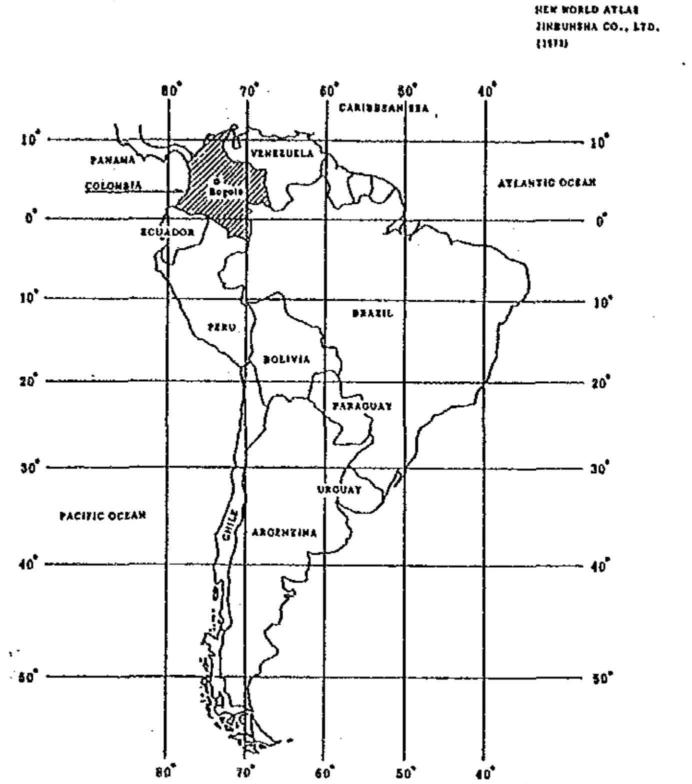
FEASIBILITY STUDY
ON
SMALL-SCALE POWER PLANTS
REHABILITATION PROJECT
IN
THE REPUBLIC OF COLOMBIA

CARACOLI HYDROELECTRIC POWER PLANT

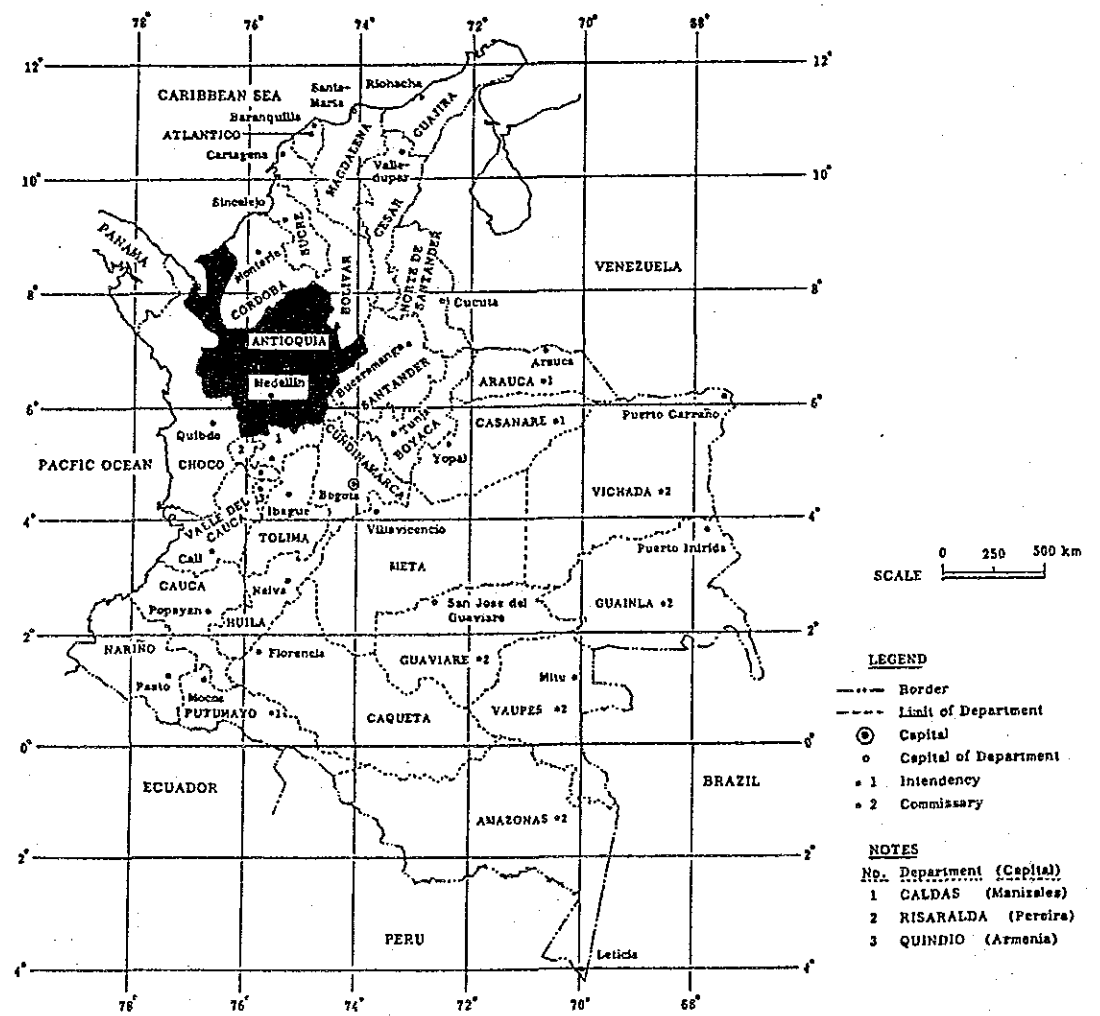
MARCH 1990

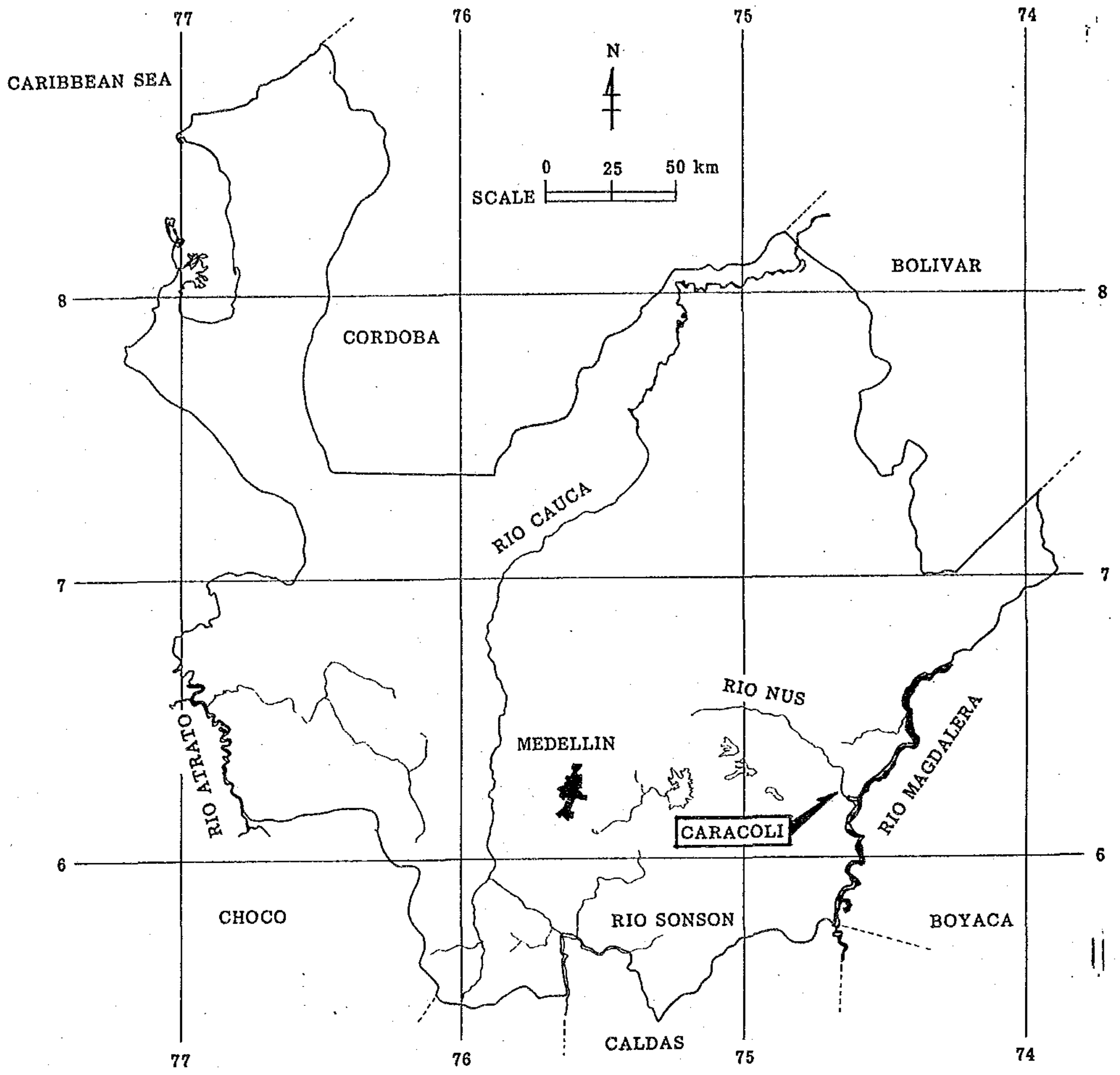
Japan International Cooperation Agency

MAP OF SOUTH AMERICA

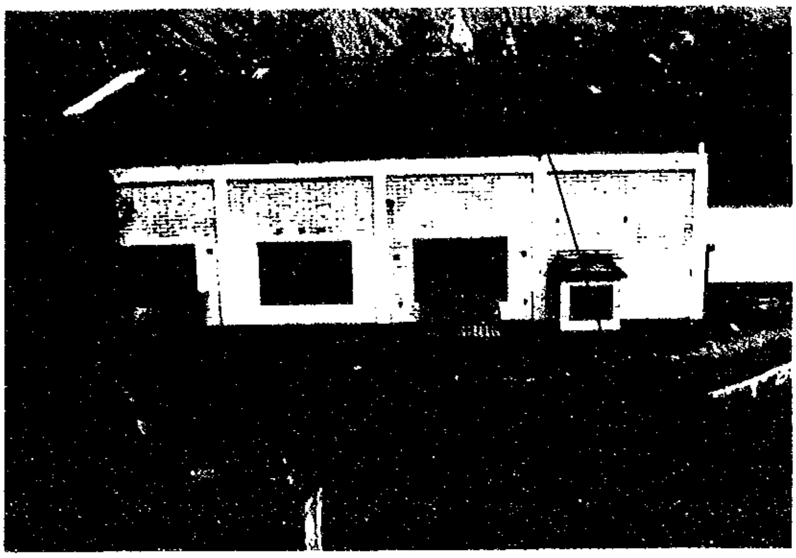


POLITICAL DIVISION IN THE REPUBLIC OF COLOMBIA

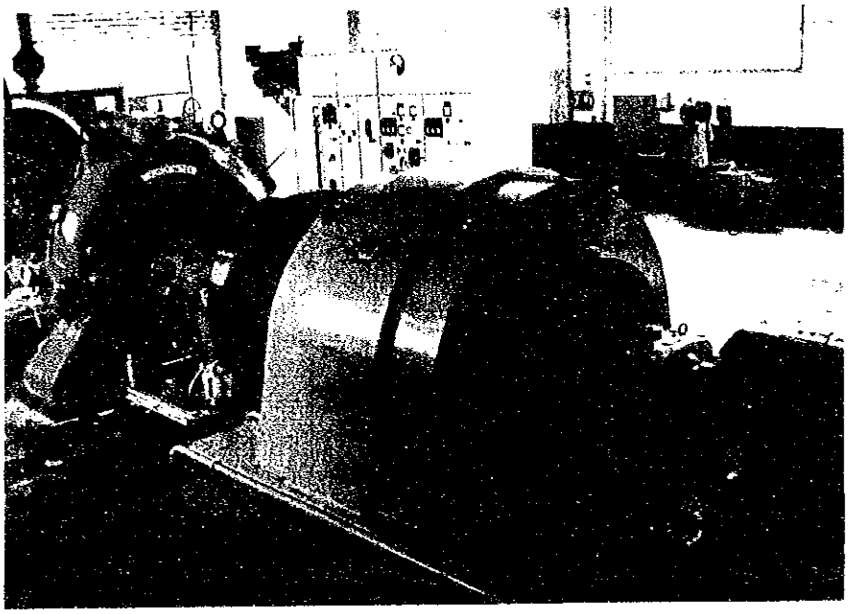




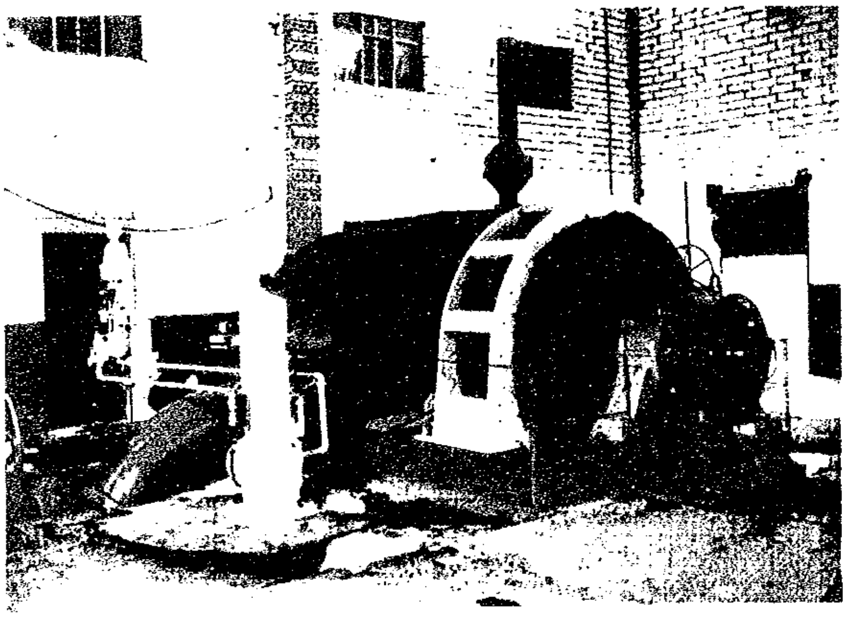
Location Map of Study Area



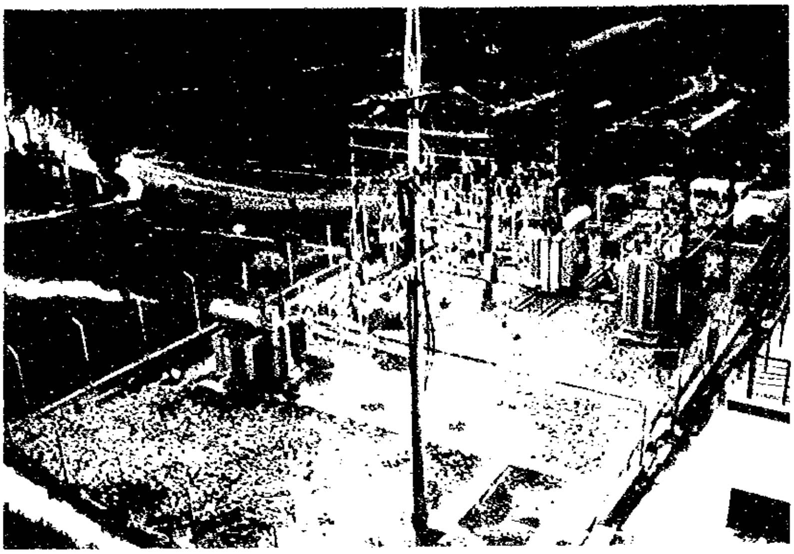
Powerhouse



Francis turbine and generator



Pelton turbine and generator



Substation

Location Map of Study Area
Photographs

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CHAPTER 1   INTRODUCTION

The feasibility study (hereinafter referred to as the FS) for the rehabilitation plan of Caracoli run-of-river type hydroelectric power plant (rated output of 3.2 MW) was conducted following the pre-FS that was carried out for eight months from November, 1987 to June, 1988. This report is prepared to summarize the results of the FS.

This FS was performed in accordance with the Scope of Work (S/W) agreed and signed in July 1988 between Japan International Cooperation Agency (JICA) and Instituto Colombiano de Energia-Electrica (ICEL). The study was conducted during 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, Caracoli hydroelectric power plant (hereinafter referred to as Caracoli P/P) was selected as a candidate for the FS for the following reasons:

1) The environmental disruption due to the rehabilitation will be minimal, and the rehabilitation of this power plant will not conflict with other already acquired water rights.

2) Since the diversion weir, the intake and the head tank have been damaged, they do not function properly.

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

- Maximum output : 6.7 MW
- Annual potential generated power : 57 GWh
- Facility utilization factor : 96 %

CHAPTER 2 SUMMARY OF STUDY RESULTS

The power plant owned by EADE is the run-of-river type (rated output: 3,200 kW), and is located along the Nus River in Antioquia Department. The maximum output recorded in February 1989 was 2,300 kW, about 900 kW (28%) lower than the rated output. The annual generated energy in 1988 was 18,285.9 MWh.

(1) Present conditions of generating facilities and their problems

In Caracoli P/P, the penstock is installed from the upstream high elevation area to the downstream low elevation area near the winding Nus River. In the existing generating facilities, the diversion weir, the intake and the desilting basin/head tank have been damaged and obsolete. On the other hand the headrace, which consists of the 80-meter-long lined pressure tunnel constructed of masonry and the 1,300-meter-long penstock (diameter: 1,350 mmϕ) is maintained in good condition.

The Pelton type turbine (rated output: 1,600 kW) manufactured in 1935 and the horizontal Francis type turbine (rated output: 1,600 kW) manufactured in 1963 are installed in parallel and in close proximity each other. The former has been used for 55 years and the latter for 27 years. The current rated output of each turbine is 1,150 kW and each turbine maintains approximately 72% of the rated output.

Any discharge observation facilities or hydrological gauging station have not been provided near the upstream or downstream area in this power plant. So in this study, the discharge data, offered by HIMAT #2308-716 Caramanta hydrological gauging station located about 5 km upstream from the intake, are used.

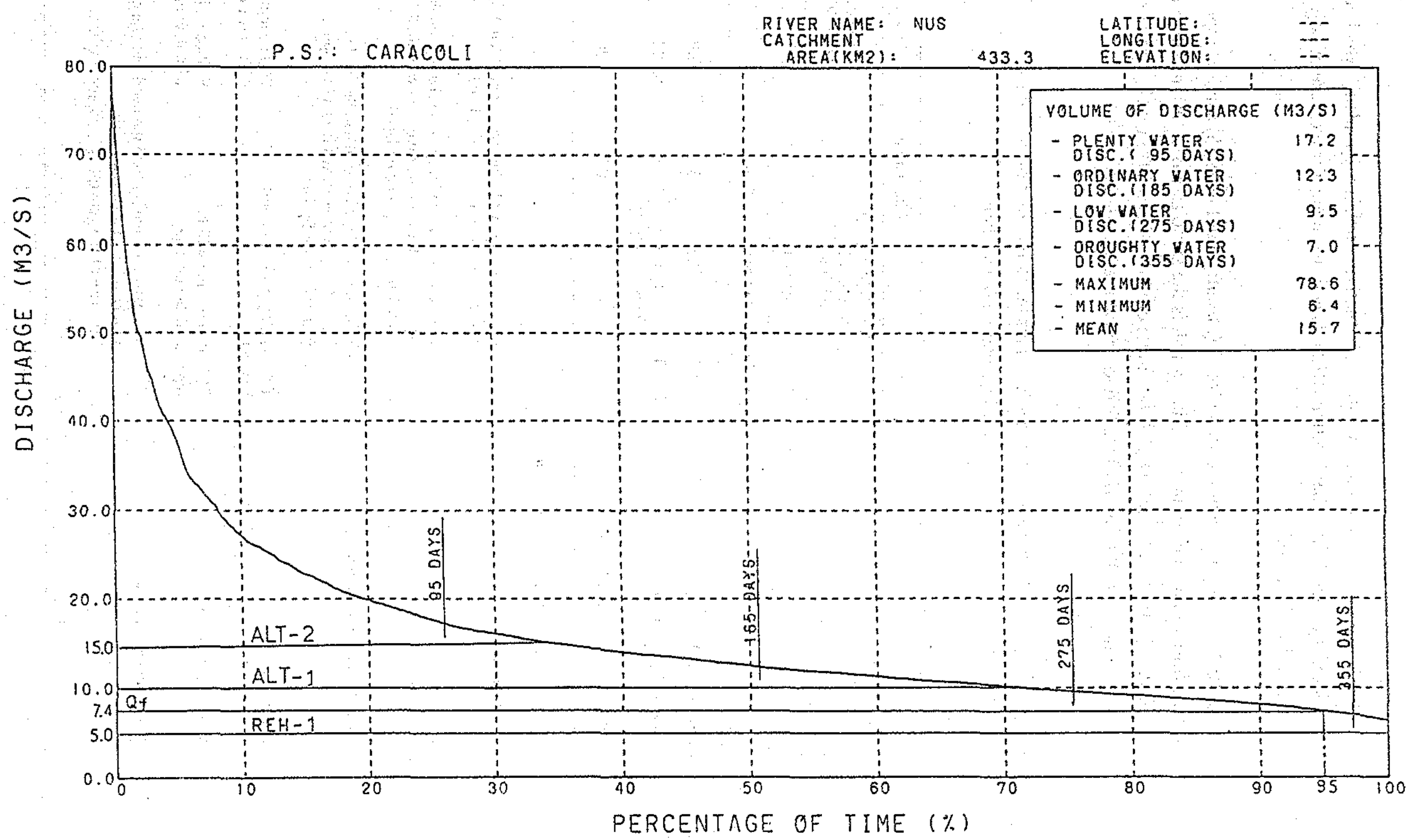
There was 94.4 km^2 difference between the value of catchment area (320.0 km^2) registered at the above gauging station and its value measured (414.4 km^2) by JICA study team.

(2) Alternative rehabilitation plans

Fig. 2.1 shows the discharge-duration curve at the intake, which is converted using the discharge data offered by HIMAT's Caramanta hydrological gauging station. The maximum available discharge of the existing power plant, Q=5.0 m^3/s, is considered rather small from the viewpoint of water utilization.

Accordingly, comparative studies shall be made for the generation-optimizing plan as well as the rehabilitation plan of the existing generating facilities. The maximum available discharge is set to three different values of 5.0 m^3/s (present maximum available discharge), 10.0 m^3/s and 15.0 m^3/s, and respective generation plans will be formulated.

Fig-2.1 TYPICAL FLOW DURATION CURVE AT INTAKE SITE



As shown in Table 2.1, there are three alternative rehabilitation plans, which are a plan for existing facilities rehabilitation (REH-1) and two plans for increase of power output (ALT-1 and ALT-2).

In REH-1, only the Pelton turbine No. 1 (manufactured in 1935) shall be replaced with the horizontal shaft Francis turbine which is the same as No. 2 turbine (manufactured in 1963). In ALT-1 and ALT-2, two units of horizontal shaft Francis turbines with the same output shall be newly installed to double the installed capacity.

Table 2.1 Alternative Plans for Rehabilitation of Caracoli Hydroelectric Power Plant

Item	Alternative				
	Rehabilitation of the existing facilities			Increase of power output	
	REH-1			ALT-1	ALT-2
	Unit No.1	Unit No.2	Total		
Discharge, Q (m^3/s)	2.5	2.5	5.0	10.0	15.0
Max. output, P (kW)	1,700	1,100 (present output)	2,800	6,700	10,200
Facility utilization factor (%)			100	96	80

Rehabilitation and improvement plan:		
Diversion weir	Because of damage to this weir, a new sandtrap will be built (common to all plans).	
Intake	Will be reconstructed into the horizontal intake type in parallel with the diversion weir modification. Screen and gate will be replaced.	
Desilting basin/head tank	Will be reconstructed to match the intake layout to prevent vortexes from developing. (Common to all plans)	
Penstocks	The existing penstocks will be used.	Will be replaced with new one.
Generating equipment	#1 Pelton turbine will be replaced with new one.	Will be replaced with new one.
Powerhouse building	The existing buildings and the overhead crane will be used after partial repair. The base of the generating equipment will be remodeled.	

(3) Selection of optimum plan

Comparative study results of alternative plans are summarized in Table 2.2.

ALT-1 for doubling the available discharge (5 m^3/s) to 10 m^3/s is well balanced and promising generation plan from the viewpoint of economical indices, e.g., construction cost per kW and generating cost per kWh.

In case of ALT-1, replacement of penstocks is necessary, but the present penstocks are maintained in good condition, and do not need to be replaced immediately.

In this report, the basic design concept for ALT-1 to be conducted in the FS stage is described in Chapter 11.

Table 2.2 Comparison of Rehabilitation Plan for the Caracoli Power Plant

Alternative Plan		(1) Specifications for Existing Generating Facilities					(2) Rehabilitation Plan							(3) Recovered or Increased Energy	
		(10)	(11)	(12)	(13) Present facility capacity		(20)	(21)	(22)	(23)	(24)	(25)	(26)	(30)	(31)
		Max. available discharge Qo (m^3/s)	Net head Ho (m)	Rated output Po (kW)	(14) Output Pe (kW)	(15) Generated energy Ee (GWh)	Max. available discharge Q_1 (m^3/s)	Standard net head H_1 (m)	Theoretical output =9.8x(20)x(21) (kW)	Resultant efficiency η	Output =(22)x(23) P_1 (kW)	Annual probable generated energy E_1 (GWh)	Facility utilization factor ε (%)	Output =(24)-(14) ΔP (kW)	Annual probable generated energy (25)-(15) ΔE (GWh)
REH-1	Pelton	2.5	86.0	1,600	1,200	9.17	2.5	82.9	2,031	0.835	1,700	14.9	100	500	5.7
REH-1	Francis	2.5	86.0	1,600	1,100	9.64	2.5	82.9	2,031	0.542	1,100	9.6	100	0	0
REH-1	Total	5.0	86.0	3,200	2,300	18.81	5.0	82.9	4,062	-----	2,800	24.5	100	500	5.7
ALT-1							10.0	82.9	8,124	0.835	6,700	57.0	96	4,400	38.1
ALT-2							15.0	82.9	12,186	0.845	10,200	72.3	80	7,900	53.5

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) Foreign currency portion C_{1f}	(42) Local currency portion $C_{1\ell}$	(43) (41)+(42) C_1	Civil work cost C_2	(43)+(44) C	Cost per ΔP =(45)/(30) C/ΔP	Cost per P_1 =(45)/(24) C/P_1	Operation and maintenance costs AOM	(62) Foreign currency portion 2.610 x (41) ÷ 25	(63) Local currency portion 2.016 x [(42)+(44)] ÷ 25	(64) (62)+(63)	(60)+(64)	per E_1 =(65)/(25) ÷0.95	per ΔE =(65)/(31) ÷ 0.95	C/B	Priority order
REH-1	Pelton	1,000	400	1,400	900	2,300	4,700	1,400	6.8	107	106	213	220	16	41	1.51	3
REH-1	Francis																
REH-1	Total																
ALT-1		2,900	1,200	4,100	2,900	7,000	1,600	1,050	26.8	305	329	634	661	12	18	0.99	1
ALT-2		3,600	1,450	5,050	3,750	8,800	1,100	860	40.8	376	420	796	837	12	16	0.96	1

(Notes)

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

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

(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) : E_1(Energia Media)

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

(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	Civil Engineering	Head of Central Eng. Div.
Jairo E. Gonzalez Morales	Civil Engineering	Central Eng. Div.
Mario Gutierrez Ospina	Civil Engineering	Central Eng. Div.
Rafael Torres Mariño	Civil Engineering	Central Eng. Div.
Rafael Gomez Florez	Civil Engineering	Central Eng. Div.
Jorge E. Hurtado Muños	Civil Engineering	Central Eng. Div.

3.1.3 Supporting Technical Staff from EADE

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

Supporting Staff	Position
Humberto Alonso Codavid A.	Manager of Planning Department
David Aguilar	Manager of Substation and Plants
Walter Leon Ospina Ortiz	Planning 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:

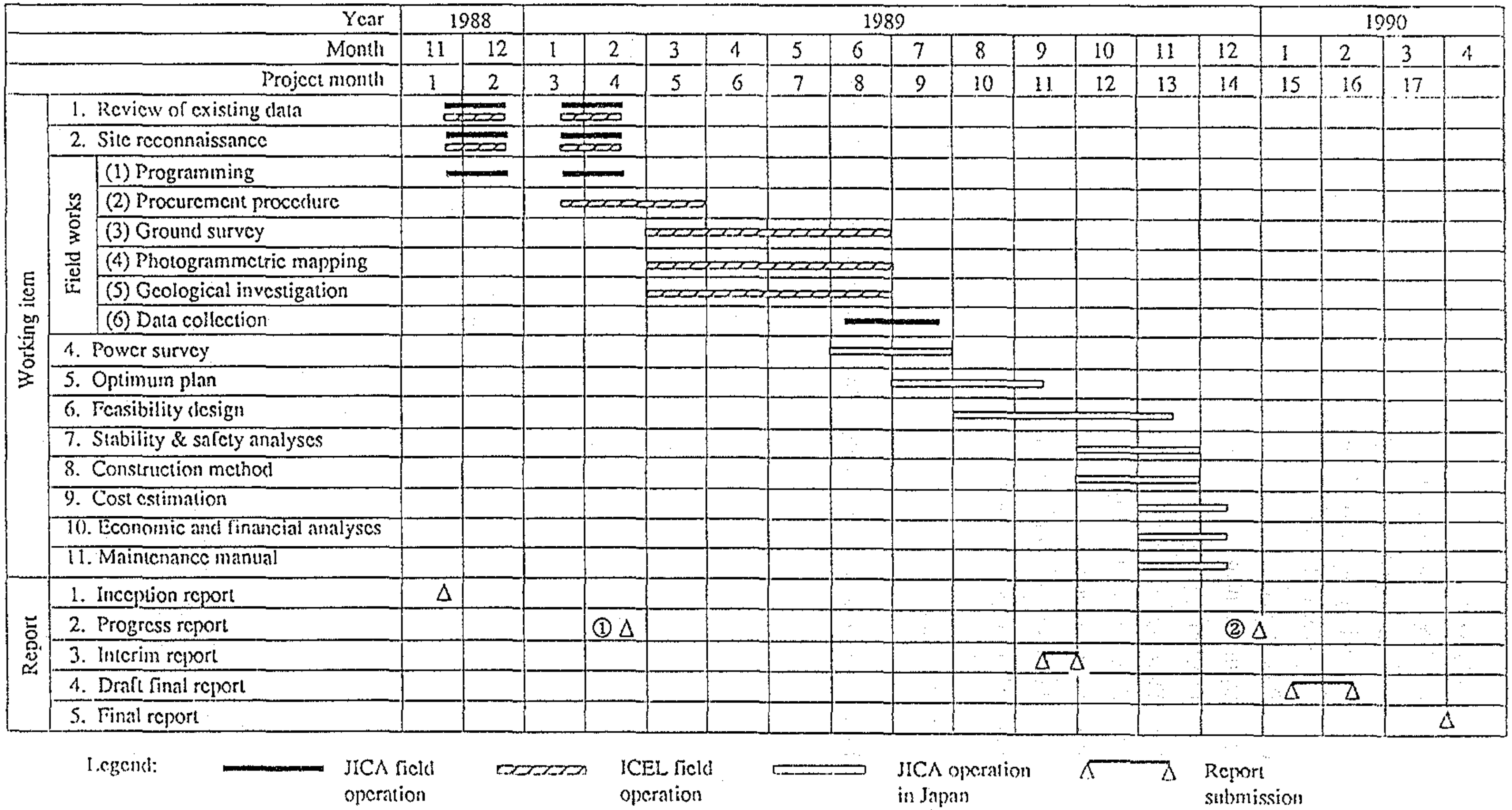
(1) Review of the existing data
(2) Site reconnaissance

(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



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

In the first site reconnaissance, two civil and electrical 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
Feb. 6	Bogota → Medellin	Discussion at EADE	R. Torres	Murao Toyama Masayuki Tamai
Feb. 7	Medellin → Caracoli	Field survey at Caracoli P/P		
Feb. 8		Same as above		
Feb. 9	Caracoli → Medellin	Travel		
Feb. 10		Discussion at EADE		

The second field survey

Date	Schedule	Detail of Study Item	Member	
			ICEL	JICA
June 27	Bogota → Medellin → Caracoli	Discussion at EADE, and travel		Yoshio Kawasaki Takashi Inoue
June 28		Field survey at Caracoli P/P		
June 29		Same as above		
June 30	Caracoli → Medellin	Discussion at EADE and travel		

3.3 Detail of Field Survey Work

The field survey work planned in consultations 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 Fig. 3.1. The scales for the topographic maps are as follows:

- Scale : 1:200
- Contour lines : 2 m pitch
- Scale in section A-A : 1:200 (H), 1:50 (V)

3.3.2 Boring Survey Work Plan

The boring survey was conducted as shown in Fig. 3.1.

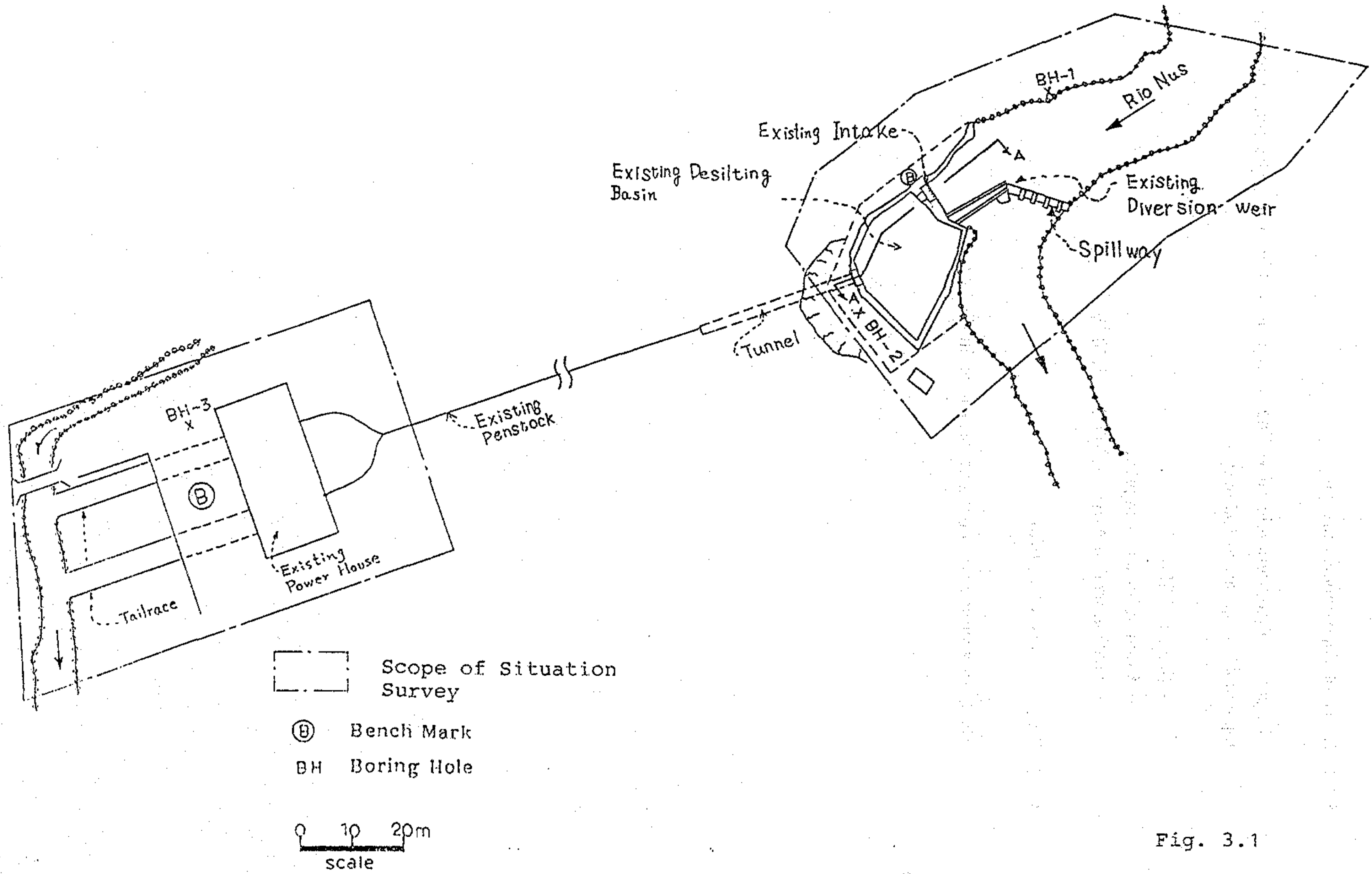

Rio Nus
BH-1
Existing Intake
A
Existing Desilting Basin
Existing Diversion weir
Spillway
BH-2
Tunnel
Existing Penstock
BH-3
Existing Power House
Tailrace
Scope of Situation Survey
Bench Mark
BH Boring Hole
0 10 20m
scale

Fig. 3.1

CHAPTER 4 PRESENT CONDITIONS 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 182 MW, while installed capacity was 13 MW (7%). In 1987, electric power was 911 GWh, while supplied power was 66 GWh, representing about 7% of total electric power. The public electric power company bought electricity to cover the remaining 1,098 GWh from another electric power company.

The breakdown of power demand in 1987 indicates that power demand for residential, commercial, industrial, and miscellaneous uses was 79%, 8%, 6% and 7% respectively. Power demand for residential was high, while that for industrial was low.

Annual average rate of increase in power demand from 1983 to 1987 was 10.3%, while that of generated energy, 31.6%, showing increase. However, 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)	113	120	130	132	182	12.7
2. Electric Energy (GWh)						
1) Residential	471	528	571	635	719	11.2
2) Commercial	59	57	60	67	63	5.5
3) Industrial	35	45	48	52	59	13.9
4) Miscellaneous	51	53	68	54	60	4.1
Total	616	683	747	808	911	10.3
SUPPLY						
1. Installed Capacity (MW)	13	13	13	13	13	0
2. Generated Energy (GWh)	22	37	45	64	66	31.6
3. Power Loss (GWh)	177	207	227	246	253	9.3

(Source: INFORME ESTADISTICO: RESUMEN 1983-1987)

* Annual average increase rate is calculated as follows:

Example: When peak demand is 12.7%, $113 \times (1 + x)^4 = 182$

$x = 0.127$ (12.7%)

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	1.7	1.7	1.3	1.3	1.3	-6.5
2. Hydroelectric	11.2	11.2	11.2	11.2	11.2	0
3. Others	0	0	0	0	0	0
Total	12.9	12.9	12.5	12.5	12.5	-0.78

(Source: INFORME ESTADISTICO: RESUMEN 1983-87)

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

Table 4.3 Conditions of Caracoli Power Plant (1984-1988)

Item	1983	1984	1985	1986	1987	1988
1) Installed capacity (kW)	3,200	3,200	3,200	3,200	3,200	3,200
2) Generated energy (MWh)	4,708	17,225	19,869	19,578	19,114	18,286
3) Utilization factor (%)	17	61	71	70	68	65
4) Operating time (%)	45	84	94	90	88	86

(Source: Data collected by EADE)

(2) Transmission facilities

The public electric power company provides 115 kV transmission lines to its transmission and substation facilities at Caracoli P/P. Voltage to be transmitted to Caracoli P/P are 13.2 kV and 44 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$/kWh)	2.25	2.73	3.51	4.58	5.70	26.0
Electric Charge (Average): (COL$/kWh)						
1. Residential	1.61	2.11	2.83	3.74	4.66	30.4
2. Commercial	4.58	5.37	9.42	12.96	16.46	37.7
3. Industrial	3.70	4.19	5.17	6.99	9.08	25.2
4. Public use	2.35	3.00	4.49	6.08	8.13	36.4
5. Average	2.06	2.57	3.61	4.83	6.05	30.9
Breakdown of Power Demand by customer						
1. Residential	132,705	151,954	166,407	188,912	208,557	12.0
2. Commercial	12,965	13,041	13,619	14,451	15,566	4.7
3. Industrial	1,134	1,204	1,265	1,488	1,653	9.9
4. Others	2,842	3,107	3,452	3,439	3,759	7.2
5. Total	149,646	169,306	184,743	208,290	229,535	11.3
Diffusion of Electricity						
1. Overall (1000 households)	1,637	1,675	1,713	1,752	1,791	2.3
2. Power demand (1000 households)	626	717	785	892	984	12.0
3. Electrification rate (%)	38	43	45	51	55	9.7

(Source: INFORME ESTADISTICO: RESUMEN 1983-87)

4.1.4 Forecast of Power Supply and Demand

Fig. 4.1 shows the future balance of power demand and supply by the year 1995, based on the present conditions of the balance described in Table 4.1. The annual average increase rates of electric power are assumed as follows.

1) Annual average increase rate of power demand: 10.3%

2) Annual average increase rate of generated energy: 0% (In Table 4.1, the annual average increase rate of generated energy is 33.8%, however, the installed capacity was set at 13 MW. Accordingly the generated energy after 1987 is assumed to be constant at 66 GWh and the increase rate is assumed to be 0%).

3) Annual average increase rate of power loss: 9.3%.

4) The power bought is obtained using the following formula.

   Power bought = (Power demand + Power loss) - Generated power

   Example: The power demand in 1995 is calculated as follows:

   $$911 \times (1 + 0.103)^8 = 1{,}996 \text{ (GWh)}$$

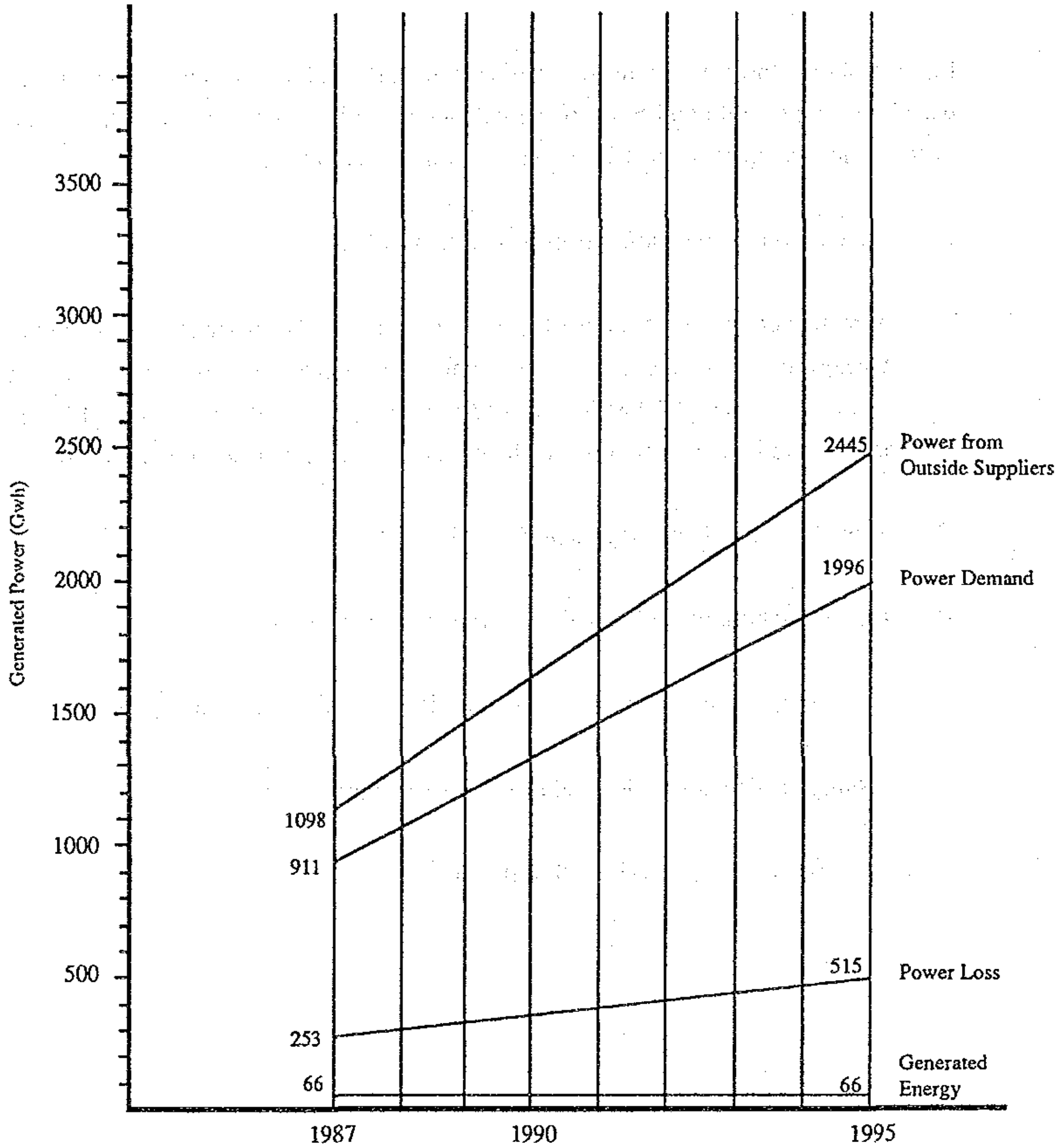


Fig. 4.1 Power Demand Forecast

4.2 Operation Record of the Existing Power Plant

4.2.1 Generated Energy

The record of generated energy and the running periods during six years from 1983 to 1988 are shown in Table 4.5.

In 1983, Unit No. 1 had been stopped from April to August, and Unit No. 2 has been stopped throughout the year. Accordingly the facility utilization factor of this year was low at 17%.

During five years from 1984 to 1988, the average facility utilization factor was 67%, showing the operation condition was normal.

Table 4.5 Record of Generated Energy and Running Periods

Year	Unit No.	Output inscribed on the name plate	Generated energy	Total generated energy	Running period	Total running period	Facility utilization factor	Total facility utilization factor	Operation factor	Total operation factor
1983	1	1.6	4,708	4,708	4,032	7,936	34	17	46	45
	2	1.6	0		3,904		0		45	
1984	1	1.6	10,350	17,224.7	8,134	14,793	74	61	93	84
	2	1.6	6,874.7		6,659		49		76	
1985	1	1.6	10,376	19,868.5	8,326	16,482	74	71	95	94
	2	1.6	9,492.5		8,156		68		93	
1986	1	1.6	10,408	19,577.5	8,197	15,806	74	70	94	90
	2	1.6	9,169.5		7,609		65		87	
1987	1	1.6	10,598	19,113.7	7,977	15,351	76	68	91	88
	2	1.6	8,515.7		7,374		61		84	
1988	1	1.6	10,631	18,285.9	8,145	15,011	76	65	93	86
	2	1.6	7,654.9		6,866		55		78	

(Note)

1. Facility utilization (%) = $\frac{\text{Generated energy (MWh)}}{\text{8,760(hr) x Output inscribed on the name plate (MW)}} \times 100$

2. Operation factor (%) = $\frac{\text{Running period (hr)}}{\text{8,760 (hr)}} \times 100$

4.2.2 Operation and Maintenance Costs

Record of Caracoli P/P's operation and maintenance costs in 1988 is shown in Table 4.6.

Average operation and maintenance costs per generated energy is 471 pesos/MWh.

Table 4.6 Record of Operation and Maintenance Costs (1988)

Month	Generated energy (MWh)	Operation and maintenance costs (Pesos)	Pesos/MWh
1	1,531.1	718,125	469
2	1,127.9	718,125	637
3	1,019.0	718,125	705
4	1,355.3	718,125	530
5	1,666.7	718,125	431
6	1,918.2	718,125	374
7	1,339.1	718,125	536
8	1,707.7	718,125	421
9	1,564.2	718,125	459
10	1,655.5	718,125	434
11	1,639.1	718,125	438
12	1,762.1	718,125	408
Total	18,285.9	8,617,500	471

4.3 General Conditions of Generating Equipment and Civil Structures

4.3.1 General Conditions of Generating Equipment

The present condition of the generating equipment is summarized below:

(1) Generating equipment

Pelton and Francis turbines are installed in this power plant.

The Pelton turbine was manufactured in 1935 and already used for 55 years. Its bucket requires repair every two years because of wear by sand.

The Francis turbine was manufactured in 1963 and already used for 27 years. Its runner requires repair every five years because of wear by sand.

Major defects in the generating equipment are shown in Table 4.7.

Table 4.7 Major Defects in Generating Equipment

	Unit No.1 (Pelton turbine)	Unit No.2 (Francis turbine)
Water turbine	1) The bucket is worn out by sand. The repair by welding is required every two years. 2) A bearing temperature alarm is not provided.	1) The casing and runner are worn out by sand. 2) The guide vane does not close completely because of wear by sand. 3) The above portions require repair every five years.
Inlet valve	1) Operation of this valve is difficult because of manual operation. 2) The valve does not close completely.	1) Same as left 2) Same as left
Governor	Frequency adjustment cannot be made without the link to 44 kV system.	-
Generator	No problem. (The winding was burnt out five years ago and replaced with a high-insulation winding.)	-
Control panel for turbine and generator	The measuring instruments and the panel wiring were modified.	Same as left

(2) Substation

The 44 kV substation is installed outside to transmit electricity from this power plant to San Jose del Nus Substation.

Main transformer No. 1 is single-phase, 667 kVA x 3 and used for 53 years.

Main transformer No. 2 is three-phase, 2000 kVA and in use for 28 years. The high-tension winding was replaced with a new one three years ago.

The circuit breaker and the disconnecting switch have been used for 22 years. No remarkable defects are found in the substation equipment.

(3) Switchgear

13.2 kV switchgears are installed in the building to distribute electricity from Caracoli, Virginia, and Cascaron on a voltage of 13.3 kV.

There is no remarkable defect in this switchgear because it was manufactured only two years ago.

(4) Transmission and distribution lines

The 44 kV transmission line and the 13.2 kV distribution line were provided 21 years ago.

The 13.2 kV distribution line has been gradually replaced.

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 45 m, 996 m and 2 m, respectively. The foundation exists on the bedrock. The piers are installed every 2 m in the 15 m-wide crest on the left bank of the river. Using the weirs inserted between the piers, water level can be adjusted.

The conduction channel (10 m in width and 15 m in length) is constructed on the right bank in parallel with the river, connected with the head tank. There is the intake on the right bank of the downstream end and the sand trap channel on its left bank. The intake entrance is constructed in parallel with the river. The width, height and floor elevation of the entrance is 3 m, 3.3 m and 992 m, respectively. The entrance has a sluice gate and a screen in front of the gate.

Two sluice gates are constructed perpendicularly to the channel wall. Both width and height of the gates are 1.0 m.

(2) Head tank (serving both as desilting basin)

The head tank is installed in parallel with the river, in an almost rectangular shape, and its average width, length, and depth are 20 m, 20 m and 4.0 m, respectively. Five sand flush gates (ϕ0.3~0.45 m) are installed on the wall on the river side. The water flow in the head tank easily causes vortexes, which prevent the desilting process in the tank. The tunnel entrance to the penstock is provided at the corner on the right bank of the tank. The coarse-stone-concrete structure of the tank partially peels off, but is comparatively in good condition.

(3) Penstock

The penstock consists of a 135 m-long tunnel from the entrance and a steel conduit pipe (ϕ1.35 m and 1,300 m long). The tunnel contains the 80 m-long concrete-lined pressure tunnel on the upperstream side, and its cross section is 3 m in width and 2.0 m in height. The exposed part of the penstock is supported by a concrete saddle which exists on the bedrock. There is no marked leakage or deformation in the penstock, and the structure is comparatively in good condition.

(4) Powerhouse and tailrace

The powerhouse building size is 13.3 m x 24.2 m and its floor elevation is 901 m. There are two tailraces and their cross sections are 2.3 m (width) x 0.8 m (height), and 2.4 m x 1.1 m. Their length are 33 m and 30 m each. The tailrace on the right bank was constructed under the concrete-lining method, the one on the left bank was constructed only by excavating the ground.

Both machine foundation and tailraces are comparatively in good condition. The building is also comparatively in good condition even though about 50 years have elapsed since the construction of the building.

CHAPTER 5 BASIC DATA COLLECTION

The pre-FS was conducted from November, 1987 to July, 1988. The FS followed in November, 1988 to collect topographical, geological, hydrometeorological and other related data as detailed below:

5.1 Topographic Maps

Caracoli Hydroelectric P/P is located about 90 km east of Medellin City.

JICA Study Team collected the following topographic data.

- Topographic maps (scale: 1/25,000 - 1/500,000) published by IGAC
- Topographic survey maps and as-built drawings that were actually measured by EADE for the study of this power plant

(1) Topographic maps published by IGAC

Scale	Drawing No.	Description
1/500,000	-	the whole area of Antioquia Department
1/100,000	132	the upstream area of Rio Nus
"	133	the downstream area of Rio Nus
1/ 25,000	132-III-A,B,C,D	the upstream area of Rio Nus
"	IV-A,C	the power plant site and vicinity

(2) Topographic maps actually measured by EADE

Scale	Drawing No.	Description
1/200	1 de 2	Plan of the intake area
"	2 de 2	Plan of the P/P area

(3) As-built drawings

Scale	Drawing No.	Description
1/200 or 1/50	PLANO No.1 Aug./64	Plan and detailed cross section of intake facilities
1/1,000	PLANO No.2 Jan./64	Plan of the P/P
1/50	PLANO No.3 Jan./64	Plan of the powerhouse building
1/20	1004	Detail of the P/P foundation

5.2 Geological Survey Data

Geologic survey data collected for this survey is as follows:

- Aerial photographs of the power plant and its environs

- Perforaciones Planta Electrica Municipio de Caracoli 1989, Tecnisuelos

- Mapa Geologico de Colombia 1988, Ingeominas

5.3 Hydrometeorological Data

Since Caracoli P/P does not have facilities for monitoring precipitation levels, JICA Study Team gathered EPM's (Empress Publicas de Medellin) hydro-meteorological data in conducting this survey.

Discharge on the Nus River which is directly related to this FS, was monitored at HIMAT's Caramanta hydrological gauging station, which is located 5 km upstream from the intake portion. The precipitation levels and discharge data that were collected are listed below:

Table 5.1 List of Data Collected Relating to Hydrometeorology

(1) Precipitation-observation record

Meteorological station No.	Meteorological station Name	Controller	Location Latitude	Location Longitude	Altitude (m)	Observation period
2308050	San Roque	EPM	0630	7501	1,450	1969-1987
2308033	Santo Domingo	"	0628	7510	1,940	1959-1987
2308701	Bella Lina	"	0633	7506	1,460	1987
2308077	Guacharacas	"	0633	7457	830	1987
2308508	San Jose Del Nus	"	0629	7450	835	1972-1987
2308058	Caracoli	"	0625	7446	600	1969-1987
2310009	La Ceiba	"	-	-	1,260	1985-1987

(2) Discharge-observation record

Hydrological gauging station No.	Hydrological gauging station Name	River	Controller	Establish-ment	Location Latitude	Location Longitude	Altitude (m)	Catchment area (km^2)	Observa-tion period
2308716	Caramanta	Nus	HIMAT	1973-07	0628	7443	820	320	1975-85

(3) Water quality data

The observation of water quality was not recorded.

(4) Sediment data

JICA Study Team collected records relating to Caramanta gauging station in Nus River for seven years from 1981 to 1987.

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 Antioquia 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 EADE (refer to Table 5.2).

5.4.2 Power Condition Data

(1) JICA Study Team collected the following data for the purpose of examining EADE's power condition.

1) Record of generated energy and electricity buying at EADE for ten years from 1978 to 1987

2) EADE's schematic power diagram

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

1) Single line diagram

2) Generating cost

3) Operation and maintenance personnel

4) Residual value

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

CHAPTER 6 PRESENT CONDITIONS OF TOPOGRAPHY AND GEOLOGY

6.1 Topography and Geology in the Area

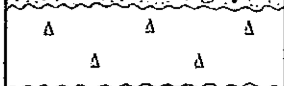
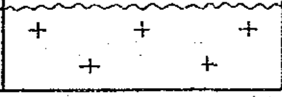
6.1.1 Topography

The fountainhead of the Nus River is at the eastern slope of the Central Cordillera which is situated 50 km northeast of Medellin, the capital of Antioquia Department. The Nus River flows east-southeast down and joins the Magdalena River near Pto Nare. The project site is situated in the mountainous area of its midstream side, and the geomorphology in the vicinity of the project site is formed of mountains in the state of medium relief.

6.1.2 Geology

The bedrock consists of cretacious granodiorite on which terrace deposit, talus deposit and debris flow deposit (riverbed deposit) overlay locally. 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	Schematic column	Lithology	Remarks
Quartanary		Riverbed dep. Debris flow dep.	
		Talus dep.	
		Terrace dep.	
Cretaceous		Granodiorite	

6.1.3 Geological Structure

Particularly large faults are not seen in the vicinity of the project site.

6.2 Geology in the Project Site

The geological condition of the foundation for the various civil structures is described below:

(1) Diversion weir and head tank

The bedrock around the head tank consists of granodiorite, and about 3-meter thick terrace deposit overlays on which further talus deposit overlays. It is presumed that a number of fresh outcrops are seen in the riverbed level, and the diversion weir and the tank lie directly on the rocks. Granodiorite above elevation higher than the surface of the terrace is considerably weathered and decomposed. The riverbed deposit in the downstream side of the diversion weir is assumed to be 2-meter thick or less.

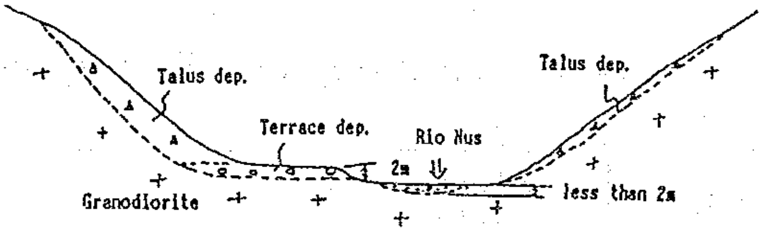


Fig. 6.1 Schematic Geological Profile

(2) Penstock

Debris flow deposit partly covers the weathered granodiorite in the Penstock portion. Debris flow deposit is generally thin, and the foundation structures for the Penstock lies directly on the bedrock.

(3) Power plant

Debris flow deposit overlays on the weathered granodiorite in the vicinity of the power plant. The bedrock is exposed at the rear and east sides of the powerhouse buildings, and the buildings are almost built on the bedrock. The mountainous side of the power plant is cut, and the weakly weathered rocks overlay on the slope and are stable.

6.3 Distribution of Concrete Aggregates

The aggregate may be produced by using gravel and debris flow deposit available in the riverbed.

6.4 Geotechnological Evaluation

(1) The overburden in the project site is generally thin, and bedrock overlays in the shallow depth. Fresh granodiorite is extremely hard and dense, and it has bearing capacity and impermeability sufficient as foundations for about 10-meter high concrete dam and for various structures.

(2) Various structures in the project site are constructed on the bedrock and there is no geological problem in such structures. However, the several places of passages along the Penstock is eroded by rainwater and a proper measure to protect foundation structures for Penstock needs to be taken for the future.

(3) There are no traces that large-scale landslides occurred in the vicinity of the project site.

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

Antioquia Department, in the northwest of Colombia, lies at 5°30' to 8°52' north latitude, near the equator.

Generally, the lowland areas enjoy a tropical climate and have a hot and very humid rainy season. It changes to a temperate climate zone with increasing altitude.

The lowland areas have a temperature of 28°C, while the temperature in highland areas (with an elevation of 1,800 to 2,800 m) ranges from 12 to 18°C. Medellin, the second biggest city in Colombia, lying in the highland with an elevation of about 1,500 m, has a temperature of 20°C.

Annual precipitation in the lowland area between West and Central Andes Mountain Ranges reaches 3,000 mm, while about 2,000 mm in the eastern and western highland of West Andes Mountain Range.

The fountainhead of the Nus River is in Central Andes Mountain Range and the Nus River flows at the eastern slope east-southeast down. It is about 43.0 km long as far as it joins the Magdalena River.

The project site, with an elevation of about 700 m above sea level, is situated in the east of Medellin, and has tropical climate. Average temperature is around 24°C. Annual precipitation is about 2,000 mm, but it fluctuates from year to year. One year is divided into two seasons, rainy and dry seasons. (Refer to Fig. 7.2.)

Observation Item	Gauging Station No	Gauging Station Name	Latitude	Longitude
Discharge	2308-716	Caramanta	0628	7443
Preciptation	2308-050	San Roque	0630	7501
	2308-033	Santo Domingo	0628	7510
	2308-701	Bella Lina	0633	7506
	2308-077	Guacharacas	0633	7457
	2308-508	San Jose Del Nus	0629	7450
	2308-053	Caracoli	0625	7446
	2310-009	La Ceiba	——	——

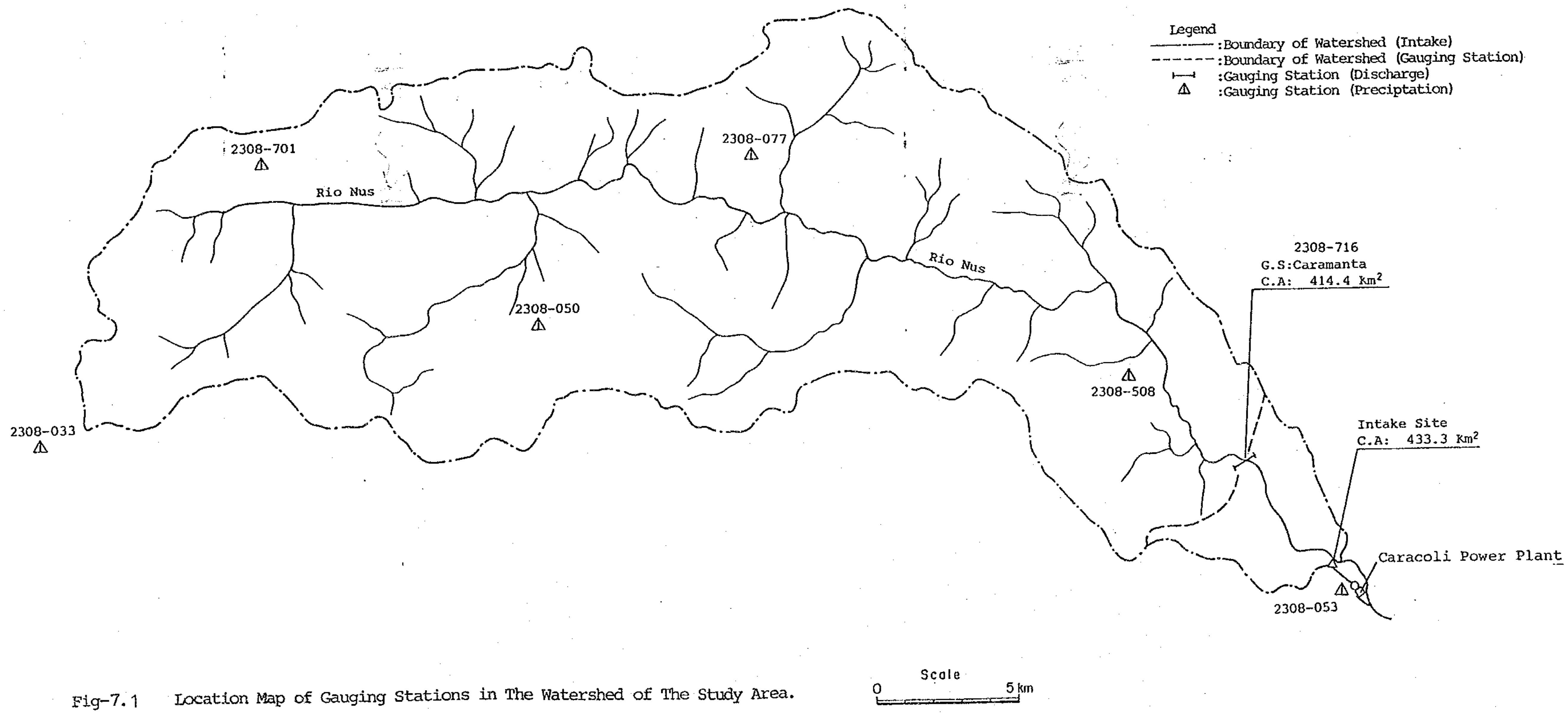


Fig-7.1 Location Map of Gauging Stations in The Watershed of The Study Area.

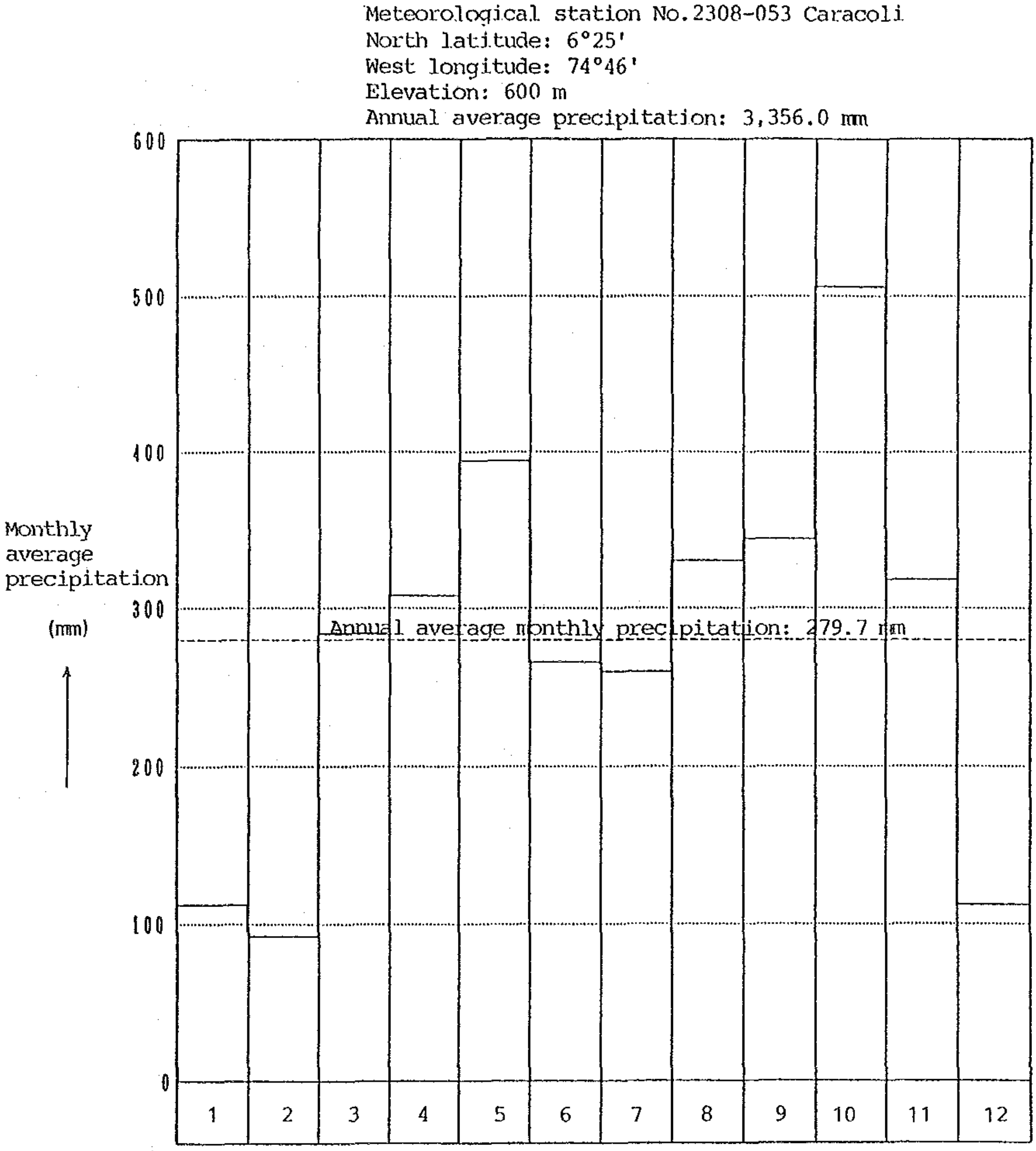


Fig.7.2 Monthly Average Precipitation in the Project Site (1969-70)

7.2 Discharge Analysis

The study team gathered, compiled observations recorded at the Caramanta gauging station during 11 years from 1975 to 1985, and then prepared discharge and flow-duration curves according to the above record by converting river basin. (Refer to Drawing No. CA-H-01.)

7.2.1 Comparing Discharge Observation Record

Though the Caramanta hydrological gauging station was established on July, 1973, the Study Team gathered the observation record for 11 years from 1975 to 1985. Non-observation dates are not included in the above record.

7.2.2 Collation to Catchment Area at the Gauging Station

To confirm the present location of the existing Caramanta 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 compared the catchment area at the gauging station using the topographic map (scale: 1/100,000) published by IGAC, and found there was a difference between the catchment area indicated on HIMAT's resister and the same measured by the study team, as shown in Table 7.1.

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

Item	Latitude	Catchment area (km^2)
HIMAT register	0628	320.0
Compared value	0627	414.4
Difference	0001	94.4

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 Caramanta Gauging Station

As shown in Table 7.2, discharge data at the Caramanta gauging station, located about 5 km upstream from the intake site of Caracoli hydroelectric power plant, are arranged using 11-year data without non-observing date.

In calculating monthly average discharge, the month in which the observing date is less than 10 days is excluded from the calculation. As can be seen from (1) of Drawing CA-H-01 representing graphically monthly average discharge, seven-month flow period (May to November) can be clearly distinguished from five-month drought period (December to April).

Typical flow-duration curves calculated from 11-year flow-duration curves from 1975 to 1985 according the parallel method are indicated in (3) of Drawing CA-H-01. As shown in Table 7.3, the plentiful, ordinary, low and droughty discharges in flow-duration curves are indicated in numerical values.

Table 7.4 shows the maximum discharge recorded at Caramanta gauging station for 11 years from 1975 to 1985.

7.2.5 Discharge and Flow-Duration Curves at the Intake Site

Discharge and flow-duration curves at the intake site of Caracoli P/P are calculated by multiplying respective catchment area ratio by record observed at the existing Caramanta gauging station located about 5 km upstream of the intake site.

Since numerical values of the catchment area at the intake site are not officially approved, the value measured by the study team is used. Therefore, a ratio of catchment area between Caracoli P/P's intake site and the Caramanta gauging station is set at 433.3/414.4 ≒ 1.04.

Discharge and flow-duration curves at the intake site converted according to the catchment area ratio are shown in Drawing CA-H-01, and representative values of monthly and daily average discharge and of plentiful, ordinary, low and droughty discharges are indicated in Table 7.5.

Table-7.2 MONTHLY FLOW TABLE OF DAILY AVERAGE FLOW AT G.S. SITE

GAUGING ST.: 2308716 CARAMANTA
RIVER NAME: NUS

(UNIT: M3/S)

GAUGING YEAR	TYPE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL TOTAL
1975	MAX.	30.4	20.6	41.9	37.4	65.4	97.7	63.1	55.1	72.2	68.3	51.2	81.2	97.7
	MEAN	16.9	15.2	20.2	21.9	28.6	27.7	31.4	23.1	26.1	29.6	24.1	25.9	24.2
	MIN.	14.5	13.3	12.9	13.2	16.4	16.6	15.9	14.3	16.7	18.1	17.4	15.5	12.9
1976	MAX.	29.9	20.5	37.5	53.2	50.2	52.1	20.7	52.5	45.0	47.7	42.3	20.4	53.2
	MEAN	14.8	12.8	18.0	20.7	17.4	15.9	10.2	14.5	16.1	23.6	18.9	11.2	16.2
	MIN.	12.5	10.9	11.5	11.4	11.2	11.0	8.2	8.3	8.4	12.5	11.7	9.0	8.2
1977	MAX.	9.0	9.5	16.1	34.7	58.0	22.4	63.0	48.6	39.7	45.8	17.3	20.9	63.0
	MEAN	8.2	7.7	8.0	9.8	18.0	13.6	19.1	22.0	15.9	21.5	12.6	11.2	14.0
	MIN.	7.6	7.0	6.8	6.9	7.9	9.8	10.1	11.4	10.3	12.4	11.3	9.8	6.8
1978	MAX.	10.0	13.5	15.1	27.2	75.5	66.8	67.8	32.7	43.9	74.0	62.3	50.5	75.5
	MEAN	7.7	7.1	7.2	13.6	23.2	19.5	22.7	11.9	19.5	17.4	16.3	12.2	14.9
	MIN.	6.9	5.5	5.5	5.9	9.2	9.0	11.4	6.7	9.9	8.8	8.8	7.9	5.5
1979	MAX.	7.1	7.7	22.9	37.3	32.5	33.4	20.4	53.9	31.3	28.9	28.1	18.3	53.9
	MEAN	6.3	5.4	5.6	11.8	13.2	13.0	9.1	16.4	14.2	13.6	14.1	10.4	11.2
	MIN.	5.5	4.4	4.0	3.6	6.2	6.4	3.5	5.7	5.8	5.8	5.8	6.0	3.5
1980	MAX.	14.2	8.6	40.2	17.3	42.5	36.7	33.9	51.3	42.1	102.4	33.4	26.8	102.4
	MEAN	8.6	6.5	8.4	6.5	12.7	12.1	10.4	12.0	12.6	23.8	15.5	9.5	11.6
	MIN.	7.0	5.5	3.2	4.2	4.9	6.3	5.4	3.3	7.0	11.1	8.8	7.1	3.2
1981	MAX.	9.1	10.3	11.3	40.7	77.8	83.5	52.9	21.9	41.4	43.3	18.3	24.7	83.5
	MEAN	7.0	6.5	6.3	15.9	25.1	26.0	17.5	11.4	13.7	18.9	10.0	9.8	14.0
	MIN.	6.3	5.5	5.0	5.4	9.4	11.5	9.3	8.5	8.7	9.1	7.9	7.4	5.0
1982	MAX.	8.8	35.4	24.8	93.0	86.1	70.1	34.3	62.1	85.3	91.5	95.3	10.7	95.3
	MEAN	6.7	11.9	8.9	18.6	22.5	13.4	10.7	18.4	29.5	20.5	18.9	8.7	15.7
	MIN.	5.2	5.9	5.0	7.8	9.7	7.5	6.8	7.1	7.5	11.9	10.0	7.1	5.0
1983	MAX.	18.7	7.8	9.0	37.4	53.2	22.9	32.5	24.5	60.0	34.8	40.9	15.3	60.0
	MEAN	7.8	6.5	5.7	12.2	16.8	8.5	10.3	10.1	18.6	12.6	11.8	9.7	10.9
	MIN.	6.5	5.9	4.7	4.5	6.4	5.9	5.9	5.8	8.2	7.5	7.3	7.3	4.5
1984	MAX.	23.1	37.4	18.8	36.7	40.0	34.5	28.2	37.5	65.0	61.2	35.2	21.1	65.0
	MEAN	11.1	13.0	9.4	11.3	19.1	17.6	17.0	14.5	25.2	21.3	20.8	16.0	16.3
	MIN.	9.0	9.0	8.0	8.0	8.3	9.7	9.7	9.7	11.5	10.4	10.0	12.8	8.0
1985	MAX.	16.0	14.9	27.5	56.2	46.0	30.0	45.4	59.5	77.5	51.8	32.9	29.3	77.5
	MEAN	11.9	10.7	13.5	16.7	20.4	11.1	10.3	19.1	23.8	22.0	16.8	12.5	15.7
	MIN.	10.4	9.4	8.3	8.3	11.1	6.4	4.4	8.3	11.5	14.2	12.2	9.0	4.4
TOTAL	MAX.	30.4	37.4	41.9	93.0	86.1	97.7	67.8	62.1	85.3	102.4	95.3	81.2	102.4
	MEAN	9.7	9.4	10.2	14.4	19.7	16.2	15.3	15.8	19.6	20.4	16.3	12.5	15.0
	MIN.	5.2	4.4	3.2	3.6	4.9	5.9	3.5	3.3	5.8	5.8	5.8	6.0	3.2

Table-7.3 FLOW DURATION TABLE AT GAUGING STATION SITE

GAUGING ST. : 2308-716 CARAMANTA
RIVER NAME: NUS

(UNIT: M3/S)

GAUGING YEAR	MAX.	MEAN	MIN.	PLENTY (95 DAYS)	ORDINARY (185 DAYS)	LOW (275 DAYS)	DROUGHTY (355 DAYS)
1975	97.7	24.3	12.9	27.2	19.4	16.4	13.4
1976	53.2	16.2	8.2	17.3	13.0	11.3	8.7
1977	63.0	14.0	6.8	14.7	11.4	8.3	7.1
1978	75.5	14.9	5.5	18.0	11.7	8.1	5.8
1979	53.9	11.2	3.5	13.3	9.3	6.3	4.4
1980	102.4	11.6	3.2	12.4	8.3	6.9	4.6
1981	83.5	14.0	5.0	15.0	10.1	7.7	5.5
1982	95.3	15.7	5.0	15.7	10.7	7.6	5.5
1983	60.0	10.9	4.5	11.7	8.3	6.6	5.0
1984	65.0	16.3	8.0	18.7	14.2	10.4	8.0
1985	77.5	15.8	4.4	16.9	12.8	10.4	5.2
MEAN	75.2	15.0	6.1	16.5	11.8	9.1	6.7

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

GAUGING ST.: 2308716 CARAMANTA
RIVER NAME: NUS

(UNIT: M3/S)

GAUGING YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL TOTAL
1975	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	
1976	56.0	63.6	176.6	126.0	104.9	170.3	41.8	157.0	114.7	136.0	154.0	44.9	176.6
1977	9.3	12.5	38.8	126.9	94.5	31.8	99.0	73.0	66.2	64.5	19.5	27.6	126.9
1978	12.6	20.3	21.6	40.5	128.5	106.7	107.6	72.4	128.9	120.0	102.5	53.5	128.9
1979	7.5	10.9	73.2	67.7	108.4	91.1	39.2	165.5	78.8	29.8	35.9	43.3	165.5
1980	22.5	15.2	58.0	59.5	138.6	97.7	94.5	151.7	175.9	231.8	106.7	74.0	231.8
1981	11.9	14.0	20.7	115.5	221.0	174.9	86.0	31.6	62.5	120.0	58.7	39.2	221.0
1982	10.2	40.5	36.6	162.5	133.0	120.0	53.5	111.0	144.2	125.1	171.7	16.0	171.7
1983	52.7	9.3	20.2	130.3	171.7	34.0	120.0	63.2	189.2	114.6	109.3	21.6	189.2
1984	32.0	57.1	26.1	48.0	54.5	61.0	31.0	54.5	88.0	68.0	54.5	24.0	88.0
1985	16.9	16.0	37.2	71.6	71.6	39.6	71.6	88.0	124.0	113.2	54.5	39.0	124.0
TOTAL	56.0	63.6	176.6	162.5	221.0	174.9	120.0	165.5	189.2	231.8	171.7	74.0	231.8

NOTE) (1) DATA MISSING

Table 7.5 Representative Discharge at the Intake Site

1) Monthly average discharge

Item	Month												
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
Max. average discharge (m^3/s)	17.7	15.8	21.0	22.8	29.7	28.8	32.6	24.0	30.6	30.7	25.0	26.9	25.1
Daily average discharge (m^3/s)	10.1	9.8	10.6	15.0	20.5	16.8	15.9	16.4	20.4	21.2	16.9	13.0	15.6
Min. average discharge (m^3/s)	6.5	5.6	5.9	6..7	13.2	8.8	9.5	10.5	13.1	13.1	10.4	9.0	11.3

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
17.2 m^3/s	12.3 m^3/s	9.5 m^3/s	7.0 m^3/s

River utilization factor of a certain available discharge to typical flow-duration curves at the intake site (a ratio of total available discharge and total river discharge flowing into the intake site) and facility utilization factor (a ratio of total discharge for which water can be taken in to the available discharge throughout the year and total water amount, in the event that available discharge is secured throughout the year) are represented graphically in (5) of Drawing CA-H-01.

7.3 Flood Runoff Analysis

The flood discharge is important to maintain the safety of existing fascilities and the repaired sections. The design flood discharge is obtained by that the observation record of the discharge at Caramanta 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 7.6 is summarized according to the discharge data.

Table 7.6 Annual Flood Discharge

Year Observed	Maximum Discharge (m^3/sec)
1976	176.6
1977	126.9
1978	128.9
1979	165.5
1980	231.8
1981	221.0
1982	171.7
1983	189.2
1984	88.0
1985	124.0

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

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

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

Fig. 7.3 and 7.4 show that maximum yearly discharge is plotted on X - axis of abscissa and that percentage of excess probability calculated is plotted on Y - axis of ordinate by using the extreme probability paper. Table 7.7 shows the probable flood discharge for major years of return period from the probability curve shown in the figure.

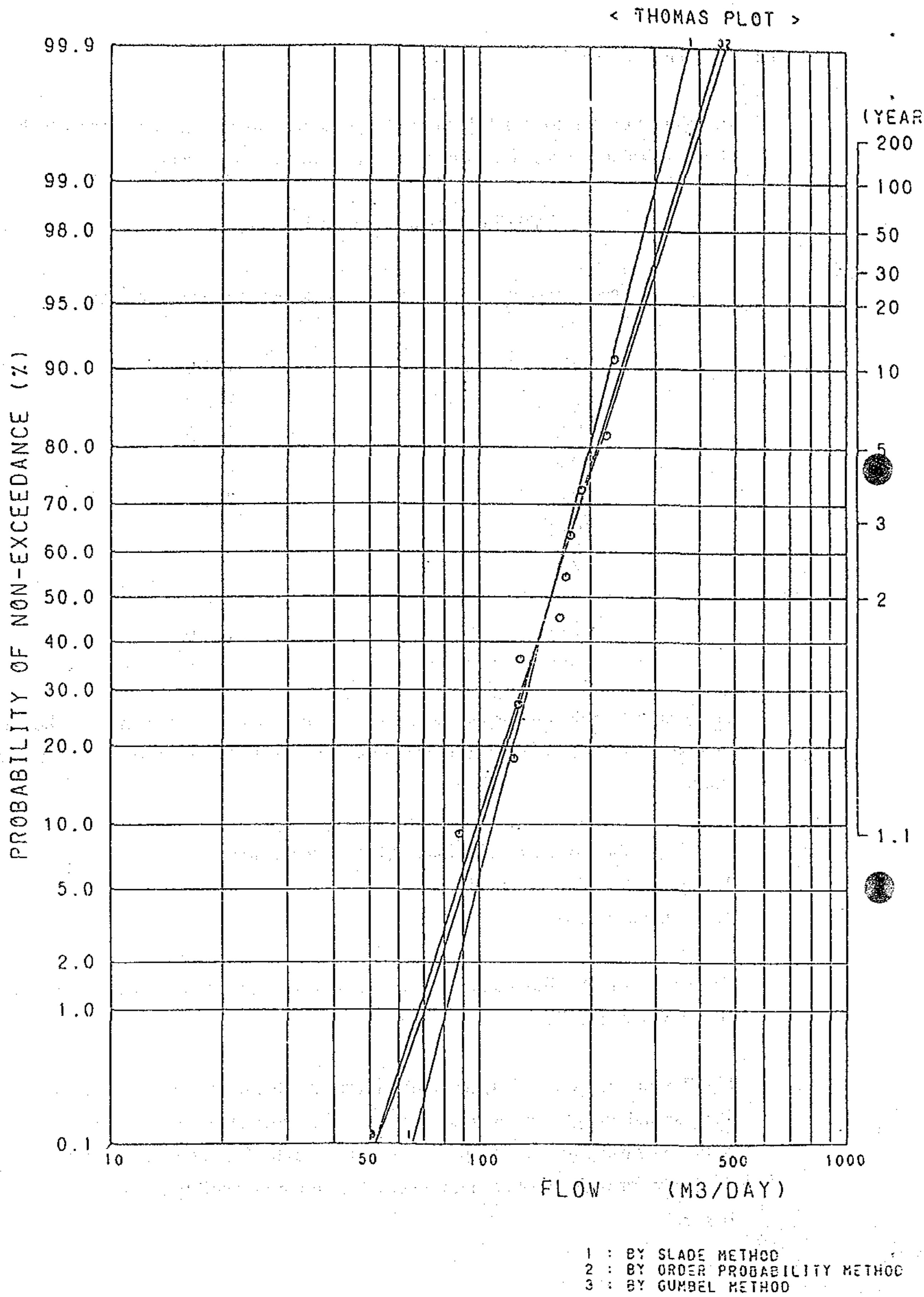


Fig. 7.3 Probability Curve of Rio Nus at Caramanta (Thomas Plot)

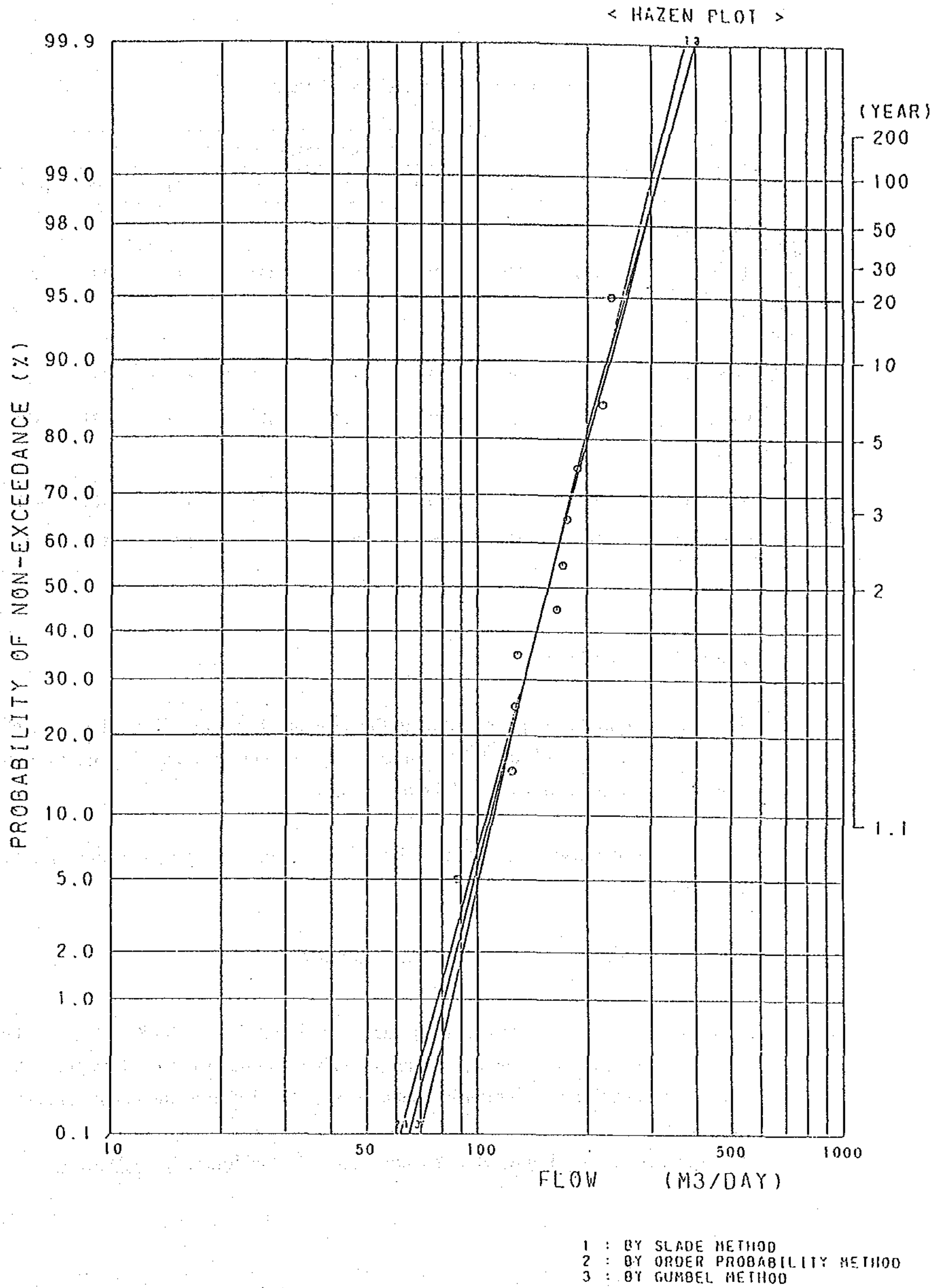


Fig. 7.4 Probability Curve of Rio Nus at Caramanta (Hazen Plot)

Table 7.7 Potential Flood Discharge

Method	Return Period in Years							
	5	10	20	50	100	200	500	1000
Logarithm normal distribution method (m^3/s)	189	225	249	280	302	324	353	375
Order probability method:								
Thomas plot (m^3/s)	211	248	282	327	361	394	439	474
Hazen plot (m^3/s)	202	230	257	291	316	340	373	397
Gumbel method:								
Thomas plot (m^3/s)	208	242	275	317	349	381	423	454
Hazen plot (m^3/s)	198	226	253	288	314	340	374	400

7.3.2 Design Flood Discharge

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

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

$$Q = 361 \times \frac{433.3}{414.4} = 377.5 \ldots\ldots 380 \text{ m}^3/\text{s}$$

The specific discharge per catchment area (km^2) will be $q = 0.88$ 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 $C = 8.0$.

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

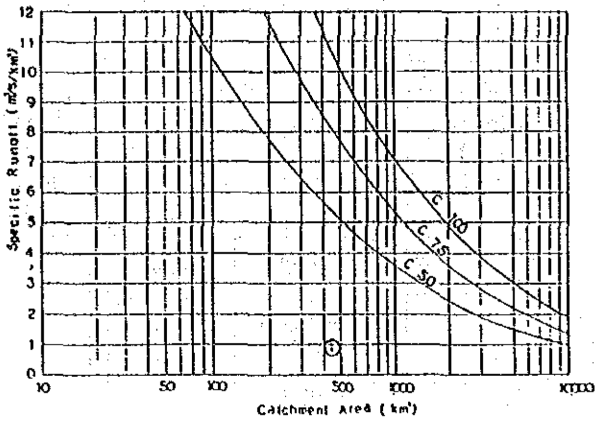


Fig. 7.5 Design Flood Discharge and Creager Curve

7.4 Sediment Analysis

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

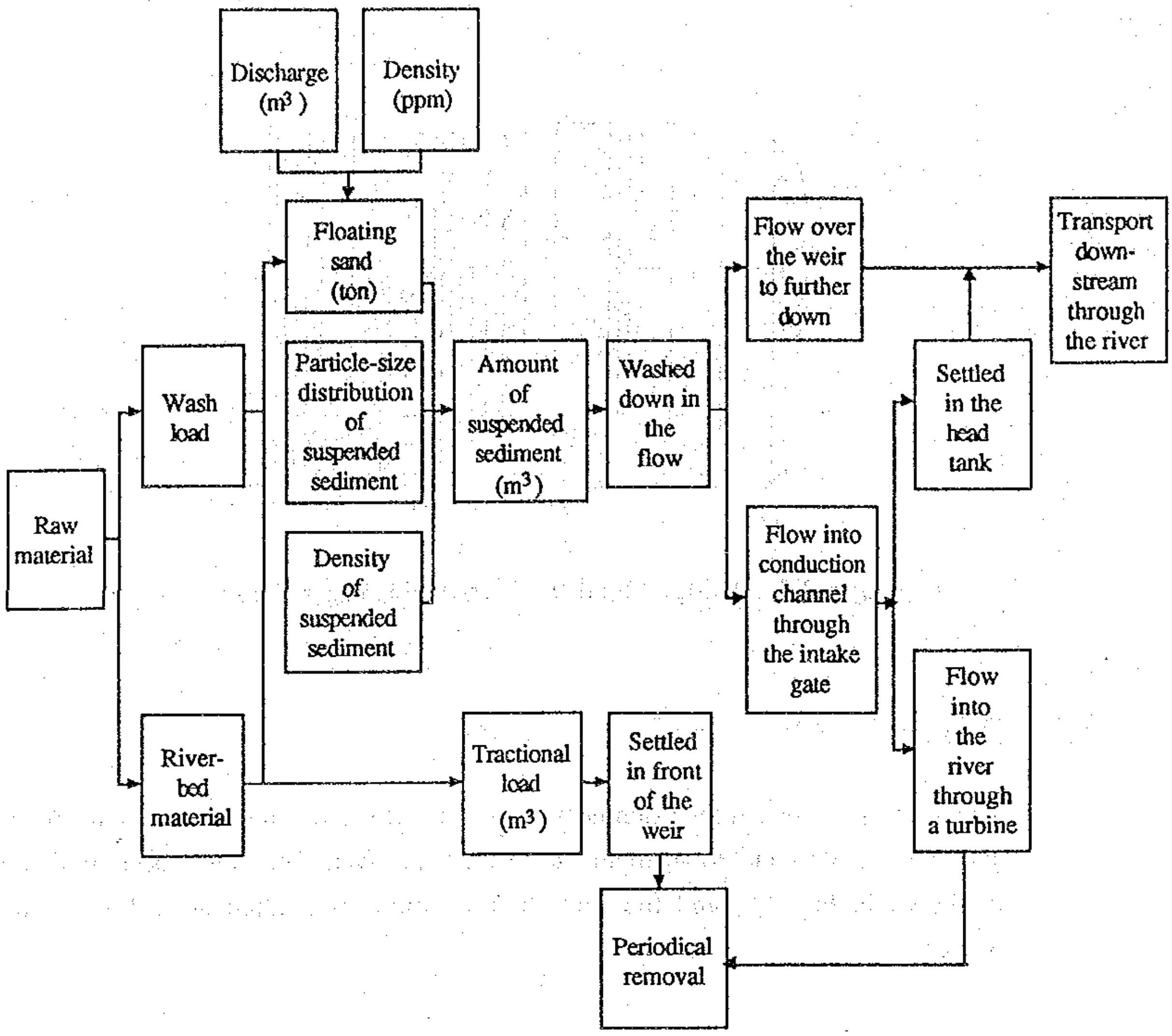


Fig. 7.6 Mechanism of Debris Flow and Calculation Flow of Debris Volume

7.4.1 Debris Flow Status

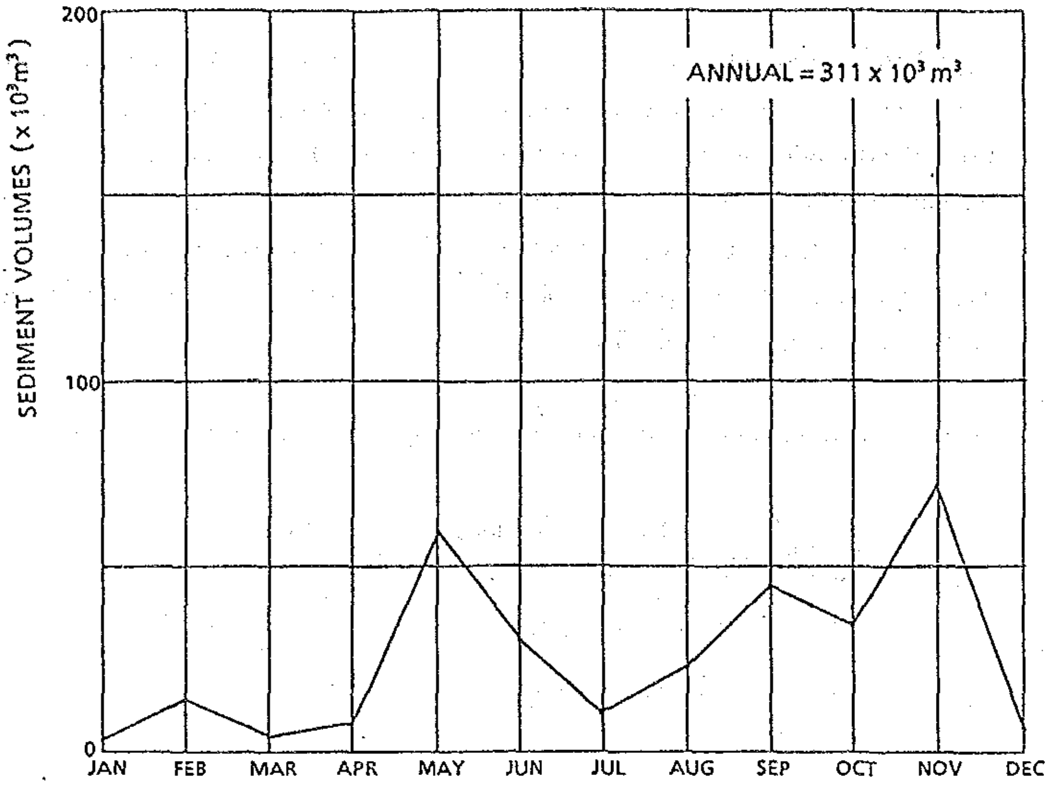
The catchment at the Nus River is formed by comparatively steep ravine. The vegetation of the catchment is good. The debris flowing from these catchments is mainly gully erosion by terrace collapse, etc.

Fig. 7.7 shows monthly sediment concentration and volume. As the figure shows, the sediment volume increases on April and September to November, and decreases during the droughty season from December to April. The transition of the sediment concentration does not show any marked characteristic, but it relatively decreases during the droughty season (on December to January and April) and on July. The suspended sediment curve prepared according to the measured values (concentration and volume) is shown in Fig. 7.8.

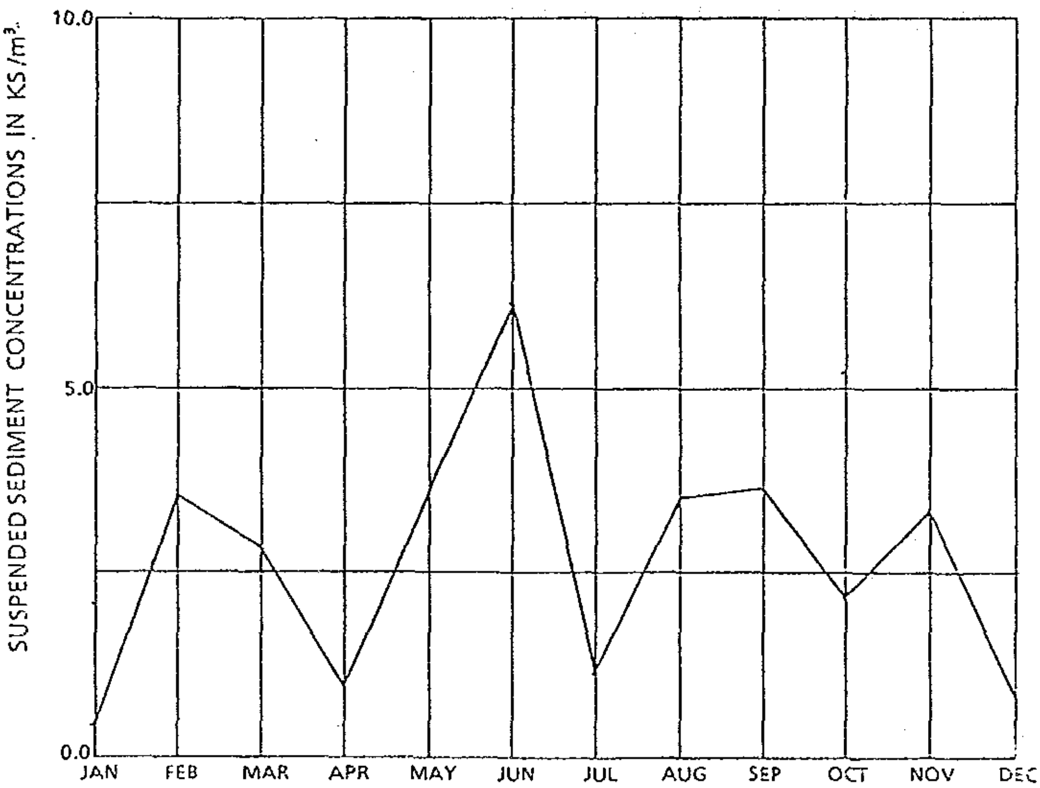
The suspended sediment (ton/year) at the gauging station spot is shown below.

River	Catchment Area (km^2)	River Discharge Rate			Concentration		Suspended Sediment Rate
		Total 103 m^3/year	Max. (m^3/s)	Min. (m^3/s)	Max. (ppm)	Min. (ppm)	10^3 tons/year
Nus	414.4	473,000	231.8	3.2	6,410	10	314

The suspended sediment flowing into the gauging station on the Nus River reaches 790 tons/km^2 per year per catchment area, and annual average suspended sediment concentration of Nus River is 700 ppm.



SEDIMENT VOLUMES



SEDIMENT CONCENTRATIONS

Fig 7.7 Sediment Concentrations and Volumes

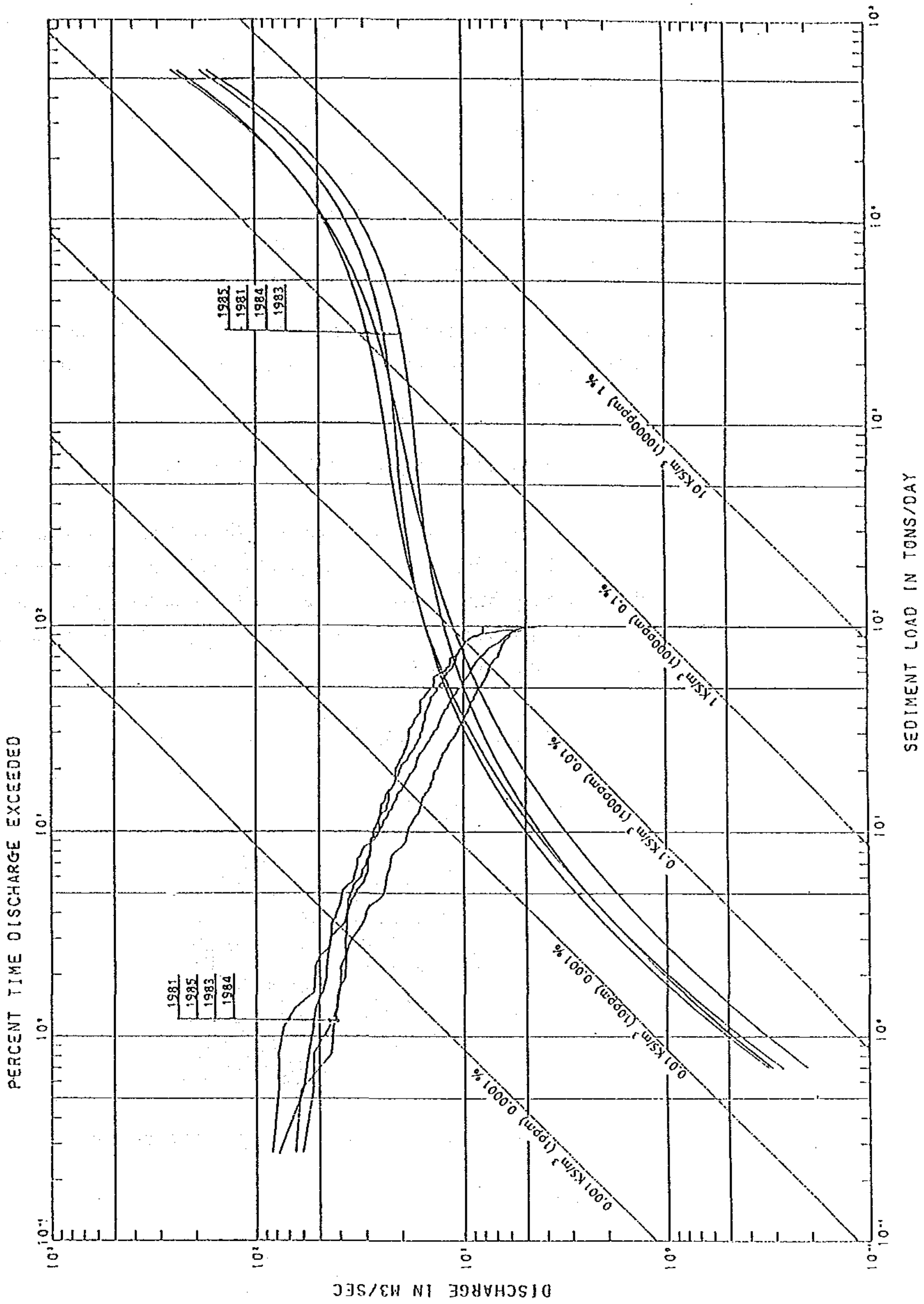


Fig. 7.8 Sediment Rating Curve

7.4.2 Assumption of Sediment Rate

(1) Major physical properties

(a) Grain size distribution

JICA study team was not able to obtain data for the gain size distribution of bed-load. Based on the related data, the grain size constitution is assumed as follows:

Gravel = 60% Sand = 30% Silt = 10%

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

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

* Handbook of Applied Hydrology

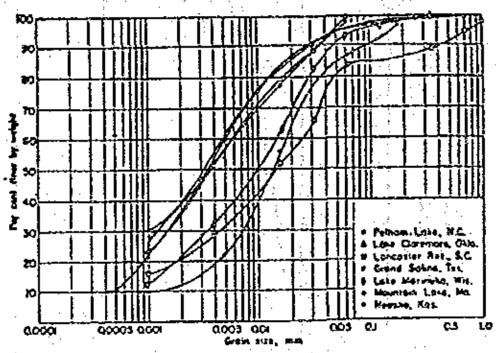


Fig. 7.9 Grain Size Constitution of Suspended Sediment *

(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. From the past case example, the unit volume weight ranges in value according to the grain size constitution of sediment at reservoir and the active conditions (under or above water) of the load at that time, as shown in Table 7.8.

Table 7.8 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)
433.3	494	314	16	330

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(%)	60	30	10	100
Unit volume weight (ton/m^3)	1.68	1.48	1.04	
Unit weight per grain size (ton/m^3)	1.008	0.444	0.104	1.556... 1.56

	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 head tank from the intake. Partial rough particles in the suspended sediment flown into the head tank are settled here, and the remaining suspended sediment is discharged into the river through penstock to water wheel together with discharge. The river discharge more than design discharge flows down the river by overflowing the weir together with the suspended sediment contained in this discharge.

Catchment area (km^2)	River discharge (10^6 (m^3)	Bed-load 10^3 (ton)	Suspended sediment 10^3 (ton)
433.3	494	16	314
		x 10^3 m^3	x 10^3 m^3
		10	311

Design discharge Q m^3/s	Sediment in head tank x 10^3 m^3	Flow down the river x 10^3 m^3
5	78	233
10	155	156
15	217	94

Design discharge Q m^3/s	Settled in the head tank 10^3 (m^3)	Flow down the channel 10^3 (m^3)
5	2	76
10	3	152
15	4	213

It is assumed from the results of the above analysis that annual average sediment in front of the diversion weir will be about 27 m^3/day and sediment settled in the head tank will be about 8 m^3/day (if available discharge is 10 m^3/s). A counterplan for removing these sediment shall be fully considered.

7.5 Water Quality Analysis

The study team could not obtained the result of water quality test. The water quality is considered to have no problem as far as the study team visually inspected the environment and water in the area.

CHAPTER 8 GENERATION PLAN

The generation plan is made based on the planned maximum available discharge at Caracoli P/P of 5.00 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 Caracoli P/P are rehabilitated, the headrace structure and its appurtenant facilities, except for the 1,300-meter-long headrace and the powerhouse building, need to be repaired or replaced. In addition, the Pelton turbine needs 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

As stated above, the planned maximum available discharge at Caracoli P/P is 5.00 m^3/s. As shown in Fig. 8.1, the annual 95%-guaranteed discharge is 7.4 m^3/s.

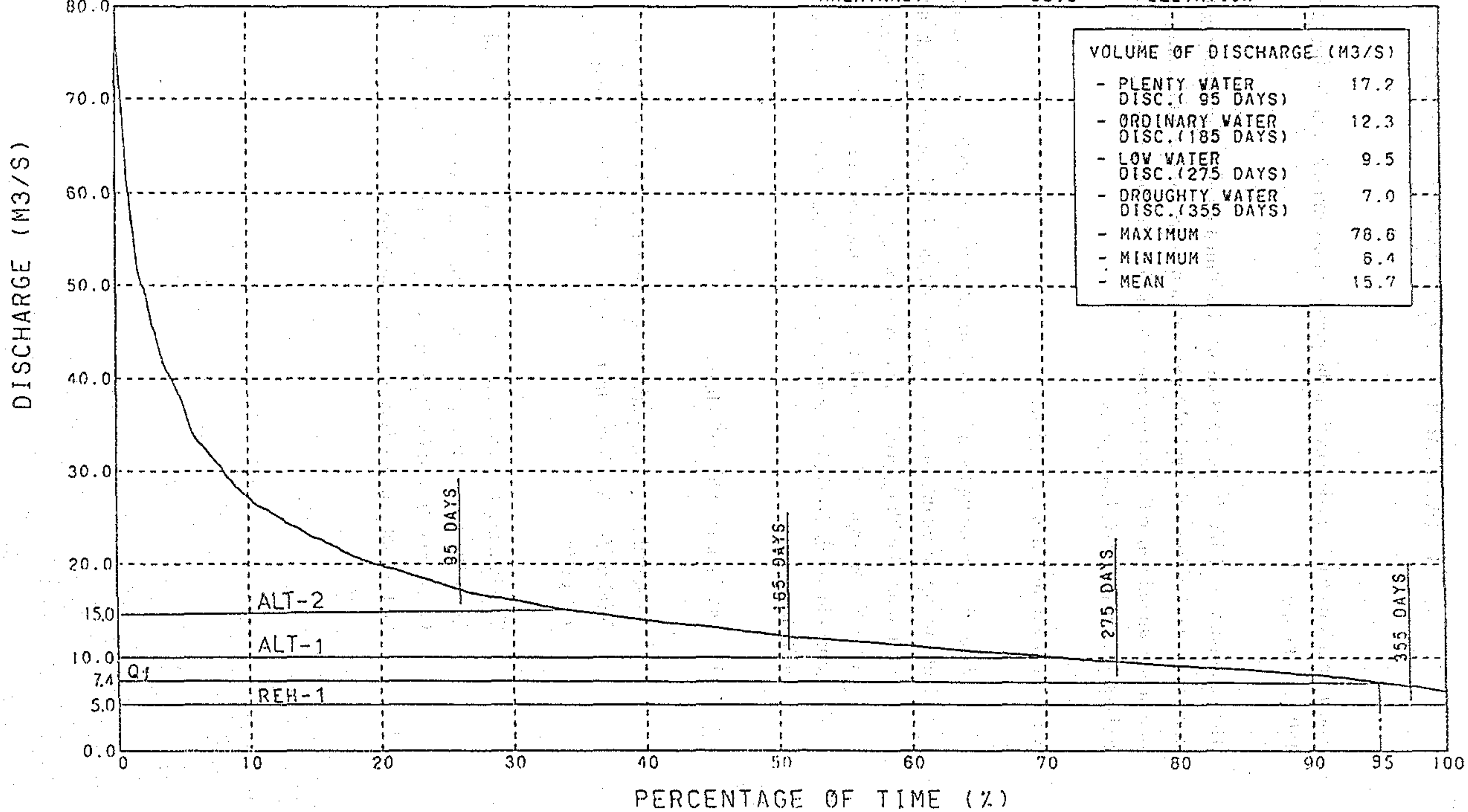
To compare the maximum available discharge, three rehabilitation plans of Q=5.00 m^3/s (maximum available discharge at the existing P/P), Q=10.00 m^3/s and Q=15.00 m^3/s were set up, and respective generated output and annual generated energy were calculated.

Fig-8.1 TYPICAL FLOW DURATION CURVE AT INTAKE SITE

P.S.: CARACOLI

RIVER NAME: NUS
CATCHMENT AREA(KM2): 433.3

LATITUDE: ---
LONGITUDE: ---
ELEVATION: ---



(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. The net head He is obtained by the formular below, which includes the loss of head between the head tank and the conduction channel.

$$He = Hg - \Delta H$$

$$\Delta H = V^2/2g\,(1+f_1 + f_2\,L/D + fm) + h = V^2/2g\,(1.85 + f_2\,L/D) + h$$

where:

ΔHg = gross head
Head tank water level (998 m) - discharge level (899.3 m) = 98.7 m

ΔH = 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; $124.6\, n^2/D^{1/3}$

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 (existing), 0.0145 (new)

Table 8.1 Calculated Result of Standard Net Head

Q (m^3/s)	D (m)	I (m)	V (m/s)	$V^2/2g$ (m)	$f_r.l/D$	Σf (m)	$V^2/2g\Sigma f$ (m)	Δh (m)	He (m)
5	1.35	1300	3.49	0.621	22.83	24.68	15.38	0.42	82.90
10	1.65	1300	4.68	1.117	11.94	13.79	15.40	0.40	82.90
15	1.95	1300	5.03	1.288	9.55	11.40	14.68	0.42	83.10

Accordingly, the standard net head is calculated to be 82.9m.

8.2 Generated Output

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

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

where:

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

Resultant efficiency (η) is the value representing total efficiency, and this value is obtained by the following formula.

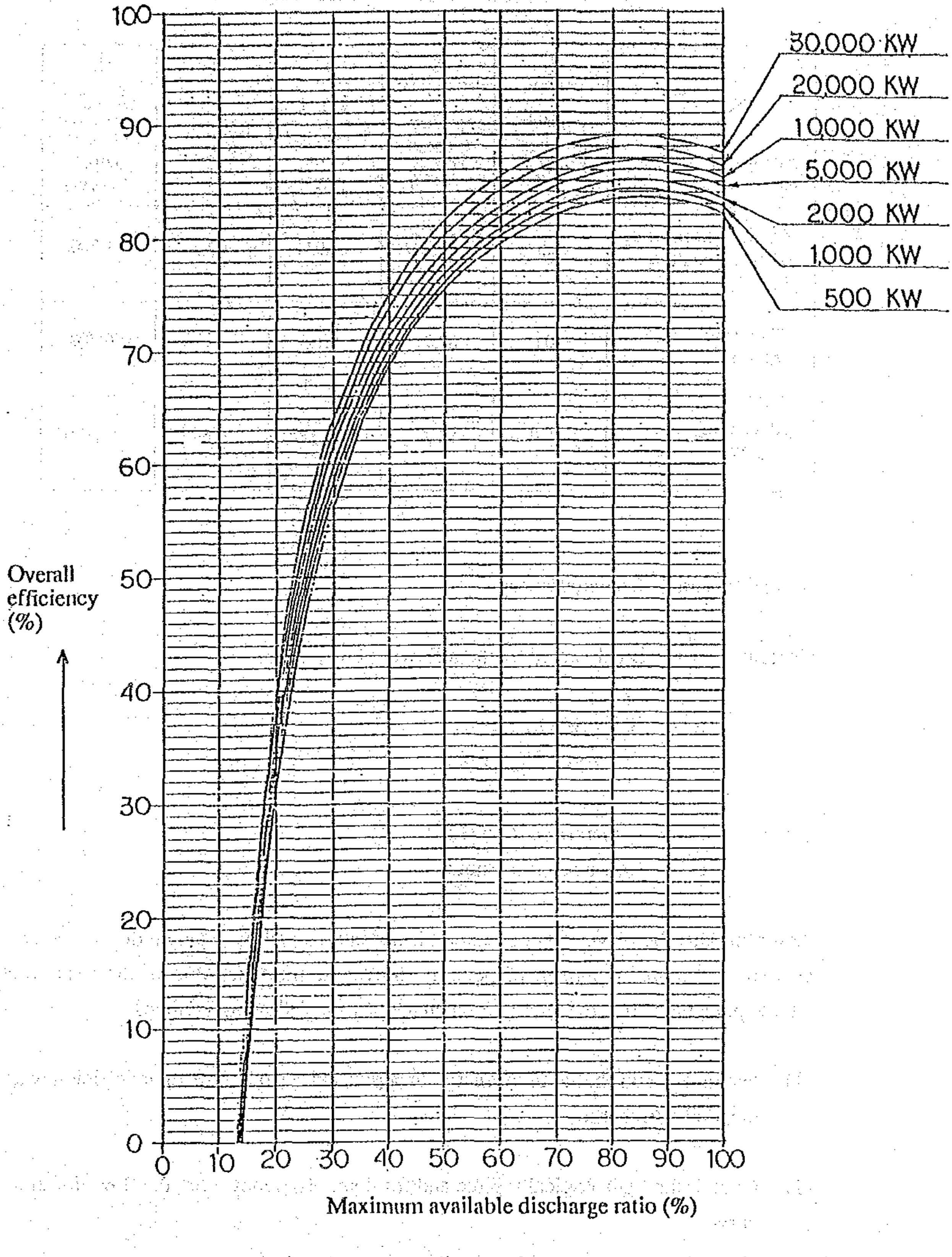
$$\eta = \eta t \times \eta g$$

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.

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_e(m)	③ 9.8 x ① x ② Theoretical output (kW)	④ Resultant efficiency η	⑤ ③ x ④ Generated output P (kW)
Plan for rehabilitating existing facilities (REH-1)	2.5	82.9	2,031	0.835	1,700
Alternative (ALT-1)	10.0	82.9	8,124	0.835	6,700
Alternative (ALT-2)	15.0	82.9	12,186	0.845	10,200

8.3 Annual Potential Generated Energy

Generated energy is calculated by the following formula.

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

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

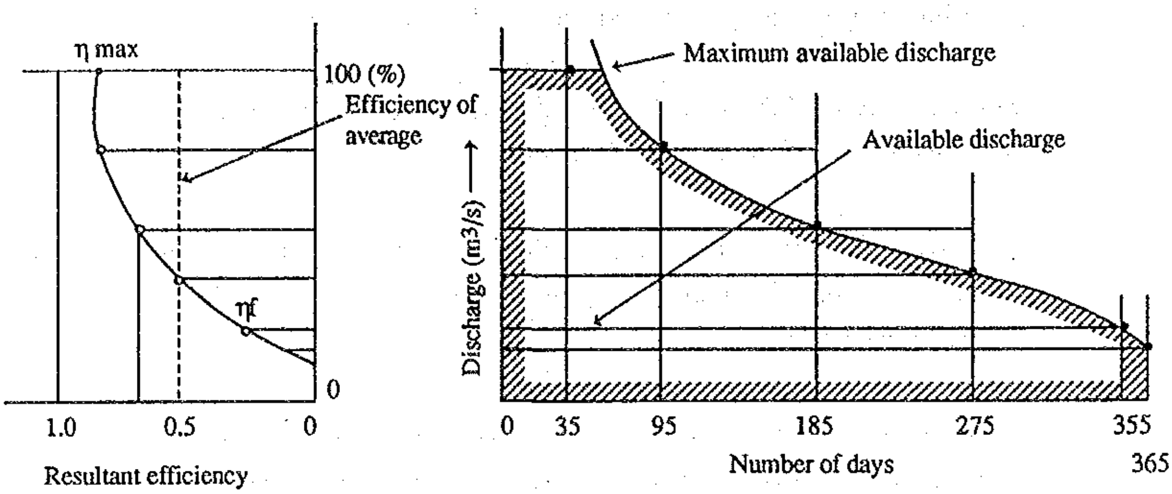
For the calculation of the annual potential generated energy at Caracoli P/P, item (2) as mentioned above is used for the following reasons.

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

② Since there are no recorded observations at the Caramanta 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, as shown below.



Max. available discharge = m³/s Net head , H_e = m

① Day	② Number of days	③ Available discharge (m³/s)	④ Burden ratio Available discharge / 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 $9.8 \times Q \times H_e \times \eta$
⑦ Mean value of generated output of calculation stage and right above stage.
⑧ ⑦ x ② x 24 is the generated energy for calculated days, and the total value becomes yearly possible generated energy.

Fig. 8.3 Procedure for calculating annual potential generated energy using the hydrological regime-efficiency method

8.3.1 Calculation of Annual Potential Generated Energy

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

(1) The annual potential generated energy in the case of the rehabilitation plan of the existing facilities (max. available discharge = 2.5 m^3/s x 1 unit):

14.9 GWh (100 %)

(2) The annual potential generated energy in the case of the alternative plan 1 (ALT1) (max. available discharge = 5.0 m^3/s x 2 units):

57.0 GWh (96 %)

(3) The annual potential generated energy in the case of the alternative plan 2 (ALT2) (max. available discharge = 7.5 m^3/s x 2 units):

72.3 GWh (80 %)

Table 8.3 Calculation of Annual Potential Generated Energy

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

Max. available discharge Q = 2.5 m^3/s x 1 unit
Standard net head H_e = 82.9 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.	365	2.5	1.0	0.835	1,700	1,700	14,892
Total	365					(1,700)	14,892

(2) Alternative plan 1 (ALT-1)

Max. available discharge: Q = 5.0 m^3/s x 2 units
Standard net head (He): 82.9m
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.	258	10.0	1.000	0.835	6,783	6,783	42,000
261	3	9.9	0.990	0.836	6,723	6,753	486
265	4	9.8	0.980	0.839	6,679	6,701	643
270	5	9.6	0.960	0.843	6,574	6,626	795
275	5	9.5	0.950	0.845	6,521	6,547	785
280	5	9.4	0.940	0.846	6,460	6,490	778
285	5	9.3	0.930	0.847	6,399	6,429	771
290	5	9.1	0.910	0.849	6,276	6,337	760
296	6	9.0	0.900	0.850	6,215	6,245	899
300	4	8.9	0.890	0.851	6,153	6,184	593
305	5	8.8	0.880	0.851	6,084	6,118	734
310	5	8.7	0.870	0.851	6,014	6,049	725
315	5	8.6	0.860	0.851	5,945	5,979	717
320	5	8.4	0.840	0.851	5,807	5,876	705
325	5	8.2	0.820	0.851	5,669	5,738	688
330	5	8.1	0.810	0.851	5,600	5,634	676
335	5	7.9	0.790	0.849	5,448	5,524	662
340	5	7.7	0.770	0.847	5,298	5,373	644
345	5	7.5	0.750	0.844	5,142	5,220	626
350	5	7.2	0.720	0.840	4,913	5,027	603
355	5	7.0	0.700	0.835	4,748	4,830	579
360	5	6.7	0.670	0.829	4,512	4,630	555
365	5	6.4	0.640	0.821	4,268	4,390	526
Total	365					(5,890)	56,950

(3) Alternative plan 2 (ALT-2)

Max. available discharge: Q = m³/s
Standard net head (H_e): 82.9m
Turbine type: Francis turbine

Day	Number of days	Available discharge (m^3/s)	Burden ratio Available discharge / Max. available discharge	Resultant efficiency η	Generating power (kW)	Average power (kW)	Generated energy (MWh)
Max.	125	15.0	1.000	0.845	10,297	10,297	30,891
130	5	14.8	0.986	0.849	10,208	10,252	1,230
135	5	14.5	0.966	0.851	10,024	10,116	1,213
140	5	14.3	0.953	0.854	9,921	9,972	1,196
145	5	14.0	0.933	0.856	9,736	9,828	1,179
150	5	13.8	0.920	0.857	9,608	9,672	1,160
155	5	13.6	0.906	0.858	9,479	9,543	1,145
160	5	13.5	0.900	0.859	9,421	9,450	1,134
165	5	13.2	0.880	0.860	9,222	9,321	1,118
170	5	13.0	0.866	0.861	9,093	9,157	1,098
175	5	12.7	0.846	0.861	8,883	8,988	1,078
180	5	12.5	0.833	0.861	8,743	8,813	1,057
185	5	12.3	0.820	0.861	8,603	8,673	1,040
190	5	12.1	0.806	0.861	8,463	8,533	1,023
195	5	12.0	0.800	0.860	8,384	8,423	1,010
200	5	11.8	0.786	0.859	8,234	8,309	997
205	5	11.7	0.780	0.857	8,146	8,190	982
210	5	11.5	0.766	0.856	7,997	8,071	968
215	5	11.4	0.760	0.855	7,918	7,957	954
220	5	11.2	0.746	0.853	7,761	7,839	940
225	5	11.0	0.733	0.852	7,614	7,687	922
230	5	10.8	0.720	0.851	7,466	7,540	904
235	5	10.6	0.706	0.847	7,294	7,380	885
241	6	10.5	0.700	0.846	7,216	7,255	1,044
245	4	10.4	0.693	0.844	7,173	7,123	688

(cont'd)

Day	Number of days	Available discharge (m^3/s)	Burden ratio Available discharge / Max. available discharge	Resultant efficiency η	Generating power (kW)	Average power (kW)	Generated energy (MWh)
250	5	10.3	0.686	0.842	7,045	7,088	850
255	5	10.1	0.673	0.841	6,900	6,972	836
260	5	10.0	0.666	0.837	6,799	6,849	821
265	5	9.8	0.653	0.835	6,648	6,723	806
270	5	9.6	0.640	0.832	6,488	6,568	788
275	5	9.5	0.633	0.830	6,405	6,446	773
280	5	9.4	0.626	0.826	6,307	6,356	762
285	5	9.3	0.620	0.825	6,233	6,270	752
290	5	9.1	0.606	0.822	6,077	6,155	738
296	6	9.0	0.600	0.818	5,981	6,029	868
300	4	8.9	0.593	0.816	5,900	5,940	570
305	5	8.8	0.583	0.814	5,819	5,859	703
310	5	8.7	0.580	0.812	5,739	5,779	693
315	5	8.6	0.570	0.808	5,645	5,692	683
320	5	8.4	0.560	0.805	5,493	5,569	668
325	5	8.2	0.546	0.798	5,316	5,404	648
330	5	8.1	0.540	0.796	5,238	5,277	633
335	5	7.9	0.526	0.791	5,076	5,157	618
340	5	7.7	0.513	0.785	4,910	4,993	599
345	5	7.5	0.500	0.777	4,734	4,822	940
350	5	7.2	0.480	0.767	4,486	4,610	922
355	5	7.0	0.466	0.755	4,293	4,389	904
360	5	6.7	0.444	0.746	4,060	4,176	885
365	5	6.4	0.426	0.730	3,795	3,927	1,044
Total	365	-	-	-	-	(7,254)	72,294