 pesos/MWh.

Table 4.6 Record of Operation and Maintenance Costs

Month/ year	Generated energy (MWh)	Operation Cost (pesos)	Maintenance Cost (pesos)	Total (pesos)	Peso MWh
Oct., 1981	577.00	312,666.95	180,538.00	493,204.95	855
Nov., 1981	493.00	284,456.70	11,972.80	296,429.50	601
Dec., 1981	497.00	482,253.34	No data available	482,253.34	970
Jan., 1982	80.00	244,259.49	111,110.46	355,369.95	4,442
Feb., 1982	82.35	233,305.00	252,098.00	485,403.00	5,894
Mar., 1982	401.00	274,137.33	194,423.25	468,560.58	1,168
Apr., 1982	436.00	276,905.56	264,006.50	540,912.06	1,241
Total	2,566.35	2,107,984.37	1,014,149.01	3,122,133.38	15,171

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

No units have been operating since February, 1984, because of perforation to the penstock. They have been left unattended, without any maintenance or inspection.

The generating equipment has been in operation for 42 to 46 years from the start of commercial operation. Since then, it has remained unused for five years. Therefore, the reliability and efficiency of the generating equipment are questionable, and operation of such generating equipment is ill-advised.

Tables 4.7 and 4.8 show defects in turbines and generators found in the survey conducted by CEDENAR. Thus, CEDENAR desires new replacements.

Table 4.7 Major Defects in Turbines and Auxiliary Equipment

Equipment	#1 unit	#2 unit	#3 unit	#4 unit
Water unit turbine	1) Nozzles and needles are excessively worn out.	#2 unit was removed.	1) Nozzles and needles are excessively worn out.	1) Same as # 3
unit	2) Buckets are rusted.		2) Buckets are rusted.	2) Same as # 3
unit	3) The shaft moves slightly.		3) Operation of the deflector is difficult.	3) Same as # 3
Inlet valve	Operation of this valve is difficult, because of manual operation.	Same as above	Operation of this valve is difficult, because of manual operation.	Same as # 3 unit
Governor	Unreliable system	Same as above	Unreliable system	Unreliable system
Hydraulic equipment	Oil leaks	Same as above	Oil leaks	Same as # 3 unit

Table 4.8 Major Defects in Generators and Auxiliary Equipment

Equipment	#1 unit	#2 unit	#3 unit	#4 unit
Stator and unit rotor	1) The stator and rotor are rusted and corroded.	#2 unit was removed.	1) Stator and rotor are rusted and corroded.	1) Same as # 3
unit	2) The coil is replaced twice a year due to burning.		2) The coil is replaced twice a year due to burning.	2) Same as # 3
unit	3) The insulation resistance value of the coil is lower than the standard value.		3) The insulation resistance value of the coil is lower than the standard value.	3) Same as # 3
Bearings	Temperature can not be measured, because the temperature-measuring instrument is not installed.	Same as above	Temperature can not be measured, because the temperature-measuring instrument is not installed.	Same as # 3 unit
Voltage regulator	Slow response	Same as above	Slow response	Same as # 3 unit
Turbine and generator control panel	The accuracy of the measuring equipment and protection relays is not good. They should be replaced.	Same as above	Same as # 1 unit	Same as # 3 unit

(2) Transformer

One transformer (6.6/13.2 kV, 2.5 MVA, self-cooled type) was installed outside to boost the generator voltage of 6.6 kV to the distribution voltage of 13.2 kV, but was transferred to Rio Mayo power plant. Currently, there is no transformer installed at Julio Bravo P/P.

(3) Switchgear

A switchgear was installed in the building to distribute 13.2 kV from Julio Bravo P/P to Catamuco and Aeropuerto. This switchgear has been in use for 17 years. According to a survey conducted by CEDENAR, no defect in the switchgear was found.

(4) Distribution line

The distribution line from Julio Bravo P/P to Pasto is rated at 6.6 kV, and to Catambuco and Aeropuerto at 13.2 kV. This distribution line has been in use for 19 years. According to a survey conducted by CEDENAR, no defects in distribution line facilities (steel towers, cables, insulators) were found. Rehabilitation of these facilities is not required.

4.3.2 General Condition of Civil Structures

(1) Intake facilities

The diversion weir is constructed of coarse stone concrete. Its crest length, crown elevation and height are 27 m, 2,357 m and 6 m, respectively. The foundation for the right-side diversion weir on the terrace is partly damaged. The dimensions of the intake entrance are 2 m x 2.2 m. The intake is installed parallel to the river. Planner shape of the diversion weir is bent like a key, and the diversion weir is installed at the intake portion so as to facilitate the inflow into the conduction channel. A concrete gate (2.0 x 1.6 m) is installed at the intake entrance and though partially peeled, it is in good condition.

(2) Conduction channel

The 2,500-meter-long conduction channel, constructed of boulder, has a trapezoidal section of 1.9 x 1.6 m. Though the conduction channel is partially damaged, with earth and sand from the mountain side flowing into the conduction channel, this channel is in comparatively good condition. Cross sections of the conduction channel show the gradient of the conduction channel bottom is not uniform.

(3) Desilting basin

The desilting basin, consisting of two concrete-made head tanks (size: 2 x 3.7 to 4.9 x 28 m, 1.25 x 1.85 to 3.05 x 28 m), is installed at a right angle to the conduction channel. A vortex has a tendency to develop within the desilting basin, impeding the flow rate. The concrete structure of the desilting basin has partially peeled off, but is in good condition.

(4) Head tank

Since the head tank is separated by a partition into two tanks (W3.8 x L46 m and W4.0 x L46 m), the tank capacity is insufficient for the operation. At capacity the tank's water level is 2,351.30 m. The mountain slope near the outlet of the spillway, eroded by the discharging water, has collapsed. The concrete structure is in good condition.

(5) Penstock

The penstock was damaged and removed.

(6) Power plant and tailrace

Powerhouse building size is 4 x 15 m, and floor elevation is 2,205.70 m. The building foundation and tailrace are in a comparatively good condition. However, the building has deteriorated since it is about 40 years old.

CHAPTER 5 BASIC DATA COLLECTION

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

5.1 Topographic Maps

Julio Bravo Hydroelectric P/P, built near the Pasto River of the Patia River system, is located about 10 km of Pasto City on the downstream side.

JICA Study Team collected the following topographic data.

- Topographic maps (scale: 1/25,000 - 1/400,000) published by IGAC

- Topographic survey maps that were actually measured by CEDENAR for the study of this power plant

(1) Topographic maps published by IGAC

Scale	Drawing No.	Description
1/400,000	-	the whole area of Nariño Department
1/100,000	429	
1/ 25,000	429-II-A,C	

(2) Topographic maps actually measured by CEDENAR

Topographic survey maps actually measured by CEDENAR from March to October, 1989 for the study of this power plant are as follows:

Topographic Survey Map	Scale
Plan of diversion weir, intake and vicinity	1/200
Transverse cross section of diversion weir, intake and vicinity	1/100
Longitudinal section of penstock and vicinity	Vertical = 1/200, Horizontal = 1/500
Transverse cross section of penstock and vicinity	1/100

CEDENAR prepared the following survey map prior to conducting this survey.

No. 273
Diagnostic de la cuenca superior del Rio Pasto y plan de ordenamiento y manejo

5.2 Geologic Survey Data

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

- Aerial photographs of this power plant and its environs
- Recuperacion y optimizacion de la central de Julio Bravo, 1989, CEDENAR
- Informe de Resultados de Perforaciones y Ensayos de Suelos para la Pequena Central, Hidroelectrica de Julio Bravo en Pasto, 1989, Estudio de SUELOS LTD
- Mapa Geologico de Colombia, 1988 1: 1,500,000 INGEOMINAS

5.3 Hydrometeorological Data

Since Julio Bravo P/P does not have the facilities for monitoring precipitation levels and discharge, JICA Study Team gathered HIMAT's hydrometeorological data in conducting this survey.

Precipitation at the existing HIMAT, precipitation station and the duration of monitoring record are listed below: Discharge on the Pasto River which is directly related to this FS was monitored at one place of Universidad; HIMAT's hydrological gauging station, and JICA Study Team obtained records monitored for 14 years from 1972 to 1985.

Table 5.1 List of Data Collected Relating to Hydrometeorology

(1) Precipitation-observation record

Meteorological station		Controller	Location		Altitude (m)	Observation period
No.	Name		Latitude	Longitude		
5204-004	BUESACO	HIMAT	0123	7709	2020	1970-87
-007	NARIÑO	"	0117	7722	2590	1970-87
5201-014	GUASCA LA	"	0136	7723	500	1970-87
5204-016	BERRUECOS	"	0130	7708	2200	1981-87
-501	OBONUCO	"	0112	7718	2710	1970-88
-502	APTO ANTONIO NARIN	"	0125	7716	1796	1970-88
-504	TAMINANGO	"	0133	7715	1875	1971-87
5205-001	PENOL EL	"	0127	7727	1620	1970-87

(2) Discharge-observation record

Hydrological gauging station		River	Controller	Establish-ment	Location		Altitude (m)	Catchment area (km^2)	Observa-tion period
No.	Name				Latitude	Longitude			
5204-701	UNIVERSIDAD	PASTO	HIMAT	1908-08	0112	7717	2590	177.0	1972-85

(3) Water quality data

The observation of water quality was recorded at the Universidad gauging station, and JICA Study Team collected observation records for three years from 1985 to 1988.

- Water quality analysis data
  Observation items: pH, specific resistance, $CaCo_3$, Ca, Mg, K, Co_3, HCO_3, C1 and SO_4

(4) Sediment data

JICA Study Team collected records relating to sediment that was observed by CEDENAR near the diversion weir at Julio Bravo P/P.

The observation period was as follows:

- Sediment observation record (sediment grain size distribution) for four months from April to July, 1989

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) in Nariño Department. 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 CEDENAR (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 CEDENAR's power conditions.

1) Records of generated energy, power loss and electricity buying at CEDENAR for five years from 1984 to 1988

2) CEDENAR's forecast of installed output and generated energy from 1990 to 2000

3) CEDENAR schematic power diagram

(2) JICA Study Team gathered the following data relating to Julio Bravo P/P.

1) Single line diagram

2) Residual value

3) Operation and maintenance personnel

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 Pasto (total length: 34 km) is at the western slope with the elevation of approximately 3,200 m on the central mountains in South Colombia from which the Rio Pasto flows northwest down and joins the Patia River through Pasto City. The project site is located about 6 km downstream of Pasto City.

The topography in the vicinity forms a plateau of andesite lava with a comparatively loose slope of volcanic scoriae.

Low terraces exist near the riverbed. From aerial photographs, unusual contours are not observed in the vicinity of the project site.

6.1.2 Geology

Tertiary volcanic rocks widely cover a long and narrow area in the NE-SW direction near Pasto City, capital of Nariño Department in South Colombia.

The bedrock in the project site consists of pyroclastic rocks composed of volcanic breccia and andesite lava. Talus overlays in the loose slope at the foot of the mountains, while low terrace deposit overlays in the riverbed. 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

Era			Lithology	Remarks
Cenozoic era	Quaternary period		Riverbed materials (Gravel & sand)	
			Talus	
			Terrace deposits (Sand & gravel)	
	Tertiary period		Volcanic breccia	Bedrock comprising the foundation for the project site
			Andesite lava	
			Volcanic breccia	

6.1.3 Geological Structure

Fig. 6.1 shows geological structure in the surrounding area of the project site. Andesite lava covers the upper parts of the slope on the mountains, forming a steep cliff. While volcanic breccia covers the lower section, forming a comparatively gentle slope. Talus deposit overlays on the foot of the mountain, and lower terrace deposit overlays on the both banks of the riverbed.

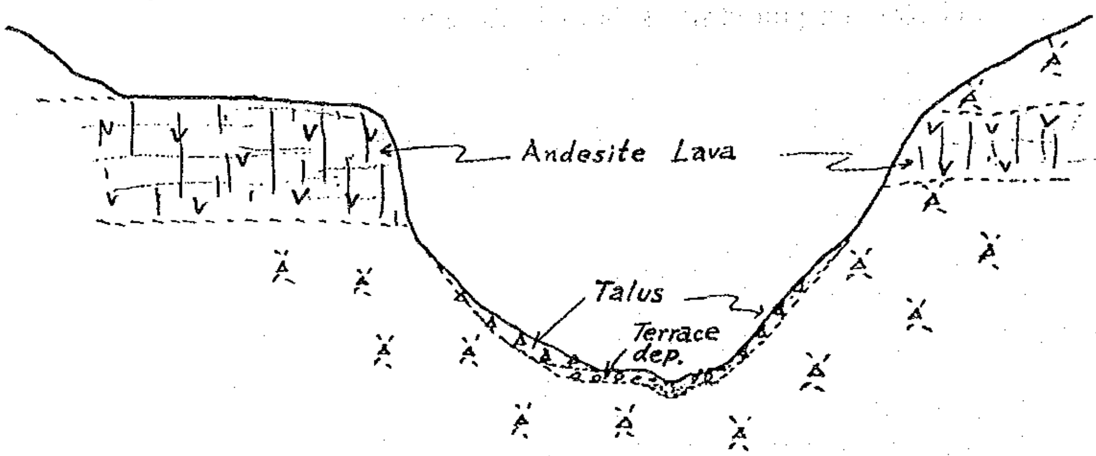


Fig. 6.1 Schematic Geological Profile

6.2 Geology in the Project Site

The geological condition of the foundation for the various civil structures is described below: (Refer to Drawing No. JB-G-01)

(1) Diversion weir

According to a boring survey (BH-1), the thickness of riverbed deposit exceeds 10 m. volcanic breccia comprising the bedrock for the diversion weir is in a fresh condition. It is possible that this volcanic breccia is used as the bedrock for the 10-meter-high concrete dam; there is no problem in strength or permeability.

(2) Conduction channel

No unusual landsliding has been observed in the whole section of the conduction channel. It is essential to take proper measures for the debris flow from a small stream crossing the conduction channel.

(3) Head tank

Bedrock consists of weathered and loose volcanic breccia. Proper measures for slope protection of excavated part must be taken.

(4) Penstock

The foundation for the penstock is considerably weathered and loose volcanic breccia. No landslide and large-scale cave-in have been observed.

(5) Power plant

This power plant site is situated on lower terrace deposit. According to a boring survey (BH-4), lower terrace deposit is about 3-meter-thick, and parts deeper than 3 m consist of volcanic breccia.

6.3 Distribution of Concrete Aggregates

The topographical and geological surveys indicate that various structures for small-scale hydroelectric power generation can be constructed at this project site.

Andesite, produced at quarry sites scattered on the left bank, upstream from the project site is suitable for concrete aggregate.

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

Nariño Department located in the southwest part of Colombia lies at 0°15' to 2°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. The lowland areas have an average temperature of 24°C, while the highland areas (with the elevation of 1,800 to 2,800 m) range from 12 to 18°C.

Pasto, lying in the highland at an elevation of 2,600 m, has a temperature of 12°C. This temperature level remains constant throughout the year.

Annual maximum precipitation in the west slope of West Andes Mountain Range (elevation: 1,000 to 2,000 m) exceeds 6,000 mm, while precipitation is low for the lowland areas; annual precipitation in the most downstream part of the Patia River is about 2,000 mm.

The project site with the elevation of about 2,000 m above sea level is situated in the northwest of Pasto, and lies on the Central Andes Range. Annual precipitation in the project site is relatively low, but it fluctuates year to year. (Refer to Fig. 7.2.)

Observation Item	Gauging Station		Latitude	Longitude
	No	Name		
Discharge	5204-701	Universidado	0112	7717
Preciptation	5204-007	Narino	0117	7722
	5204-501	Obounco	0112	7718

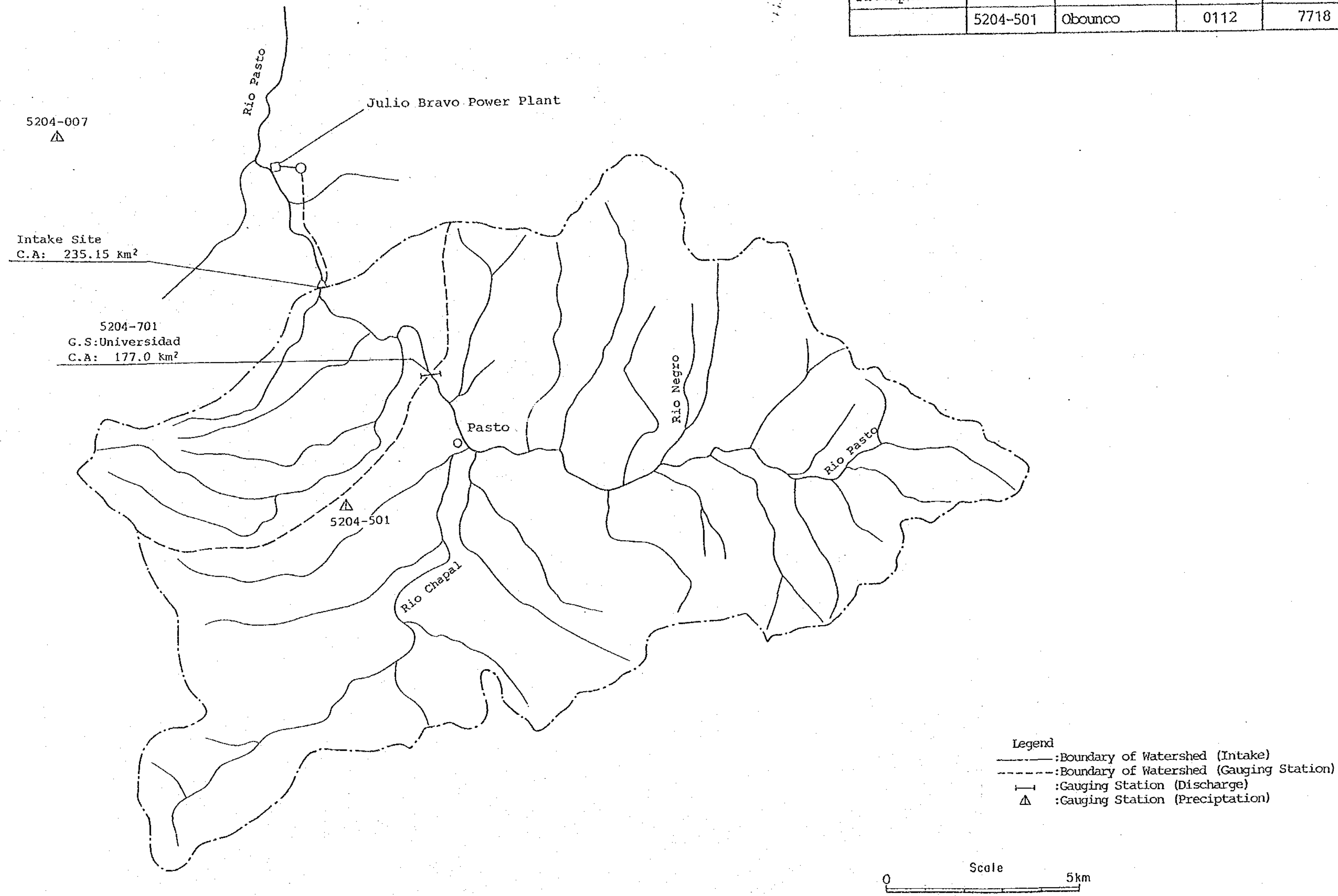


Fig-7.1 Location Map of Gauging Stations in The Watershed of The Study Area.

Meteorological station No.5204-501 Obonuco
North latitude: 1°12'
West longitude: 77°18'
Elevation: 2,710 m
Annual average precipitation: 816.4 mm

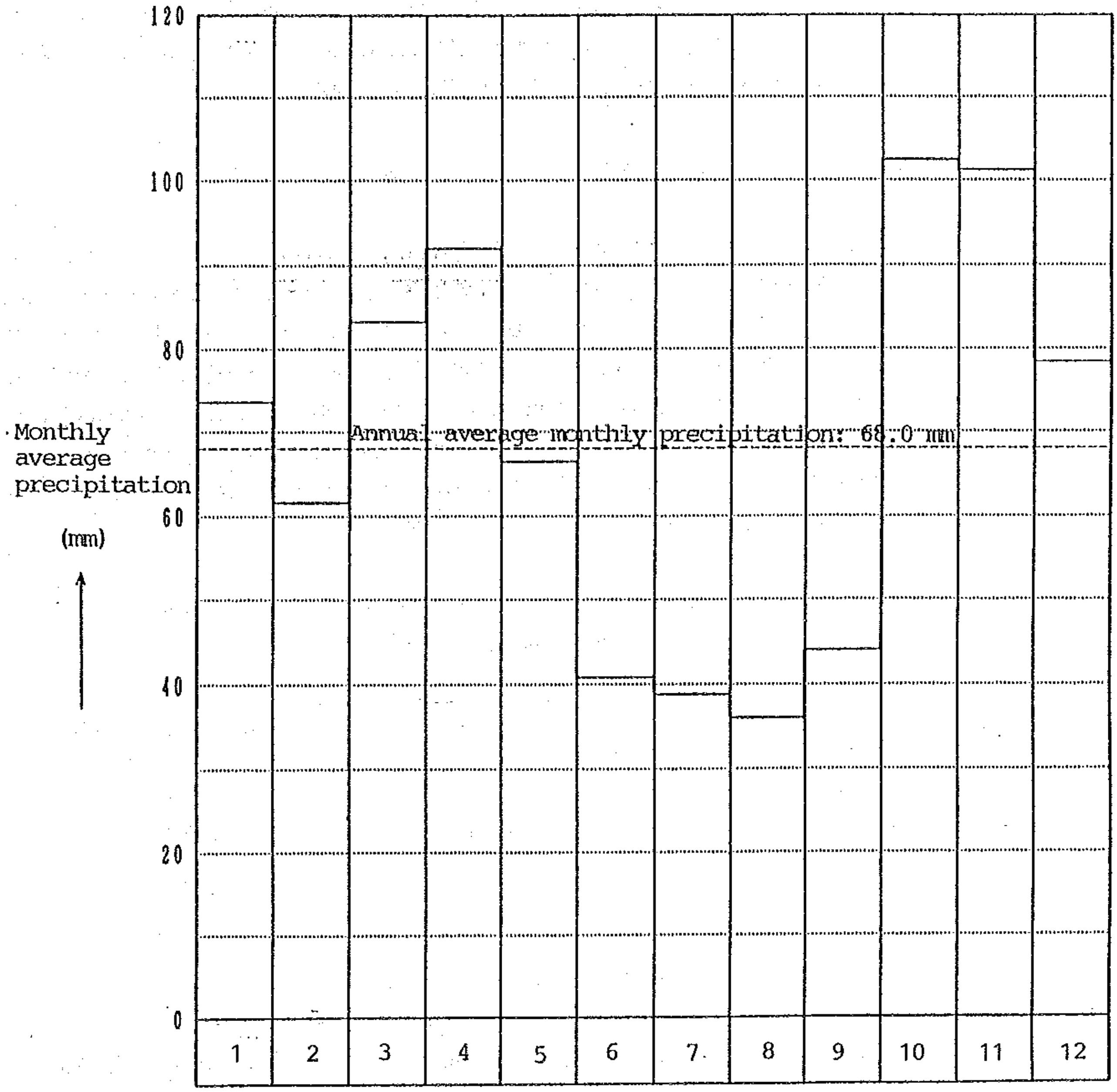


Fig.7.2 Monthly Average Precipitation in the Project Site (1970-88)

7.2 Discharge Analysis

The study team gathered, compiled observations recorded at the Universidad gauging station during 14 years from 1972 to 1985, and then prepared discharge and flow-duration curves according to reliable 10-year data out of the above record by converting river basin. (Refer to Drawing No. JB-H-01.)

7.2.1 Comparing to Catchment Area at the Gauging Station

To confirm the present location of the existing Universidad gauging station, the longitude and latitude indicated on HIMAT's gauging register are plotted on the topographic maps (1/100,000) published by IGAC. However, there was a 2' difference in latitude from the location of gauging station observed by the Study Team through a field reconnaissance. Therefore, the Study Team compares the catchment area at the gauging station (CA=177.0 km^2) using the topographic map (scale: 1/400,000) published by IGAC. However, there is no great difference in the catchment area, as shown in Table 7.1.

Table 7.1 Result of Comparison of Gauging Station Location and Catchment Area

Item	Latitude	Catchment area (km^2)
HIMAT register	1°12'	177.0
Compared value	1°14'	178.8
Difference	02'	1.8

7.2.2 Comparing Discharge Observation Record

Though the Universidad hydrological gauging station was established in August, 1970, the Study Team gathered observations recorded during 14 years from 1972 to 1985. As shown in Table 7.2, non-observation dates are included in the above record. Annual discharge observation records are complete for 6 years from 1973 to 1978 and 5 years from 1981 to 1985. Since discharge was not monitored continuously, these recorded observations are unreliable as year-round data.

Non-observation years: January to May, 1972
January to February, 1979
March to July, 1980
January to March, 1981

Table 7.2 Dates in which Discharge Record was not Observed

Year	Non-observation Date
1972	Jan. 1 to May 31
1979	Jan. 1 to Feb. 7
1980	Mar. 1 to July 31
1981	Jan. 15 to Mar. 31

7.2.3 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.

7.2.4 Discharge and Flow-duration Curve at Universidad Gauging Station

Discharge data at the Universidad gauging station, located about 6 km upstream from the intake site of Julio Bravo hydroelectric power plant, are arranged using the 10-year observation data, excluding non-observation days, as shown in Table 7.3.

In calculating monthly average discharge, months in which the observation time was less than 10 days are excluded. As seen in Drawing JB-H-01 number (1), a graphic representation of monthly average discharge, three-month flow periods can not be clearly distinguished from drought periods.

However, the three months from July to September (in summer) and the four months from December to March (winter) are considered drought periods.

Typical flow-duration curves calculated from the 10-year flow-duration curves from 1973 to 1978 and from 1982 to 1985 according to the parallel method are indicated in Drawing JB-H-01 number (3). Periods of three-month flow, ordinary water discharge and nine-month flow in flow-duration curves are indicated in numerical values, as shown in Table 7.4.

Table 7.5 shows the maximum discharge recorded at Universidad gauging station during 14 years from 1972 to 1985.

Table-7.3 MONTHLY FLOW TABLE OF DAILY AVERAGE FLOW AT G.S. SITE

GAUGING ST.: 5204-701 UNIVERSIDAD
RIVER NAME: PASTO

(UNIT: M3/S)

GAUGING YEAR	TYPE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL TOTAL
1972	MAX.	(1)	(1)	(1)	(1)	(1)	14.9	17.0	6.2	12.6	16.8	16.9	8.9	17.0
	MEAN	(1)	(1)	(1)	(1)	(1)	4.3	5.5	3.2	3.2	3.8	6.3	3.7	4.3
	MIN.	(1)	(1)	(1)	(1)	(1)	2.7	3.5	2.3	2.1	0.5	3.2	2.4	0.5
1973	MAX.	3.8	11.6	3.9	6.4	6.9	15.8	5.7	7.9	2.5	11.6	16.2	13.2	16.2
	MEAN	2.3	3.3	1.9	2.4	2.2	2.7	2.5	3.0	1.7	3.3	3.7	4.6	2.8
	MIN.	1.5	1.9	1.3	1.6	1.5	1.5	1.4	1.7	1.3	1.2	1.8	2.8	1.2
1974	MAX.	7.9	22.6	17.5	11.3	19.3	4.0	31.3	4.3	3.0	12.6	22.0	23.3	31.3
	MEAN	3.1	7.6	6.7	4.9	3.7	2.2	5.3	2.5	1.9	4.2	10.6	7.6	5.0
	MIN.	2.0	2.9	3.0	2.6	1.8	1.5	2.0	1.6	1.4	2.2	[illegible]	4.1	1.4
1975	MAX.	7.5	12.6	14.1	6.3	6.7	18.4	9.7	8.5	6.0	7.8	19.8	16.1	19.8
	MEAN	4.2	4.8	5.4	3.5	3.8	5.9	5.0	4.5	3.7	3.7	[illegible]	7.8	4.9
	MIN.	2.8	2.8	3.1	2.6	3.2	3.1	3.0	3.5	2.6	2.7	2.8	4.5	2.6
1976	MAX.	9.7	13.3	11.8	24.6	7.7	8.6	29.4	10.4	10.2	11.7	18.9	15.8	29.4
	MEAN	4.9	5.0	4.1	7.0	4.2	4.4	6.8	4.6	4.0	5.1	[illegible]	5.4	5.1
	MIN.	3.2	4.0	3.1	3.4	3.5	3.0	3.3	3.1	2.7	2.9	[illegible]	3.0	2.7
1977	MAX.	4.6	5.0	24.6	14.0	8.0	7.0	5.0	7.5	15.1	9.6	[illegible]	11.0	24.6
	MEAN	2.5	2.4	3.3	5.2	3.3	3.0	2.9	3.2	3.1	3.0	[illegible]	2.0	3.0
	MIN.	2.0	1.4	0.8	0.8	2.0	1.8	2.0	2.0	1.1	2.0	[illegible]	1.2	0.8
1978	MAX.	10.1	2.2	2.9	22.7	7.5	11.3	4.8	2.7	2.5	2.7	[illegible]	22.2	22.7
	MEAN	2.6	2.0	2.1	7.2	3.7	4.7	2.8	2.3	2.0	1.9	2.2	3.9	3.1
	MIN.	1.4	2.0	2.0	2.4	1.8	2.8	2.0	1.8	1.6	1.8	1.4	1.0	1.0
1979	MAX.	(1)	1.7	8.8	15.8	5.2	9.2	11.3	8.1	10.0	6.9	5.8	13.1	15.8
	MEAN	(1)	1.3	2.6	4.7	3.2	3.6	3.1	3.2	2.9	2.4	2.9	3.8	3.1
	MIN.	(1)	1.0	0.3	0.6	2.2	2.4	2.0	1.8	2.0	1.5	1.7	2.2	0.3
1980	MAX.	15.3	18.6	(1)	(1)	(1)	(1)	(1)	3.3	7.7	14.8	7.4	3.3	18.6
	MEAN	4.4	4.7	(1)	(1)	(1)	(1)	(1)	1.8	2.3	4.9	2.7	1.8	3.2
	MIN.	2.6	2.6	(1)	(1)	(1)	(1)	(1)	1.5	1.5	1.5	1.5	1.5	1.5
1981	MAX.	2.4	(1)	(1)	3.4	5.4	5.4	15.9	11.9	8.5	2.8	10.6	25.1	25.1
	MEAN	1.6	(1)	(1)	2.6	2.5	2.2	3.7	1.5	1.7	1.5	3.5	4.4	2.5
	MIN.	1.5	(1)	(1)	2.0	1.5	1.4	1.5	1.0	1.0	0.9	0.9	1.5	0.9
1982	MAX.	10.8	10.0	8.9	19.9	17.7	4.6	7.1	2.8	1.8	2.2	4.3	10.1	19.9
	MEAN	4.5	2.5	4.3	4.5	3.9	2.5	3.5	2.2	1.5	1.5	1.8	2.7	3.0
	MIN.	2.2	1.5	2.3	2.1	1.9	2.2	2.3	1.8	1.5	1.5	1.5	1.5	1.5
1983	MAX.	3.2	5.4	5.5	9.4	10.2	2.9	5.2	3.6	2.2	6.1	3.0	12.3	12.3
	MEAN	1.6	2.3	2.4	3.6	2.7	1.9	2.6	1.9	1.4	2.3	2.0	5.0	2.5
	MIN.	1.5	1.5	1.3	1.5	1.5	1.4	1.5	1.3	1.1	0.9	1.4	2.2	0.9
1984	MAX.	12.2	12.6	9.9	17.4	11.6	4.3	2.2	1.9	9.6	14.0	8.9	12.1	17.4
	MEAN	5.3	5.8	2.5	5.3	4.6	2.5	1.6	1.5	3.1	4.3	4.0	2.8	3.6
	MIN.	2.8	2.3	1.8	2.3	2.2	1.8	1.3	1.4	1.0	2.0	2.2	1.7	1.0
1985	MAX.	6.2	3.6	6.3	9.1	5.8	6.5	1.5	2.7	2.8	4.9	8.3	5.9	9.1
	MEAN	2.7	2.3	2.5	2.5	2.8	2.2	1.4	1.4	2.0	2.3	2.7	1.5	2.2
	MIN.	1.5	1.4	1.2	1.4	1.4	1.2	1.3	1.2	1.5	1.5	1.5	1.1	1.1
TOTAL	MAX.	15.3	22.6	24.6	24.6	19.3	18.4	31.3	11.9	15.1	16.8	22.0	25.1	31.3
	MEAN	3.3	3.7	3.4	4.5	3.4	3.2	3.6	2.6	2.5	3.2	4.1	4.1	3.5
	MIN.	1.4	1.0	0.3	0.6	1.4	1.2	1.3	1.0	1.0	0.5	0.9	1.0	0.3

NOTE) (1) ALL DATA MISSING

Table-7.4 FLOW DURATION TABLE AT GAUGING STATION SITE

GAUGING ST.: 5204-701 UNIVERSIDAD
NAME: PASTO

(UNIT: M3/S)

GAUGING YEAR	MAX. (1-ST DAY)	PLENTY (95 DAY)	ORDINARY (185 DAY)	LOW (275 DAY)	DROUGHTY (355 DAY)	MIN. (LAST DAY)	MEAN
1973	16.2	3.1	2.2	1.8	1.4	1.2	2.8
1974	31.3	5.8	3.5	2.4	1.6	1.4	5.0
1975	19.8	5.7	3.9	3.4	2.7	2.6	4.9
1976	29.4	5.4	4.1	3.6	2.9	2.7	5.1
1977	24.6	3.2	2.2	2.0	1.2	0.8	3.0
1978	22.7	3.2	2.2	2.0	1.6	1.0	3.1
1982	19.9	2.8	2.3	1.5	1.5	1.5	3.0
1983	12.3	2.8	1.9	1.5	1.1	0.9	2.5
1984	17.4	4.3	2.5	2.0	1.1	1.0	3.6
1985	9.1	2.4	1.8	1.5	1.1	1.1	2.2
MEAN	20.3	3.9	2.7	2.2	1.6	1.4	3.5

Table-7.5 MONTHLY ABSOLUTE MAXIMUM FLOW TABLE AT G.S. SITE

GAUGING ST.: 5204701 UNIVERSIDAD
RIVER NAME: PASTO
(UNIT: M3/S)

GAUGING YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL TOTAL
1972	(1)	(1)	(1)	(1)	(1)	21.1	22.5	7.3	15.5	19.6	55.0	19.1	55.0
1973	5.1	22.5	4.7	13.0	12.5	25.7	11.3	13.0	3.8	17.2	41.6	26.6	41.6
1974	18.5	46.3	43.3	34.1	44.1	8.4	58.3	7.3	5.1	29.7	68.1	55.8	68.1
1975	10.0	22.5	24.5	11.9	13.2	33.7	32.1	22.0	13.2	11.0	59.8	18.1	59.8
1976	17.8	20.9	17.5	26.4	15.1	15.1	59.5	16.9	17.8	29.9	33.0	38.4	59.5
1977	7.8	7.8	56.6	32.6	11.7	10.6	9.3	8.5	39.3	22.8	11.4	18.1	56.5
1978	21.2	2.2	3.2	33.8	8.5	11.7	4.9	3.0	2.8	5.1	7.8	64.1	64.1
1979	(1)	(1)	14.4	21.0	5.2	15.6	11.9	10.7	15.6	7.5	6.1	15.6	21.0
1980	21.0	32.8	3.5	(1)	(1)	(1)	(1)	5.0	8.9	18.8	8.5	3.8	32.8
1981	2.6	11.0	1.5	9.8	6.4	6.4	37.6	22.4	10.7	4.5	29.1	61.2	61.2
1982	13.1	13.1	29.1	29.1	21.0	6.4	8.5	2.9	1.8	2.9	4.5	10.7	29.1
1983	4.5	7.7	10.7	12.9	11.9	4.5	6.0	9.4	2.9	15.6	5.8	19.4	19.4
1984	27.3	31.9	14.6	36.2	29.1	5.0	2.2	2.0	10.7	19.4	20.7	37.6	37.6
1985	10.7	6.7	6.9	11.9	10.7	12.6	1.8	2.9	2.9	6.4	8.5	9.6	12.6
TOTAL	27.3	46.3	56.6	36.2	44.1	33.7	59.5	22.4	39.3	29.9	68.1	64.1	68.1

NOTE) (1) DATA MISSING

7.2.5 Discharge and Flow-Duration Curves at the Intake Site

Discharge and flow-duration curves at the intake site of Julio Bravo P/P are calculated by multiplying respective catchment area ratio by recorded observations at the existing Universidad gauging station located about 6 km upstream from the intake site.

Since numerical values of the catchment area at the intake site are not officially approved, the value, 235.15 km^2, from the "Recuparacion y Optimizacion de la Central de Julio Bravo", prepared by CEDENAR in November, 1988, is adopted. Therefore, a ratio of catchment area between Julio Bravo P/P's intake site and HIMAT's Universidad gauging station is set at 235.15/177.0=1.33.

Discharge and flow-duration curves at the intake site converted according to the catchment area ratio are shown in Drawing JB-H-01, and 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 in Table 7.6.

Table 7.6 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)	7.0	10.1	8.9	9.5	6.1	7.6	9.0	6.1	5.3	6.8	14.1	10.4	6.8
Daily average discharge (m^3/s)	4.5	4.9	4.6	5.9	4.5	4.3	5.8	3.5	3.3	4.2	5.4	5.4	4.6
Min. average discharg (m^3/s)	2.2	1.7	2.5	3.2	3.0	2.5	1.9	1.9	1.8	2.0	2.4	2.1	2.9

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
5.1 m^3/s	3.5 m^3/s	2.9 m^3/s	2.2 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, if available discharge is secured throughout the year) are represented graphically in Drawing JB-H-01 number (5).

7.3 Flood Runoff Analysis

Flood discharge is important to maintain the safety of existing facilities and the repaired sections. The designed flood discharge is obtained by that the observation record of the discharge at Universidad 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 which is shown in Table 7.7 is summarized according to the discharge data.

Table 7.7 Annual Flood Discharge

Year Observed	Maximum Discharge (m^3/sec)
1972	55.0
1973	41.6
1974	68.1
1975	59.8
1976	59.5
1977	56.5
1978	64.1
1979	21.0
1980	32.8
1981	61.2
1982	29.1
1983	19.4
1984	37.6
1985	12.6

The observation data recorded over 14 years is a comparatively short sample. There are several methods to calculate potential flood, but the following three methods are considered.

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. 7.3 and 7.4 show that maximum yearly discharge is plotted on the X-axis and that percentage of excess probability calculated is plotted on the Y-axis by using the extreme probability paper. Table 7.8 shows the potential flood discharge for major years of return period from the probability curve shown in the figure.

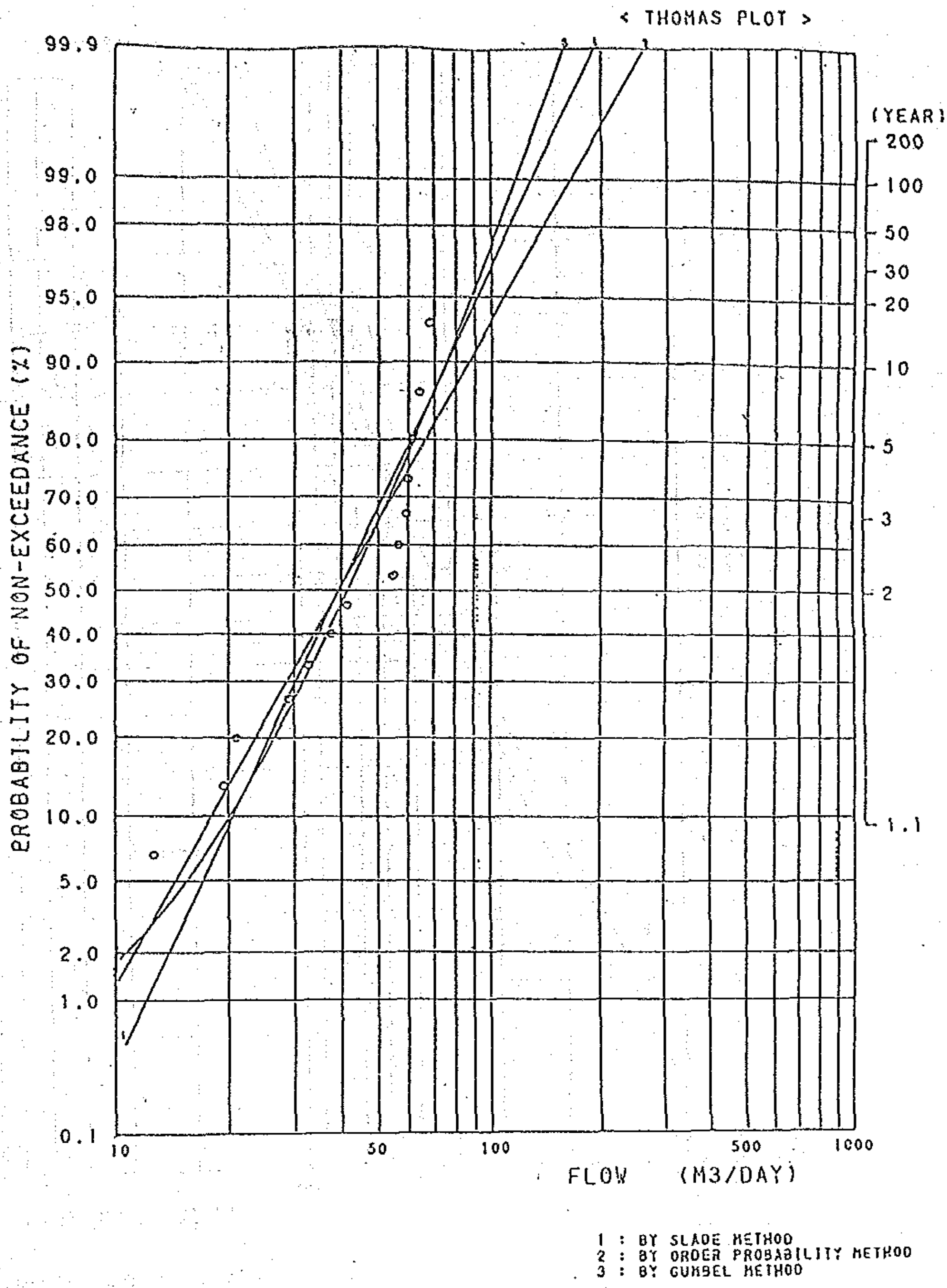


Fig. 7.3 Probability Curve of Rio Pasto at Universidad (Thomas Plot)

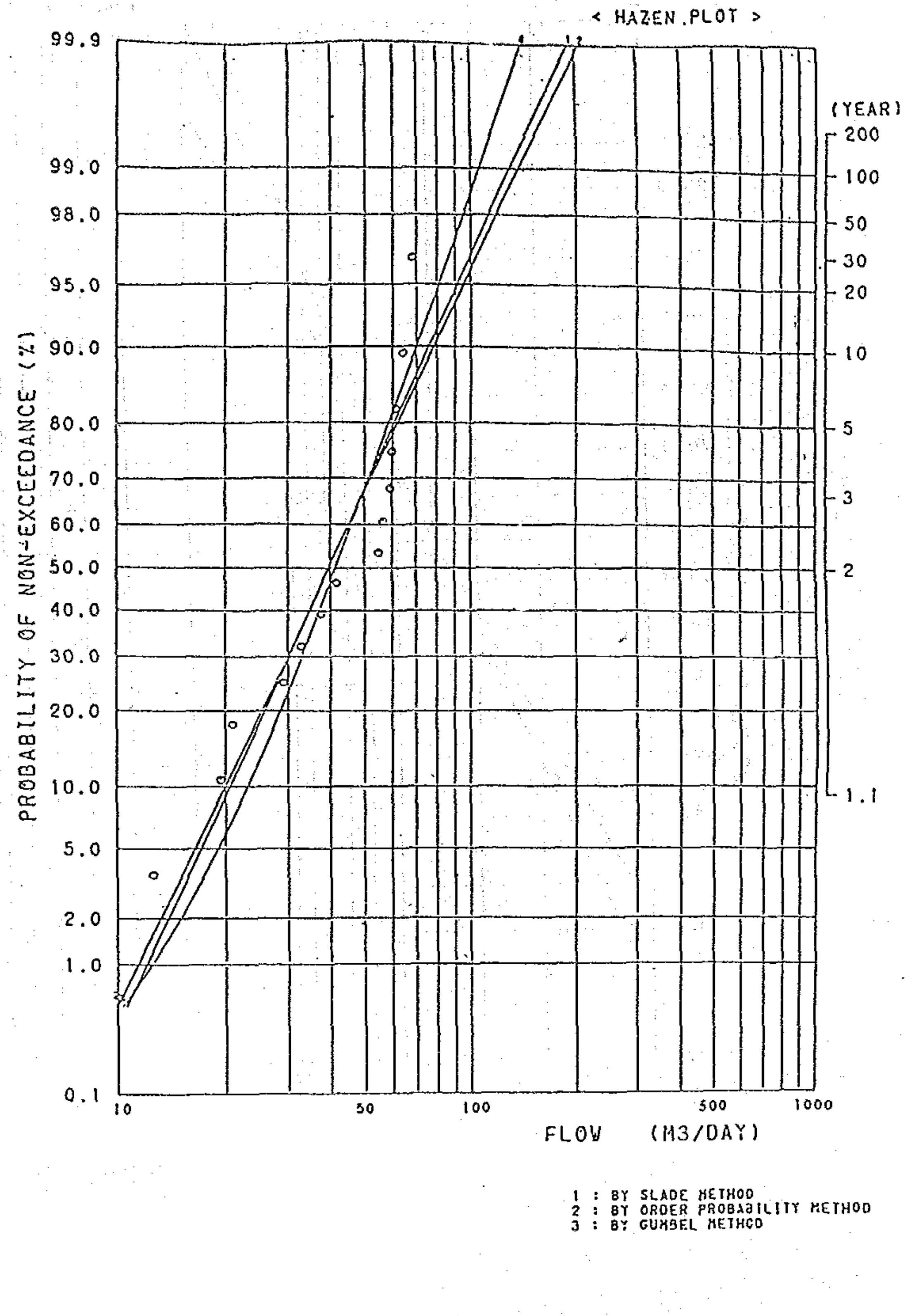


Fig. 7.4 Probability Curve of Rio Pasto at Universidad (Hazen Plot)

Table 7.8 Potential Flood Discharge

Method	Return Period in Years							
	5	10	20	50	100	200	500	1000
Logarithm normal distribution method (m^3/s)	61	76	92	113	130	147	172	191
Order probability method:								
Thomas plot (m^3/s)	66	87	108	139	165	192	231	263
Hazen plot (m^3/s)	62	78	95	118	137	156	184	206
Gumbel method:								
Thomas plot (m^3/s)	62	75	88	105	117	130	146	159
Hazen plot (m^3/s)	59	70	81	95	106	116	130	141

7.3.2 Design Flood Discharge

The design flood discharge is applied in case the life danger is comparatively small, 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 calculated with the catchment area ratio.

$$Q = 165 \times \frac{235.15}{177} = 219 \ldots\ldots 220 \text{ m}^3/\text{s}$$

The specific discharge per catchment area (km^2) will be $q = 0.94$ 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 $C = 6.6$.

* Applied Hydrology Editor Ven Te Chow

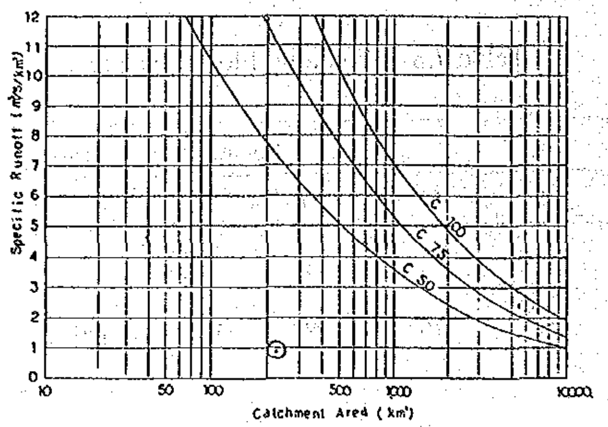


Fig. 7.5 Design Flood Discharge and Creager Curve

7.4 Sediment Analysis

The debris produced at the catchment mountain flows down up to the intake point, and flows further to downstream via channel and river. The debris flow process is shown in Fig. 7.6, and the debris flow volume is studied by this process.

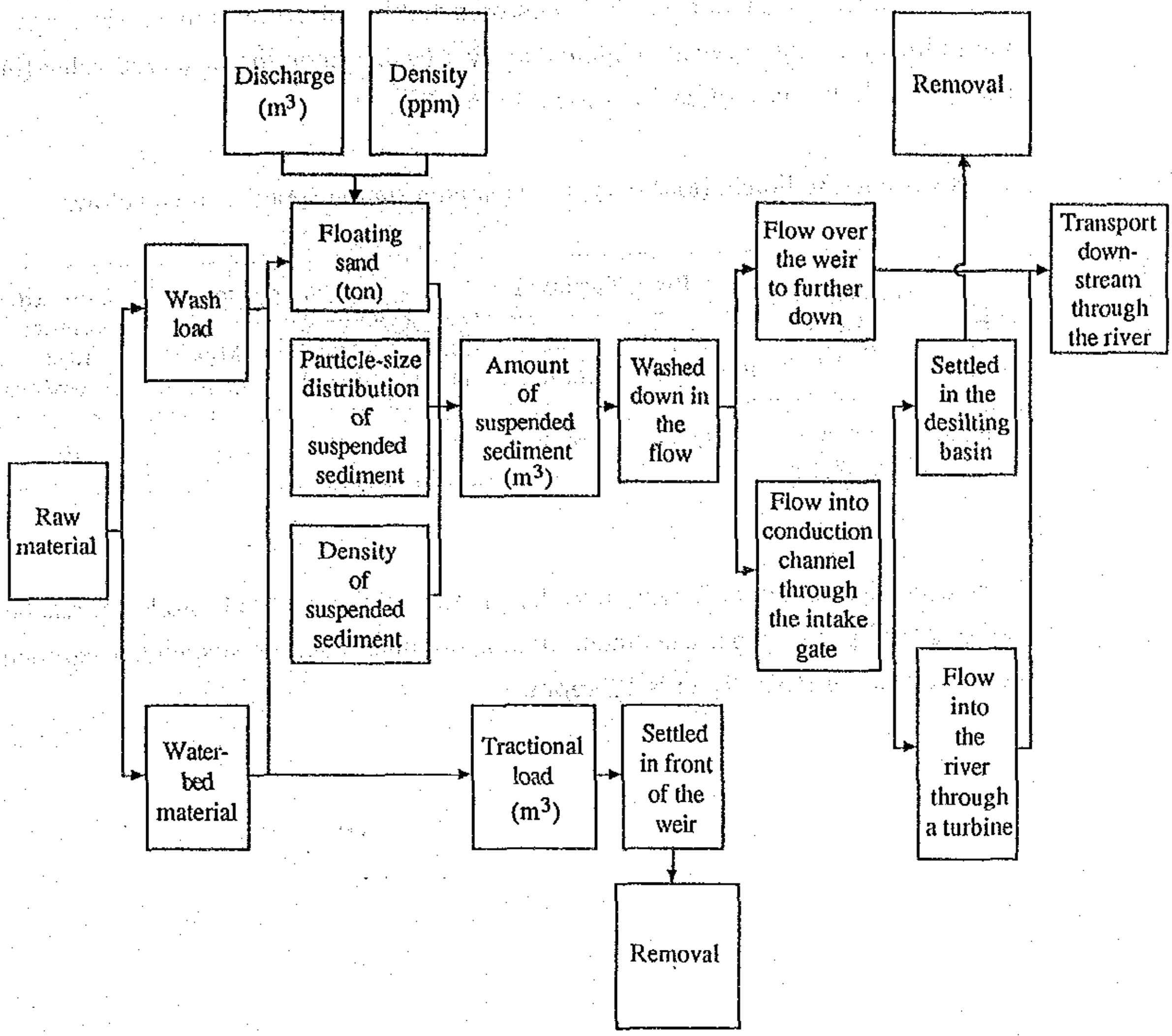


Fig. 7.6 Mechanism of Debris Flow and Calculation Flow of Debris Volume

7.4.1 Debris Flow Status

The catchment at the Pasto River includes the urban district of Pasto City, and the upstream area near the watershed is a comparatively steep ravine. The vegetation of the catchment is good. The debris flowing from these catchments is mainly debris and city waste generated by city development, erosion of river bed and bank, gully erosion by terrace collapse, etc.

The suspended sediment curve has been prepared by referring to the basic shape of the sediment rating curve at neighboring river basing upon the observed value (but short period) at this spot, and is shown in Fig. 7.7.

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
		Total 103 m^3/year	Max. (m^3/s)	Min. (m^3/s)	Max. (ppm)	Min. (ppm)	10^3 tons/year
Pasto	178.8	111,000	68.1	0.8	-	-	10

The suspended sediment flowing into the gauging station on the Pasto River reaches 60 tons/km^2 per year per catchment area, and annual average suspended sediment concentration of Pasto River is 100 ppm.

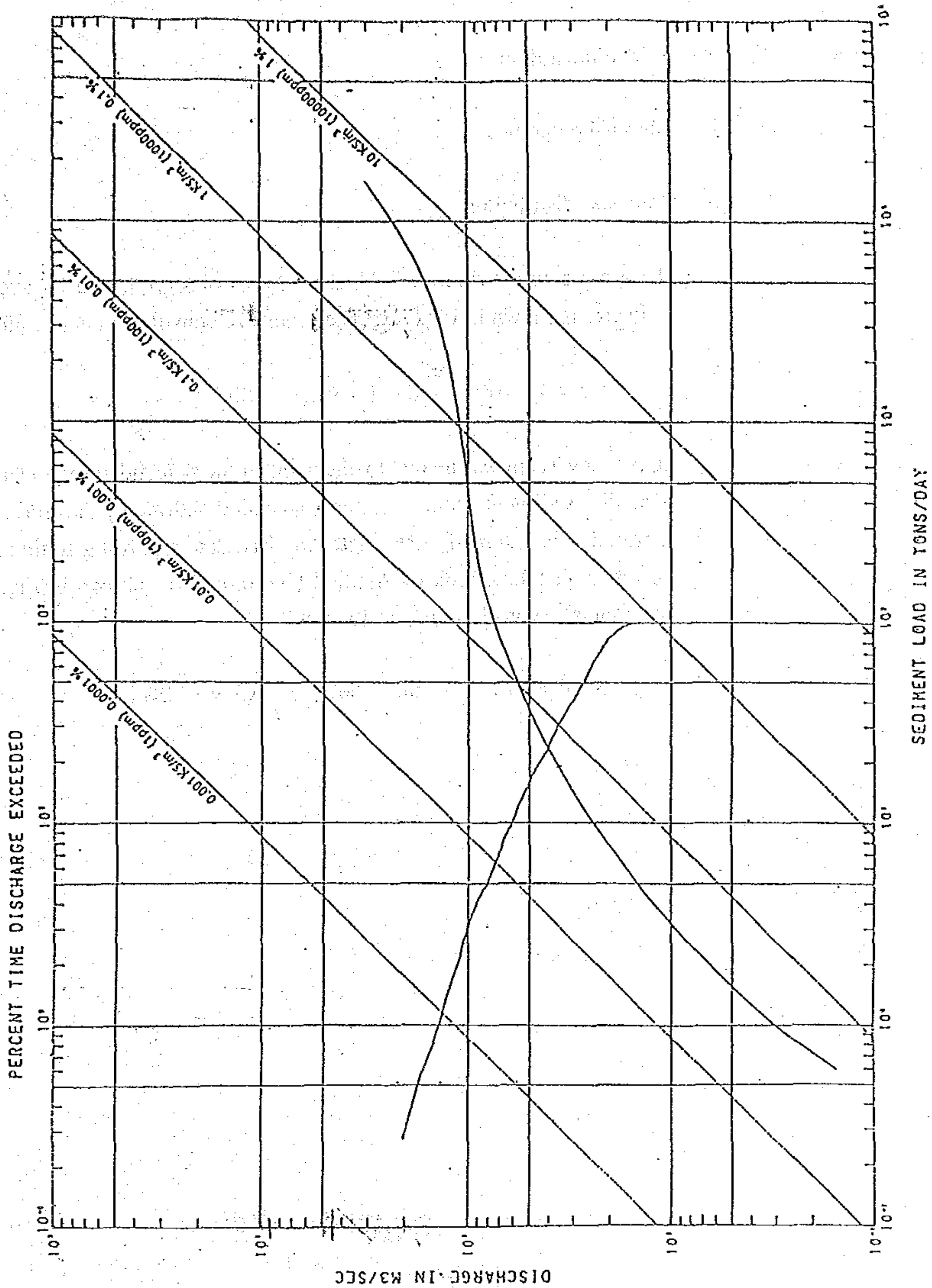


Fig. 7.7 Sediment Rating Curve

7.4.2 Assumption of Sediment Rate

(1) Major physical properties

(a) Grain size distribution

The grain size distribution of bed-load was observed, and its average grain size is shown in Fig. 7.8. The grain size constitution is as follows:

Gravel = 10% Sand = 80% Silt = 10%

JICA Study Team was unable to obtain either the suspended sediment data or settled sediment data. For the suspended sediment, the grain size distribution is assumed, referring to the past data regarding sediment of the reservoir (Handbook of Applied Hydrology), as shown in Fig. 7.9. The grain size constitution is as follows:

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

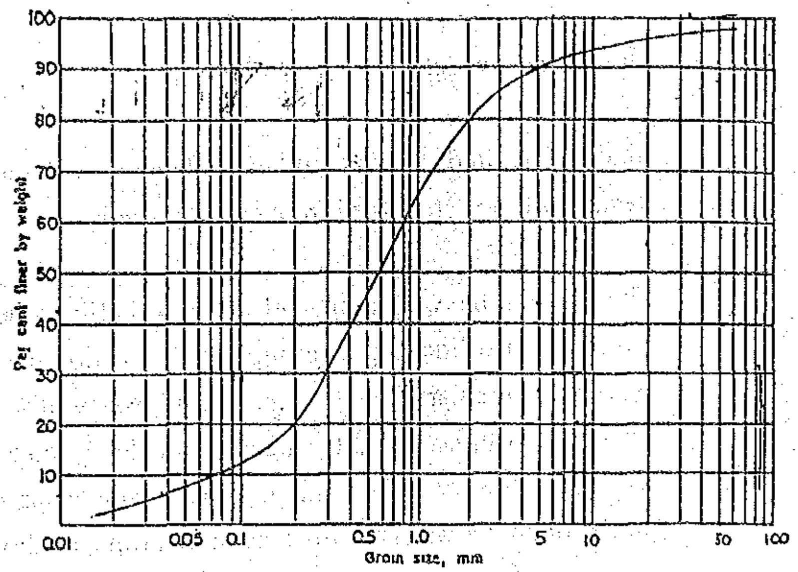


Fig. 7.8 Grain Size of Bed-load

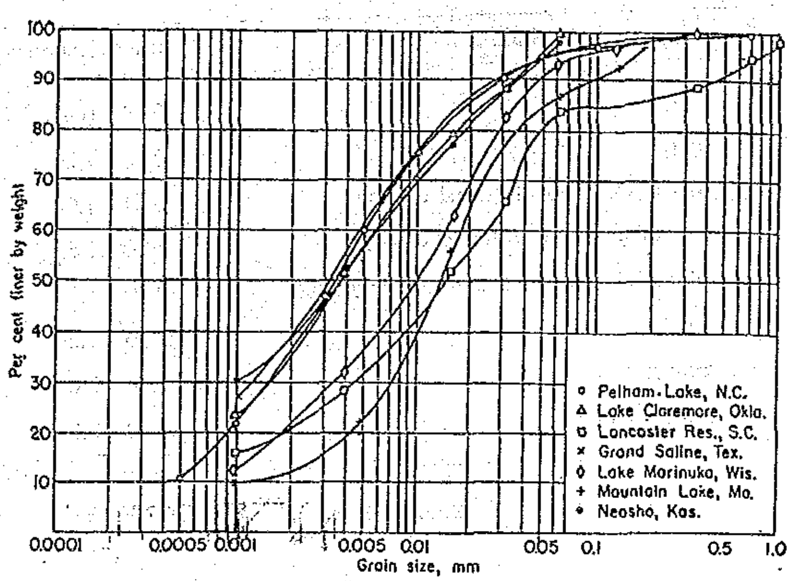


Fig. 7.9 Grain Size Constitution of Suspended Sediment *

* Handbook of Applied Hydrology (17-16)

(b) Unit volume weight

Since JICA Study Team could not obtain unit volume weight of sediment data, it will be determined from existing studies.

The unit volume weight of sand and gravel affects the consolidation load, but the consolidation is completed in a comparatively short time. However, fine particles of clay, colloid, etc. will require a longer time. The unit volume weight ranges 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, as shown in Table 7.9.

Table 7.9 Range of Unit Volume Weight*

(units: ton/m^3)

Grain	Almost submerged	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. Suspended sediment can be calculated from the sediment record (concentration measurement) and the discharge record. The quantitative record for the flown sand has not been obtained.

Generally flowing sand is 10 to 50% of total sediment rate, and the flowing sand of the Colorado River is 12 to 50% of total sediment rate. The World

Bank study team estimates the flowing sand of the Indus River at the Tarubera dam (Pakistan) spot will be 5% of 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)
235.15	146	13	1	14

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

	Flown Sand			
	Gravel	Sand	Silt	Total
Grain size constitution(%)	10	80	10	100
Unit volume weight (ton/m^3)	1.68	1.48	1.04	
Unit weight per grain size (ton/m^3)	0.168	1.184	0.104	1.456... 1.46

	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 flowing sands are deposited at the diversion weir and in front of the intake, and do 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 flowing into the channel are settled at the desilting basin, and the remaining suspended sediment is discharged into the river by a water wheel with discharge. Suspended sediment contained in the river discharge exceeding the designed discharge overflows the diversion weir and flows down the river.

Catchment area (km^2)	River discharge (10^6 (m^3))	Bed-load 10^3 (ton)	Suspended sediment 10^3 (ton)
235.15	146	1	13
		↓	↓
		x 10^3 m^3	x 10^3 m^3
		0.7	13

Design discharge Q m^3/s	Sediment in water channel x 10^3 m^3	Flow down the river x 10^3 m^3
2	5	8
3	8	5
4	9	4

Design discharge Q m^3/s	Settled in the desilting basin (m^3)	Flow down the channel (m^3)
2	500	4500
3	800	7200
4	900	8100

From this analysis the annual average sediment in front of the diversion weir is assumed to be about 2 m^2/day and sediment settled in the desilting basin 2 m^3/day (if available discharge is 3 m^3/s). A counterplan for removing the sediment will be fully considered.

7.5 Water Quality Analysis

The acidity, etc. and specific resistance of water which most greatly affect facilities are studied.

7.5.1 Criteria of Judgement

(1) Acidity, etc.

To judge the effect of acidity, etc., the criteria shown in Table 7.10 and the past instances shown in Table 7.11 will be referenced.

Table 7.10 Judgement Criteria of Erosion of Water (DIN 4030)

Item	Grade of Erosion		
	Weak Erosion	Strong Erosion	Very Strong Erosion
pH	6.5 - 5.5	5.5 - 4.5	Less than 4.5
CO_2 mg/l	15 - 30	30 - 60	More than 60
NH_4^+ mg/l	15 - 30	30 - 60	More than 60
Mg^{2+} mg/l	100 - 300	300 - 1500	More than 1500
SO_4^{2-} mg/l	200 - 600	600 - 3000	More than 3000

Table 7.11 Damage Example of Concrete in Erosive Environment of Water

Item	Water Characteristics	Damage Status
Groundwater	pH : 2.3 - 6.7	Tunnel concrete Indication of leakage is observed 4 years after construction. Peeling of mortar and cracks in concrete are noted after 7 years.
River water (Azuma River)	pH : 3.1 - 2.7 Mg^{2+} : 13.5 ppm SO_4^{2-} : 316.8 ppm Cl^- : 101.8 ppm	Dipping test concrete specimen (ϕ15cm) When unit cement volume 320 kg/m^3, W/C=35.1% and 3-month old material was palced into the river, the diameter reduced to 14.6 cm after 15 months. About 2 mm of the surface was dissolved, and another 2-3 mm was weakened.

(2) Specific resistance

Since water with a low specific resistance includes many soluble salt which will accelerate corrosion of steel, the corrosive nature is strong. The effect of specific resistance to corrosion is clear from the result of the National Bureau of Standard (NBS) investigation shown in Table 7.12; but there are some exceptions, and evaluation of the corrosive nature from only specific resistance is not often done.

Table 7.12 Specific Resistance and Corrosive Nature

Corrosive nature	Degree of acidity		Specific resistance Ω-cm	Maximum hole corrosion depth for 12 years (mm)
	pH	Total acidity		
	7.8	3.0	1770	0.74
	4.5	4.6	11200	1.19
	7.3	2.6	2980	0.99
	5.9	12.8	45000	1.02
	7.6	alkaline	350	3.02
	7.4	ditto	263	3.48
	9.4	ditto	278	4.39
	6.8	36.0	800	2.62

7.5.2 Evaluation of Water Quality

The results of the water quality test are given below:

Year observed	pH	Specific resistanca (Microohms)	Total $CaCO_3$	Fe (mg/l)	Cl (mg/l)	So4 (mg/l)
1985	6.3 - 4.0	345 - 166	62.8 - 99.4	-	-	-
1986	6.8 - 4.4	346 - 162	48.9 - 144.0	-	-	-
1987	6.8 - 4.2	302 - 182	36.7 - 89.0	-	-	-
1988	5.2 - 4.6	460 - 315	79.2 - 116.3	-	-	-
1989	7.7 - 4.3	258 - 127	-	0.2 - 27	16 - 37	2 - 50

The minimum pH value slightly increased from 4.0 in 1985 to 4.3 in 1989, but corrosive nature is presently high because of municipal wastewater upsteam. It is difficult to improve water quality within a short period of time.

Specific resistance is low and corrosive nature is strong. When compared to the WHO water quality standards, hardness of chloride and sulfate is low and iron content is high. The values of pH and specific resistance indicate that acid-resistance materials should be used for penstocks and turbines.

CHAPTER 8 GENERATION PLAN

The generation plan is made based on the planned maximum available discharge at Julio Bravo P/P of 2.00 m^3/s.

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

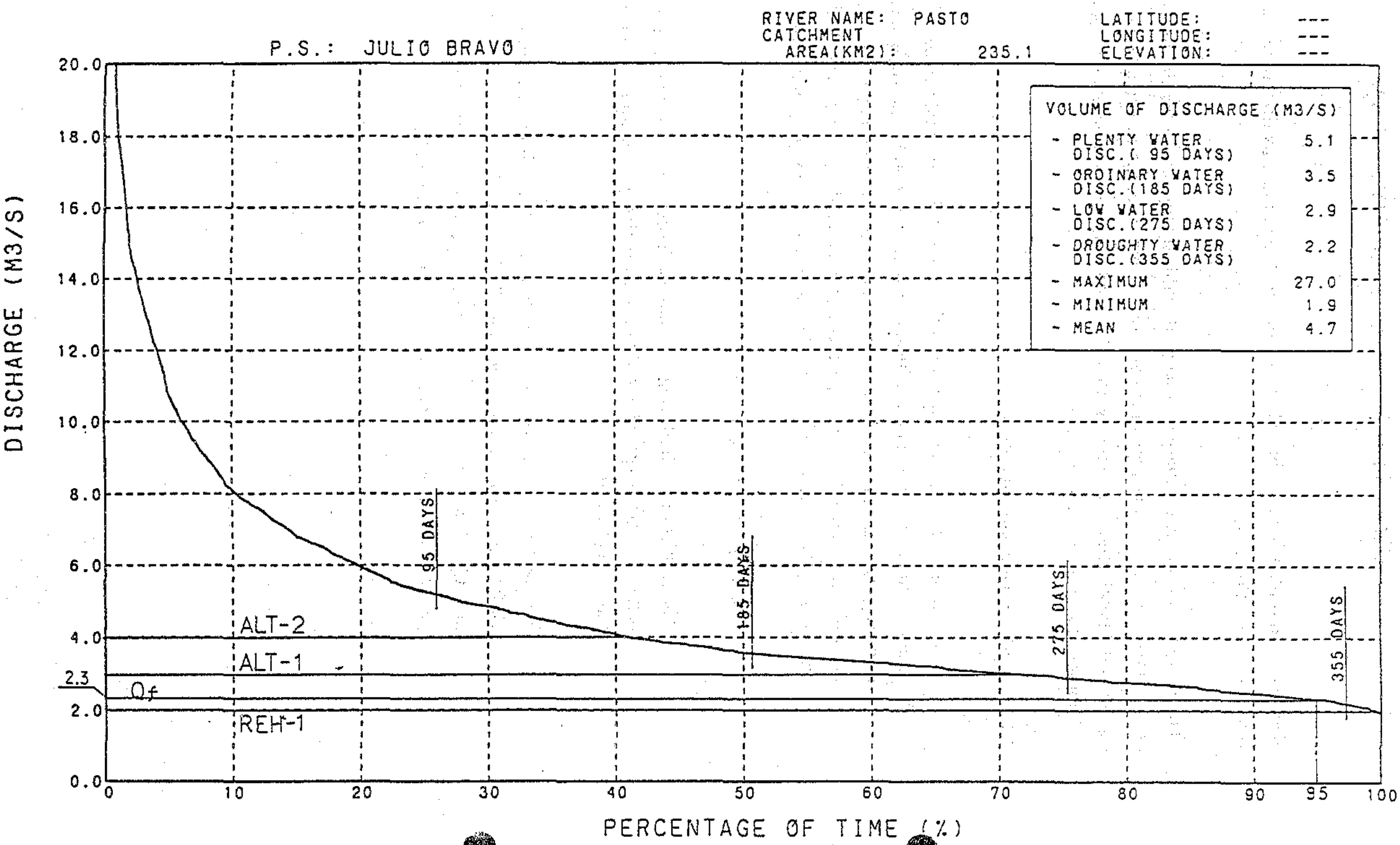
When the generating facilities for Julio Bravo P/P are rehabilitated, the headrace structure and its appurtenant facilities, except for the 2,500-meter-long headrace, need to be repaired or replaced. In addition, generating facilities and transformers need to be replaced. Therefore, comparative studies shall be made for the generation-optimizing plan as well as the rehabilitation plan of the existing generating facilities.

(1) Maximum available discharge

The results of the hydrological analysis indicate that the existing open channel can safely discharge up to 4.00 m^3/d by partially expanding the cross section of the open channel.

To compare the maximum available discharge, three rehabilitation plans of Q=2.00 m^3/s (maximum available discharge at the existing P/P), Q=3.00 m^3/s and Q=4.00 m^3/s were set up, and respective generated output and annual generated energy were calculated, as shown in Fig. 8.1.

Fig-8.1 TYPICAL FLOW DURATION 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.

$$He = Hg - H$$

$$H = V^2/2g\ (1.0 + f_1 + f_2\ L/D + fm) + h = V^2/2g\ (1.85 \times f_2 \times L/D) + h$$

where:

Hg = gross head
Head tank water level (2351.20m) - tailrace water level (2204.00m)=147.20m

H = total loss of head (m)

$V^2/2g$ = velocity head (m)

f_1 = coefficient of inflow loss; 0.1

f_2 = coefficient of frictional loss; 124.6 n/D

L = penstock length (m)

D = penstock diameter (m)

fm = loss coefficient at the branched part, 0.75

h = margin (m)

n = coefficient of roughness, 0.012

Table 8.1 Calculated Result of Standard Net Head

Q (m^3/s)	D (m)	I (m)	V (m/s)	$V^2/2g$ (m)	f.I/D	$V^2/2g(\Sigma f)$ (m)	h (m)	H (m)	He (m)
2	0.90	230	3.15	0.506	4.75	3.34	0.86	4.20	143.00
3	1.10	230	3.16	0.509	3.64	2.79	0.91	3.70	143.50
4	1.20	230	3.54	0.639	3.24	3.25	0.95	4.20	143.00

Accordingly, the standard net head is calculated to be 143.00 m.

8.2 Generated Output

Theoretical output obtained from available discharge (Q) and the standard net head (H) is multiplied by the resultant efficiency coefficient 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)
He	=	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, and this value is obtained by the following formula.

$$\eta = \eta t \times \eta g$$

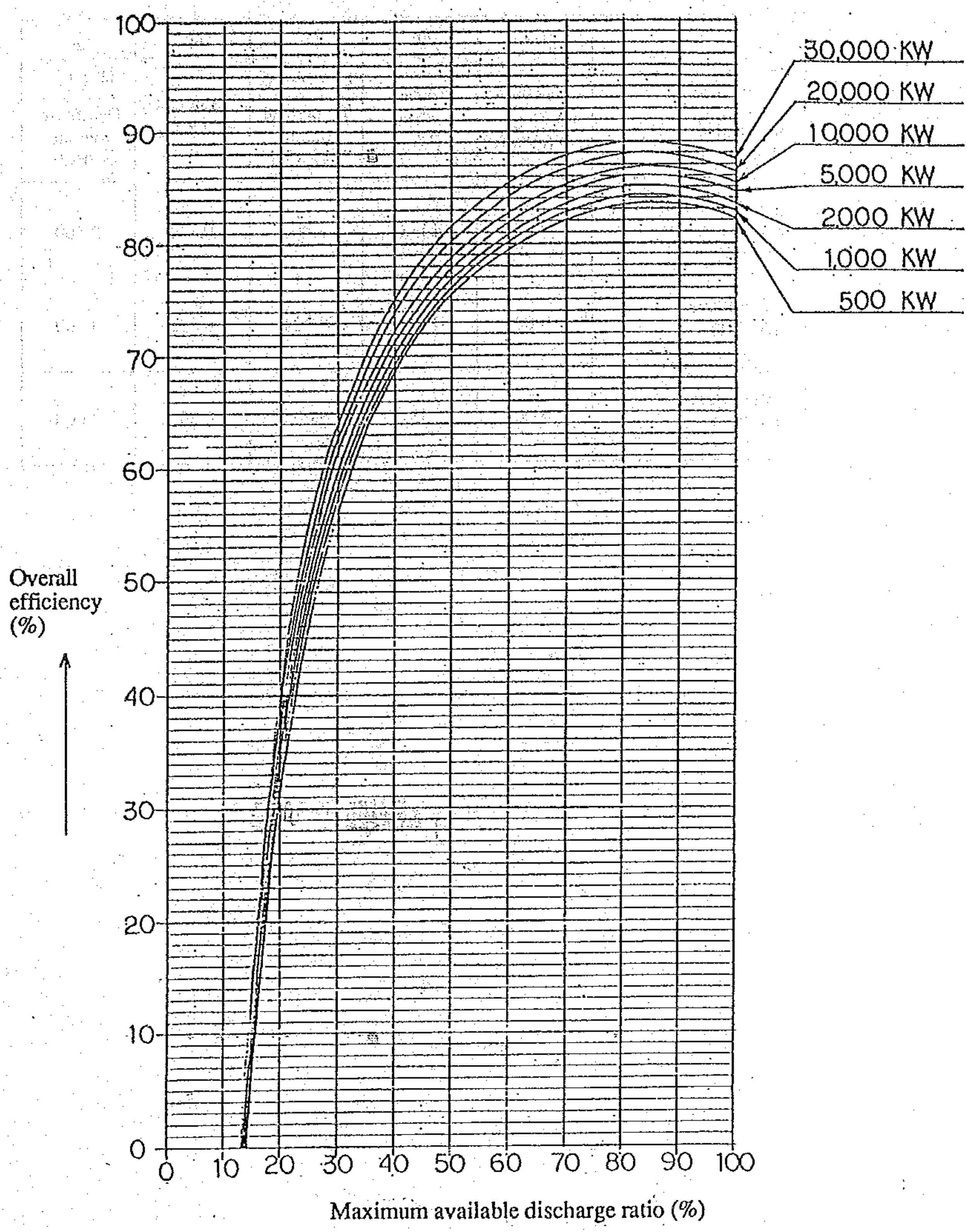
where:

ηt	=	turbine efficiency
ηg	=	generator efficiency

Resultant efficiency corresponds to the value of the maximum available discharge ratio, 100%, in the resultant efficiency curve as shown in Fig. 8.2. Table 8.2 shows the calculation result of the generated output for the alternative plans.

In the plan for rehabiliting the existing facilities (Q = 2.00 m^3/s), plus correction of about 23 m in the standard net head was recognized. Along with the improvement of the resultant efficiency, the existing generating equiment's capacity increased from 1,500 kW to 2300 kW.

Fig. 8.2 Resultant Efficiency Curve of Francis Turbine and Generator



(Source: The above curve is drawn according to the study standard for formulation of hydroelectric development plan (March, 1981).

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 ρ (kW)
Plan for rehabilitating existing facilities (REH-1)	2.00	143.0	2,802	0.830	2,300
Alternative (ALT-1)	3.00	143.0	4,204	0.835	3,500
Alternative (ALT-2)	4.00	143.0	5,605	0.835	4,600

8.3 Annual Potential Generated Energy

Generated energy is calculated by the following formula.

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

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

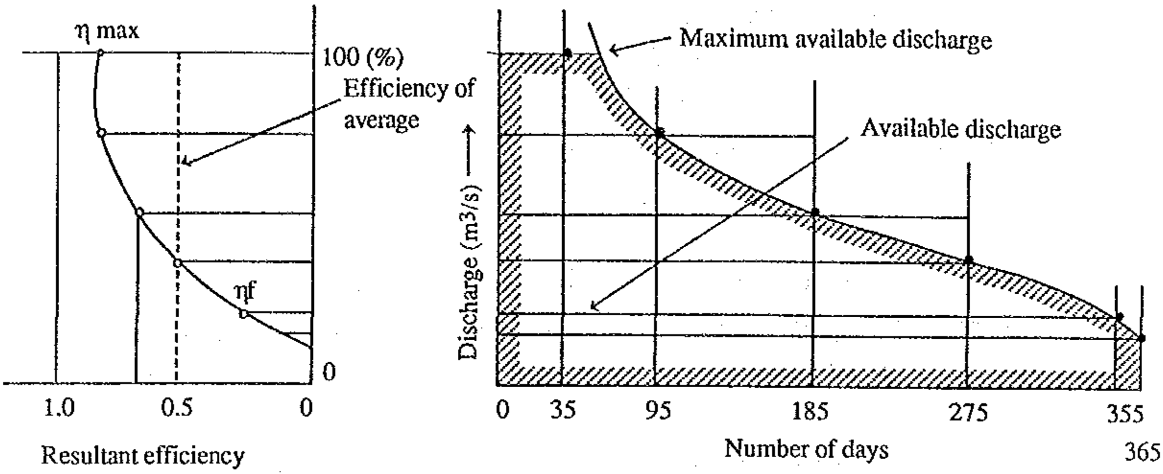
For the calculation of the annual potential generated energy at Julio Bravo P/P, item (2) as mentioned above is used for the following reasons.

① Instead of recorded observations at the intake site of this power plant, converted data from the Universidad gauging station operated by HIMAT is used as discharge data.

② Since there are no recorded observations at the Universidad 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 average generating output-to-available discharge ratio of (3) and flow-duration curve are used for the calculation. 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 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.

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 the case of the rehabilitation plan of the existing facilities (max. available discharge = 2.00 m^3/s):

20.4 GWh (100%)

(2) The annual potential generated energy in the case of the alternative plan 1 (ALT-1) (max. available discharge = 3.00 m^3/s):

29.4 GWh (97%)

(3) The annual potential generated energy in the case of the alternative plan 2 (ALT-2) (max. available discharge = 4.00 m^3/s)

34.6 GWh (85%)

Table 8.3 Calculation of Annual Potential Generated Energy

(1) Rehabilitation plan of existing facilities (REH-1)

Max. available discharge Q = 1.00 m^3/s x 2 units
Standard net head He = 143.0 m
Turbine type: Francis 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.	362	2.0	1.00	0.830	2,326	2,326	20,208
365	3	1.9	0.95	0.835	2,223	2,274	163
Total	365	365				(2,300)	20,371

(2) Alternative plan 1 (ALT-1)

Max. available discharge: Q = 1.5 m^3/s x 2 units
Standard net head (He): 143.0 m
Turbine type: Francis 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.	255	3.0	1.0	0.835	3,421	3,510	21,481
260	5	2.9	0.966	0.842	3,323	3,465	415
275	15	2.8	0.933	0.847	3,097	3,372	1,213
290	15	2.7	0.900	0.849	3,212	3,267	1,176
305	15	2.6	0.866	0.85	3,097	3,154	1,135
315	10	2.5	0.833	0.85	2,977	3,037	728
330	15	2.4	0.800	0.849	2,855	2,916	1,049
340	10	2.3	0.766	0.847	2,730	2,792	670
355	15	2.2	0.733	0.842	2,595	2,662	958
360	5	2.1	0.700	0.835	2,457	2,526	303
365	5	1.9	0.633	0.819	2,180	2,318	278
Total	365					(3,001)	29,406

(3) Alternative plan 2 (ALT-2)

Max. available discharge: Q = 2.0 m^3/s x 2 units
Standard net head (He): 143.0 m
Turbine type: Francis 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.	146	4.0	1.0	0.835	4,680	4,680	16,398
150	4	3.9	0.975	0.84	4,590	4,635	444
160	10	3.8	0.950	0.847	4,499	4,544	1,090
165	5	3.7	0.925	0.849	4,391	4,445	533
175	10	3.6	0.900	0.85	4,283	4,445	1,040
185	10	3.5	0.875	0.85	4,169	4,337	1,014
190	5	3.4	0.850	0.849	4,050	4,226	493
205	15	3.3	0.825	0.847	3,930	4,109	1,436
220	15	3.2	0.800	0.845	3,807	3,990	1,392
240	20	3.1	0.775	0.84	3,679	3,868	1,796
245	5	3.0	0.750	0.835	3,552	3,743	433
260	5	2.9	0.725	0.83	3,413	3,615	1,253
275	15	2.8	0.700	0.823	3,276	3,482	1,203
290	15	2.7	0.675	0.817	3,140	3,208	1,154
305	15	2.6	0.650	0.808	2,862	3,069	1,104
315	10	2.5	0.625	0.85	2,717	2,930	703
330	15	2.4	0.600	0.80	2,578	2789	1,004
340	10	2.3	0.575	0.79	2,435	2,647	635
355	15	2.2	0.550	0.78	2,25	2,506	902
360	5	2.1	0.525	0.755	2,010	2,365	283
365	5	1.9	0.475	0.819	2,180	2,152	258
Total	365					(3,556)	34,568

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, the headrace structure, with the exception of a 2,500-meter-long segment, needs 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 three rehabilitation plans are shown in Table 9.1.

$Q = 2.00\ m^3/s$
$Q = 3.00\ m^3/s$
$Q = 4.00\ m^3/s$

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
Discharge, Q (m^3/s)	2.0	3.0	4.0
Max. output, P (kW)	2,300	3,500	4,600
Facility utilization factor (%)	100	97	85

Rehabilitation and improvement plan:

Diversion weir	Because of damage to this weir, a new sandtrap will be built (common to all plans).
Intake	Improvement to allow for stable intake of maximum available discharge. A regulating gate will be built.
Desilting basin	A new, suitable-sized one will be constructed (common to all plans).
Conduction channel	The cross section at certain sections will be enlarged to allow for stable intake of maximum available discharge. Placement of covers for full length of channel (common to all plans).
Head tank	Will be expanded at its present location to increase regulating capacity. Spillway will be newly installed.
Penstocks	New construction
Generating equipment	Will be replaced with 2 new units.
Powerhouse building	Expansion of building at the present location. An overhead crane will be installed.
Outlet	Outlet will be improved.
Substation	The transformer will be replaced.

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 costs 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		
	Rehabilitation of the existing facilities	Increase of power output	
	REH-1	ALT-1	ALT-2
1. Specifications			
Design discharge (m^3/s)	1.00	1.50	2.00
Net head (m)	143	143	143
Theoretical output (kW)	1,400	2,100	2,800
Turbine type	H.F.	H.F.	H.F.
Turbine output (kW)	1,230	1,850	2,470
Generator power factor	0.9	0.9	0.9
Generator output (kVA)	1,300	1,950	2,600
Main transformer capacity (kVA)	2,600	3,900	5,200
2. FOB costs (U.S.$1,000)			
Generating equipment			
(1) Turbine etc.	442.85	557.15	642.85
(2) Generator etc.	145.70	164.25	182.15
(3)=(1)+(2) Sub-total:	588.55	721.4	825
(4) Number of units	2	2	2
(5)=(3)x(4) Subtotal:	1,177.1	1,442.8	1,650
(6) 4.16 kV switchgear etc.	122.1	122.1	122.1
(7) transformer	31.4	46.5	60.7
(8)=(5)+(6)+(7) Total:	1,330.6	1,611.4	1,832.8

(H.F. : Horizontal Francis)

Table 9.3 Implementation Costs of Generating Equipment

(units: US$1,000)

Item		Alternative					
		Rehabilitation of the existing facilities		Increase of power output			
		REH-1		ALT-1		ALT-2	
		A	B	A	B	A	B
1) FOB cost		1,330.6	-	1,611.4	-	1,832.8	-
2) Transportation costs, insurance							
	1) x 0.12	159.7	-	193.4	-	219.9	-
3) Tax	1) x 0.223	-	296.7	-	359.3	-	408.7
4) Value-added tax	1) x 0.134	-	178.3	-	215.9	-	245.6
5) Others	1) x 0.22	-	292.8	-	354.5	-	403.2
6) Total		1,490.4	767.8	1,804.8	929.7	2,052.8	1,057.5
7) Contingency	1) x 0.17	226.2	-	273.9	-	311.6	-
8) Eng. fee	1) x 0.149	198.3	-	240.1	-	273.1	-
9) Total	6) + 7) + 8)	1,914.9	767.8	2,318.8	929.7	2,637.5	1,057.5
10) Grand total		2,682.7		3,248.5		3,695	

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

9.2.2 Estimation of Civil Work 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 CEDENAR. 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		
	Rehabilitation of the existing facilities	Increase of power output	
	REH-1	ALT-1	ALT-2
Diversion weir and intake construction	61.7	61.7	61.7
Desilting basin construction	10.0	21.7	30.4
Conduction channel construction	33.5	36.1	43.4
Head tank construction	16.5	17.2	19.0
Penstock construction	98.2	113.1	148.0
Foundation of equipment construction	17.4	17.4	17.4
Powerhouse building construction	18.3	18.3	18.3
Temporary construction	14.3	14.3	14.3
Other construction	0	0	0
① Subtotal	269.9	299.8	352.5
② Contingency (① x 0.15)	40.5	45.0	52.9
③ Engineering fees ((① + ②) x 0.10)	31.0	34.5	40.5
④ Total (① + ② + ③)	341.4	379.3	445.9
⑤ Output loss	0	0	0
⑥ Grand total (④ + ⑤)	341.4	379.3	445.9

9.3 Comparison of Economic Indices

To compare the economic indices; 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-2 plan calls for US$1,070.00/kW per increase in power output, which is the lowest cost.

Table 9.5 Comparison of Construction Costs per kW

Item		Alternative		
		Rehabilitation of the existing facilities	Increase of power output	
		REH-1	ALT-1	ALT-2
Existing equipment output (kW)				
Rated output	Po	1,500	1,500	1,500
Available output	Pe	0	0	0
Post-rehabilitation output	P_1 (kW)	2,300	3,500	4,600
Recovered/increased output $\Delta P = P_1 - Pe$ (kW)		2,300	3,500	4,600
Rehabilitation work cost	(US$1,000)			
Foreign currency portion Cf		1,900	2,300	2,650
Local currency portion C		1,700	2,000	2,250
Total C = Cf + C		3,600	4,300	4,900
Construction cost per kW	(US$/kW)			
C/P_1		1,570	1,220	1,070
$C/\Delta P$		1,570	1,220	1,070

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 intrest payments for the construction are totaled and divided by 25 years.

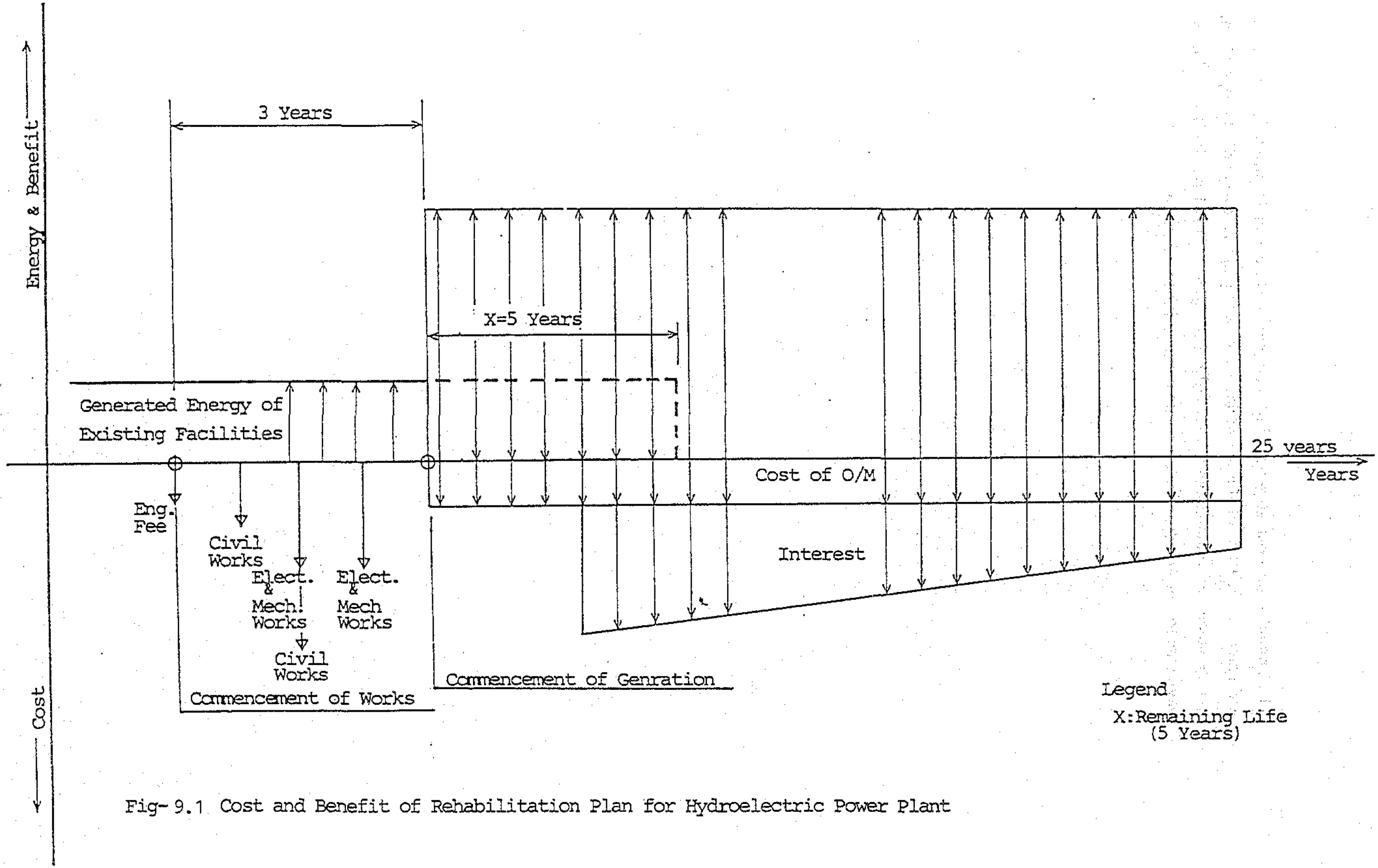


Fig- 9.1 Cost and Benefit of Rehabilitation Plan for Hydroelectric Power Plant

The results of calculation of generating costs per kWh are shown in Table 9.6. Since the generating capacity at Julio Bravo P/P was lost in 1984, a new generating plan at the existing site is under consideration. The generating cost per annually supplied energy is 15.0 mills/kwh according to ALT-2. The optimum plan is selected from ALT-1 or ALT-2.

Table 9.6 Comparison of Generating Cost per kWh

Item		Alternative		
		Rehabilitation of the existing facilities	Increase of power output	
		REH-1	ALT-1	ALT-2
Existing equipment capacity:				
Power output	Pe (kW)	0	0	0
Energy	Ee (GWh)	0	0	0
Rehabilitation plan:				
Power output	P_1 (kW)	2,300	3,500	4,600
Generated energy	E_1 (GWh)	20.4	29.4	34.6
Recovered/increased power				
Output	$\Delta P = P_1 - Pe$ (kW)	2,300	3,500	4,600
Energy	$\Delta E = E_1 - Ee$ (GWh)	20.4	29.4	34.6
Total of expenses at generating terminal: (US$1,000)				
Construction work cost				
Foreign currency portion Cf_1		1,900	2,300	2,600
Local currency portion $C\ell_1$		1,700	2,000	2,250
Construction cost total $C_1 = Cf_1 + C\ell_1$		3,600	4,300	4,900
Interest payment C_2				
Foreign currency portion Cf_2		3,083	3,733.3	4,246.4
Local currency portion $C\ell_2$		1,719	1,987.8	2,300.8
Total $C_2 = Cf_2 + C\ell_2$		4,802	5,721.1	6,547.2
AOM C_3 = US$4 x Pl x 25 years		230	350	460
Total $\Sigma Ci = C_1 + C_2 + C_3$		8,639	10,346.4	11,909.3
Average annual cost $C = \Sigma Ci/25$		345.5	413.9	476.4
Generating cost per annually supplied energy (mills/kWh)				
Per E_1	$C/(E_1 \times 0.95)$	18	15	15
Per ΔE	$C/(\Delta E \times 0.95)$	18	15	15

9.3.3 Overall Evaluation

Figure 9.2 shows a graph of the construction costs per kWh for each alternative plan. Taking these figures into account in the cost-benefit analysis, ALT-1 plan is thus selected as the optimum plan.

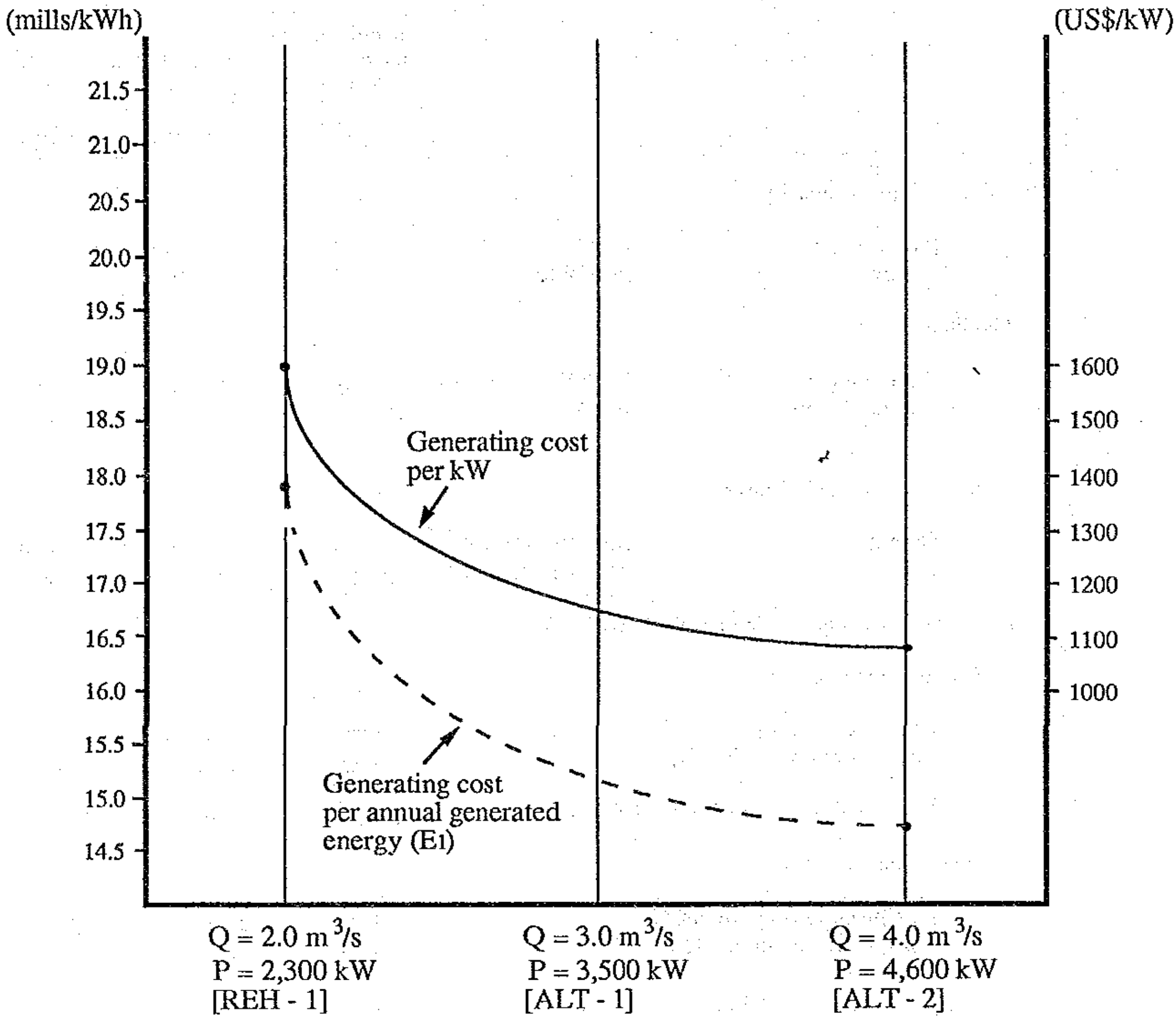


Fig.9.2 Generating Cost

CHAPTER 10 FINANCIAL ANALYSIS

To evaluate the profitability of rehabilitation plans, a cost-benefit analysis is adopted. The difference between revenue after the existing facilities are rehabilitated and the revenue when the existing facilities are not rehabilitated is regarded as the profitability of the investment. Then the financial analysis of the selected rehabilitation plan is made for the planning of the balance of revenue and expenditure in accordance with the cash balance. For the evaluation of the investment propriety within the national economy, refer to the economic analysis described in the main report.

10.1 Preconditions for the Financial Analysis

Preconditions set up for the financial analysis are summarized below:

(1) Residual life of existing power plant

In case of unchanging the existing facilities with new ones, residual life of the existing power plant is tentatively set at five years after the installation of new equipment.

(2) Estimation of construction cost

The construction cost is estimated in both foreign and local currency portion according to the market price as of September, 1989. Currency exchange rate between foreign currency (US$) and local currency (Col.$) is set at US$1.00 = Col.$369.4, as determined by DNP.

The construction cost includes the contingency and technical management expense. The land acquisition cost is not accounted because the plan is for rehabilitating the existing power plant. The FOB price of the generating facilities is taken from Japanese market price. The CIF price is calculated in the ratio of CIF price to FOB price which ISA usually applies to a hydroelectric power generation project. The ratio of CIF price to FOB price is 1.00 : 1.12.

(3) Service life

The service life of the project is set at 25 years after rehabilitation for evaluating the profitability.

The annual depreciation of facilities will be based on the fixed amount method adopted by ICEL. The service life, as described below, is determined according to the facility. The residual price will be set at zero.

1) Service life of civil structure : 50 years
2) Service life of generating facilities : 25 years

(4) Operation and maintenance costs

Operation and maintenance costs consist of the fixed cost which depends upon the scale invested in the facilities, and the variable cost which fluctuates in proportion to generated electric power. This study adopts the average cost, i.e., US$4.0 per installed capacity (kW) per year, which ISA usually applies to make an estimate of operation and maintenance costs of a hydroelectric power plant.

(5) Estimation of revenue

ICEL's electricity-selling unit price of US$13.36/MWh (Col $4,936,18/MWh) and US$2,942.36/MW (Col$1,086,909.69/MW) in December, 1988 is adopted as the financial unit price. The estimation of annual revenue can be made by multiplying the rated capacity and the annual supplied power at generating terminal.

(6) Discount rate

The discount rate which is used to calculate the net present value (NPV) and the cost-benefit ratio (C/B Ratio) is set at 7.6% per year. It is determined by the real interest rate in Colombia.

(7) Conditions for borrowing capital on investment

The loan conditions for borrowing capital in foreign and local currency are as follows:

1) Loan conditions of foreign currency

- Annual interest : 10%
- Period for principal repayment : 25 years (including a 4-year grace period)
- Terms of payment : Repayment of the principle in equal, annual amounts

2) Loan conditions of local currency

- Annual interest : 21%
- Period for principal repayment : 8 years (including a 1-year grace period)
- Terms of payment : Repayment of the principal in equal, annual amounts

(8) Constant price

The annual inflation rate in Colombia varied from 24 to 30%, but the prices used in the cost and benefit stream are set at the constant price in 1989.

(9) Evaluation index

For evaluating profitability, the following three indices, which are commonly used, are adopted.

(1) Cost-benefit ratio (C/B ratio)
(2) Net present value (NPV)
(3) Internal rate of return (IRR)

These indices are calculated by using "with" and "without" the project.

10.2 Comparison of Profitability

The profitability of the generating plans is calculated using the cash flow for each alternative plan, as shown in Table 10.1.

Table 10.1 Profitability Index of Alternative Plans

Alternative	C/B	NPV (US$1,000)	IRR (%)
REH-1	1.16	- 277	6.0
ALT-1	0.96	100	8.1
ALT-2	0.94	175	8.3

From the results of the financial analysis according to cash generation of the project, ALT-2 is determined to be the most profitable plan.

The rehabilitation plan, ALT-1 is selected as the optimum plan, which is described in Section 9.3.3, since it has a high profitability amongst the alternatives.

10.3 Financial Planning

The cash balance of the selected rehabilitation plan is prepared as a projected financial statement. The projected Profit-Loss Statement and Fund Flows Statement are shown in Table 10.2. According to the financial plan, the selected rehabilitation plan will show a profit from the year 2002, with a projected aggregate profit of US$777,000 at the end of service life.

Table - 10.2 PROJECTED FINANCIAL STATEMENTS

(1) PROJECTED REVENUE AND EXPENDITURE STATEMENT (at 1989 Price)

== Julio Bravo : ALT-1 ==

Year	Year in Order	(A) Total Operating Revenue	(B) Operating Expenditure (US$:1000) O/M Cost	Depreci-ation	Interest on Investment	Total	(C) Net Benefit (A)-(B)
1989	-6	0.0	0.0	0.0	0.0	0.0	0.0
1990	-5	0.0	0.0	0.0	0.0	0.0	0.0
1991	-4	0.0	0.0	0.0	0.0	0.0	0.0
1992	-3	0.0	0.0	0.0	12.7	12.7	-12.7
1993	-2	0.0	0.0	0.0	25.3	25.3	-25.3
1994	-1	0.0	0.0	0.0	31.6	31.6	-31.6
1995	0	0.0	0.0	0.0	154.4	154.4	-154.4
1996	1	191.8	7.0	133.6	441.8	582.3	-390.5
1997	2	383.6	14.0	133.6	618.5	766.1	-382.5
1998	3	383.6	14.0	133.6	565.2	712.7	-329.1
1999	4	383.6	14.0	133.6	500.1	647.7	-264.1
2000	5	383.6	14.0	133.6	435.1	582.7	-199.1
2001	6	383.6	14.0	133.6	370.1	517.6	-134.0
2002	7	383.6	14.0	133.6	198.3	345.8	37.7
2003	8	383.6	14.0	133.6	186.6	334.2	49.4
2004	9	383.6	14.0	133.6	175.0	322.5	61.1
2005	10	383.6	14.0	133.6	163.3	310.9	72.7
2006	11	383.6	14.0	133.6	151.6	299.2	84.4
2007	12	383.6	14.0	133.6	140.0	287.5	96.1
2008	13	383.6	14.0	133.6	128.3	275.9	107.7
2009	14	383.6	14.0	133.6	116.6	264.2	119.4
2010	15	383.6	14.0	133.6	105.0	252.5	131.1
2011	16	383.6	14.0	133.6	93.3	240.9	142.7
2012	17	383.6	14.0	133.6	81.6	229.2	154.4
2013	18	383.6	14.0	133.6	70.0	217.5	166.1
2014	19	383.6	14.0	133.6	58.3	205.9	177.7
2015	20	383.6	14.0	133.6	46.7	194.2	189.4
2016	21	383.6	14.0	133.6	35.0	182.6	201.0
2017	22	383.6	14.0	133.6	23.3	170.9	212.7
2018	23	383.6	14.0	133.6	11.7	159.2	224.4
2019	24	383.6	14.0	133.6	0.0	147.6	236.0
2020	25	383.6	14.0	133.6	0.0	147.6	236.0
TOTAL		9398.1				8621.3	776.8
						C/B:	0.92

(2) PROJECTED FUNDS FLOW STATEMENT (Constant Price at 1989)

== Julio Bravo : ALT-1 ==

(US$:1000)

Year	Year in Order	(A) Source: Benefit before Interest	Depreci-ation	Balance Brought Forward	Long/Short Term Borrowing	Total	(B) Application: Construc-tion Progress	Debt Service Interest	Debt Service Principal	Total	Cash Balance (A)-(B)
1989	-6	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0
1990	-5	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0
1991	-4	0.0	0.0		126.5	126.5	126.5	0.0	0.0	126.5	0.0
1992	-3	0.0	0.0		126.5	126.5	126.5	12.7	0.0	139.2	-12.7
1993	-2	0.0	0.0		63.3	63.3	63.3	25.3	0.0	88.6	-25.3
1994	-1	0.0	0.0		696.2	696.2	696.2	31.6	0.0	727.8	-31.6
1995	0	0.0	0.0		1871.4	1871.4	1871.4	154.4	0.0	2025.7	-154.4
1996	1	51.2	133.6		1344.6	1529.4	1344.6	441.8	254.2	2040.5	-511.1
1997	2	236.0	133.6			369.6		618.5	254.2	872.7	-503.1
1998	3	236.0	133.6			369.6		565.2	370.8	936.0	-566.4
1999	4	236.0	133.6			369.6		500.1	370.8	870.9	-501.3
2000	5	236.0	133.6			369.6		435.1	370.8	805.9	-436.3
2001	6	236.0	133.6			369.6		370.1	370.8	740.9	-371.3
2002	7	236.0	133.6			369.6		198.3	370.8	569.1	-199.5
2003	8	236.0	133.6			369.6		186.6	116.6	303.3	66.3
2004	9	236.0	133.6	66.3		435.9		175.0	116.6	291.6	144.3
2005	10	236.0	133.6	144.3		513.9		163.3	116.6	279.9	234.0
2006	11	236.0	133.6	234.0		603.6		151.6	116.6	268.3	335.3
2007	12	236.0	133.6	335.3		704.9		140.0	116.6	256.6	448.3
2008	13	236.0	133.6	448.3		817.9		128.3	116.6	244.9	573.0
2009	14	236.0	133.6	573.0		942.5		116.6	116.6	233.3	709.3
2010	15	236.0	133.6	709.3		1078.9		105.0	116.6	221.6	857.2
2011	16	236.0	133.6	857.2		1226.8		93.3	116.6	210.0	1016.9
2012	17	236.0	133.6	1016.9		1386.5		81.6	116.6	198.3	1188.2
2013	18	236.0	133.6	1188.2		1557.8		70.0	116.6	186.6	1371.2
2014	19	236.0	133.6	1371.2		1740.8		58.3	116.6	175.0	1565.8
2015	20	236.0	133.6	1565.8		1935.4		46.7	116.6	163.3	1772.1
2016	21	236.0	133.6	1772.1		2141.7		35.0	116.6	151.6	1990.1
2017	22	236.0	133.6	1990.1		2359.7		23.3	116.6	140.0	2219.7
2018	23	236.0	133.6	2219.7		2589.3		11.7	116.6	128.3	2461.0
2019	24	236.0	133.6	2461.0		2830.6		0.0	0.0	0.0	2830.6
2020	25	236.0	133.6	2830.6		3200.2		0.0	0.0	0.0	3200.2