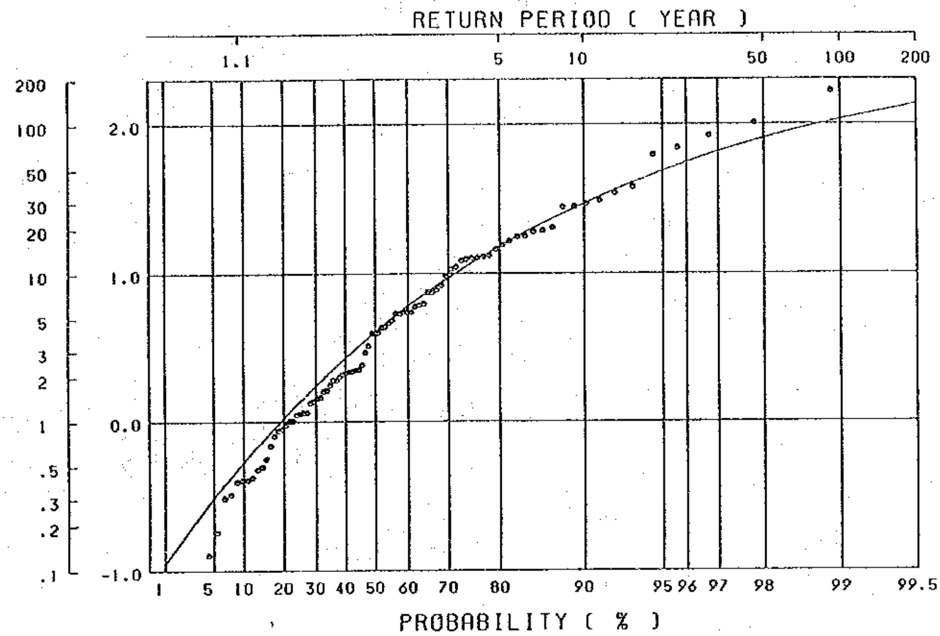
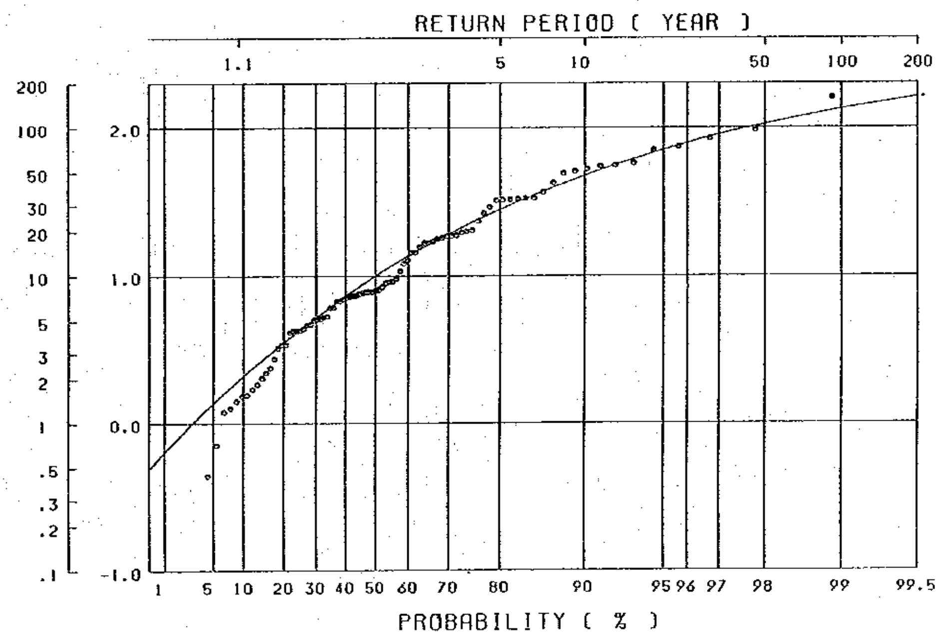


3: LOG A=2.041+0.347M-1.6LOG(R)

(L.ESTEVA & E.ROSENBLUETH)

Fig. 8-7 Return Period for Maximum Acceleration calculated by Eq. (3)



4: LOG A=2.308+0.411M-1.637LOG(R+30)

(T.KATAYAMA)

Fig. 8-8 Return Period for Maximum Acceleration calculated by Eq. (4)

8.2.3 Maximum Acceleration Estimated for Pirris Project Site

The results of stochastically estimating the maximum accelerations at the Pirris project site for return periods of 50, 100, 200, 500, 1,000, and 10,000 years applying the equations of Oliveira, McGuire, Esteva-Rosenblueth, and Katayama based on historical earthquakes are shown in Table 8-7.

The evaluation results based on McGuire's and Katayama's equations, indicate larger maximum accelerations compared with those based on Oliveira's and Esteva-Rosenblueth's equations. It is thought these differences resulted because the earthquake data which served as the basis from which the attenuation model was derived depended on the ground conditions of the site.

In other words, Oliveira's equation was proposed based on earthquake data obtained at the surface of hard bedrock. As for the equation of Esteva-Rosenblueth, an equation for the surface of hard ground was modified into an equation for the surface of bedrock and proposed.

On the other hand, McGuire's equation and Katayama's equation were based on earthquake data obtained at the surfaces of various kinds of ground from hard to soft. Because of this, McGuire's and Katayama's equations tend to give maximum accelerations of larger values compared with Oliveira's and Esteva-Rosenblueth's equations.

In this way, the results will be different depending on the attenuation model applied. With regard to the Pirris project site, it can be judged that because of the seismic activity of Costa Rica being fundamentally high, it will be appropriate to assume a value enveloping the results obtained here, that is, "290 gal."

This value of 290 gal corresponds roughly to a return period of 10,000 years from the standpoint of stochastic analysis results.

Table 8-7 Maximum Accelerations for Six Return Periods

Attenuation Model (Eq. No.)	Return Period (Year)					
	50	100	200	500	1000	10000
(1) C. Oliveira	75.3	99.7	124.7	156.7	179.1	236.8
(2) R. K. McGuire	150.1	178.0	203.6	233.1	251.8	294.9
(3) Esteva & Rosenblueth	79.9	107.0	135.2	172.1	198.3	268.1
(4) T. Katayama	104.8	132.5	168.3	195.2	219.4	282.3

(Unit: gal)

8.2.4 Design Horizontal Seismic Coefficient

(1) Design Horizontal Seismic Coefficient of Ground

Regarding the relationship between the maximum horizontal acceleration of earthquake motion and the design horizontal seismic coefficient, the following equation will generally be valid:

$$Kh = R \frac{A_{max}}{980} \quad \cdots\cdots \quad (5)$$

where, Kh : Design horizontal seismic coefficient

R : Conversion factor

A_{max} : Maximum horizontal acceleration of earthquake motion (gal)

The design horizontal seismic coefficient of the above equation is what is called effective seismic coefficient or equivalent seismic coefficient, and the following proposals have been made in research in Japan.

(1) Kh = (0.35 ~ 0.42) $A_{max}/980$ (effective value of steady sine wave) . . . . (6)

(2) Kh = 0.33 $(A_{max}/980)^{1/3}$ (Noda[4], 1975) . . . . (7)

(3) Kh = 0.072 + 0.332 (A_{max}/980) (Matsuo[5], 1984) . . . . . (8)

(4) Kh = (0.13 ~ 0.34) A_{max}/980 (Hakuno[6], 1984) . . . . . . (9)

(5) Kh = (0.50 ~ 0.60) A_{max}/980 (Watanabe[7], 1984) . . . . (10)

In the Technical Guideline for Aseismic Design of Nuclear Power Plants[8] published in 1987, the following equation is proposed as a result of overall evaluation and consideration taking into account these cases of study.

Kh = (0.40 ~ 0.60) A_{max}/980 . . . . (11)

The concept of effective seismic coefficient (equivalent seismic coefficient) was derived so that the largeness of stresses produced in ground and structures by earthquake motions will be equivalent for cases of handling dynamically (dynamic analysis by input of earthquake motion) and for cases of handling statically (static analysis using design seismic coefficient). The conversion factor which will be required for calculating effective seismic coefficient (equivalent seismic coefficient) is thought to be largely dependent on the frequency characteristics of design input earthquake motions. That is, for an earthquake motion with long-period components predominant, a large value (for example; 0.6) should be taken for the conversion factor. And for an earthquake motion with short-period components predominant, a small value (for example; 0.4) can be taken for the conversion factor.

In the second progress report (July, 1991), the seismic risk analysis was made based on the earthquake records during the period of 49 years between 1939 and 1987. On the other hand, the seismic risk analysis in this report was executed using the earthquake records during the period of 92 years between 1900 and 1991.

Applying Eq. (5) and supposing R = 0.5, the design ground horizontal seismic coefficient for the Pirris project site can be estimated to be Kh = 0.15, since the maximum acceleration at the site is 290 gal.

(2) Design Horizontal Seismic Coefficient for Dam

Regarding the design horizontal seismic coefficient for dam, as shown in Table 8-8, the same value as the design horizontal seismic coefficient of ground is adopted for fill dam and gravity dam. For arch dam, a value twice the design horizontal seismic coefficient of ground is adopted.

Table 8-8 Design Horizontal Seismic Coefficient for Dam

Dam Type	Design Horizontal Seismic Coefficient
Fill Dam	0.15
Gravity Dam	0.15
Arch Dam	0.30

8.3 Afterword

The determination of optimum configuration and cross section of dam, and the basic stability evaluation of dam during earthquake are normally made according to the seismic coefficient method. The design seismic coefficient to be used in the seismic coefficient method, as previously mentioned, is evaluated considering a conversion factor for the maximum acceleration of earthquake motion assumed for the site. The value of the conversion factor can be thought to depend on the frequency characteristics of the earthquake motions assumed, and the dynamic characteristics of dam and foundation rock to be considered in the earthquake-resistant design. Therefore, it is desirable for the seismic stability of dam to be ascertained by dynamic analyses. The appropriateness of the design seismic coefficient can be verified by comparison of dynamic and static analyses.

For the reference, general procedure of earthquake resistant design for dams is shown in Fig. 8-9.

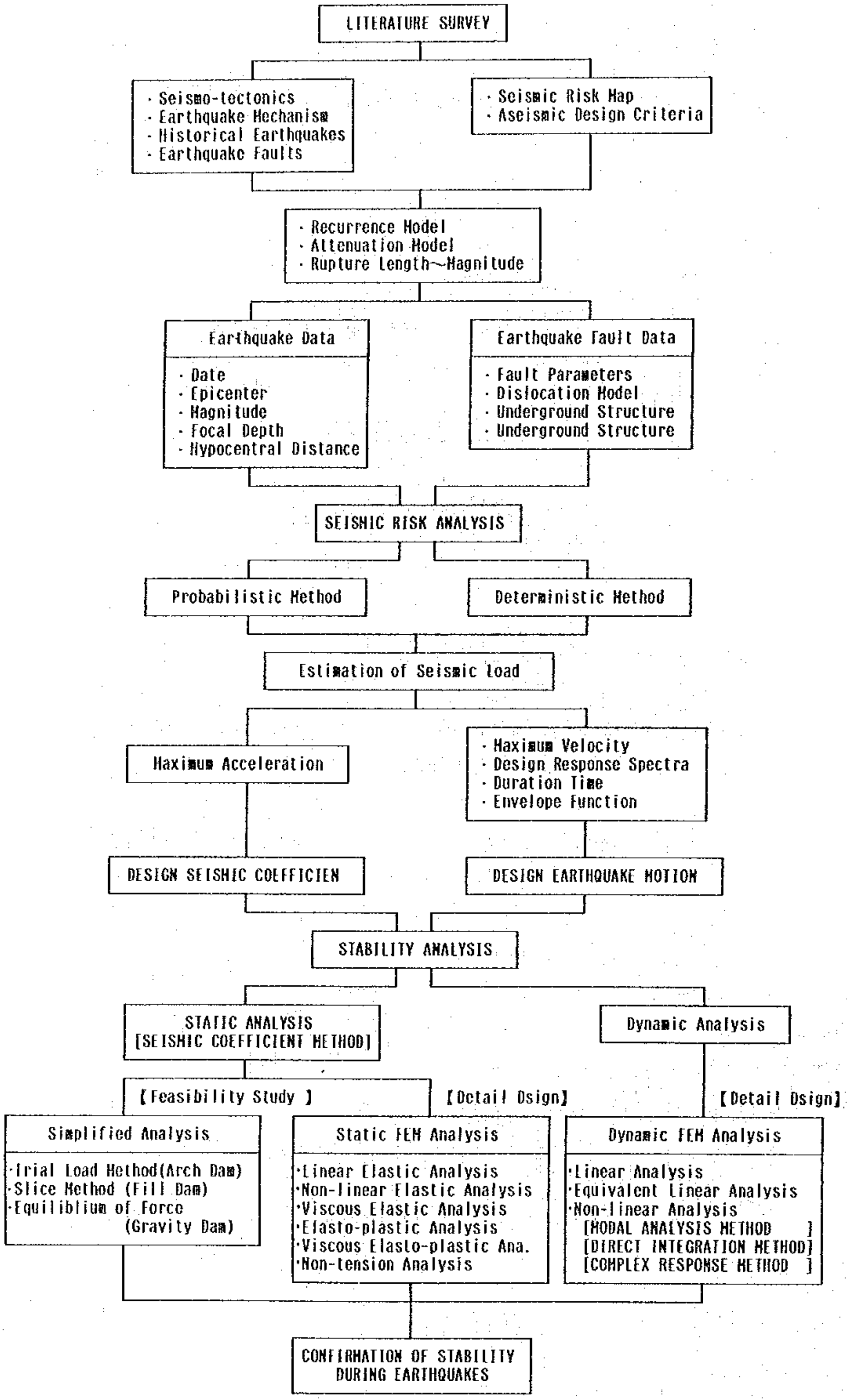


Fig. 8-9 General Procedure of Earthquake Resistant Design for Dam

Reference

1) Vasconez K.C.; Costa Rica : A Country profile, V.S. Department of Commerce Report, No. PB-89-105829.

2) Bourgsis J, J. Azema, J. Tournon, J. Aubouin, *The geological history of the Caribbean-Cocos plate boundary with* special reference to the Nicoya ophiolite complex (Costa Rica) and D.S.D.P. results (Legs 67 and 84 off Guatemala) : A synthesis, Tectonophysics, Vol.108, pp1-32, 1984.

3) Instituto Costarricense de Electricidad P.H. PIRRIS Estudio Seismologico Preliminar, Noviembre 1989.

4) Noda, S., Kambe, T., and Chiba, T., "Seismic Coefficient of Gravity-type Quaywall and Ground Acceleration," Report of Port and Harbour Technical Research Institute, Ministry of Transport, Vol. 14, No. 4, PP.67-111, 1975

5) Matsuo, M., and Itabashi, K., "Study on Evaluation of Aseismicity of Slopes and Soil Structures," Transactions of the Japan Society of Civil Engineers, No. 352, III-2, Dec. 1984

6) Hakuno, M., and Morikawa, O., "A Simulation Concerning Earthquake Acceleration and Failure of Structures," Transactions of the Japan Society of Civil Engineers, No. 344, I-1, pp.299-302, Apr. 1984

7) Watanabe, H., Sato, S., and Murakami, S., "Evaluation of Earthquake-Induced Sliding in Rockfill Dams," Soil and Foundation, Vol. 24, No. 3, pp. 1-14, Sept. 1984

8) Japan Electric Association, "Technical Guide to Aseismic Design of Nuclear Power Plants," 1987

CHAPTER 9 DEVELOPMENT PLAN

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9.1 Reviews of Existing Development Schemes and Development Scales

9.1.1 Water Supply Intake Project in Upstream Area

The Pirris Water Supply Intake Project is one of seven plans formulated by ICAA for supplying water to the city of San Jose. This plan is for providing Copey Dam upstream of the city of Santa Maria in the upstream basin of the Pirris River and supplying water of a quantity of 2.0 m^3/s as far as San Rafael Town in the suburbs of San Jose by a waterway of 38.6 km. According to "Plan Maestro de Abastecimiento de Agua y Saneamiento y Alcantarillado Sanitario de la Gran Area Metropolitana" prepared in August 1989, the outline of the water supply intake project is as follows:

The dam site is at a point 5 km upstream of Santa Maria City with a catchment area of 62.4 km^2 with water collected from the upstream basin of the Pirris River. The dam would be a rockfill dam 67 m in height and having a concrete facing, the dam crest length being 520 m and the dam volume 2,592 $\times 10^3$ m^3. The reservoir gross storage capacity would be 38.5 $\times 10^6$ m^3, of which the effective storage capacity would be 36.5 $\times 10^6$ m^3, and with an effective depth of 41.5 m, a maximum of 2.0 m^3/s is to be drawn at an intake water level of 1,818 m. The waterway is to consist of a pressurized steel pipeline 31.3 km in length and a pressure tunnel 7.3 km in length, a total of 38.6 km, installed on a route from the dam and going by Santa Maria City, San Cristobal Town, and Corralillo Town to reach San Rafael Town. A concrete water tank of capacity 5,000 m^3 would be installed at water level elevation of 1,215 m at the terminal point of the waterway, and this wold be connected to the water mains of San Jose City.

The total construction cost of this project is estimated to be US$114,600 $\times$ 10^3 with the unit cost per cubic meter US$0.598/$m^3$. ICAA has formulated 6 plans other than this project and the unit costs per cubic meter of these plans are as follows:

	Project	Intake Volume	Unit Cost per Cu. M
1.	Heredia District Ground Water Pump-up Project	2.3 m^3/s	US$0.134/$m^3$
2.	Rio Macho Intake Project	2.3 m^3/s	US$0.193/$m^3$
3.	Rio Sarapiqui Pump-up Project	0.6 m^3/s	US$0.254/$m^3$
4.	Rio Patrio Intake Project	1.0 m^3/s	US$0.238/$m^3$
5.	Rio Sarpiqui-Rio Patris Project	2.3 m^3/s	US$0.247/$m^3$
6.	Rio Trendas Intake Project	1.0 m^3/s	US$0.274/$m^3$
7.	Rio Pirris Intake Project	2.0 m^3/s	US$0.598/$m^3$

As shown above, the Pirris River Intake Project has the poorest economics of the seven schemes and it is evaluated that there is very little possibility of this project being implemented. Therefore, after discussion with ICE, it was decided not to consider above mentioned Intake Project in the study of Pirris Project.

9.1.2 Stepped Development Plan and Basic Development Concept

(1) Selection of Alternative Development Plan

According to the present plan proposed by ICE, the Pirris Hydroelectric Power Development Project is at a project site of a high head as much as approximately 900 m.

When a study is made regarding this high head considering geology and topography based on field investigations, three alternative plans including the single-step development and two-step development proposed by ICE, as shown in Fig. 9-1, are conceivable.

Plan I is a dam-and-conduit type scheme connecting from Pirris Dam to the powerhouse with a single pressure headrace tunnel, aiming for development of the head obtained in one stroke. In this case the headrace tunnel is long at approximately 9 km, and studies of the construction method and construction period giving consideration to economics will be problems.

Plans II and III are for development dividing the head into two steps. The basic layout for these plans is that of connecting Pirris Dam and a powerhouse by a pressurized headrace with a No. 1 power plant provided downstream of the dam. Further, with a small dam constructed downstream of the No. 1 power plant, a No. 2 power plant is provided as a regulating pond type power station combining and re-regulating the discharge of the No. 1 power plant and the runoff of the remaining catchment area. The location of the No. 2 powerhouse would be that of the powerhouse in the case of single-step development.

With these plans, by dividing the whole into two power development projects, the costs of the individual development projects would become smaller to make stepped development more feasible, but it will be unavoidable for the overall cost to become higher.

(2) Basic Development Concept

The specifications of the individual comparison plans selected according to the preceding section and the results of comparison studies of the economics are given in Table 9-1. From these results, Plan I with the best economic nature was selected as the basic development scheme.

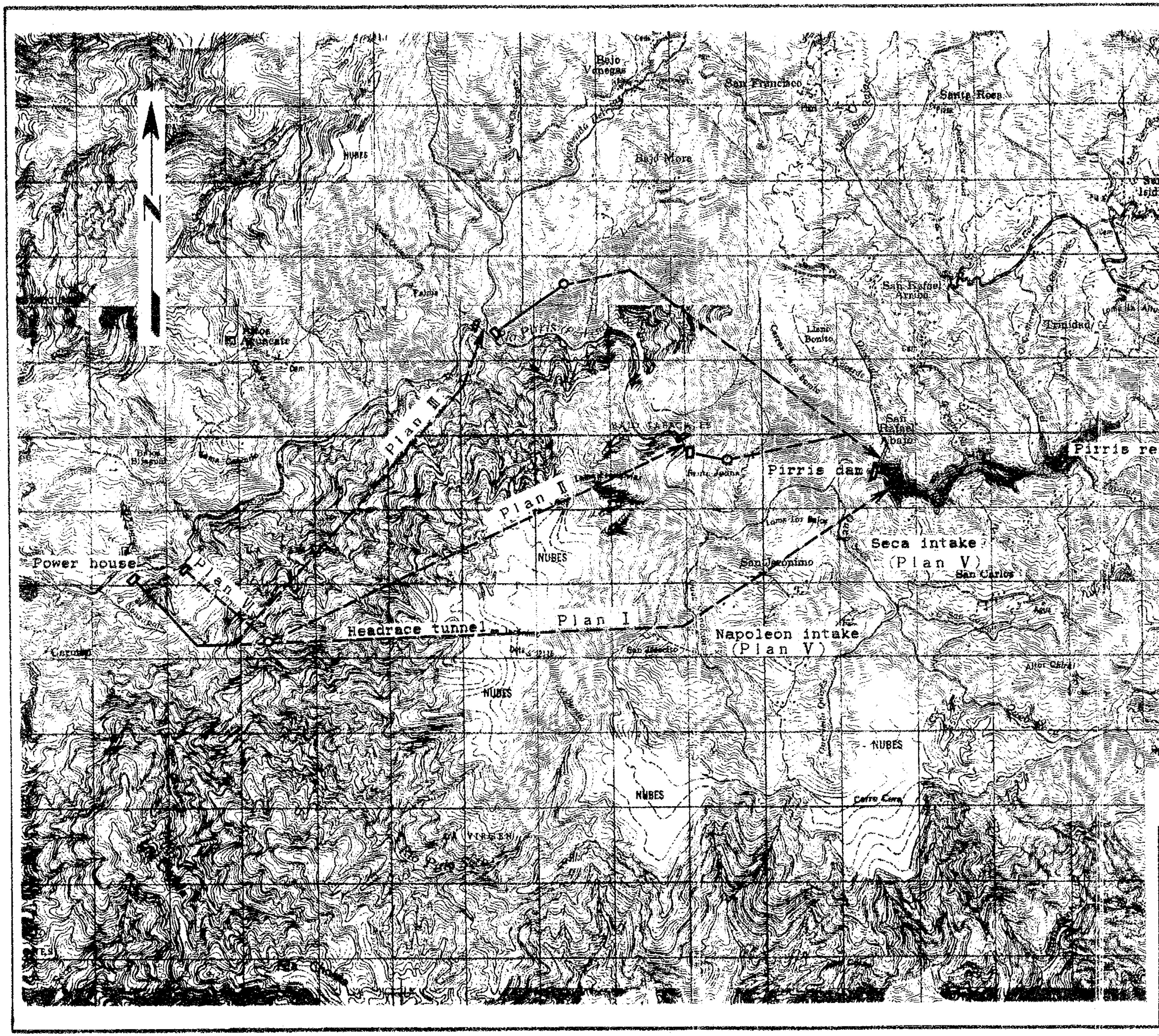
9.1.3 Mode of Demand and Development Scale

(1) Mode of Demand

The power demand in Costa Rica becomes a maximum around November and December in the summer. The daily variation in power demand shows many kinds of modes throughout the year. The representative examples of daily load curves of Wednesday, April 4 and Monday, November 19, 1990 are shown in Fig. 9-2.

Table 9-1 Alternative Study (1) (Lower dam site)

Item	Unit	Plan I	Plan II			Plan III		
			No.1 PS	No.2 PS	Total	No.1 PS	No.2 PS	Total
			ダム水路式	同　左	——	ダム水路式	同　左	——
Catchment Area	km^2	250.8	250.8	277.4 (250.8+26.6)	——	250.8	313.9 (250.8+63.1)	——
Annual Inflow	$10^6 m^3$	351.61	351.61	388.90	——	351.61	440.07	——
Annual Power Discharge	$10^6 m^3$	284.62	284.62	323.65	——	284.62	369.79	——
Annual Spill	$10^6 m^3$	66.99	66.99	65.25	——	66.99	70.28	——
Dam Type	—	Concrete Arch	Concrete Arch	Concrete	——	Concrete Arch	Concrete	——
High Water Level	m	1,184.7	1,184.7	915	——	1,184.7	677	——
Low Water Level	m	1,149	1,149	914	——	1149	674	——
Avaialble Drawdown	m	35.7	35.7	1	——	35.7	3	——
Gross Storage Capacity	$10^6 m^3$	26.88	26.88	0.40	——	26.88	0.65	——
Effective Storage Capacity	$10^6 m^3$	20	20	0.10	——	20	0.25	——
Dam Height	m	108	108	30	——	108	35	——
Tunnel Length	m	8,590	2,200	6,700	——	5,200	5,700	——
Tunnel Diameter	m	2.5	2.5	2.7	——	2.5	2.9	——
Tunnel Type	—	Pressure	Pressure	Pressure	——	Pressure	Pressure	——
Maximum Discharge	m^3/s	15	15	17	——	15	20	——
Standard Intake Water Level	m	1,173	1,173	915	——	1,173	677	——
Tail Water Level	m	303	915	303	——	677	303	——
Gross Head	m	870	258	612	870	496	374	870
Rated Effective Head	m	816.59	244.14	575.48	819.62	466.89	346.17	813.06
Installed Capacity	MW	105.1	31.4	83.9	115.3	60.1	59.4	119.5
Head Loss	m	53.41	13.86	36.52	50.38	29.11	27.83	56.94
Firm Peak Power	MW	102.39	30.61	82.80	113.41	58.54	54.80	113.34
Annual Firm Energy	GWh	186.86	55.87	151.12	206.99	106.84	100.00	206.84
Annual Secondary Energy	GWh	370.01	110.62	292.53	403.15	211.56	204.91	416.47
Annual Total Energy	GWh	556.87	166.49	443.65	610.14	318.40	304.91	623.31
Investment Cost	10^6 ¢	12,951	15,001			16,220		
Annual Cost (C)	10^6 ¢	1,684	1,950			2,109		
Annual Benefit (B)	10^6 ¢	2,286	2,519			2,544		
Benefit Cost Ratio (B/C)	—	1.36	1.29			1.21		
Surplus Benefit (B-C)	10^6 ¢	603	569			435		
Unit Cost of Energy	¢/kWh	23	25			26		



REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT

Alternative Layout

Fig. 9-1

The load of ICE starts to increase from about 04:00 around dawn with a morning peak at 11:00 to 12:00. The load decreases to around the same level as at 07:00 during the afternoon siesta period. It increases again from around 16:00 to reach a lighting time peak at around 18:00. There are momentary peaks once each in the morning and in the evening, while peak durations amount to 4 to 6 hours. Roughly 35 ~ 40 percent of demand is for 24 hours, and 70 percent of demand for approximately 14 hours with the remainder as peak hours.

The maximum demand of 682 MW was recorded on November 19, 1990. The daily load factor is approximately 65 percent. From this daily load curve, it is thought that almost all of the demand is composed of meter rate lighting and power load.

(2) Development Scale

In making a study of development scale, it is also necessary to examine the required peak duration from the standpoint of electric power demand and supply balance.

Here, a rough study was made of the necessary peak duration based on the load forecast and power development plan prepared by ICE. [Sistema Nacional Interconectado de Costa Rica, Plan de Expanción de la Generación, según Modelo LOGOS]. The hydroelectric power facilities are predominant in ICE. According to the power development plan of ICE, the predominance in hydro will not change in the future.

In general, the necessary peak duration as seen from power demand is examined by the "deduction method" in case of a hydro-main, thermal-subordinate system. The examination was made by the "deduction method" in this case.

Study Conditions

The examinations by the "deduction method" were made based on the following conditions:

1) The commissioning year of Pirris Power Station is 2001 A.D.

2) The daily load curve is converted to 2001 with November 19, 1990 (Monday) as the basis.

3) Operation is 24 hours with geothermal and thermal power as the base load.

4) Hydroelectric power stations scheduled to be commissioned prior to the year 2000 not including existing hydroelectric power stations and Pirris Project are allotted to supply intermediate load.

5) Gas turbines are used to supply momentary peak load.

6) Electric power still short after the measures of 3) to 5) is supplemented from Pirris Power Station.

7) Existing thermal plants commissioned prior to 1970 are considered inoperable due to deterioration by the year 2001 and are abolished. (Colima, 19 MW, commissioned in 1958 and 1962: San Antonio, 10 MW, in 1954)

8) Importation of electric power from foreign countries is out of scope of the study.

9) The planned output of 128 MW was adopted for the power supply capacity of Pirris Power Station.

As a result, it is considered that Pirris Power Station should be developed as a peaking power plant as shown in Fig. 9-3. Consequently, the peak duration time required of Pirris Power Station will be 5 hours.

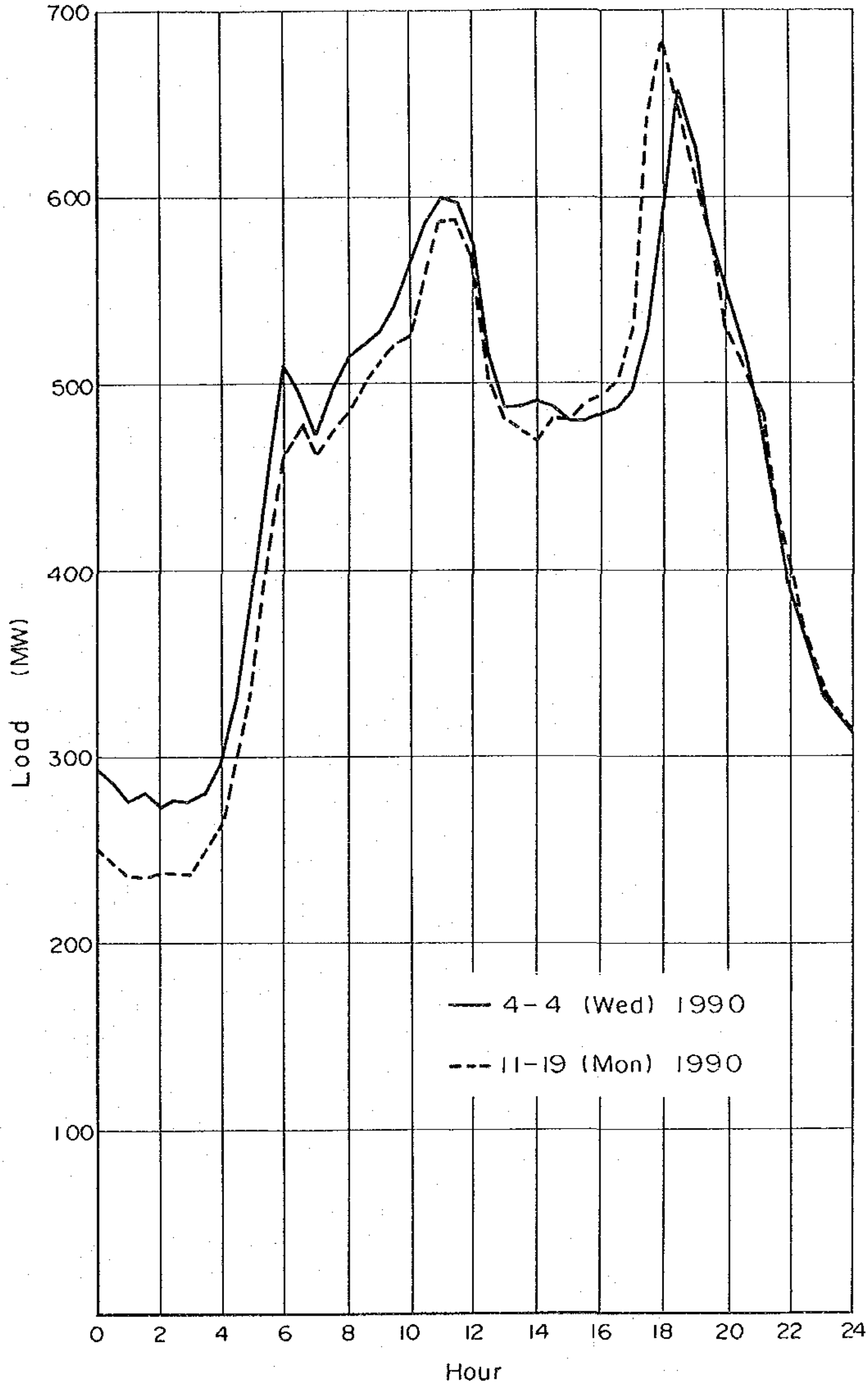
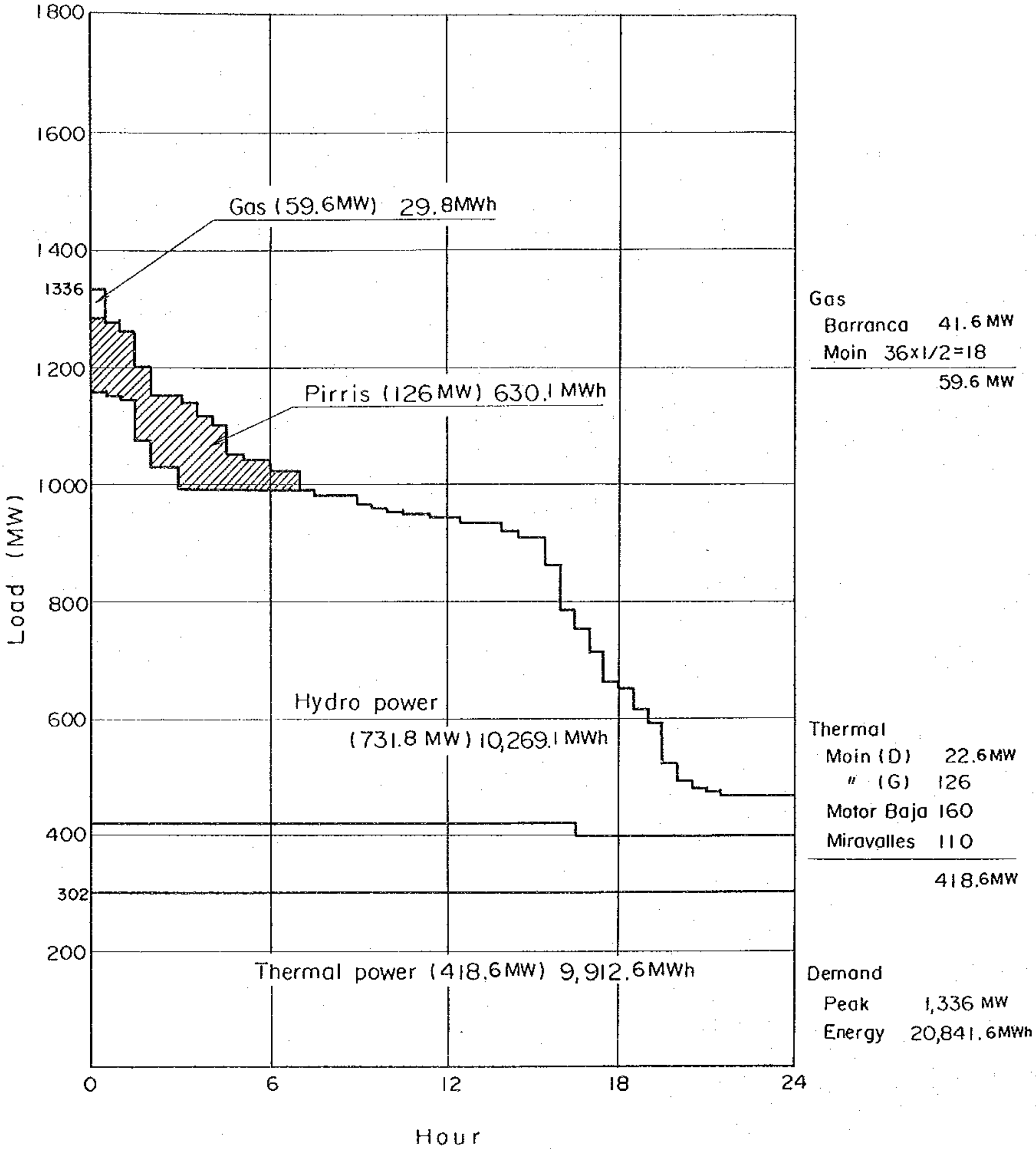


Fig. 9-2 Daily Load Curve



＊1. It was assumed that 5% of the total thermal installed capacity in 1990 will gradually and linearly diminish until year 2010.

＊2. A half capacity of a gas turbine at Moin Power Station will be used for peak load supply. Others are used for base load supply.

Fig. 9-3 Power Demand and Supply Balance (2001)

9.2 Comparison Studies of Development Plans (Primary Study)

9.2.1 Selection of Alternative Plans

(1) Dam Site and Dam Type

There are two alternative locations for the dam site in the Project: an entrance point of Pirris Gorge (downstream dam site) and a point 500 m further upstream (upstream dam site). The reason is that there are limitations in selecting dam height because of the geological conditions of the right-bank side of the downstream dam site.

Looking at these two dam sites from the standpoint of topography, the downstream site has a V-shaped topography with the ratio of dam crest length (L) to dam height (H) being 1.8. Generally speaking, a site with such a valley shape is most suitable for a concrete arch dam. On the other hand, the upstream site is of a gentle U-shape, and the ratio of L to H is 2.5. This topography is appropriate for a rockfill dam for which dam construction materials is cheap. The transverse sections of the two dam sites are shown in Fig. 9-4.

As a result of field reconnaissances, it was found that the Terraba Formation at the site originally assumed does not exist and the rock is uniformly andesitic lava. As a consequence, it was judged that there would be an easing of constraints concerning the limit to dam height, although definite statement must await the results of the geological investigation works to be carried out in 1990. On the other hand, the upstream dam site consists of alternations of sandstone and slate, with weathering prominent with higher elevation, especially at the left-bank side, and it was assumed that the line of sound rock would be deep inside.

In view of the above, the three types of concrete arch dam, concrete gravity dam, and rockfill dam were taken up for the dam at the downstream site, and rough comparison studies were made. However, in case of a rockfill dam, it is conceivable that there would be problems in carrying out construction in view of the topography. As for the upstream site, it was restricted to a rockfill dam.

The approximate dam layouts with the effective storage capacity of 20×10^6 m^3 are shown in Figs. 9-5 to 9-8.

As a result of the studies, it was judged that a rockfill dam would be suitable for the upstream site irrespective of dam height, while an arch dam would be suitable for the downstream site.

In view of this, it was decided to carry out the study of the primary optimum development scale based on the layout with the location of the dam at the downstream site. However, with the upstream site, in case of equal effective storage capacity, there will be an increase in head, and an increase in annual energy production can be expected. Therefore, a study on the development scale for the upstream dam will also be made. The result will be compared with the downstream site.

Accordingly, for the upstream site, it was decided to make a comparison with the primary optimum scale determined for the downstream site including a development plan having an equivalent effective storage capacity scale.

The comparison studies are described in 9.2.2.

(2) Catchment Area and Effective Storage Capacity

- The total catchment area of the dam site of the Project (downstream site) is 250.8 km^2.

  ICAA has a plan to carry out intake at a point (catchment area 62.4 km^2) approximately 5 km upstream of Santa Maria City and conducting this for city water supply to San Jose. Therefore, the study of the Pirris Hydroelectric Power Development Project made previously by ICE used an actual catchment area of 188.4 km^2 (250.8 - 62.4 = 188.4 km^2). However, as mentioned in 9.1.1, it was decided not to consider the water supply intake plan upstream of Pirris Dam in the feasibility study of the Project. Consequently, in the examination of the Project, approximately 250.80 km^2 is to be considered as the total catchment area of the dam site (downstream site).

- There are two alternative dam sites for the Project: the downstream site and the upstream site (a site approximately 500 m upstream from the downstream dam site).

A review was made of the storage capacity curves of the two sites based on 1/5,000-scale topographical maps. The capacity curves reviewed are shown in Figs. 9-9 and 9-10. The minimum water level (LWL) of the reservoir was set up taking into consideration sedimentation at the dam site (see 6.4) and the minimum intake water level for the pressure tunnel. As a result, the reservoir high water levels (HWL) and low water levels (LWL) in the cases of effective reservoir storage capacities of 20×10^6, 30×10^6, 40×10^6 are obtained as given below.

Effective Capacity	20×10^6 m^3	30×10^6 m^3	40×10^6 m^3
Upstream dam site			
HWL	1,195.0 m	1,204.7 m	1,213.1 m
LWL	1,161.0 m	1,161.0 m	1,162.0 m
Downstream dam site			
HWL	1,184.7 m	1,194.7 m	1,203.1 m
LWL	1,149.0 m	1,149.0 m	1,150.0 m

(3) Mountain Stream Intake

There are two possible mountain stream intake sites at Queb. Seca (catchment area 10.6 km^2) and Queb. Napoleon (catchment area 3.9 km^2).

There are tributaries of the Pirris River, and water intake may be considered in the section passed by the headrace tunnel.

Intake from these mountain streams will be possible with only small-scale intake dams, short connecting waterways, and combining tanks. Thus, studies were made whether the economics of the Project could be improved by carrying out intake from these streams. The studies were made for two cases: intake from Queb. Seca only, and intake from Queb. Seca and Napoleon. Details of the studies are given in 9.2.2.

(4) Penstock Route and Tail Water Level

There are two alternative routes for penstock considered by ICE. The field reconnaissances revealed that there are landslide topographies seen at the left-bank slope where pass the penstock routes. Therefore, alternative penstock route and powerhouse site were newly selected. The topography and geology of these routes and sites are described in 7.6.3.

The upstreammost alternative by ICE for the penstock route and the downstreammost alternative by JICA survey team were compared and studied.

The tail water levels of these alternatives are as given below since the river gradient is topographically steep:

Tail water level in case of upstreammost alternative:
340.0 m (river-bed elevation 334 m)

Tail water level in case of downstreammost alternative:
303.0 m (river-bed elevation 296 m)

Details of the comparison studies on these two alternatives are as given in 9.2.2.

9.2.2 Comparison Studies of Development Plan

(1) Basic Conditions for Study

(a) Comparison Studies by Annual Cost Method

In making examinations of the comparative study for the Project and of the development scale, the technique of taking a standard type of thermal power station that would have been constructed if the Project did not exist as an alternative facility and considering the cost of that thermal as the benefit was used.

The alternative facility selected was a combination of gas turbine and diesel engine power plants that could be considered as an alternative to Pirris Power Station.

In making the examination, market prices were used with the annual surplus benefit (B - C) and the benefit-cost ratio (B/C) as indices. Cost (C) is the equalized annual cost for the service life (50 years) of the hydro power facilities and Benefit (B) is the equalized cost of the alternative thermal.

The particulars of the alternative thermal power station are given in Table 9-2. It was decided that the costs of the transmission lines from Pirris Power Station and the alternative power station to the capital city of San Jose would not be considered at the stage of the comparison study.

(b) Annual Cost

The equalized annual cost of a hydropower facility consists of depreciation, interest and operation and maintenance cost. The cost is obtained by multiplying the construction cost by the annual cost factor.

Annual Cost = Annual Cost Factor × Construction Cost
= Depreciation + Interest + Operation and Maintenance Cost

Depreciation + Interest = Construction Cost × Capital Recovery Factor

$$\text{Capital Recovery Factor} = \frac{i\ (1 + i)^n}{(1 + i)^n - 1}$$

where, n: service life,

civil structure	50 yr
hydropower equipment and facilities	35 yr
electrical equipment and facilities	35 yr

i: discount rate, 12 percent

Capital Recovery Factor:

Civil structure	12.0%
Hydropower equipment and facilities	12.2%
Electrical equipment and facilities	12.2%

Depreciation + Interest ≒ Construction Cost × 12%
Operation and Maintenance Cost ≒ Construction Cost × 1%

Therefore, Annual Cost ≒ Construction Cost × 13%

(c) Conception of Benefit

The benefit of the Project is to be the total of the overall depreciation, interest, maintenance and administration cost, and fuel cost of the alternative thermal power station. The output and energy production of the Project used for benefit calculations are obtained according to the conditions indicated below. These are respectively defined as effective output and effective electric energy. Transmission line losses are not considered in the study below.

i) The effective output is the dependable peak output less the station power ratio of 0.3 percent, accident ratio of 0.3 percent, and repair ratio of 2.0 percent.

The dependable peak output is the average value of monthly minimum peak outputs during the energy calculation period (25 years).

Effective Output = (1 - 0.003) x (1 - 0.003) x (1 - 0.02) x Dependable Peak Output

ii) The effective electric energy is the annual energy production less the station service ratio of 0.3 percent.

Effective Electric Energy = (1 - 0.003) x Annual Effective Energy

Furthermore, firm electric energy is defined as the electric energy produced during the necessary equivalent peak duration. The secondary electric energy is defined as all other electric energy.

iii) Benefit = Effective Output x kW Value + Firm Energy x Firm kWh Value + Secondary Energy x Secondary kWh Value

(See Table 4-2 for kW value and Firm/secondary kWh Value)

(2) Reservoir Operation Plan

The annual average inflow at the Pirris Dam site is approximately 11 m^3/s. In the rainy season from May to November, approximately 84 percent of the inflow occurs, and especially, the runoffs of September and October are large.

Time-dependent-wise, the variation in runoff is not very great. The inflow of 499×10^6 m^3 in the highest water year (1969) of a 25-year period was approximately 2.2 times the inflow of the lowest water year

(1986). In case the effective storage capacity is taken as 20 $\times$ 10^6 m^3, the reservoir regulating ratio will be approximately 6 percent.

The electric energy calculations in making the study were done by electronic computer using the monthly average inflow for the 25-year period from May 1964 to April 1989.

The firm discharge is defined as the runoff that can be used at any time during the 25 years. The mass curve of the inflow was used for determinations to make available discharge a maximum.

The mass curve of the reservoir inflow is shown in Fig. 9-11. The relation between the reservoir effective storage capacity and the firm discharge is shown in Fig. 9-12.

In calculation of electric energy, the standard intake water level serving as the basis for design of turbines and generators was given by (High Water Level - 1/3 $\times$ Water Level Variation Width) which is the operating water level on average.

Calculations of electric energy were made for reservoir operation using the principle of optimum scale according to the dynamic programming technique. Discharges of the individual months were set so that the total electric energy would be a maximum. And the influence of evaporation from reservoir was ignored.

(3) Examination of Development Scale

(a) Reservoir Scale

As described in 9.2.1, "Selection of Alternative Proposal", the examination of the reservoir scale was done first on the downstream dam site. For the comparison studies, a concrete arch dam was considered as the common condition, and for effective storage capacity, five cases centered on the sizes of 20, 30 and 40 $\times$ 10^6 m^3 were selected. The conditions considered in carrying out the comparison studies were as follows:

i) Based on the study in 9.1.3, "Mode of Demand and Development Scale", the minimum peak duration of Pirris Power Station was put as 5 hours.

ii) The maximum power discharge of Pirris Power Station was set to match the peak duration of 5 hours for the firm discharge determined from the inflow and effective storage capacity of the reservoir.

iii) The number of main electric equipment units was made two Pelton turbines.

iv) The locations of the penstock and powerhouse were taken to be those of the downstreammost alternative (discharge water level 303 m) selected based on field reconnaissances.

Table 9-3, Fig. 9-13 show the study results. According to this, the surplus benefit B - C, becomes maximum in the vicinity of effective capacity of 40 x 10^6 m^3. The investment efficiency B/C becomes maximum between 20 to 30 x 10^6 m^3. Therefore, the scale of Pirris Reservoir cannot be specifically determined at this stage. A scale of effective capacity between 20 to 40 x 10^6 m^3 was taken as the primary optimum development scale.

(b) Dam Site and Dam Type

The study described above was on the downstream dam site; a similar study was made on the upstream dam site (rockfill dam) also. The results were used as data for a comparison study for selection of the dam site.

The study results are shown in the same Table 9-3 and Fig. 9-14. According to these, the highest points of B - C and B/C for the upstream dam site are in a broad range of effective capacity of 20 to 40 x 10^6 m^3 similarly to the downstream site, and it is difficult to specifically determine the optimum scale.

However, when compared with the downstream dam site, the economics of the upstream dam site are inferior. Consequently, although depending on the results of further detailed investigations, it was thought that the downstream dam site will be superior. Further, regarding study of the dam type, three dam types, concrete arch, concrete gravity, and rockfill, were conceivable for application to the downstream dam site.

As a result, it was found that a concrete arch dam is found to be the most economical dam type for the downstream site with dam height ranging from 108 to 126 m which corresponds to effective reservoir storage capacity of 20 to 40 x 10^6 m^3. The next economical one is a concrete gravity dam, followed by a rockfill dam.

Rockfill dam at upstream dam site is economically inferior to concrete arch dam at lower dam site, but is economically little different from concrete gravity dam at lower dam site.

The economics of Plan IV applying a rockfill dam and a concrete gravity dam to Plan I (downstream dam site, concrete arch dam, effective storage capacity 20 × 10^6 m^3, 40 × 10^6 m^3) are given in Table 9-4.

(c) Mountain Stream Intake

The study of the pros and cons of mountain stream intake was made on Plan V in which intake of water from the respective mountain stream was considered for Plan I. The study of Plan V consisted of two cases: intake from the Pirris River tributary Queb. Seca (catchment area 10.6 km^2) and intake from the tributary Queb. Napoleon (catchment area 3.9 km^2) in addition.

The results of examination are as given in Table 9-4. It was thought there would not be very much economic merit in gully water intake.

At the present stage, river flow maintaining water is not considered for the section of reduced water flow downstream of the dam. Therefore, it was judged that the runoffs from the two tributaries would be necessary for river flow maintenance of the reduced flow section.

As a result of discussions with ICE concerning this point, it was decided that gully water intake would not be considered in this Study from the judgement that intake of water from the two tributaries is undesirable from an environmental point of view.

(d) Penstock Route and Tail Water Level

In view of the confirmation of landslide topography at the left-bank slope which was where the scheduled route of the penstock was, the penstock-powerhouse route newly set up downstream (downstreammost proposal) and the upstreammost alternative newly offered by ICE and including also the study of the tail water level are as shown in Table 9-4.

The downstreammost alternative is Plan I, while the upstreammost alternative of ICE is Plan VI.

According to this, there is a difference of nearly 37 m in the tail water level between Plans I and VI.

The economics of the two plans are approximately of the same degree. This is thought to be the result of the increase in construction cost of the extra length of penstock due to switching the powerhouse downstream being offset by the increase in electric energy produced.

As described in 7.6.3, the topographical and geological conditions, and the safety of the penstock and powerhouse in the future are considered and the layout of the downstreammost side free of risk of landslide was to be adopted.

Table 9-2 Standard Alternative Thermal Power Plant

Item	Unit	Description	
Type	—	Gas Turbine	Diesel (Slow Speed Internal Combustion Engine)
Installed Capacity	MW	2 × 36 MW	1 × 32 MW
Annual Plant Factor	%	30	80
Thermal Efficiency	%	29.97	34.33
Annual Energy Production	GWh	189	224
Construction Cost (Interest During Construction Included)	$	2 × 15, 583, 589	30, 132, 158
Service Life	year	15	25
Construction Period	year	2	2
Capital Recovery Factor	—	0.14682	0.12750
Diesel Calorific Value	kcal/kg	10,248	—
Bunker Calorific Value	kcal/kg	—	10,207
Fuel Consumption Rate $\left(\frac{860\ \text{kcal/kWh}}{\text{Thermal Efficiency} \times \text{Colorific Value}}\right)$	kg/kWh	0.280	0.245
O & M Cost	%	3.41	1.85
Unit Fuel Cost	$/l (1989 CIF)	0.1482 (Diesel)	0.0876 (Bunkeroil)

Type		Gas Turbine		Diesel (Slow Speed Internal Combustion Engine)	
Annual Cost	Unit	Fixed Cost	Variable Cost	Fixed Cost	Variable Cost
Capital Recovery	10^6 $	4.576	——	3.842	——
O & M Cost	10^6 $	0.957(90%)	0.106(10%)	0.502(90%)	0.056(10%)
Fuel Cost	10^6 $	——	9.426 [1]	——	4.896 [2]
Total	10^6 $	5.533	9.532	4.344	4.952
Annual Cost at Receving end					
kW Cost	$/kW	[3] kW value		119.57 $/kW	
kWh Cost	$/kWh	[4, 5] Firm energy value		0.0373 $/kWh	
		Secondary energy value		0.0235 $/kWh	

1) $189 \times 10^6 \times 0.280/0.832 \times 0.1482 = 9.426 \times 10^6$ \$

2) $224 \times 10^6 \times 0.245/0.982 \times 0.0876 = 4.896 \times 10^6$ \$

Adjustment Factor for kW & kWh

Item	kW (%)	kWh (%)
Loss of Station Service	6	6
Loss of Stoppage	4	-
Loss of Repair	12	-
Loss of Transmission	0	0

$$\text{kW Adjustment Factor} = \frac{1}{(1-0.06)\times(1-0.04)\times(1-0.12)\times(1-0.0)} = 1.259$$

$$\text{kWh Adjustment Factor} = \frac{1}{(1-0.06)\times(1-0.0)} = 1.064$$

3) $$\frac{(5.533+4.344)\times 10^6}{(2\times 36+1\times 32)\times 1{,}000} \times 1.259 = 119.57 \text{ \$/kW}$$

4) $$\frac{(9.532+4.952)\times 10^6}{(189+224)\times 10^6} \times 1.064 = 0.0373 \text{ \$/kWh}$$

5) $$\frac{4.952 \times 10^6}{224\times 10^6} \times 1.064 = 0.0235 \text{ \$/kWh}$$

Firm energy value synthesized costs of both the gas turbine generator and the diesel engine generator. The secondary energy value was estimated from the diesel engine generator cost considering reducing the operation during high stream flow season, because in this season, the energy will be produced by the hydroelectric power stations instead of the diesel engine generator plants.

Fig. 9-3 Study on Reservoir Storage Volume (1)

Item	Unit	Upper Dam Site (Plan I) CA = 243.1km²					Lower Dam Site (Plan I) CA = 250.8km²				
		U - 1	U - 2	U - 3	U - 4	U - 5	L - 1	L - 2	L - 3	L - 4	L - 5
Dam Type		Rockfill Dam					Concrete Arch Dam				
High Water Level	m	1,181.7	1,195.0	1,204.7	1,213.1	1,220.2	1,170.7	1,184.7	1,194.7	1,203.1	1,210.1
Low Water Level	m	1,160	1,161	1,161	1,162	1,162	1,148	1,149	1,149	1,150	1,150
Available Drawdown	m	21.7	34.0	43.7	51.1	58.2	22.7	35.7	45.7	53.1	60.1
Gross Storage Capacity	$10^6 m^3$	16.32	26.69	36.69	47.06	57.06	16.61	26.88	36.88	47.16	57.16
Effective Storage Capacity	$10^6 m^3$	10	20	30	40	50	10	20	30	40	50
Dam Height	m	92	105	115	123	130	94	108	118	126	133
Annual Inflow	$10^6 m^3$	340.82					351.61				
Annual Power Discharge	$10^6 m^3$	220.55	280.36	307.18	326.49	333.02	223.40	284.62	313.11	334.38	342.05
Annual Spill	$10^6 m^3$	120.27	60.46	33.64	14.33	7.79	128.22	66.99	38.50	17.24	9.56
Firm Discharge	m^3/s	2.07	3.03	3.82	4.51	5.06	2.10	3.07	3.86	4.59	5.13
Maximum Discharge	m^3/s	10	15	18	22	24	10	15	18	22	24
Standard Intake Water Level	m	1,174	1,184	1,190	1,196	1,201	1,163	1,173	1,179	1,185	1,191
Tail Water Level	m	303									
Gross Head	m	871	881	887	893	898	860	870	876	882	888
Rated Effective Head	m	802.74	826.29	836.54	851.94	857.57	793.21	816.59	826.56	841.83	848.47
Installed Capacity	MW	68.9	106.3	129.2	160.8	176.5	68.0	105.1	127.6	158.9	174.7
Head Loss	m	68.26	54.71	50.46	41.06	40.43	66.79	53.41	49.44	40.17	39.53
Firm Peak Power	MW	67.87	103.28	124.76	154.50	170.07	66.92	102.39	123.96	154.82	168.33
Annual Firm Energy	GWh	123.85	188.49	227.69	281.96	310.38	122.14	186.86	226.22	282.55	307.20
Annual Secondary Energy	GWh	300.42	366.18	387.60	386.43	377.47	302.41	370.01	393.84	394.05	391.16
Annual Total Energy	GWh	424.28	554.66	615.29	668.40	687.86	424.55	556.87	620.06	676.60	698.36
Investment Cost	10^6 ¢	11,631	14,932	17,046	20,441	22,226	9,704	12,951	15,167	17,931	19,518
Annual Cost (C)	10^6 ¢	1,512	1,941	2,216	2,657	2,889	1,262	1,684	1,972	2,331	2,537
Annual Benefit (B)	10^6 ¢	1,623	2,292	2,663	3,115	3,336	1,612	2,286	2,663	3,135	3,336
Benefit Cost Ratio (B/C)	—	1.07	1.18	1.20	1.17	1.16	1.28	1.36	1.35	1.35	1.32
Surplus Benefit (B-C)	10^6 ¢	111	351	447	458	447	351	603	691	804	799
Unit Cost of Energy	¢/kWh	27	27	28	31	32	23	23	24	27	28

Table 9-4 Alternative Study (2)

Item	Unit	Plan IV				Plan V		Plan VI
		Rockfill Dam		Concrete Gravity Dam		Queb Seca	Queb Seca + Queb Napoleon	Concrete Arch Dam
Catchment Area	km^2	← 250.8 →				261.4	265.3	250.8
Annual Inflow	10^6 m^3	← 351.61 →				366.47	371.94	351.61
Annual Power Discharge	10^6 m^3	← 284.62 →				296.02	299.99	284.62
Annual Spill	10^6 m^3	← 66.99 →				70.45	71.95	66.99
Dam Type	—	Rockfill		Concrete Gravity		←	Concerte Arch	→
High Water Level	m	1,184.7	1,203.1	1,184.7	1,203.1	←	1,184.7	→
Low Water Level	m	1,149	1,150	1,149	1,150	←	1,149	→
Available Drawdown	m	35.7	53.1	35.7	53.1	←	35.7	→
Gross Storage Capacity	10^6 m^3	26.88	47.16	26.88	47.16	←	26.88	→
Effective Storage Capacity	10^6 m^3	20	40	20	40	←	20	→
Dam Height	m	110	128	108	126	←	108	→
Dam Volume	m^3	1,850.000	3,350,000	340,000	530,000	←	172,000	→
Tunnel Length	m	← 8,590 →						
Tunnel Diameter	m	2.5	3.1	2.5	3.1	2.6	2.6	2.5
Maximum Discharge	m^3/s	15	22	15	22	15.6	15.8	15
Standard Intake Water Level	m	1,173	1,185	1,173	1,185	1,173	1,173	1,173
Tail Water Level	m	← 303 →				303	303	340
Gross Head	m	870	882	870	882	870	870	833
Rated Effective Head	m	816.59	841.83	816.59	841.83	817.95	816.69	787.29
Installed Capacity	MW	105.1	158.9	105.1	158.9	109.4	110.7	101.3
Firm Peak Power	MW	102.39	154.82	102.39	154.82	103.02	103.23	98.72
Annual Firm Energy	GWh	186.86	282.55	186.86	282.55	188.00	188.38	180.16
Annual Secondary Energy	GWh	370.01	394.05	370.01	394.05	391.17	398.56	356.73
Annual Total Energy	GWh	556.87	676.60	556.87	676.60	579.17	586.94	536.89
Investment Cost	10^6 ¢	15,074	20,766	14,740	20,676	13,270	13,394	12,319
Annual Cost (C)	10^6 ¢	1,960	2,700	1,916	2,688	1,725	1,741	1,601
Annual Benefit (B)	10^6 ¢	2,286	3,135	2,286	3,135	2,337	2,354	2,204
Benefit Cost Ratio (B/C)	—	1.17	1.16	1.19	1.17	1.36	1.35	1.38
Surplus Benefit (B-C)	10^6 ¢	327	436	370	447	612	613	603
Unit Cost of Energy	¢/kWh	27	31	26	31	23	23	23

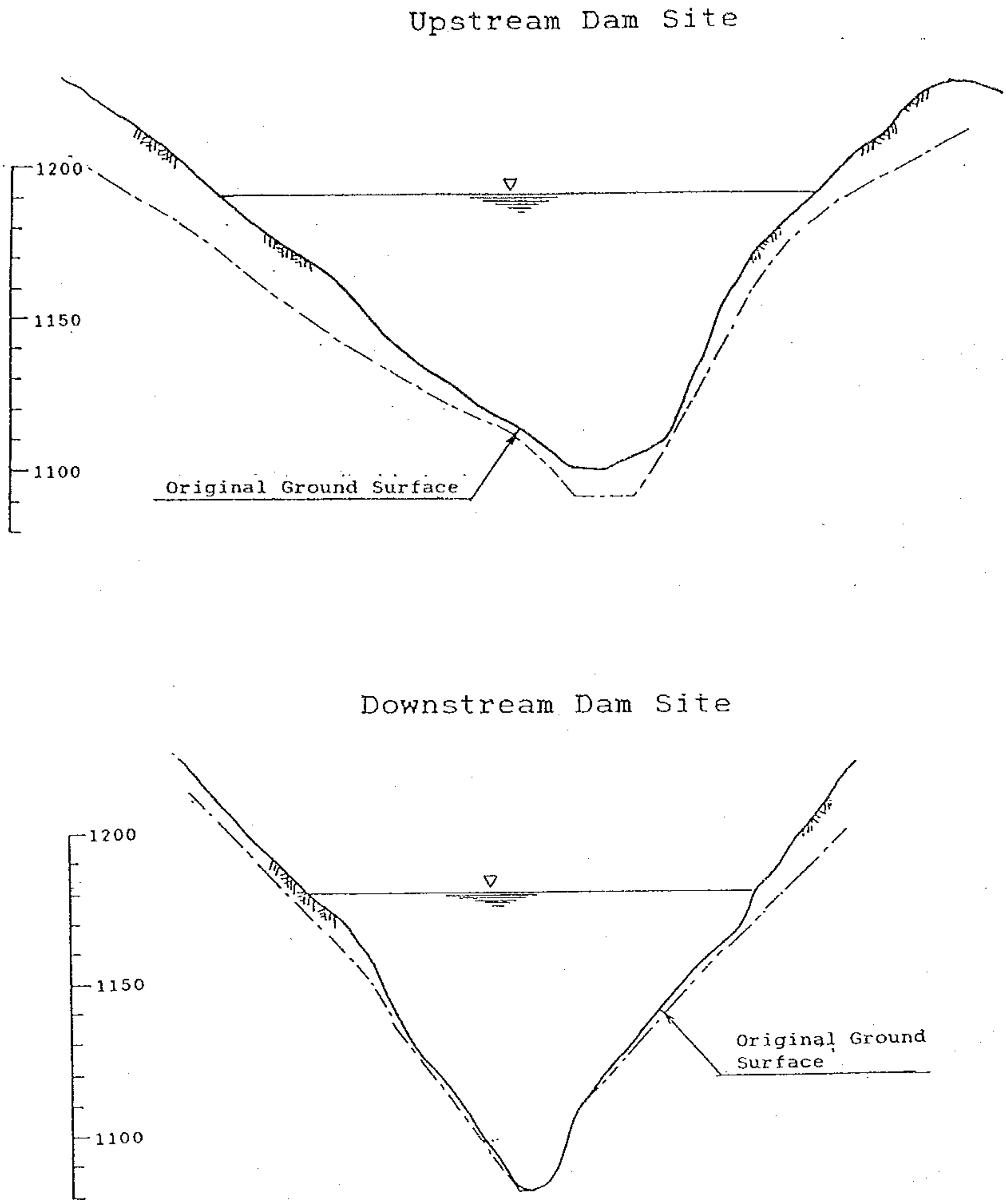
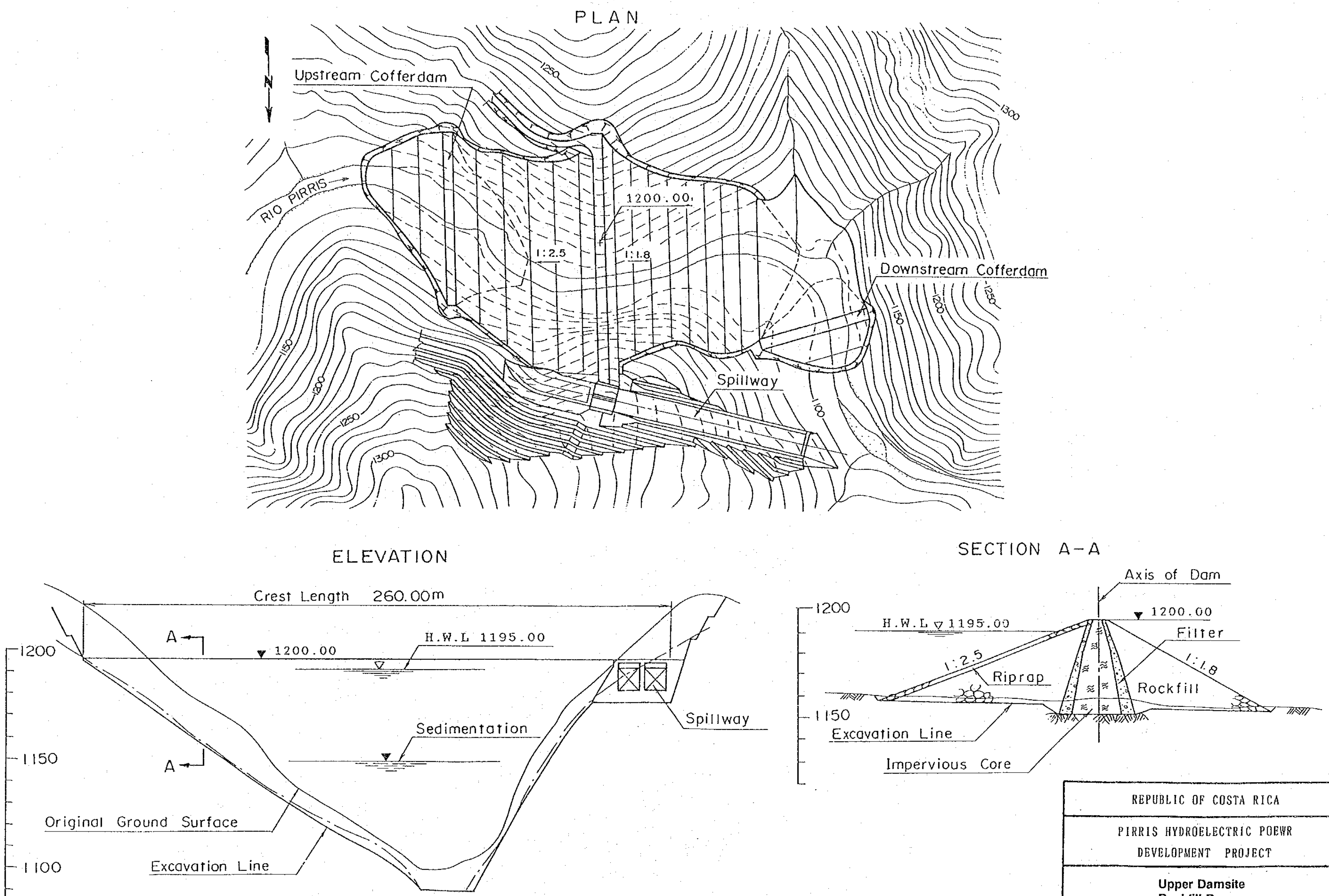


Fig. 9-4 Cross Section at Two Dam Sites

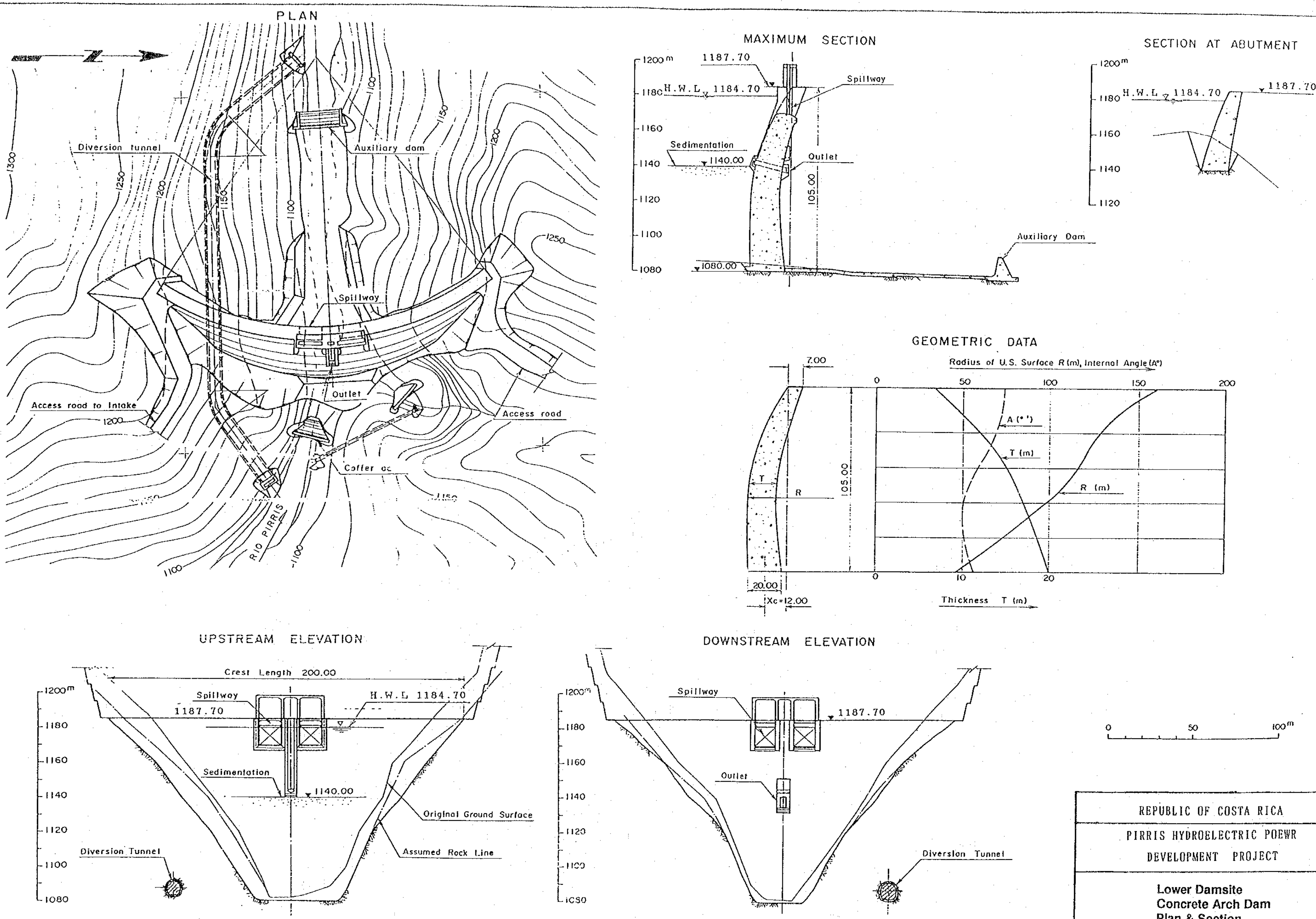


REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT

Upper Damsite Rockfill Dam Plan & Section (HWL 1195.0)

Fig. 9-5

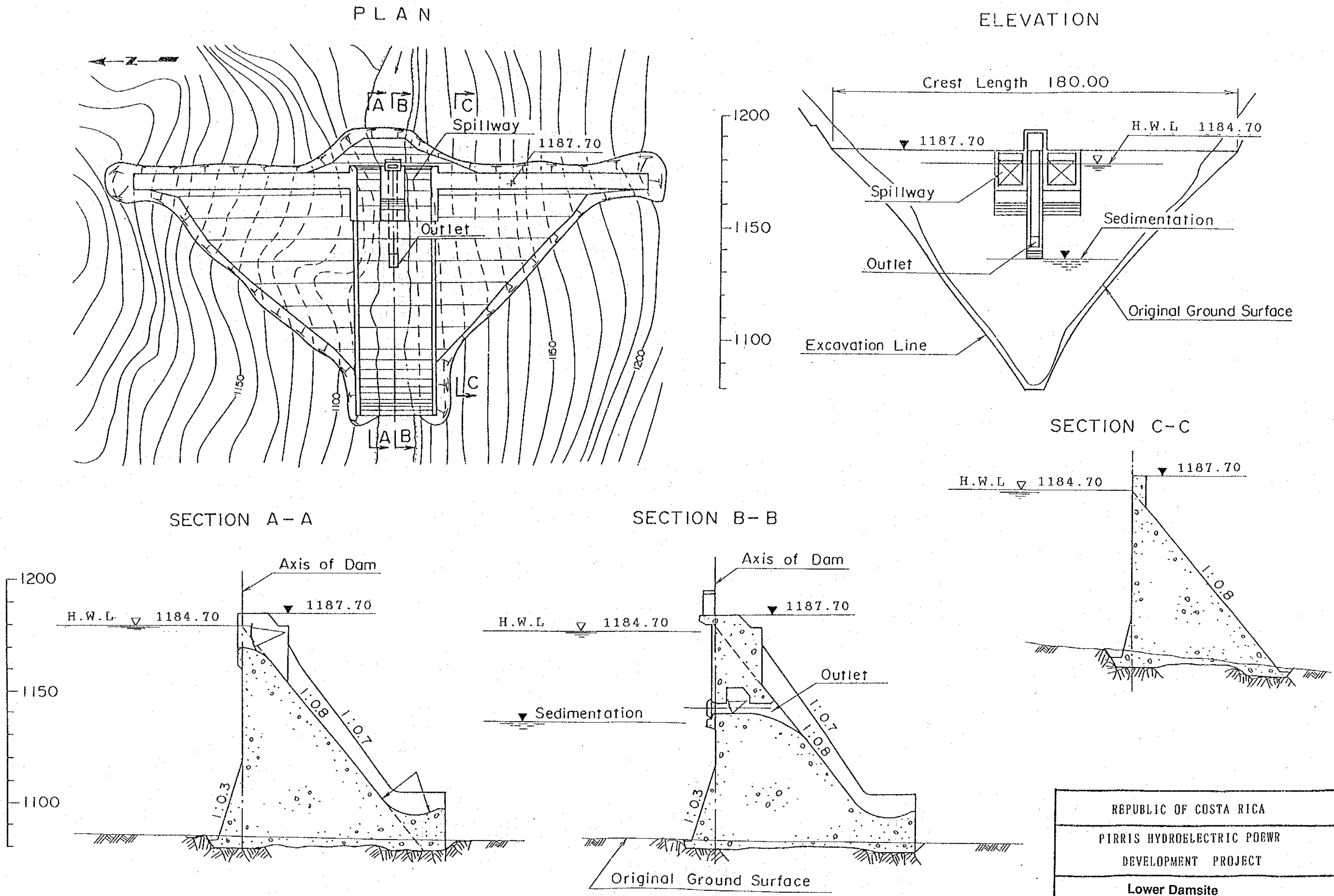


REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT

Lower Damsite
Concrete Arch Dam
Plan & Section
(HWL 1184.7)

Fig. 9-6

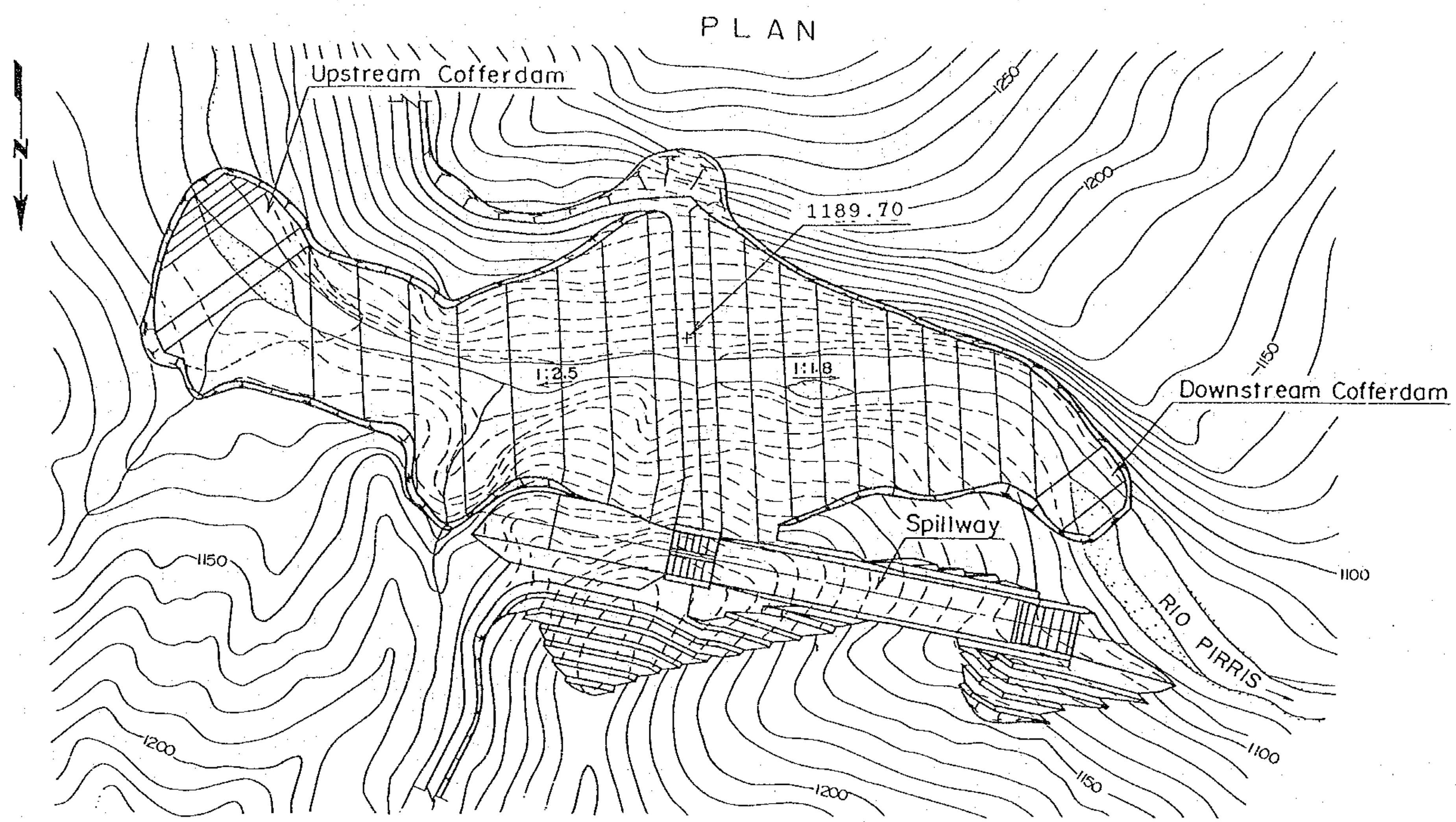


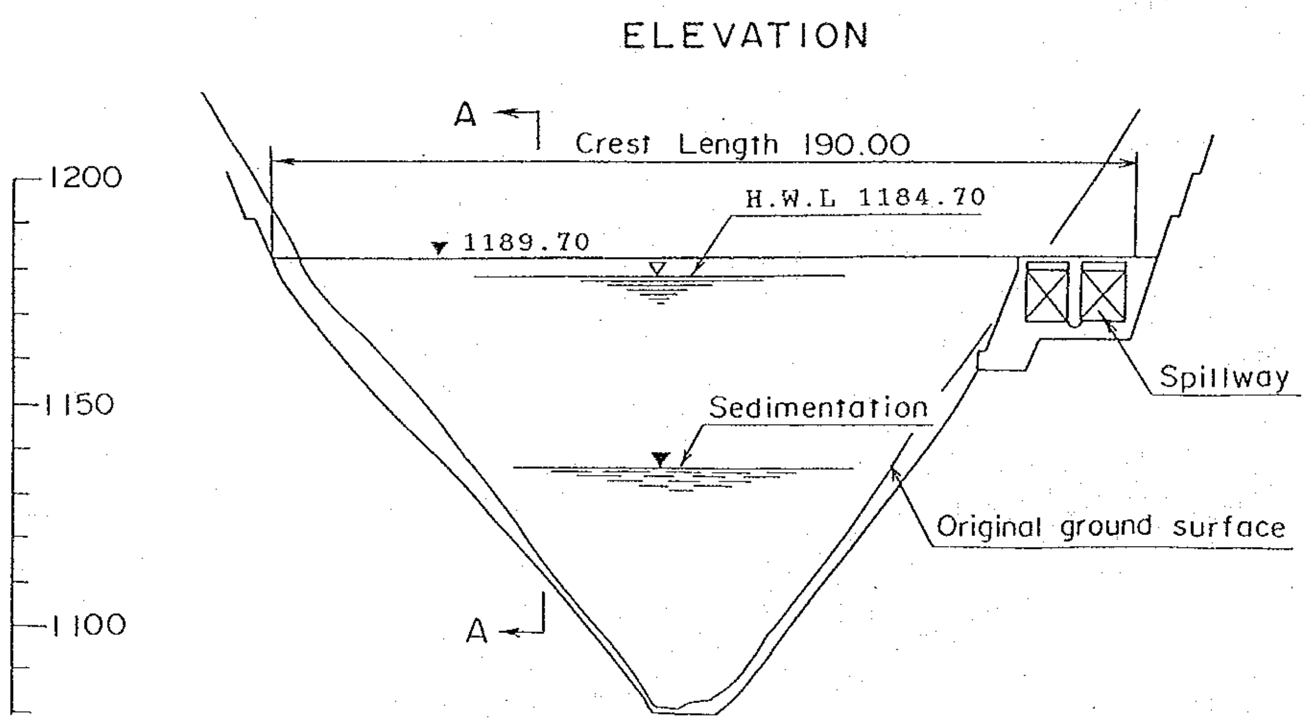
REPUBLIC OF COSTA RICA

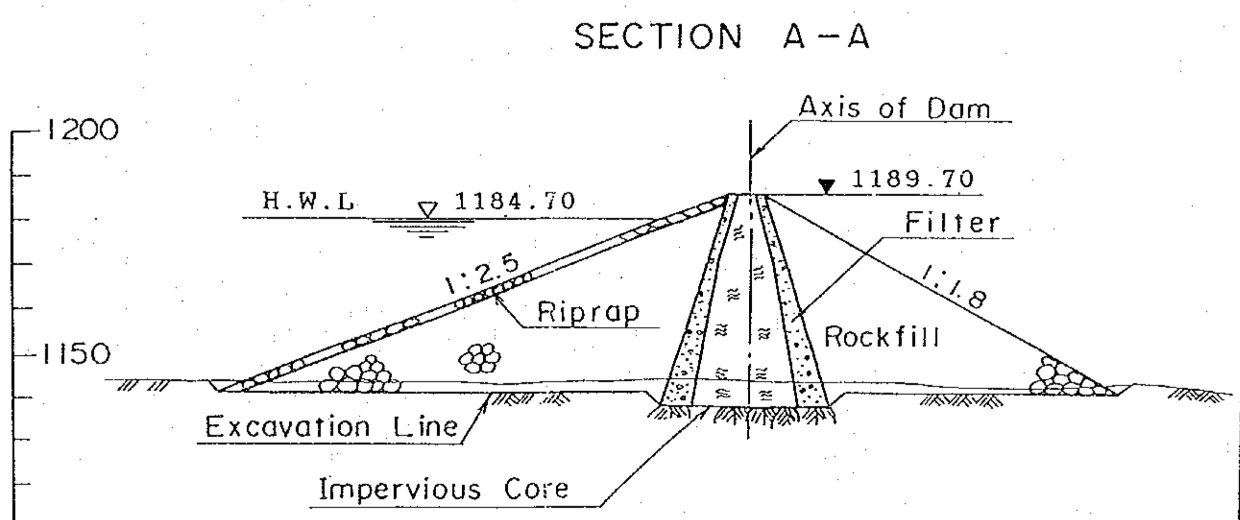
PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT

Lower Damsite
Concrete Gravity Dam
Plan & Section
(HWL 1184.7)

Fig. 9-7





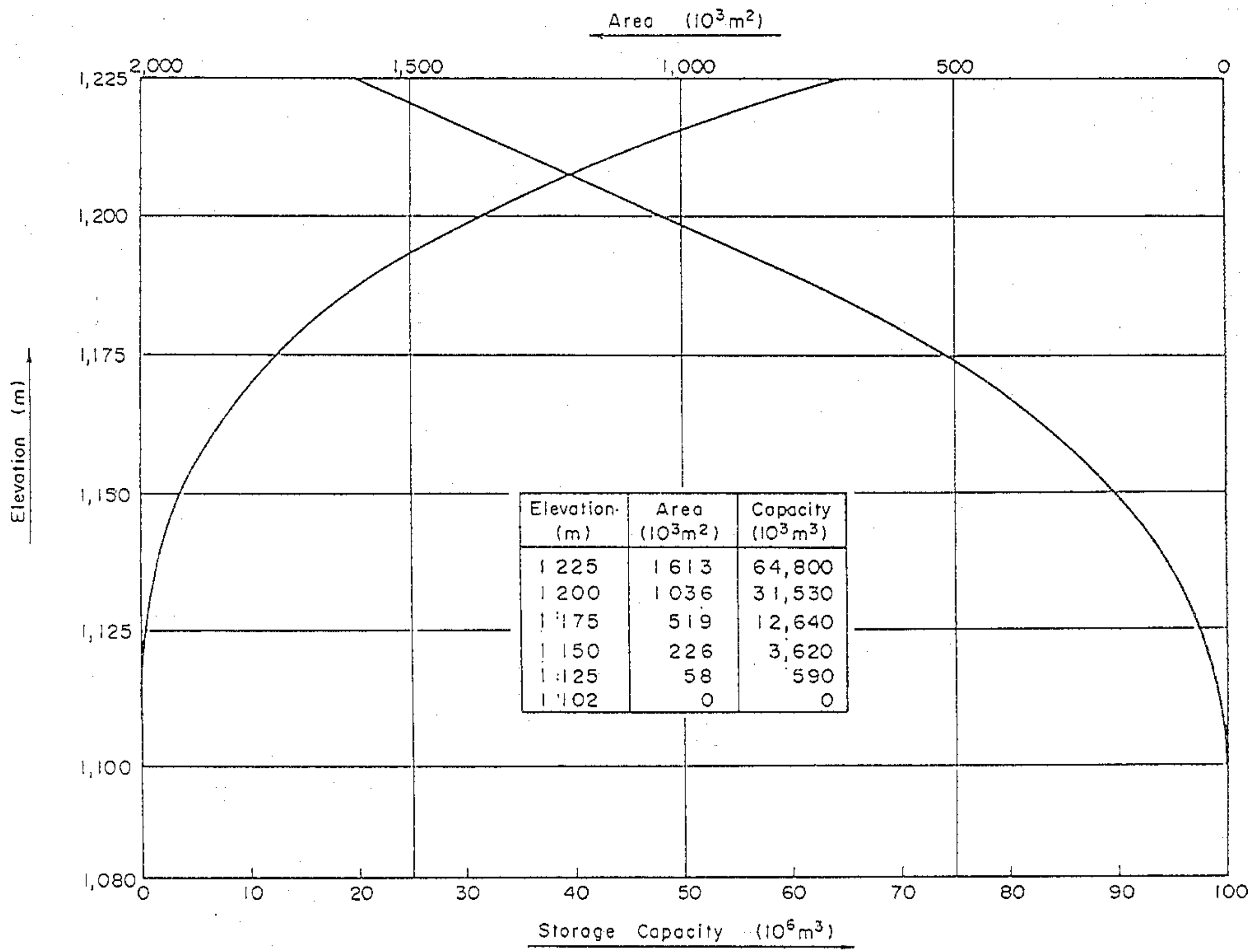


REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT

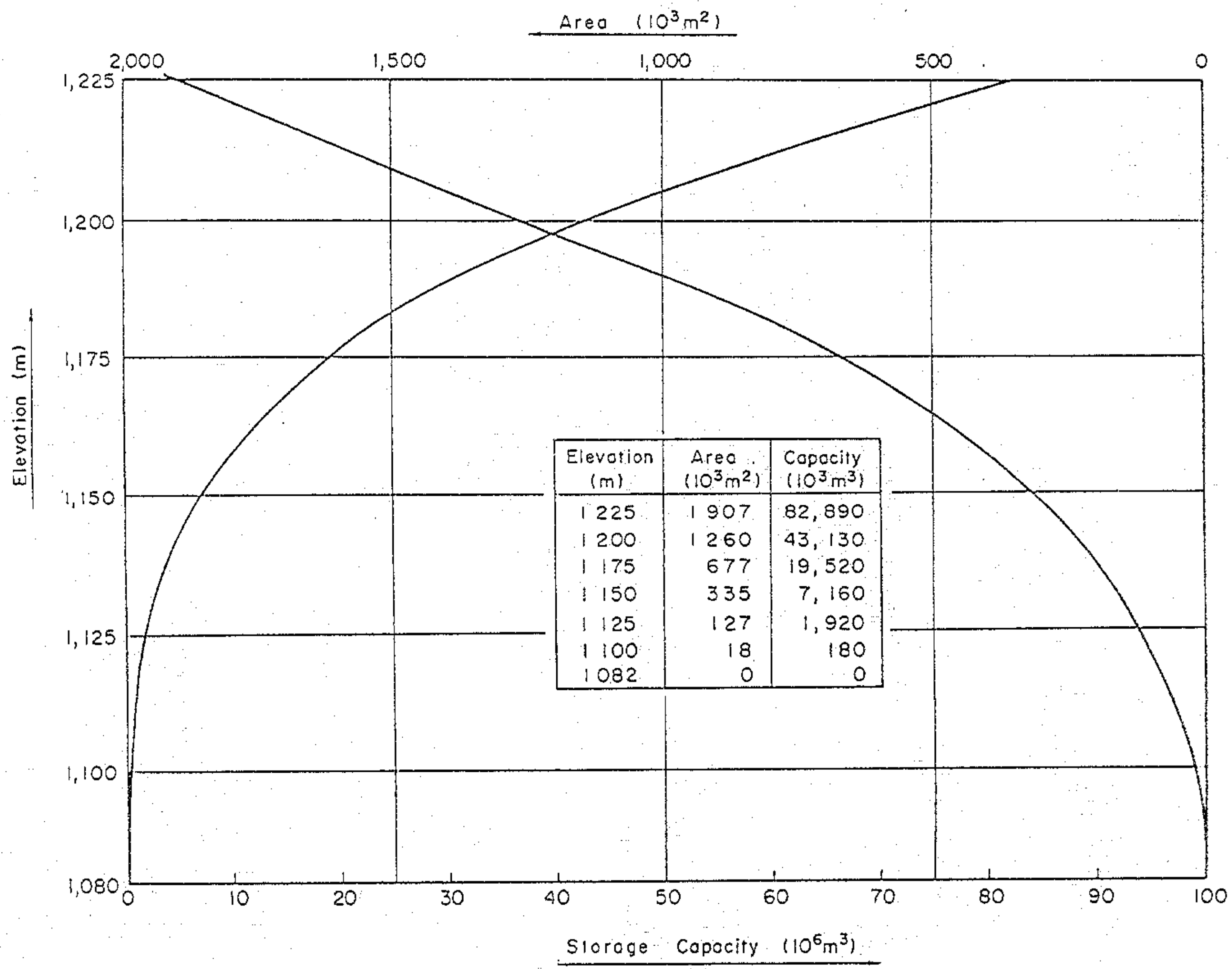
Lower Damsite
Rockfill Dam
Plan & Section
(HWL 1184.7)

Fig. 9-8



Elevation (m)	Area ($10^3 m^2$)	Capacity ($10^3 m^3$)
1 225	1 613	64,800
1 200	1 036	31,530
1 175	519	12,640
1 150	226	3,620
1 125	58	590
1 102	0	0

Fig. 9-9 Area-Capacity Curve (Upper dam site)



Elevation (m)	Area ($10^3 m^2$)	Capacity ($10^3 m^3$)
1 225	1 907	82,890
1 200	1 260	43,130
1 175	677	19,520
1 150	335	7,160
1 125	127	1,920
1 100	18	180
1 082	0	0

Fig. 9-10 Area-Capacity Curve (Lower dam site)

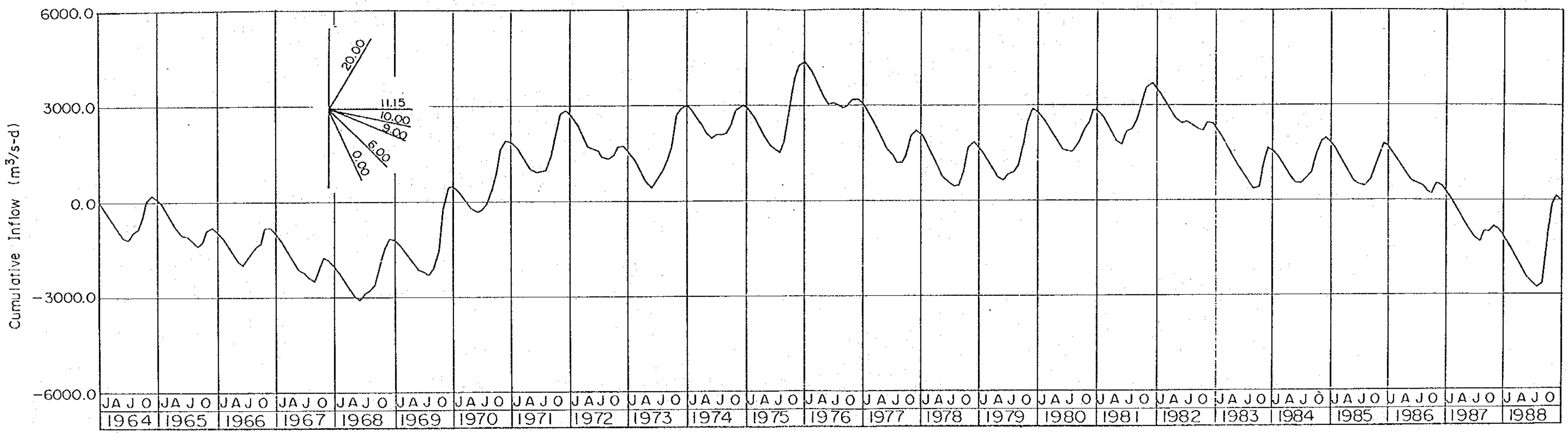


Fig. 9-11 Mass Curve at Dam Site (Lower dam site)

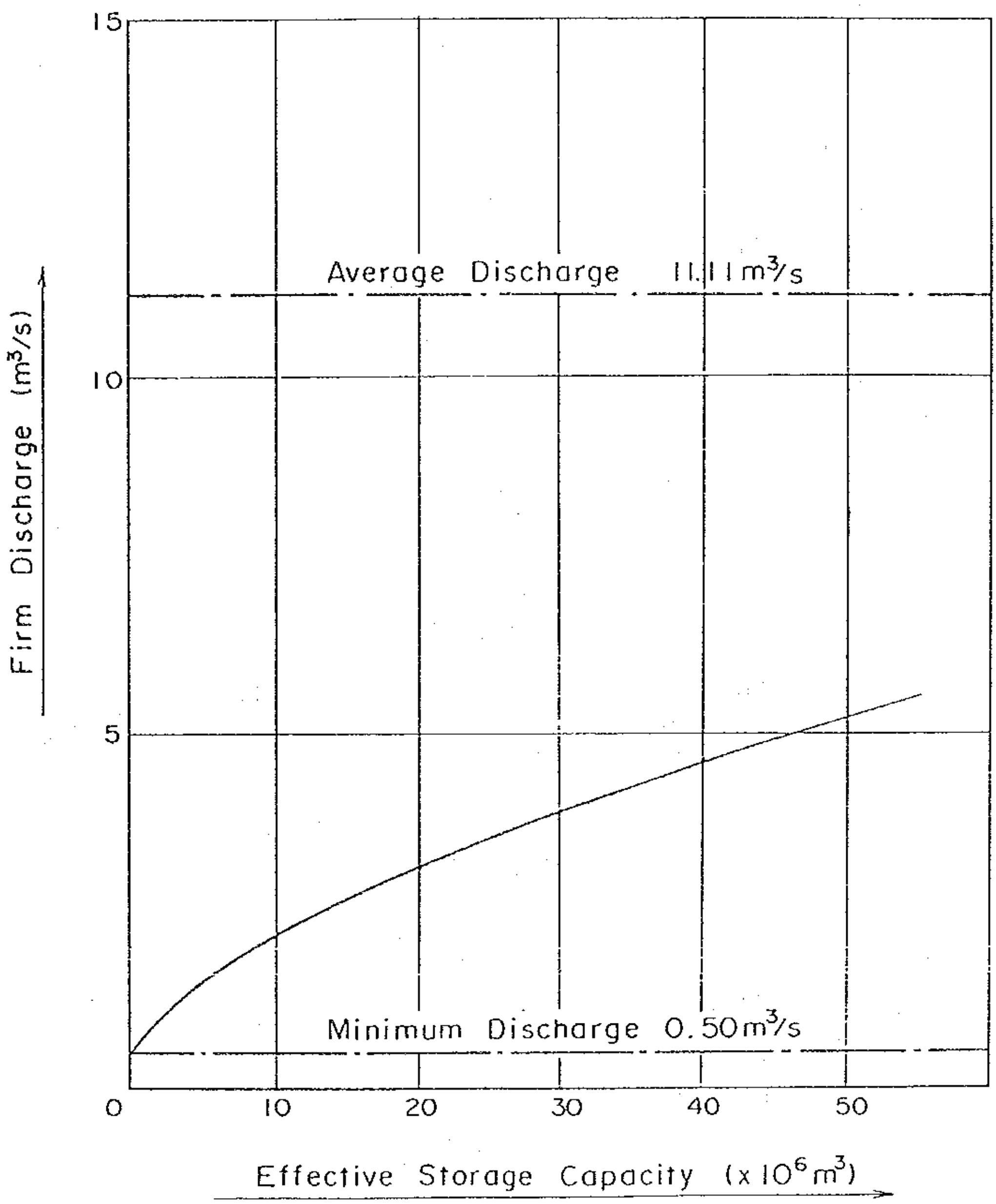


Fig. 9-12 Firm Discharge and Effective Storage Capacity (Lower dam site)

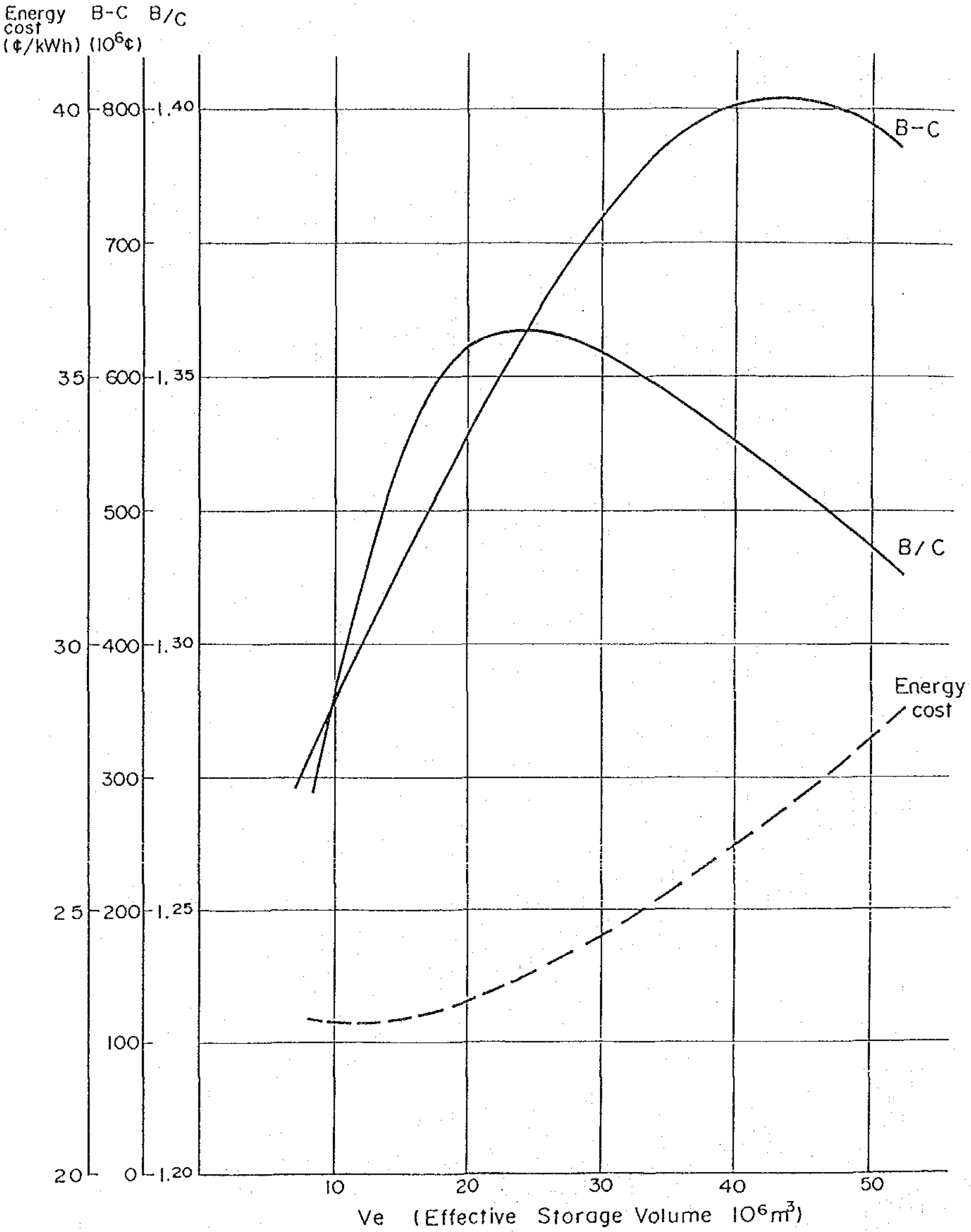


Fig. 9-13 Study on Reservoir Storage Volume (1)
(Lower dam site, Concete Arch Dam)

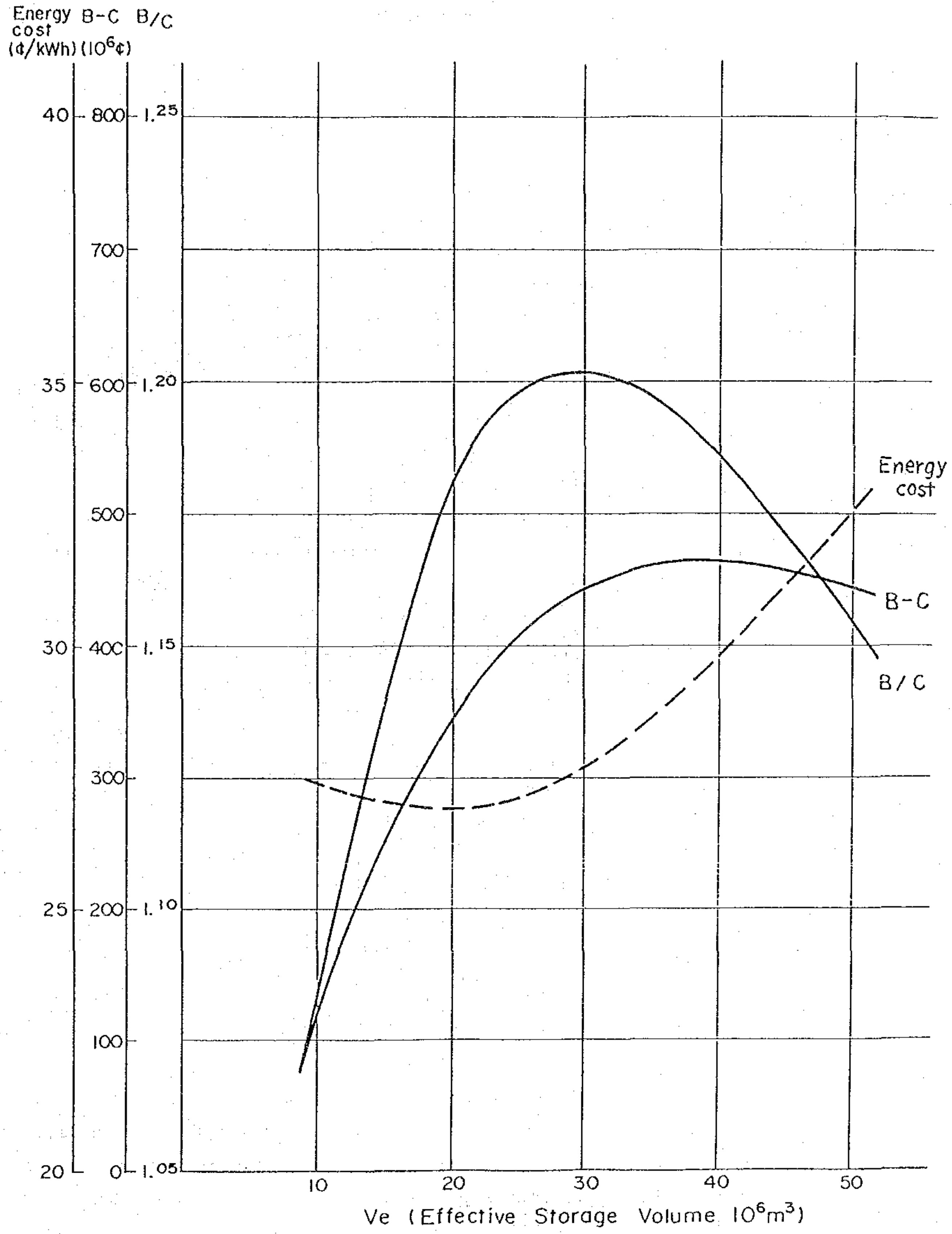


Fig. 9-14 Study on Reservoir Storage Volume (1)
(Upper dam site, Rockfill Dam)

9.3 Comparison Studies of Development Plan (Second Study)

9.3.1 Basic Conditions for Study

(1) Comparison Study by Equivalent Discount Rate (EDR)

In the Second Study, equivalent discount rate (EDR) according to the discounted cash flow (DCF) method was used as an index in addition to the annual cost method used in the Primary Study. The basic concept, similarly to the examination in the Primary Study, considered a combined power plant of gas turbine and diesel engine as the alternative facilities, and an alternative thermal capable of furnishing equivalent service (effective output and effective electric energy) to the hydro was assumed for each case, and the cost of this was taken to be the benefit of hydro.

The concrete calculations of EDR were carried out seeking a discount rate (equivalent discount rate = EDR) with which, from the cash flow developing construction cost and annual expense by year for the project life (50 years), the total of the present values of the cost of hydro and the cost of thermal (benefit) in the initial year of the Project will be equal.

Equivalent Discount Rate (EDR)

$$\sum_{i=1}^{n} \frac{Bi - Ci}{(1 + r)^{i}} = 0$$

where, Bi	:	benefit in i-th year
Ci	:	cost in i-th year
r	:	discount rate (EDR)
n	:	period of calculation

The costs in the cash flow consist of the cost of the Project incurred during the construction period expressed in terms of market prices and the operation and maintenance cost and fuel cost after start of operation, while the costs of invested capital such as interest and depreciation are omitted.

The basic conditions for calculation of EDR are as indicated in the table below.

Item		Hydro	Thermal (Gas)	Thermal (Diesel)
Station Service Rate	kW	0.3 %	6.0 %	6.0 %
	kWh	0.3	6.0	6.0
Forced Outgage Rate		0.3	4.0	4.0
Scheduled Outgage Rate		2.0	12.0	12.0
Transmission Loss Rate		-	-	-
(not considered)	kWh	-	-	-
Construction Period		5 years	2 years	2 years
Service Life (Civil Structure)		50	15	25
Service Life (Hydraulic Equipment)		35	-	-
Service Life (Electric Equipment)		35	-	-
O & M Cost Rate		1%	3.41%	1.85%
Thermal Unit Construction Cost		-	1) 397 $/kW*	2) 892 $/kW*
Thermal Unit Fuel Cost		-	3) 0.0499 $/kWh	4) 0.0219 $/kWh
Year of Cost Calculated		1989 : 1 $ = 83 ¢		

* Excluding interest during construction

1) Standard thermal gas turbine (2 x 36 MW) construction cost: $28,593,741 (excluding interest during construction)

Standard thermal gas turbine construction cost per kW $397/kW (= 28,593,741/2 x 36,000)

2) Standard thermal diesel (32 MW) construction cost:

$28,534,241 (excluding interest during construction)

Standard thermal diesel construction cost per kW
$892/kW (= 28,534,241/1 x 32,000)

3) Standard thermal (Gas turbine)

Fuel cost	:	\$9.426 x 10^6 (189 GWh)
Fuel cost/kWh	:	\$0.0499/kWh (= 9.426/189)

4) Standard thermal (Diesel)

Fuel cost	:	\$4.896 x 10^6 (224 GWh)
Fuel cost/kWh	:	\$0.0219/kWh (= 4.896/224)

9.3.2 Dam Site and Dam Type

It is necessary to carry out a comprehensive geological evaluation taking into consideration the results of detailed investigation works in selecting the dam site and the dam type.

The geology at the right bank of the downstream dam site was found to be poorer than first expected, with numerous cracks and joints existing, judging from the results of geological evaluations at the intermediate stage of detailed investigation works. Further, the downstream side of the thin ridge corresponding to the right-bank abutment presents a topography similar to a valley, and it was found that the rock mass of this mountain body was thinner than expected. Consequently, a concrete arch gravity dam is considered to be suitable as the dam type in place of a concrete arch dam. Concrete arch gravity dam will cover the topographical and geological defects of the rightbank abutment of the downstream dam site. The economics of the Project in case of adopting this dam type were examined as shown in attached Table 9-5 and Figure 9-18. According to the results of this examination, in case a concrete arch gravity dam is adopted at the downstream dam site, it will be more economical than when a concrete gravity dam is adopted at the downstream dam site or in a case of a rockfill dam at the upstream dam site.

Though the possibility for adoption of a concrete arch dam at the downstream dam site is limited, it needs to be judged based on the results of further geological investigations and geological evaluations. Consequently, at the present feasibility study stage, it was judged suitable for considerations to be made on the conservative side by adopting a concrete arch gravity dam for the downstream dam site.

9.3.3 Examination of Development Scale

The examination of the development scale was made varying the scale at different levels in the course of studies for the dam site and dam type.

In case of adopting a concrete arch gravity type dam at the downstream dam site, it is thought an effective capacity of 30 to 40 x 10^6 m^3 would be economically suitable as the apparent development scale. However, it is thought a reservoir scale of effective capacity about 30 x 10^6 m^3, H.W.L. about 1,195 m, and dam height about 120 m will be suitable, judging from the conditions of topography and geology at the right-bank of the dam site, as stated in the preceding section.

9.3.4 Studies of Maximum Discharge and Peak Hours

It is necessary for the maximum available discharge and installed capacity of Pirris Power Station to be selected for maximum economy giving consideration to the site characteristics and peak duration. If installed capacity were to be made too large, the equipment would be excessively large compared with the dependable peak output, and with capacity becoming latent, the economics would be worsened. If the installed capacity were to be excessively small, the dependable peak output would be restricted by the installed capacity, and the peak hours would be too long. The minimum peak duration was made 5 hours considering the electric power demand and supply balance, and for Plan I of effective storage capacity of about 30 x 10^6 m^3 (HWL 1,195 m), studies were made of 5 cases varying maximum discharge from 12 to 24 m^3/s. Examinations were also made for peak durations of 7 hours and 9 hours.

The results of study are shown in Table 9-6 and Figs. 9-19 and 9-20. As a result, it was found that when specifying reservoir scale at an effective storage capacity of about 30 x 10^6 m^3 (HWL 1,195 m), a maximum discharge of 18 m^3/s would be optimum.

Therefore, maximum discharge 18 m^3/s and installed capacity 128 MW were decided as the scale of Pirris power plant.

9.3.5 Selection of Main Equipment

The study described below was made in deciding on the number of main units to be provided for the installed capacity of Pirris power plant of 128 MW.

The maximum output per unit of main equipment as seen from the standpoint of power system operation is determined by the capacity of the power system. When a main equipment has interrupted out from the power system due to repairs or trouble, it is necessary for the drop in frequency caused by the interruption not to seriously affect the stability of the system.

With thermal units in the electric power system of ICE, continuous operation will be possible if frequency drop will not be greater than 1.5 Hz. However, at more than this, when unit capacity is 125 MW or less, operation will be possible for about several minutes at -2.5 Hz, but at -3.0 Hz, it will be necessary for that generator to be cut off from the power system.

Frequency change in an electric power system is calculated by the following equation:

$$\Delta F = \frac{-1}{K} x \frac{\Delta P}{P} x\, 100 \ (Hz)$$

where, ΔF : Frequency change of system (Hz)

ΔP : Unit capacity or load of generator concerned (MW)

P : Overall load of system (MW)

k : System constant (KG + KL) (1% MW/0.1 Hz) 100% MW/10 Hz

KG : Frequency characteristic of generator (1% MW/0.1 Hz) 100% MW/10 Hz

KL : Frequency characteristic of load

For the frequency drop to stay within 1.5 Hz when the above equation is used, it will be necessary for the allowable unit capacity to be less than approximately 10% of the system load. However, system load is something which is varying all of the time, and cases of midnight loads in the off-peak season dropping below 30% of the maximum load of the year are not rare. Consequently, it is not reasonable for the unit capacity to be made approximately 10% of the annual maximum load.

In the scenario for the year 2001 when Pirris power plant is to be commissioned in the system, if the frequency drop were to be limited to 1.5 Hz for the case of annual maximum load and minimum midnight load in off-peak season, with interruption of a unit generator from the system due to accident during operation at 50% of unit capacity, the unit capacity of the generator may be determined as follows:

(1) At Peak Load

$$\Delta P = \frac{\Delta F \times K \times P}{100} \ (MW) = 200.4 \ (MW)$$

where, P : 1,336 (MW)

ΔF : 1.5 (Hz)

K : 1% MW/0.1 Hz 100%/10 Hz

(2) At Off-peak Load

$$\Delta P = \frac{\Delta F \times K \times P}{100} \ (MW) = 60.1 \ (MW)$$

where, P : 400.8 (MW)

ΔF : 1.5 (Hz)

K : 100%/10 Hz

Therefore, the unit capacity of not more than 60.1 x 2 = 120.2 (MW) is preferable.

Based on the above calculation results, the checkpoints regarding unit capacity of the projected power plant as seen from the point of view of system stability will be as follows:

(a) Whether or not the frequency drop will be in the allowable range when the particular generator in operation interrupting from the system not only at peak load, but also at off-peak load.

(b) When not in the allowable range, whether or not measures to be taken in operation (partial load operation, localized load shutdown) are clearly specified.

(c) Whether or not the above measures can be implemented in a normal manner for the particular country.

The following may be said based on the results of the above examination.

A plan for 1 unit (128 MW) would provide economy of scale, but it is desirable to have a unit capacity of not more than 120.2 MW from the standpoints of system stability and power supply operation.

A plan for 3 units would pose no problem regarding system stability, but construction cost will be higher compared with proposals for 1 and 2 units.

A plan for 2 units will pose no problem regarding system stability, while there will be no problems about degree of freedom in operation at low load, manufacture, transportation limits, power supply operation, etc.

Hence, 2 units will be suitable as the number of units of main equipment.

9.3.6 Optimum Development Plan

The specifications of the optimum development plan decided as a result of studies up to this point are as given below.

Reservoir high water level	1,195.0 m
Reservoir low water level	1,149.0 m
Available drawdown	46.0 m
Sedimentation level	1,140.0 m
Total storage capacity	37.47×10^6 m^3
Effective storage capacity	30.59×10^6 m^3
Normal intake water level	1,179.7 m
Normal tail water level	304.5 m
Maximum discharge	18 m^3/s
Firm discharge	3.9 m^3/s
Gross head	875.2 m
Normal effective head	830.7 m
Installed capacity	128 MW
Firm peak power	126 MW
Number of main equipment units	2 units
Annual total energy	609.3 GWh (Rule curve)
Annual firm energy	230.0 GWh
Annual secondary energy	379.3 GWh

Electric energy production with this optimum development plan was again calculated. The electric energy production calculation at the time of making comparison studies of development plans was done for reservoir operation using the principle of optimality according to the dynamic programming technique, determining electric energy production applying the ideal reservoir operation of deciding the discharge of each month for total electric energy production under the conditions given to be a maximum for each case.

However, in actual reservoir operation, it is unavoidable for the operation to be done setting up some kind of rule concerning unknown future inflows, and the electric energy production obtained here would be small in comparison with what is obtained by ideal operation.

Here, electric energy production calculations close to actual were carried out on the optimum development plan setting up a rules curve for reservoir operation.

The regulating rate of Pirris Reservoir is low (approximately 8%), and the reservoir operation is close to that of an annual storage-type reservoir.

The seasonal variation in dam inflow is great, and in the low-water season (December-April) it is necessary to secure a firm discharge using the reservoir storage, along with which it is necessary to minimize overflow during the high-water season (May-November) as much as possible.

In electric energy calculations in comparison studies of development plans, the reservoir water level will annually fluctuate between high water level and low water level, and large amounts of overflow will occur during the flood season.

Therefore, in setting up rules, securing firm discharge in the low-water season is considered, and at the start of supply, reservoir operation is done bringing the reservoir water level as close as possible to high water level, and in the high-water season, effectively utilizing inflow from around low water level, to keep overflow in September and October as small as possible. The procedure of electric energy production calculations is shown in Fig. 9-21 and the rules curve in Fig. 9-22.

Evaporation from the reservoir surface was ignored.

The standard discharge water level was set considering flood water level of 301.3 m for outlet flood discharge (2,270 m^3/s) and a water level to clear this was set against the Pelton turbine center elevation (304.5 m).

The inflows, available discharges, overflow quantities when operating Pirris Reservoir during a 25-year period from May 1964 to April 1989 are given in Table 9-7. And the monthly storage quantities, and supply quantities are shown in Fig. 9-23, the monthly electric energy productions and monthly outputs of Pirris Power Station in Tables 9-8 and 9-9, and variations in monthly electric energy productions in Fig. 9-24.

Fig. 9-5 Study on Reservoir Storage Volume (2)

Item \ Site and Dam Type	Unit	Rock Fill Dam (Upper Dam Site)					Concrete Arch Dam (Lower Dam Site)				
		U-1	U-2	U-3	U-4	U-5	L-1	L-2	L-3	L-4	L-5
Reservoir											
Effective Storage Volume	$10^6 m^3$	10	20	30	40	50	10	20	30	40	50
High Water Level	m	1,181.7	1,195.0	1,204.7	1,213.1	1,220.2	1,170.7	1,184.7	1,194.7	1,203.1	1,210.1
Dam Height	m	92	105	115	123	130	94	108	118	126	133
Power Generation											
Installed Capacity	MW	68.9	106.3	129.2	160.8	176.5	68.0	105.1	127.6	158.9	174.7
Firm Peak Power	MW	67.87	103.28	124.76	154.50	170.07	66.92	102.39	123.96	154.82	168.33
Annual Energy	GWh	424.28	554.66	615.29	668.40	687.86	424.55	556.87	620.06	676.60	698.36
Annual Firm Energy	GWh	123.85	188.49	227.69	291.96	310.38	122.14	186.86	226.22	282.55	307.20
Annual Secondary Energy	GWh	300.42	366.18	387.60	386.43	377.47	302.41	370.01	393.84	394.05	391.16
Economy											
Construction Cost	10^6 ¢	11,631	14,932	17,046	20,441	22,226	9,704	12,951	15,167	17,931	19,518
Annual Cost (C)	10^6 ¢	1,512	1,941	2,216	2,657	2,889	1,262	1,684	1,972	2,331	2,537
Annual Benefit (B)	10^6 ¢	1,623	2,292	2,663	3,115	3,336	1,612	2,286	2,663	3,135	3,336
Surplus Benefit (B-C)	10^6 ¢	111	351	447	458	447	351	603	691	804	799
Benefit Cost Ratio (B/C)	–	1.07	1.18	1.20	1.17	1.16	1.28	1.36	1.35	1.35	1.32
E D R	%	15.6	18.2	18.8	18.7	18.5	20.0	22.2	22.3	22.9	22.5
Unit Construction Cost	¢/kWh	27.4	26.9	27.7	30.6	32.3	22.9	23.3	24.5	26.5	28.0

Item \ Site and Dam Type	Unit	Concrete Arch Gravity Dam (Lower Dam Site)					Concrete Gravity Dam (Lower Dam Site)				
			LAG-2	LAG-3	LAG-4	LAG-5	LC-1	LC-2	LC-3	LC-4	LC-5
Reservoir											
Effective Storage Volume	$10^6 m^3$		20	30	40	50	10	20	30	40	50
High Water Level	m		1,184.7	1,194.7	1,203.1	1,210.1	1,170.7	1,184.7	1,194.7	1,203.1	1,210.1
Dam Height	m		108	118	126	133	94	108	118	126	133
Power Generation											
Installed Capacity	MW		105.1	127.6	158.9	174.7	68.0	105.1	127.6	158.9	174.7
Firm Peak Power	MW		102.39	123.96	154.82	168.33	66.92	102.39	123.96	154.82	168.33
Annual Energy	GWh		556.87	620.06	676.60	698.36	424.55	556.87	620.06	676.60	698.36
Annual Firm Energy	GWh		186.86	226.22	282.55	307.20	122.14	186.86	226.22	282.55	307.20
Annual Secondary Energy	GWh		370.01	394.05	394.05	391.16	302.41	370.01	393.84	394.05	391.16
Economy											
Construction Cost	10^6 ¢		14,455	16,852	19,915	21,850	10,545	14,740	17,380	20,676	22,720
Annual Cost (C)	10^6 ¢		1,879	2,191	2,589	2,841	1,371	1,916	2,259	2,688	2,954
Annual Benefit (B)	10^6 ¢		2,286	2,663	3,135	3,336	1,612	2,286	2,663	3,135	3,336
Surplus Benefit (B-C)	10^6 ¢		407	472	546	496	241	370	403	447	383
Benefit Cost Ratio (B/C)	–		1.22	1.22	1.21	1.18	1.18	1.19	1.18	1.17	1.13
E D R	%		19.1	19.3	19.7	19.1	17.9	18.6	18.6	18.7	18.0
Unit Construction Cost	¢/kWh		26.0	27.2	29.4	31.3	24.8	26.5	28.0	30.6	32.5

Table 9-6 Study on Optimum Maximum Discharge and Peak Duration

Item	Unit	Lower Dam Site (Plan I)														
		LAG-6	LAG-7	LAG-8	LAG-9	LAG-10	LAG-11	LAG-12	LAG-13	LAG-14	LAG-15	LAG-16	LAG-17	LAG-18	LAG-19	LAG-20
Maximum Discharge	m³/s		12			15			18			21			24	
Minimum Peak Hour	hr	5	7	9	5	7	9	5	7	9	5	7	9	5	7	9
Annual Inflow	10⁶ m³								351.61							
Annual Power Discharge	10⁶ m³		264.80			292.39			313.38			327.11			336.17	
Annual Spill	10⁶ m³	71.33	71.33	72.65	46.24	47.14	48.31	27.83	30.78	29.31	19.80	21.17	19.72	16.74	16.05	14.21
Dam Type	—								Concerte Arch Gravity Dam							
High Water Level	m								1,195							
Low Water Level	m								1,149							
Available Drawdown	m								46							
Gross Storage Capacity	10⁶ m³								37.47							
Effective Storage Capacity	10⁶ m³								30.59							
Dam Height (Concrete Arch Gravity)	m								120							
Gross Head	m								875.20							
Rated Effective Head	m		817.00			825.90			830.70			835.40			836.00	
Installed Capacity	MW		84.00			106.00			128.00			150.00			172.00	
Head Loss	m		58.20			49.30			44.50			39.80			39.20	
Firm Peak Power	MW	81.03	80.48	73.77	102.63	95.27	76.13	123.33	96.91	76.36	134.62	98.05	77.98	135.85	99.28	76.52
Annual Firm Energy	GWh	147.87	205.63	242.33	187.30	243.41	250.09	225.07	247.61	250.86	245.68	250.51	256.18	247.93	253.66	251.36
Annual Secondary Energy	GWh	370.98	313.25	277.73	391.44	336.26	331.06	398.30	377.81	375.37	410.80	407.42	402.10	429.68	424.77	427.11
Annual Total Energy	GWh	518.85	518.87	520.06	578.74	579.68	581.15	623.37	625.43	626.23	656.48	657.93	658.27	677.61	678.44	678.46
Investment Cost	10⁶ ¢		14,196			15,634			16,899			18,514			19,236	
Annual Cost (C)	10⁶ ¢	1,845	1,845	1,845	2,032	2,032	2,032	2,197	2,197	2,197	2,407	2,407	2,407	2,501	2,501	2,501
Annual Benefit (B)	10⁶ ¢	1,961	2,022	2,001	2,332	2,326	2,152	2,662	2,436	2,242	2,859	2,513	2,327	2,914	2,569	2,346
Benefit Cost ratio (B/C)	—	1.06	1.10	1.08	1.15	1.14	1.06	1.21	1.11	1.02	1.19	1.04	0.97	1.17	1.03	0.94
Surplus Benefit (B-C)	10⁶ ¢	116	176	156	299	294	119	465	239	46	452	107	△ 80	413	68	△154
Unit cost of energy	¢/kWh	27	27	27	27	27	27	27	27	27	28	28	28	28	28	28

Table 9-7 Summary of Operation Study of Pirris Reservoir (1/2)

UNIT : 10**6 M3

YEAR		MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	TOTAL
1964	QIN	9.4	25.2	48.8	41.3	56.7	79.7	41.6	17.6	12.1	8.2	7.9	7.3	355.8
	QUS	13.4	13.0	47.1	47.2	45.6	46.7	41.6	17.6	12.1	9.4	13.4	13.0	320.1
	QOV	0.0	0.0	0.0	0.0	0.0	31.0	0.0	0.0	0.0	0.0	0.0	0.0	31.0
1965	QIN	13.1	26.1	16.2	16.7	41.8	63.0	33.6	18.2	12.7	9.1	8.3	7.7	266.5
	QUS	13.4	13.4	43.0	20.2	28.2	47.2	32.7	18.2	12.7	9.4	13.4	13.0	264.8
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1966	QIN	17.5	46.6	44.7	47.3	37.9	73.1	28.9	16.3	11.8	8.4	7.9	8.7	349.0
	QUS	13.4	40.0	46.7	46.7	45.4	46.9	28.9	16.3	11.8	9.4	13.4	13.0	331.9
	QOV	0.0	0.0	0.0	0.0	0.0	17.2	0.0	0.0	0.0	0.0	0.0	0.0	17.2
1967	QIN	7.4	21.6	15.7	19.3	58.3	62.1	24.1	15.6	10.4	9.1	7.4	7.5	258.7
	QUS	13.4	13.0	34.5	20.9	46.3	47.2	22.6	15.6	10.4	9.8	13.4	13.0	260.1
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1968	QIN	21.1	44.5	38.9	44.6	80.4	76.3	56.3	26.4	16.5	12.2	12.2	10.7	439.9
	QUS	13.4	40.0	46.9	47.2	45.4	46.7	45.2	26.4	16.5	12.2	13.4	13.0	366.0
	QOV	0.0	0.0	0.0	0.0	24.4	29.6	11.1	0.0	0.0	0.0	0.0	0.0	65.1
1969	QIN	12.5	23.4	20.4	51.1	68.9	152.6	82.1	35.4	17.2	11.1	10.9	13.5	499.1
	QUS	13.4	19.1	47.2	46.6	45.8	46.7	45.2	35.4	17.2	11.1	13.4	13.0	354.0
	QOV	0.0	0.0	0.0	0.0	0.7	105.9	37.0	0.0	0.0	0.0	0.0	0.0	143.6
1970	QIN	22.5	35.2	42.5	62.7	77.4	100.4	50.9	23.9	17.9	9.6	9.3	8.1	460.4
	QUS	20.6	35.2	46.8	46.8	45.2	46.7	45.2	23.9	17.9	9.6	13.4	13.0	364.1
	QOV	0.0	0.0	0.0	11.6	32.2	53.8	5.7	0.0	0.0	0.0	0.0	0.0	103.3
1971	QIN	20.5	30.6	33.6	64.7	89.3	87.0	39.3	19.2	11.9	7.6	4.7	4.5	413.0
	QUS	13.4	28.7	47.0	47.0	45.2	46.7	39.3	19.2	11.9	9.8	13.4	13.0	334.6
	QOV	0.0	0.0	0.0	4.2	44.2	40.3	0.0	0.0	0.0	0.0	0.0	0.0	88.7
1972	QIN	23.7	24.8	12.4	24.4	36.2	55.5	32.9	13.5	12.6	8.5	4.7	3.6	253.0
	QUS	13.4	15.9	39.3	20.9	29.6	47.4	24.2	13.5	12.6	9.4	13.4	13.0	252.7
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1973	QIN	13.9	53.8	50.1	57.0	67.0	116.7	49.6	36.9	16.9	10.3	7.3	3.9	483.4
	QUS	13.4	35.3	46.7	46.7	45.2	46.7	45.2	36.9	16.9	10.3	13.4	13.0	369.4
	QOV	0.0	0.0	3.5	10.3	21.8	70.0	4.5	0.0	0.0	0.0	0.0	0.0	110.1
1974	QIN	17.2	38.3	28.9	34.7	50.9	71.7	38.7	31.2	12.3	6.7	5.2	3.5	339.1
	QUS	13.4	27.0	47.1	38.7	45.3	47.2	38.7	31.2	12.3	9.4	13.4	12.4	336.0
	QOV	0.0	0.0	0.0	0.0	0.0	7.8	0.0	0.0	0.0	0.0	0.0	0.0	7.8
1975	QIN	8.9	18.8	23.2	64.1	110.7	112.1	67.9	38.5	12.9	4.3	1.7	2.9	465.9
	QUS	13.4	13.0	31.5	48.1	45.5	46.7	45.2	38.5	12.9	9.8	10.4	10.1	324.9
	QOV	0.0	0.0	0.0	0.0	54.4	65.4	22.7	0.0	0.0	0.0	0.0	0.0	142.6
1976	QIN	9.2	32.1	23.1	26.1	34.8	46.5	28.0	16.5	8.6	4.7	2.3	1.2	233.2
	QUS	13.4	13.0	43.4	21.6	29.2	38.9	20.2	15.1	10.4	9.4	10.4	10.1	235.2
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1977	QIN	3.5	16.1	7.5	32.8	50.0	79.8	43.5	19.8	8.0	4.7	3.2	3.0	271.9
	QUS	10.3	13.0	10.7	24.8	44.5	47.2	43.5	19.8	10.4	9.4	10.4	10.1	254.0
	QOV	0.0	0.0	0.0	0.0	0.0	15.9	0.0	0.0	0.0	0.0	0.0	0.0	15.9

Table 9-7 Summary of Operation Study of Pirris Reservoir (2/2)

UNIT : 10**6 M3

YEAR		MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	TOTAL
1978	QIN	8.0	16.2	20.4	32.2	60.8	101.0	41.5	16.4	11.1	7.2	6.8	9.0	330.5
	QUS	12.4	13.0	24.5	27.6	46.1	46.9	41.5	16.4	11.1	9.4	13.4	13.0	275.3
	QOV	0.0	0.0	0.0	0.0	0.0	46.5	0.0	0.0	0.0	0.0	0.0	0.0	46.5
1979	QIN	20.8	46.2	34.6	49.2	81.1	95.5	63.8	21.9	13.1	8.9	8.0	6.8	449.8
	QUS	13.4	40.8	47.0	47.3	45.4	46.7	45.2	21.9	13.1	9.8	13.4	13.0	356.8
	QOV	0.0	0.0	0.0	0.0	25.1	48.8	18.7	0.0	0.0	0.0	0.0	0.0	92.6
1980	QIN	9.0	25.1	24.9	50.4	58.5	54.5	65.9	30.0	15.7	7.3	6.6	5.8	353.6
	QUS	13.4	13.0	47.1	45.8	46.6	47.0	45.2	30.0	15.7	9.4	13.4	13.0	339.6
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	17.7	0.0	0.0	0.0	0.0	0.0	17.7
1981	QIN	20.1	61.8	35.8	54.2	72.3	79.5	37.5	15.9	10.8	7.4	7.2	6.7	409.0
	QUS	13.4	45.4	46.9	47.1	45.3	46.7	37.5	15.9	10.8	9.4	13.4	13.0	344.8
	QOV	0.0	6.9	0.0	0.0	22.9	32.8	0.0	0.0	0.0	0.0	0.0	0.0	62.7
1982	QIN	20.2	33.3	22.3	18.1	28.6	48.6	25.6	14.7	10.5	6.9	6.6	6.5	241.8
	QUS	13.4	25.4	47.3	20.9	20.2	38.3	20.2	10.8	10.5	9.4	13.4	13.0	243.0
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1983	QIN	7.0	9.8	11.2	10.8	34.0	90.0	72.6	24.7	14.9	10.3	9.6	8.1	302.9
	QUS	13.4	13.0	16.1	10.8	20.4	47.2	45.2	24.7	14.9	10.3	13.4	13.0	242.2
	QOV	0.0	0.0	0.0	0.0	0.0	26.1	27.5	0.0	0.0	0.0	0.0	0.0	53.6
1984	QIN	14.8	29.1	44.8	44.0	80.0	62.8	37.1	17.4	10.8	7.7	6.7	6.7	362.0
	QUS	13.4	21.9	46.7	46.8	45.3	46.7	37.1	17.4	10.8	9.4	13.4	13.0	321.8
	QOV	0.0	0.0	0.0	0.0	30.0	16.2	0.0	0.0	0.0	0.0	0.0	0.0	46.2
1985	QIN	8.2	21.4	25.2	43.6	58.5	69.1	65.3	19.8	12.3	8.7	8.8	7.6	348.4
	QUS	13.4	13.0	40.6	39.1	46.6	47.0	45.2	19.8	12.3	9.4	13.4	13.0	312.7
	QOV	0.0	0.0	0.0	0.0	0.0	11.7	20.1	0.0	0.0	0.0	0.0	0.0	31.8
1986	QIN	13.5	23.4	23.9	14.7	20.6	58.8	22.4	14.0	9.9	7.4	6.6	6.6	221.8
	QUS	13.4	13.0	47.3	20.9	20.2	38.6	20.2	10.4	10.4	9.4	12.1	13.0	229.0
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1987	QIN	7.7	11.0	18.7	58.5	28.3	43.9	20.2	11.8	8.7	5.4	4.9	4.3	223.4
	QUS	13.4	13.0	20.9	47.9	27.9	36.3	20.2	10.4	10.4	9.8	10.4	10.1	230.8
	QOV	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1988	QIN	6.1	17.1	18.0	41.8	149.6	119.5	55.1	19.6	12.8	7.0	6.4	5.3	458.4
	QUS	10.4	13.0	20.9	35.8	45.8	46.7	45.2	19.6	12.8	9.4	13.4	13.0	285.9
	QOV	0.0	0.0	0.0	0.0	81.5	72.9	9.9	0.0	0.0	0.0	0.0	0.0	164.2
TOTAL	QIN	335.8	731.5	685.8	1004.2	1532.5	1999.5	1124.4	535.1	312.6	198.6	171.1	159.3	8790.4
	QUS	335.0	543.6	983.3	913.4	996.3	1136.5	920.2	524.9	319.1	243.8	321.8	312.0	7549.8
	QOV	0.0	6.9	3.5	26.2	337.3	691.8	174.9	0.0	0.0	0.0	0.0	0.0	1240.6
AVE	QIN	13.4	29.3	27.4	40.2	61.3	80.0	45.0	21.4	12.5	7.9	6.8	6.4	351.6
	QUS	13.4	21.7	39.3	36.5	39.9	45.5	36.8	21.0	12.8	9.8	12.9	12.5	302.0
	QOV	0.0	0.3	0.1	1.0	13.5	27.7	7.0	0.0	0.0	0.0	0.0	0.0	49.6

< TITLE EXPLANATION > QIN---INFLOW, QUS---POWER DISCHRAGE, QOV---SPILL

Table 9-8 Energy Generation of Pirris Power Plant

E UNIT:GWH

YEAR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	TOTAL
1964	26.54	25.91	95.19	95.16	92.15	95.23	84.91	35.90	24.73	19.24	27.20	26.13	648.30
1965	26.88	27.20	86.25	39.11	55.46	95.23	66.75	37.05	25.95	19.25	27.24	26.18	532.56
1966	27.04	81.25	95.22	95.22	92.12	95.23	59.06	33.17	24.10	19.25	27.21	26.16	675.04
1967	26.83	26.02	68.18	40.60	91.32	95.22	46.08	31.92	21.32	19.94	27.21	26.14	520.76
1968	27.05	81.38	95.19	95.17	92.16	95.23	92.16	53.79	33.68	24.81	27.31	26.38	744.33
1969	27.21	38.92	94.74	91.24	92.16	95.23	92.16	72.17	35.13	22.69	27.29	26.38	715.33
1970	41.96	71.90	95.21	95.23	92.16	95.23	92.16	48.82	36.55	19.57	27.27	26.24	742.30
1971	27.15	58.57	95.16	95.23	92.16	95.23	80.26	39.27	24.35	19.92	27.12	25.90	680.35
1972	26.81	32.18	78.76	40.89	58.69	95.18	49.18	27.63	25.81	19.25	27.16	25.94	507.46
1973	26.54	71.23	95.23	95.23	92.16	95.23	92.16	75.24	34.50	20.99	27.23	26.08	751.83
1974	26.85	54.67	95.14	76.63	89.76	95.23	78.91	63.63	25.04	19.23	27.10	24.50	676.69
1975	26.40	25.58	62.03	95.23	92.16	95.23	92.16	78.51	26.34	19.88	21.05	20.01	654.59
1976	26.31	25.61	85.78	42.26	57.96	79.02	41.00	30.80	21.30	19.16	21.02	20.04	469.27
1977	20.03	25.10	20.63	48.40	88.10	95.23	88.74	40.43	21.29	19.15	21.02	20.01	508.12
1978	24.40	25.41	48.04	54.15	92.11	95.23	84.73	33.56	22.64	19.23	27.15	26.08	552.74
1979	27.02	82.93	95.17	95.17	92.16	95.23	92.16	44.66	26.70	19.93	27.22	26.14	724.48
1980	26.79	26.08	94.07	89.82	92.16	95.22	92.16	61.13	31.96	19.23	27.15	26.01	681.77
1981	26.87	92.16	95.18	95.21	92.16	95.23	76.54	32.55	22.02	19.23	27.16	26.05	700.37
1982	26.94	51.69	95.09	40.74	39.71	76.71	40.94	22.05	21.49	19.23	27.14	26.01	487.73
1983	26.54	25.45	31.34	20.71	40.08	95.23	92.16	50.49	30.44	20.99	27.27	26.25	486.96
1984	27.06	44.45	95.22	95.21	92.16	95.23	75.71	35.53	21.99	19.24	27.17	26.05	655.02
1985	26.50	25.88	80.43	76.65	92.16	95.23	92.16	40.36	25.11	19.25	27.24	26.19	627.15
1986	26.96	26.28	95.10	40.72	39.04	76.49	40.85	21.18	21.23	19.14	24.49	25.92	457.39
1987	26.47	25.39	40.71	94.35	55.13	72.89	40.78	21.09	21.09	19.63	20.66	19.91	458.11
1988	20.31	25.19	40.68	69.97	92.16	95.23	92.16	39.94	26.19	19.23	27.14	25.98	574.17
AVE	26.78	43.86	78.95	72.73	79.90	92.16	75.04	42.84	26.04	19.87	26.09	25.07	609.31

EF = 230.0 <GWH>

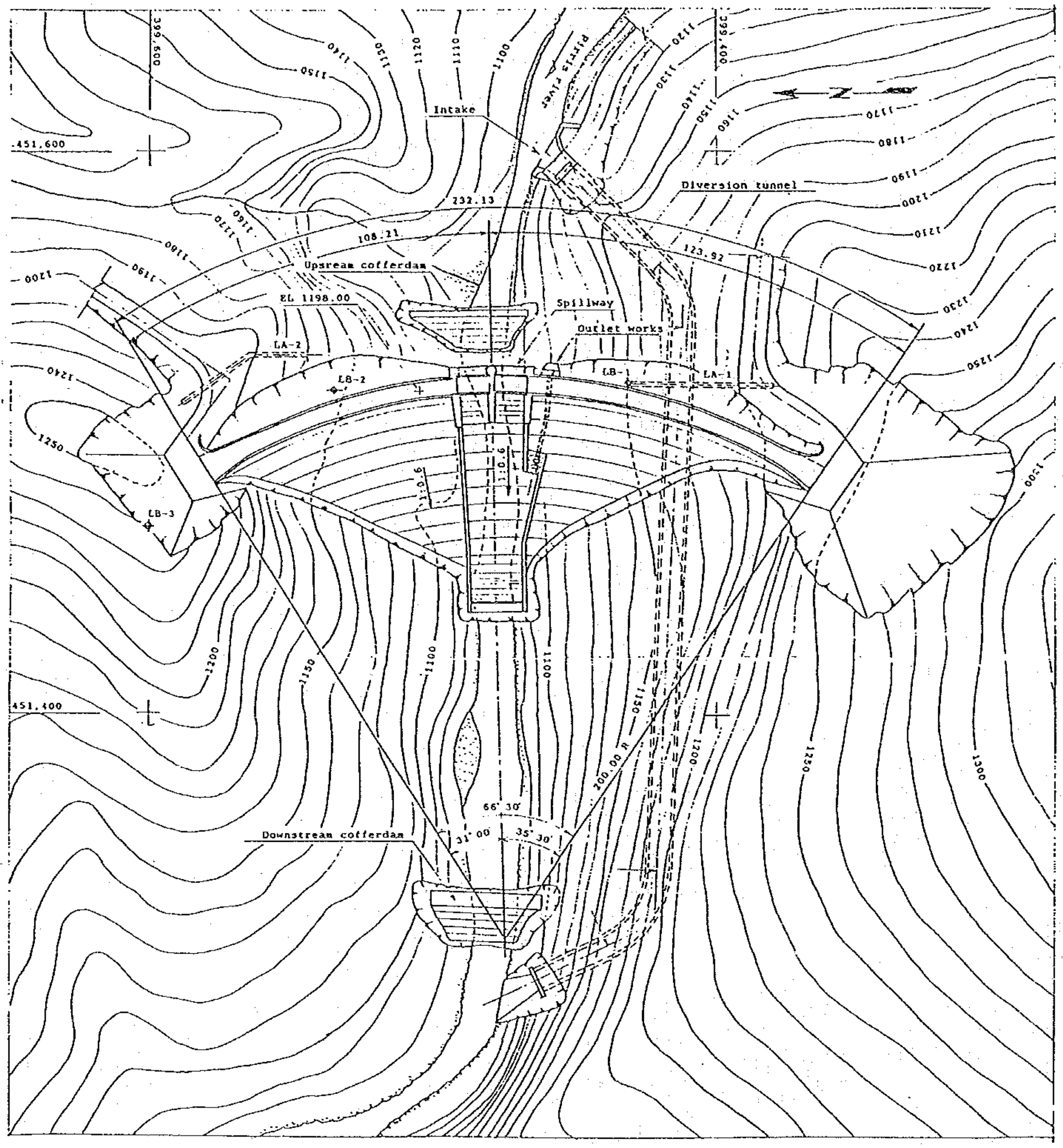
ES = 379.3 <GWH>

Table 9-9 Monthly Peak Power of Pirris Power Plant

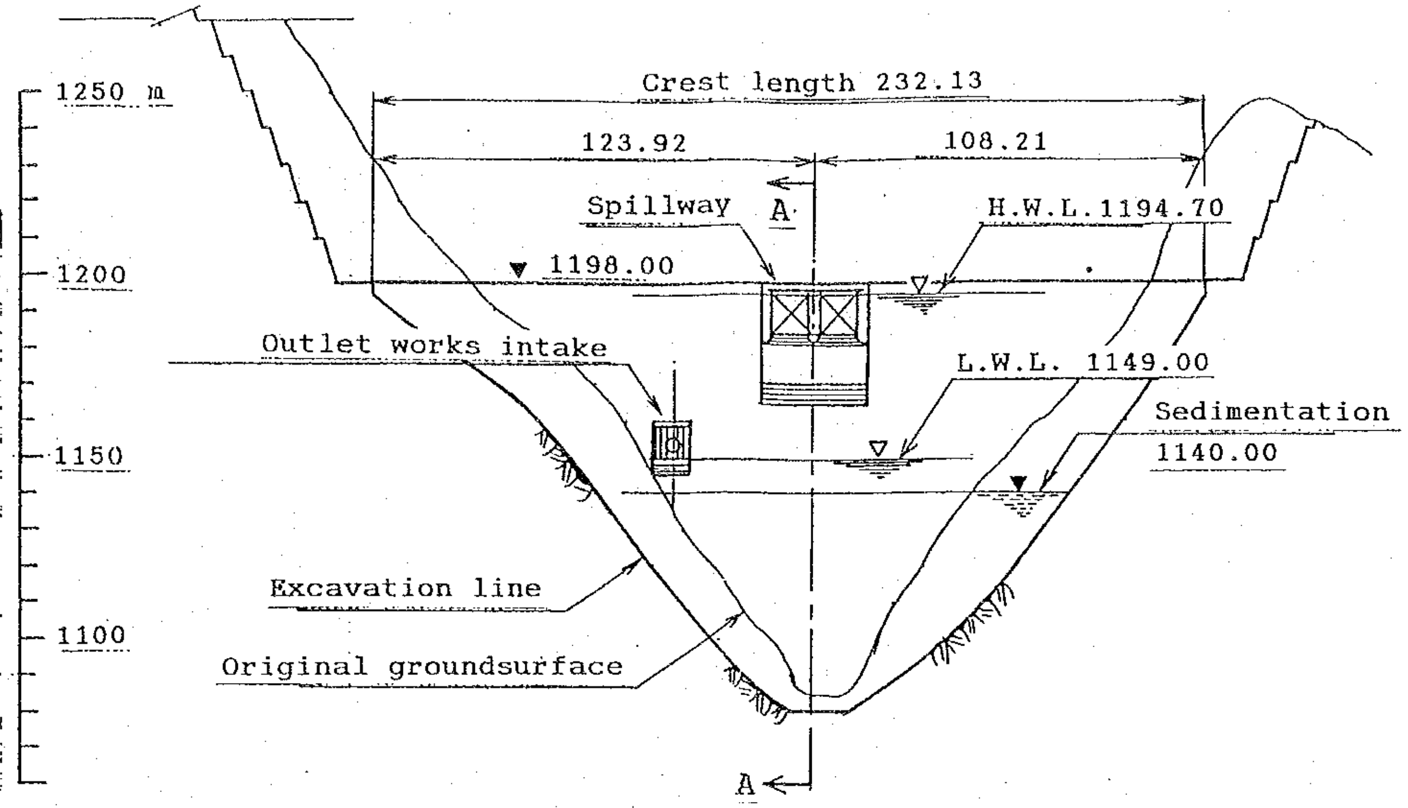
P UNIT:MW

YEAR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	TOTAL
1964	128.00	127.92	127.95	127.90	127.98	128.00	128.00	128.00	128.00	127.99	127.96	127.91	1535.60
1965	127.90	128.00	127.79	123.61	126.21	127.99	128.00	128.00	128.00	128.00	127.96	127.92	1529.39
1966	127.95	128.00	127.99	127.99	127.94	128.00	128.00	128.00	128.00	127.99	127.96	127.92	1535.74
1967	128.00	127.93	128.00	124.17	126.84	127.99	128.00	128.00	128.00	128.00	127.96	127.91	1530.78
1968	127.97	128.00	127.95	127.92	128.00	128.00	128.00	128.00	128.00	128.00	127.99	127.98	1535.81
1969	127.97	128.00	127.80	125.81	128.00	128.00	128.00	128.00	128.00	128.00	127.98	127.99	1533.55
1970	128.00	128.00	127.97	128.00	128.00	128.00	128.00	128.00	128.00	128.00	127.97	127.94	1535.88
1971	127.99	128.00	127.91	128.00	128.00	128.00	128.00	128.00	128.00	127.99	127.92	127.97	1535.78
1972	127.92	128.00	127.79	125.50	127.77	127.93	128.00	128.00	128.00	127.99	127.93	127.98	1532.82
1973	128.00	128.00	128.00	128.00	128.00	128.00	128.00	128.00	128.00	128.00	127.96	128.00	1535.96
1974	127.90	128.00	127.87	127.69	127.97	128.00	128.00	128.00	128.00	127.98	127.92	128.00	1535.34
1975	126.89	127.14	126.71	128.00	128.00	128.00	128.00	128.00	128.00	127.96	128.00	128.00	1532.70
1976	126.24	128.00	128.00	125.74	127.97	127.92	127.99	128.00	127.99	127.96	128.00	128.00	1531.80
1977	124.57	123.49	123.51	124.86	127.97	128.00	128.00	128.00	127.98	127.95	128.00	128.00	1520.33
1978	126.15	125.85	125.66	125.81	127.93	128.00	128.00	128.00	128.00	127.99	127.94	127.90	1527.22
1979	127.96	128.00	127.92	127.92	128.00	128.00	128.00	128.00	128.00	127.99	127.96	127.91	1535.65
1980	127.99	127.96	127.78	125.81	128.00	127.98	128.00	128.00	128.00	127.99	127.94	128.00	1533.44
1981	127.92	128.00	127.93	127.97	128.00	128.00	128.00	128.00	128.00	127.99	127.94	128.00	1535.74
1982	127.94	128.00	127.82	124.84	126.25	127.92	127.97	128.00	128.00	127.98	127.94	128.00	1530.65
1983	128.00	126.16	124.07	122.44	126.21	128.00	128.00	128.00	128.00	128.00	127.98	127.94	1522.79
1984	127.95	128.00	127.99	127.97	128.00	128.00	128.00	128.00	128.00	127.99	127.94	128.00	1535.83
1985	128.00	127.89	128.00	125.81	128.00	128.00	128.00	128.00	128.00	128.00	127.96	127.92	1533.58
1986	127.92	128.00	127.83	124.74	122.99	127.96	127.95	127.98	127.97	127.96	127.91	127.98	1527.18
1987	127.44	125.74	124.69	126.82	128.00	127.92	127.93	127.94	127.93	128.00	128.00	126.73	1527.14
1988	124.24	124.17	124.55	125.43	128.00	128.00	128.00	128.00	128.00	127.98	127.93	128.00	1522.31
AVE	127.47	127.37	127.18	126.35	127.52	127.98	127.99	128.00	127.99	127.99	127.96	127.92	1531.72

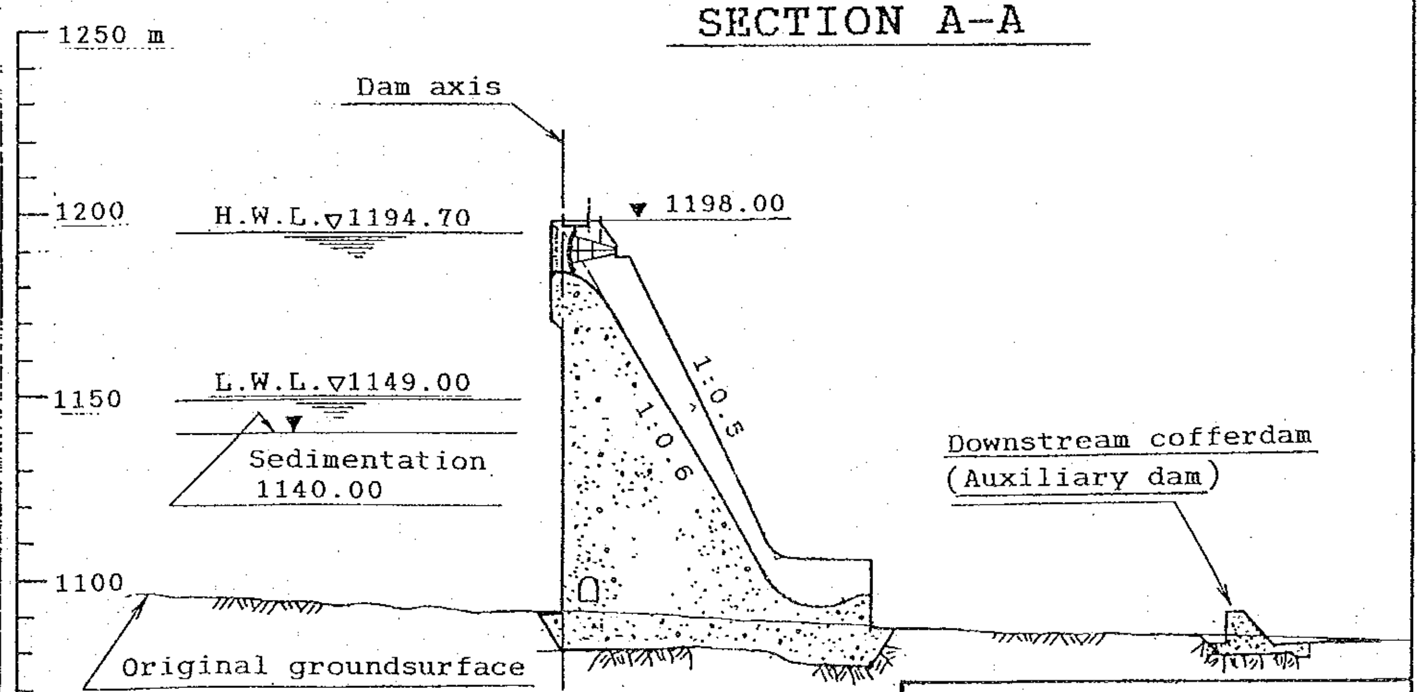
PLAN



UPSTREAM ELEVATION



SECTION A–A



0 50 100 m

Plan , Section

REPUBLIC OF COSTA RICA
PIRRIS HYDROELECTRIC POEWR DEVELOPMENT PROJECT
Lower Damsite Concrete Arch Gravity Dam Plan & Section (HWL 1194.7)
Fig. 9-15

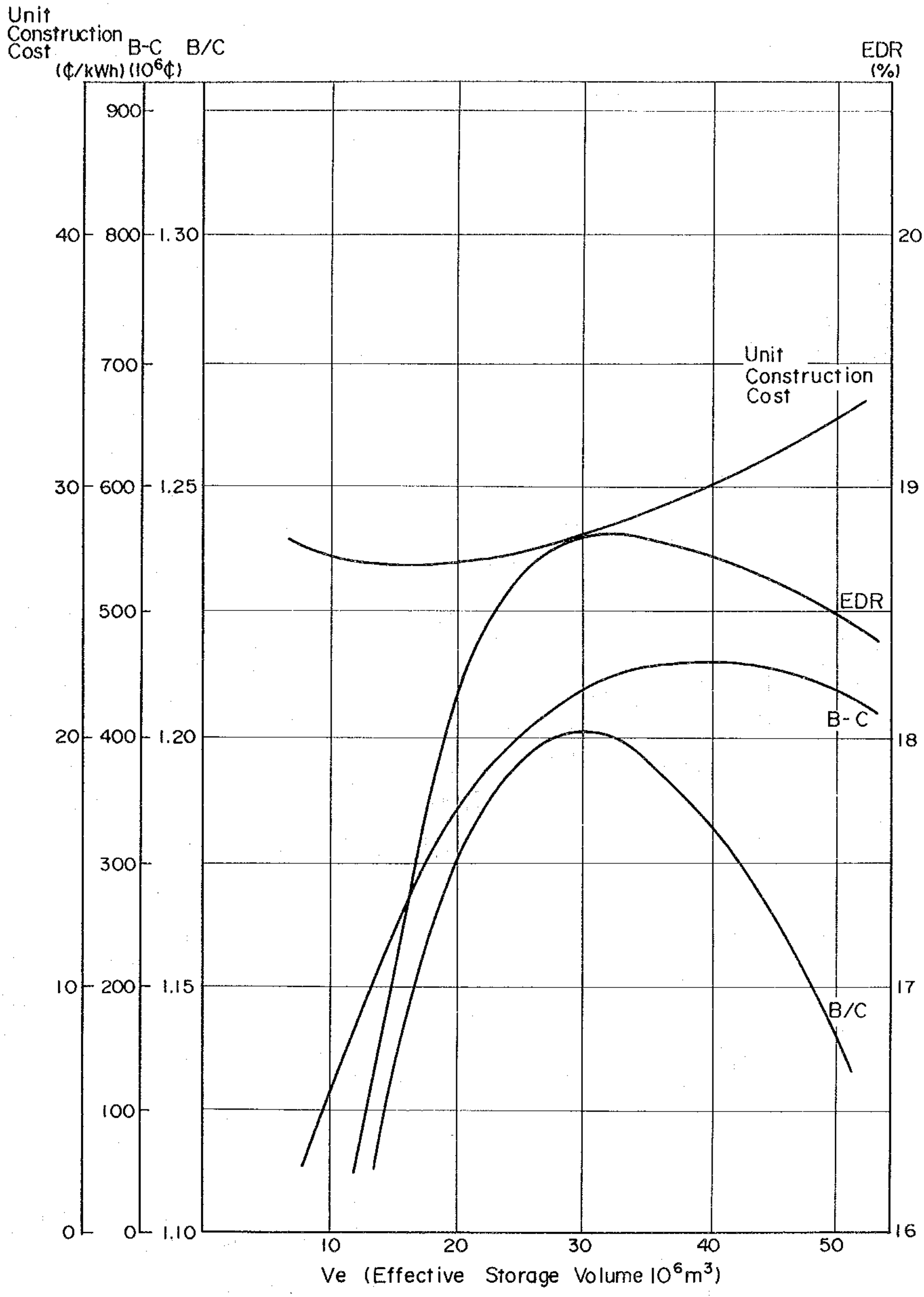


Fig. 9-16 Study on Reservoir Storage Volume (2)
(Upper dam site, Rockfill Dam)

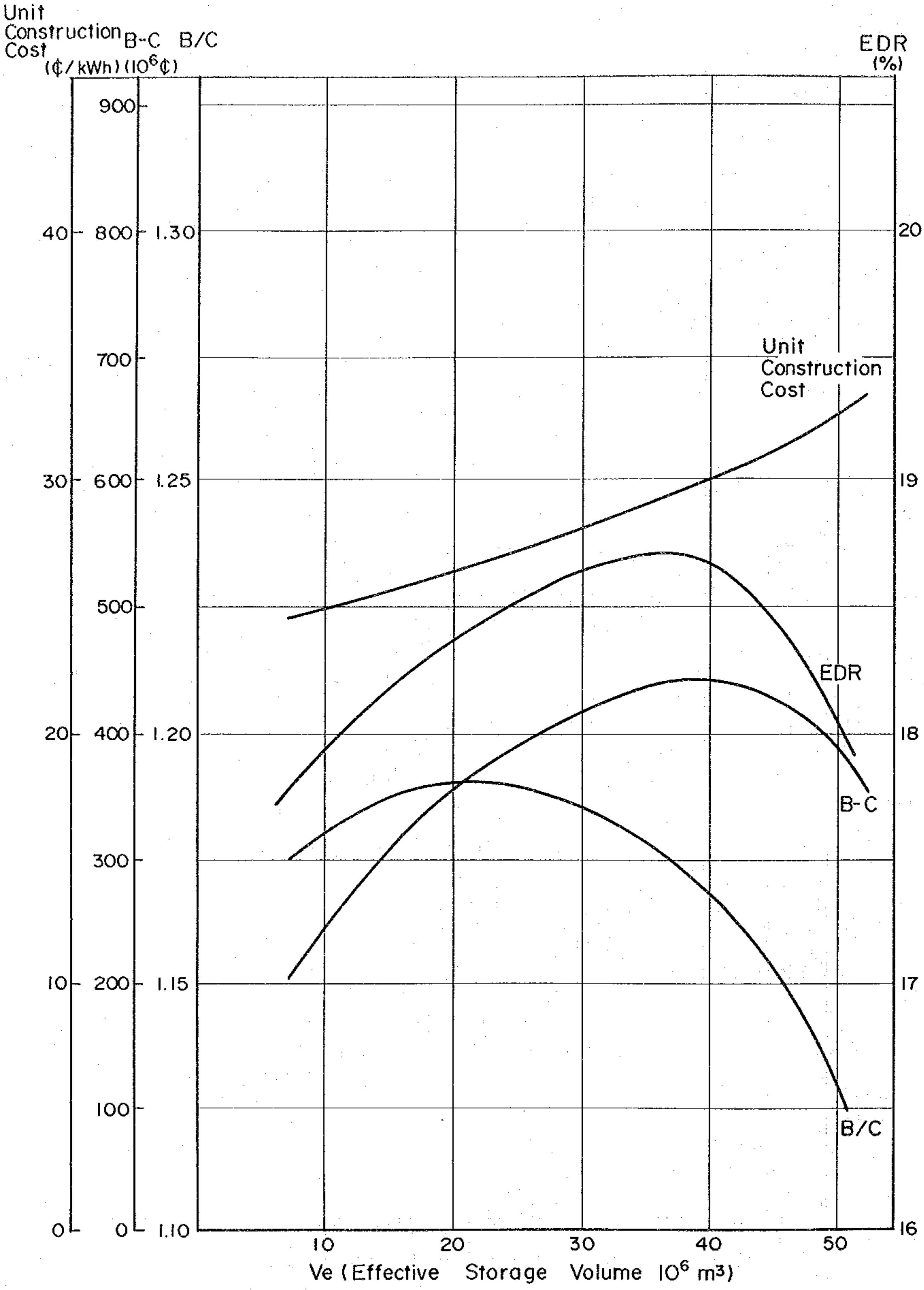


Fig. 9-17 Study on Reservoir Storage Volume (2)
(Lower dam site, Concrete Gravity Dam)

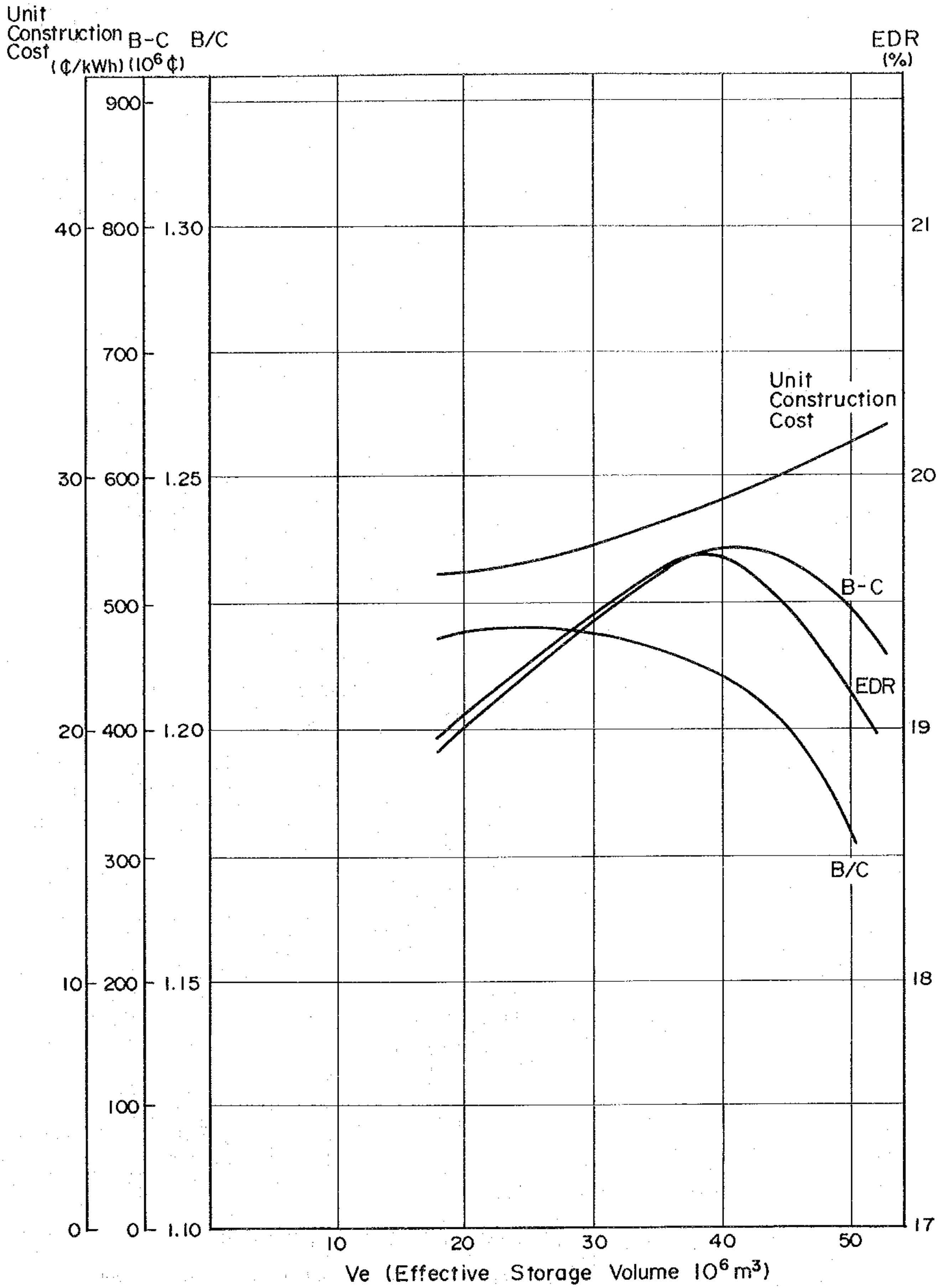


Fig. 9-18 Study on Reservoir Storage Volume (2)
(Lower dam site, Concrete Arch Gravity Dam)

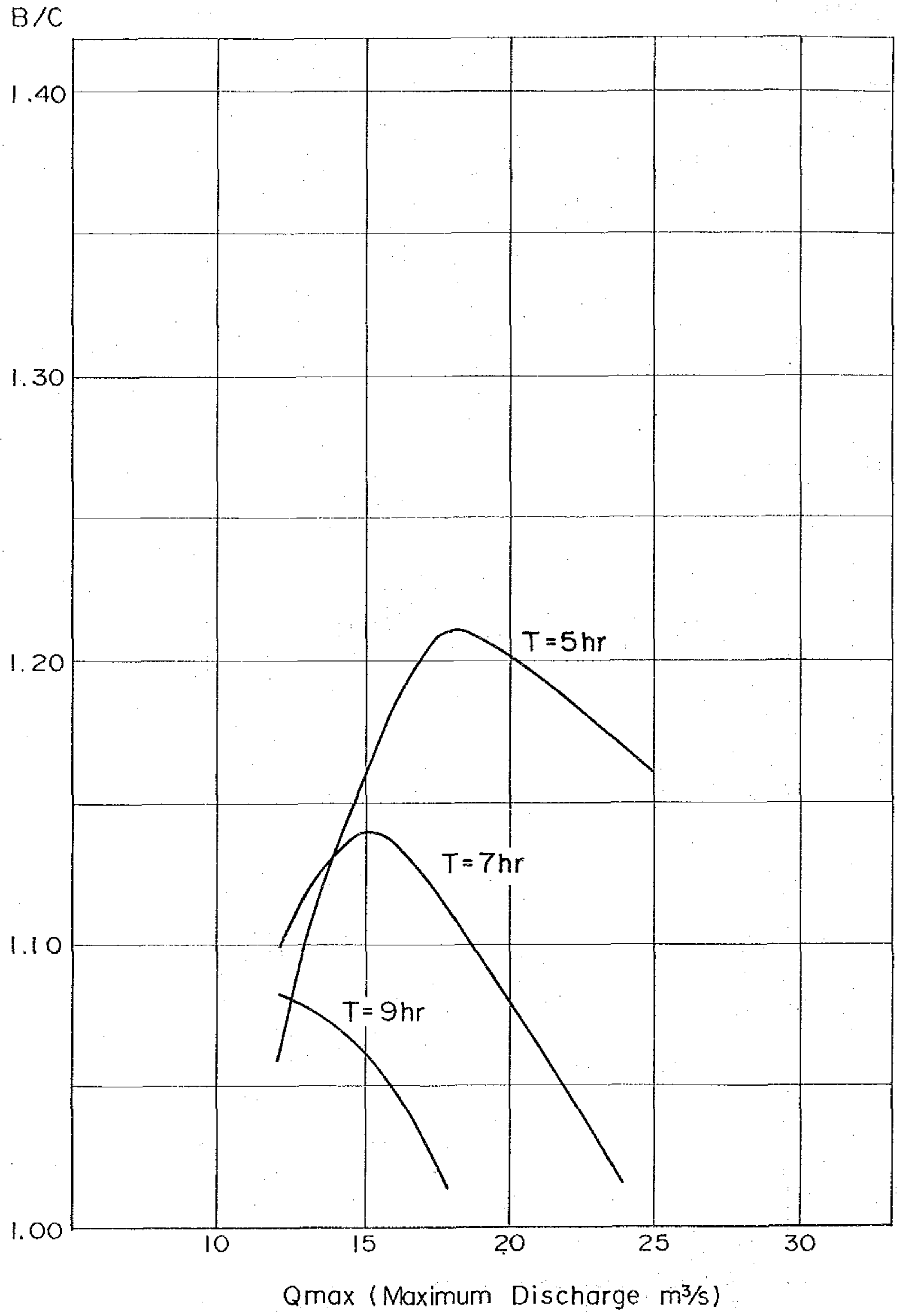


Fig. 9-19 Study on Optimum Maximum Discharge and Peak Duration (1) (B/C)
(Lower dam site, Concrete Arch Gravity Dam, HWL 1195.0)

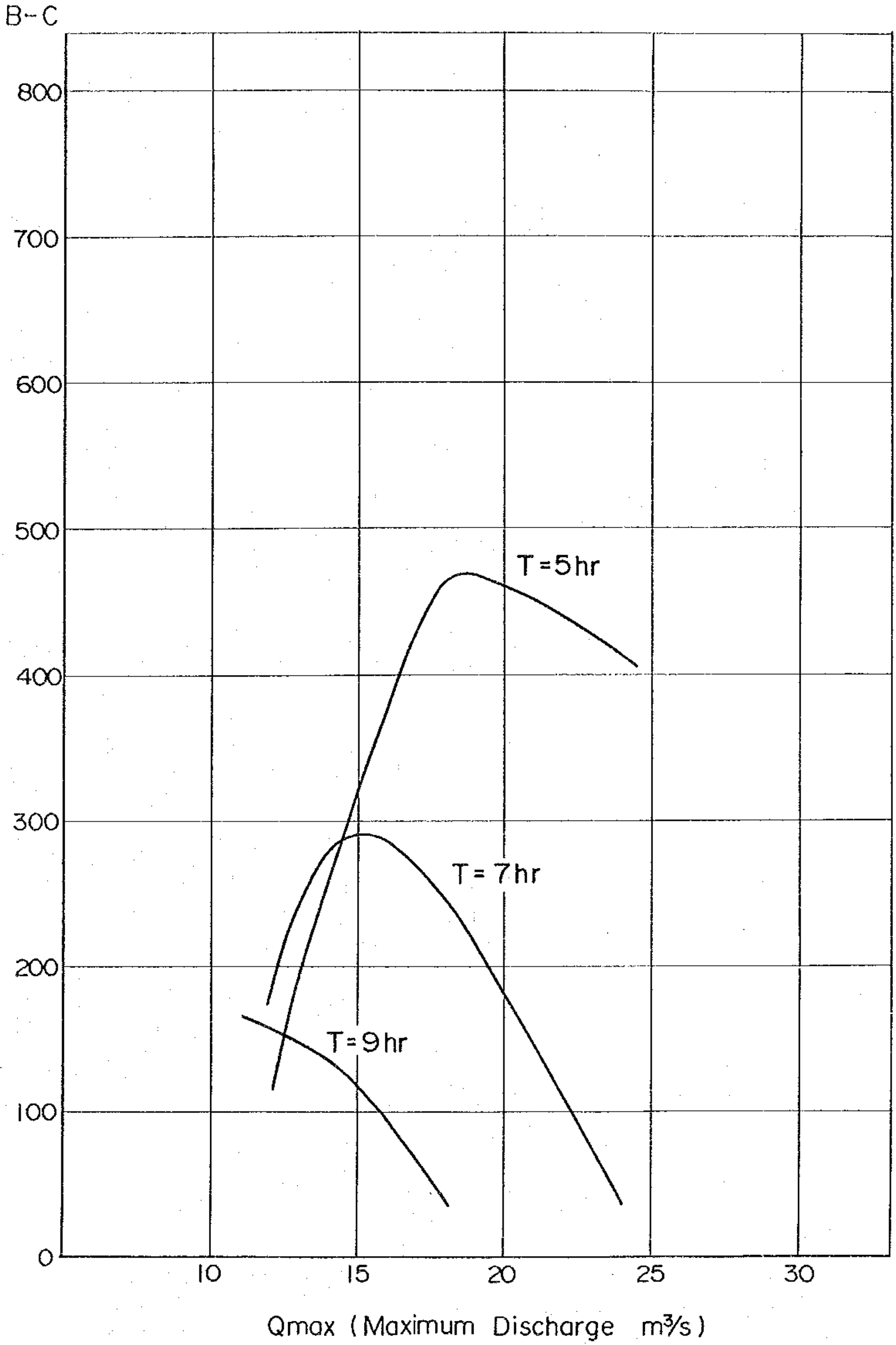


Fig. 9-20 Study on Optimum Maximum Discharge and Peak Duration (2) (B-C) (Lower dam site, Concrete Arch Gravity Dam, HWL 1195.0)

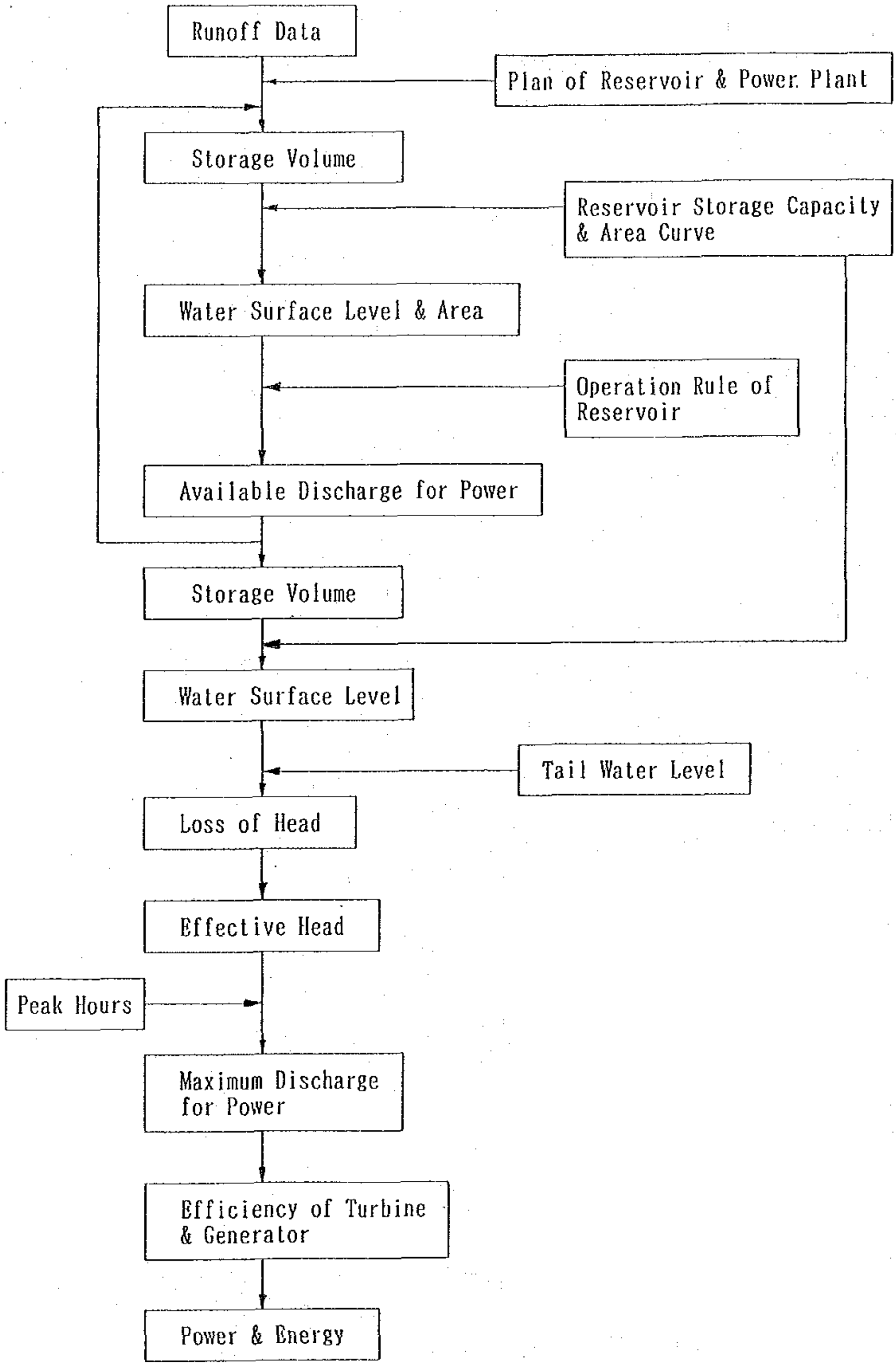
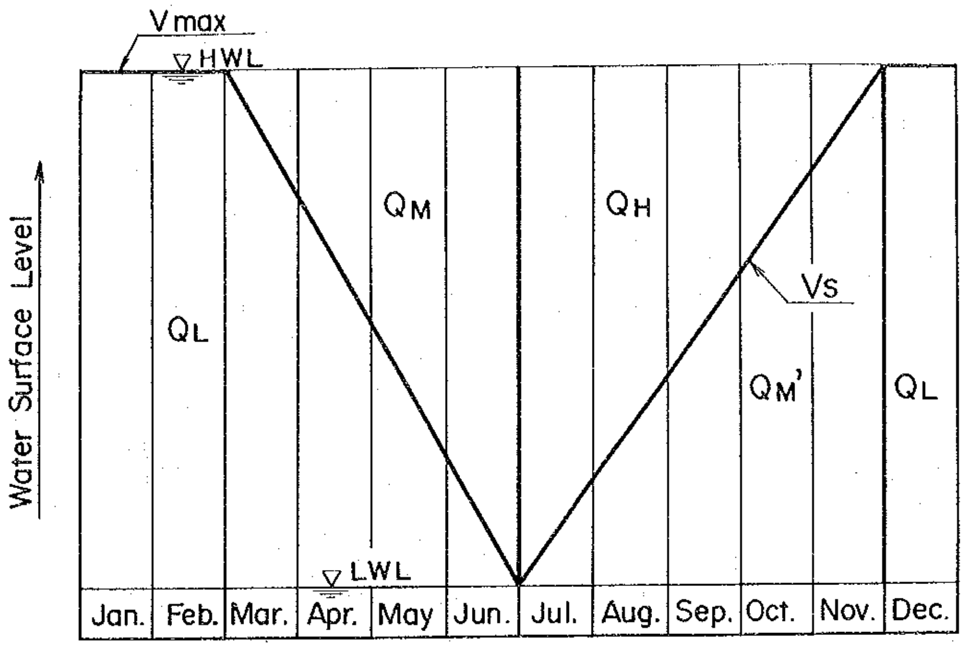
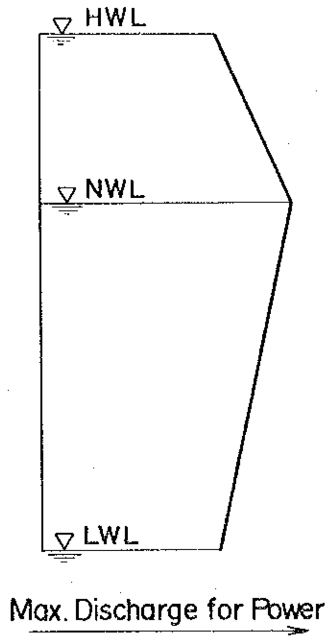


Fig. 9-21 Flow Chart of Power and Energy Calculation

Fig. 9-22 Operation Rule of Reservoir





Symbols

- V_{n-1} : Storage at the end of previous month
- V_n : Storage at the end of current month
- V_n' : Temporary storage at the end of current month
- V_{max} : Maximum storage (Effective storage capacity)
- V_s : Secured storage for firm discharge
- f_n : Spill in current month
- q_n : Inflow in current month
- Q_n : Available discharge for power in current month
- Q_M : Medium discharge for power (Q_M, Q_H)
- Q_L : Firm discharge for power (Q_L, Q_M')
- Q_H : Maximum discharge for power, variable depending on water level
- E : Evaporation, variable depending on water surface area

Operation Rule

$V_n' = V_{n-1} + q_n$

1. $V_n' \geqq V_{max}$
   - (1) $V_n' - V_{max} \geqq Q_H$ $\longrightarrow$ $Q_n = Q_H$
   - (2) $Q_H > V_n' - V_{max} \geqq Q_M$ $\longrightarrow$ $Q_n = V_n' - V_{max}$
   - (3) $Q_M > V_n' - V_{max}$ $\longrightarrow$ $Q_n = Q_M$
2. $V_{max} > V_n' \geqq V_s$
   - (1) $V_n' - V_s \geqq Q_M$ $\longrightarrow$ $Q_n = Q_M$
   - (2) $Q_M > V_n' - V_s \geqq Q_L$ $\longrightarrow$ $Q_n = V_n' - V_s$
   - (3) $Q_L > V_n' - V_s$ and $V_n' \geqq Q_L$ $\longrightarrow$ $Q_n = Q_L$
   - (4) $Q_L > V_n'$ $\longrightarrow$ $Q_n = V_n'$
3. $V_s > V_n'$
   - (1) $V_n' \geqq Q_L$ $\longrightarrow$ $Q_n = Q_L$
   - (2) $Q_L > V_n'$ $\longrightarrow$ $Q_n = V_n'$

$V_n = V_n' - Q_n$

$V_n' - V_{max} - Q_n \geqq 0$ $\longrightarrow$ $f_n = V_n' - V_{max} - Q_n$

$V_n' - V_{max} - Q_n < 0$ $\longrightarrow$ $f_n = 0$

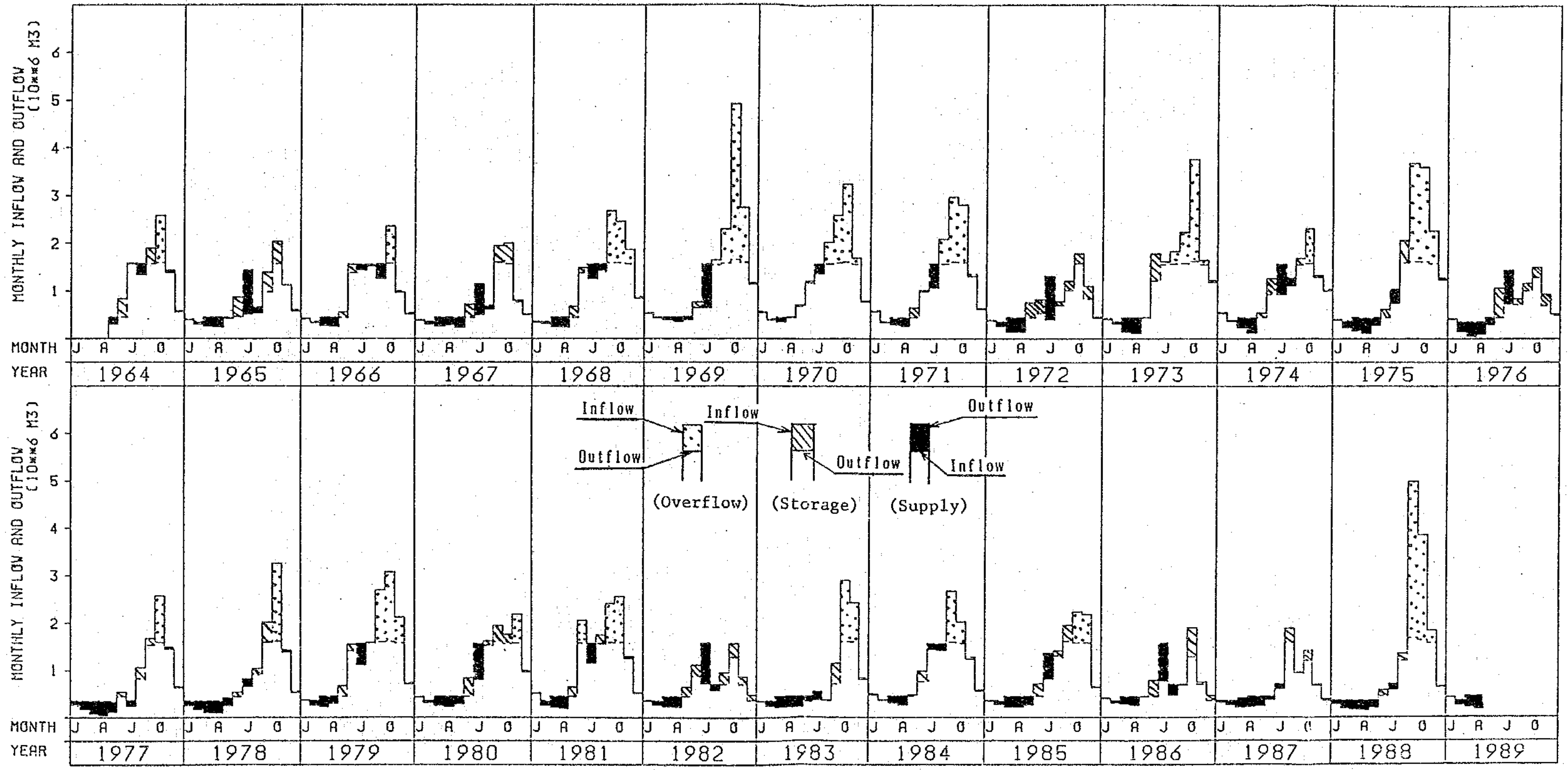


Fig. 9-23 Pirris Reservoir Operation

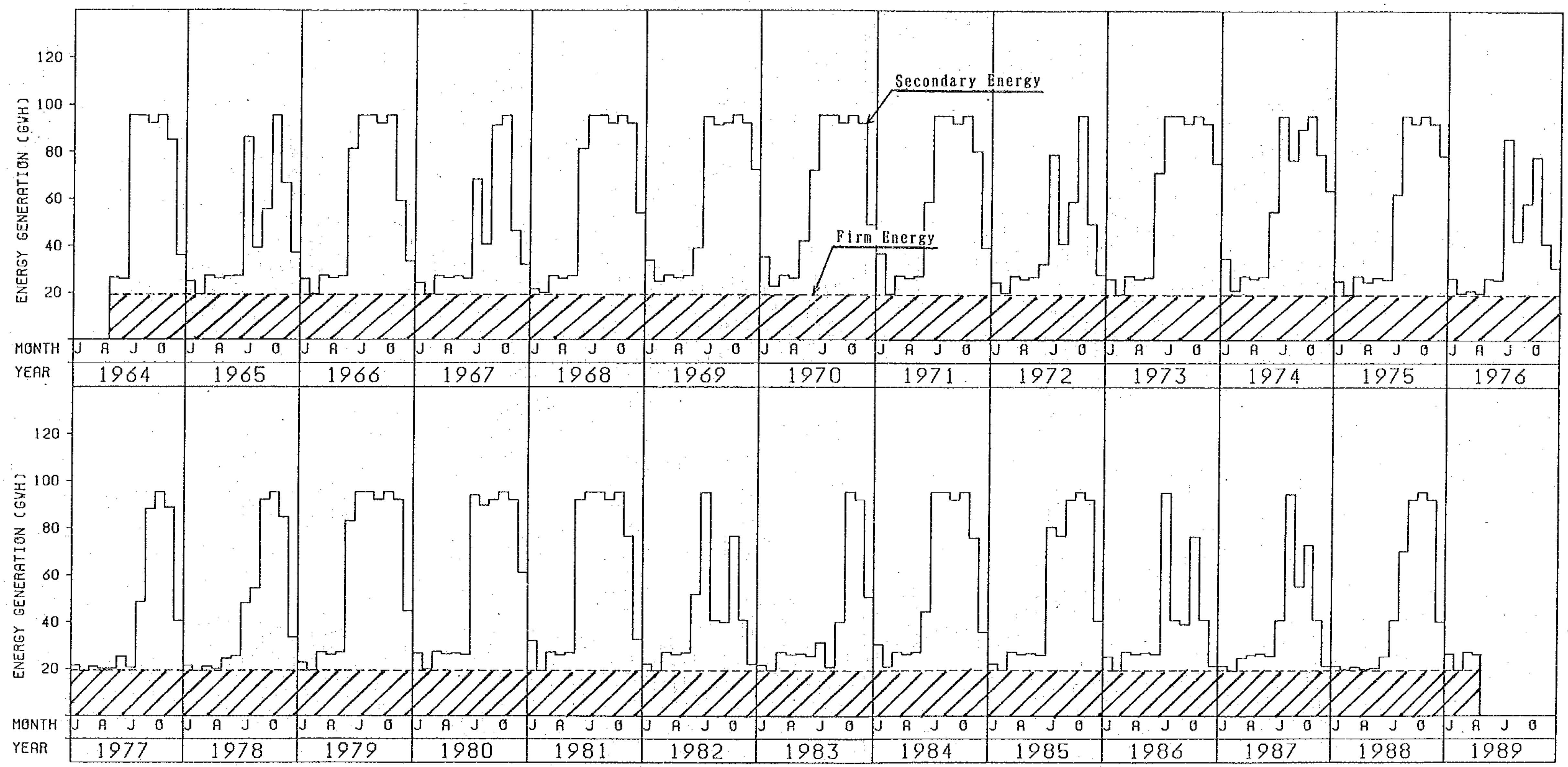


Fig. 9-24 Monthly Energy Generation

CHAPTER 10 POWER TRANSMISSION PLAN AND POWER SYSTEM ANALYSIS

CHAPTER 10 POWER TRANSMISSION PLAN AND POWER SYSTEM ANALYSIS

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CHAPTER 10 POWER TRANSMISSION PLAN AND POWER SYSTEM ANALYSIS

10.1 Outline of Power Transmission Systems

The power transmission system of Costa Rica is composed of 230 kV and 138 kV power transmission lines. Total lengths are 667 km and 670 km, respectively.

At the time when Corobici Power Plant was completed (1982), the 230 kV power transmission system was interconnected to the neighboring countries of Nicaragua and Panama. This made it possible to exchange power with these two countries. The power system is further interconnected to Honduras via Nicaragua.

In the central valley area where the capital city of San Jose is located and the power demand is largest in Cost Rica, a ring power transmission line system of 138 kV is constituted. This contributes much to improve power supply reliability.

ICE plans to provide a 230 kV interconnection for San Jose district in the near future. This 230 kV interconnection is scheduled to be completed before commissioning of Pirris Power Plant. The 230 kV and 138 kV power transmission system diagram of Costa Rica is presented in Fig. 10-1. As a long range plan, the construction of a 500 kV power transmission line, which interconnects six countries of Central America from Guatemala via Costa Rica to Panama, is being contemplated.

10.2 Power Transmission Line Route

According to the plan of ICE, the electric power generated at Pirris Power Plant will be transmitted to the capital city area which is the largest load center of the country. This electric power will be transmitted to Escazu Substation which will be constructed by ICE near San Jose City.

As for the power transmission line route, Route A and Route B, which run from the power plant upstream along Pirris River, have been considered. It was found out that a power transmission line that runs to San Jose City along Route A has the shorter distance. Therefore, the power transmission line route which is technically and economically advantageous has been surveyed along this course.

The draft plan of preliminary study is presented in Fig. 10-2. In constructing a power transmission line, presence of existing roads which can be utilized for transportation of equipment and materials can substantially reduce the construction cost. For this reason, the power transmission line route has been selected along existing roads as much as possible. It was found out in the field survey that Pirris River runs through deep canyon to the upstream of the power plant site. In particular, passage along the left bank is blocked by steep glens and hills, while there is a road, on which a vehicle can travel (dry season only), is provided on the right bank. By constructing the power transmission line along the right bank, the line distance becomes shorter and it can be built along a road.

The power transmission line runs mountainous areas from the dam site to Escazu Substation. But there will be no problem in construction of the power transmission line if passage through some national park area is avoided.

Based on the above study, Route A was definitively selected as the power transmission line route. The power transmission line length along Route A from Pirris switchyard to Escazu Substation is approximately 44 km.

10.3 Switchyard Site

The proposed power plant site is a flat place along Pirris River. There is sufficient space between the proposed power plant and Pirris River. Therefore, it is planned to locate the switchyard on the tailrace side of the power plant.

The power transmission line coming out of the switchyard must cross Pirris River in order to reach the right bank of the river. This arrangement is simple, and there is no substantial problem concerning the geography of the switchyard site.

10.4 Substation Site

The proposed site of Escazu Substation is at the periphery of the central district of Escazu. The land has some slope but it faces a two lane highway. Two circuits of existing 138 kV power transmission line (running from Caja Substation to Alajuelita Substation) pass along the substation site, and it is an ideal place for a substation. Owing to the existing road, there is no problem in transportation of heavy equipment.

The outline of Escazu Substation has not yet been determined, but the general concept is as described below. Firstly a 230 kV bus will be provided to connect the power transmission line from Pirris Power Plant and the 230 kV interconnection line to the adjacent Caja Substation. Interconnection transformers rated 230 kV/138 kV will be installed to provide interconnection with the existing 138 kV line passing nearby. It is also planned to provide four 34.5 kV feeder to supply power to the new district of Gam to be constructed in the vicinity of the substation.

10.5 Power Transmission Plan for Pirris Project

10.5.1 Basic Conditions

In the plan, in addition to the electric power generated at Pirris Power Plant, the output of Savegre hydro project (165 MW), Los Llanos hydro project (96 MW) and other hydroelectric projects which are being planned in the vicinity of Pirris Project will be transmitted to the metropolitan area of San Jose on this power transmission line. The reason is explained below.

(1) According to the plan of ICE, not only the power generated at Pirris Power Plant but also the output of Savegre and other projects, which are being contemplated for development after completion of Pirris Project, will be transmitted to the central valley region around San Jose City, which is the center of electric power consumption. (Refer to Fig. 10-3.)

(2) There is no existing 230 kV power transmission line, substation or switchyard near Savegre Project (165 MW) or Los Llanos Project (96 MW). For this reason, ICE plans to transmit the power generated at these projects to San Jose City via the switchyard of Pirris Power Plant.

(3) In formulating this power transmission plan, the following criteria was applied, the heat capacity of transmission line and stability of the line are not endangered even when a single circuit of line or one bank of transformer fails.

10.5.2 Power Transmission Line Voltage and Number of Circuits

It is more economical and advantageous in power system operation to have a power transmission line voltage coordinated to that of existing power system and select the voltage from the existing voltage classes.

Considering that the power transmitted from Pirris Power Plant only and the amount of power including the future projects supposed to be 293 MW at the maximum (Pirris 128 MW + Savegre 165 MW) as well as the transmission distance of 44 km, the capacity of the power transmission line is insufficient with 138 kV lines. Therefore, one step higher voltage, which is 230 kV was selected. As for the number of circuits, the single circuit plan and the double circuit plan are conceivable, and the economic comparison of the these two plans is illustrated in Table 10-2. As can be seen in Table 10-2, although the construction cost is cheaper for the single circuit plan, the double circuit plan is more economical when the annual expense including the transmission loss is considered. In addition, since Pirris Power Plant is a major power supply source in the power system of ICE, the total interruption of this power transmission line could cause power supply failure in the wide extent of the interconnected power system. Therefore it would be required to maintain high reliability for this power transmission line.

Based on the above reasoning, the double circuit power transmission line was selected.

10.6 Analysis of ICE Power System

The heat capacity, voltage regulation, short circuit capacity and stability of the power transmission line were studied for each 230 kV transmission line.

This study was conducted for the year 2001 when Pirris Power Plant is scheduled to be commissioned.

10.6.1 Power Flow Calculation

(1) Study Conditions

Total power demand of ICE Power System:	1,336 MW (year 2001)
Power factor of load:	95% (lag) at substation bus
Generator output:	Power stations other than Arenal and Corobici are operated at full output. Arenal and Corobic generators are used to control total system output.
Voltage control target:	Voltage is kept within 95 - 105% at each power plant and substation.

(2) Study Result

The results of power flow calculation is illustrated in the power flow diagram of Fig. 10-4. From the diagram, it can be seen that there is no need for additional reactive power control equipment, and there is no problem in power flow.

10.6.2 Short Circuit Capacity

(1) Study Conditions

Time cross section:	Year 2001 when Pirris is commissioned.
Generator:	All generators are connected to the system. Subtransient reactance X"d is used for calculation.

(2) Result of Study

Three-phase short circuit current capacity at each site is given below:

- 230 kV bus at Pirris Power Station : 4.6 kA (1,800 MVA)
- 230 kV bus at Escazú Substation : 5.7 kA (2,300 MVA)

The result of study shows no problem. The short circuits current is within the IEC standard of 31.5 kA. Therefore, there is no need to consider the breaking capacity in selecting circuit breaker.

10.6.3 Stability

(1) Assumed Fault

It is assumed that a 3-phase grounding short circuit (3LG) fault occurs on a single circuit of the line at the bus of Pirris Power Plant switchyard, which is cleared in 6 cycles (100 ms).

(2) Study Results

The results of simulation study is presented in Fig. 10-5. Result of stability study is shown in Table 10-1. The system is stable in all cases.

Table 10-1 Table of Stability Study Result

Case (Fault Point)	Year 2001	
	230 kV 1 Circuit Plan	230 kV 2 Circuit Plan
Pirris Bus	Stable	Stable
Escazu Bus	Stable	Stable

10.7 Economic Study

The power transmission line plan for Pirris Project has been formulated with the consideration on the power transmission capacity and coordination with existing power system. Finally, the 230 kV single circuit plan and the 230 kV double circuit plan have been compared by economic comparison study. The result of this study is given in Table 10-2.

The economic comparison study was conducted by comparing the capital cost expenses of power transmission line and switchyard facilities (for connection of transmission line) and the expense caused by transmission loss.

(1) Study Conditions

- Transmission loss unit cost: (Refer to 9.2.2)
  - kW cost: 119.57 $/kW
  - kWh cost: 0.0304 $/kWh

- Annual expense ratio of power transmission/substation facilities: 10.81% (R.F. = 0.0931, O.M. = 0.015)

- Power transmission loss factor; L_f:
  - Pirris: 36.9% (plant factor pf: 54.3%)
  - Savegre: 47.2% (plant factor pf: 63.4%)

Loss calculated by
Buller-Woodrow's equation: $L_f = 0.3\ pf + 0.7\ pf^2$

(2) Study Results

As seen in Table 10-2, the construction cost of the 230 kV single circuit plan is cheaper, but 230 kV double circuit plan is more economical when the annual expenses including transmission loss are taken into account.

10.8 Conclusion

The double circuit, 230 kV plan is recommended for the transmission of electric power of Pirris Project based on this study. The power transmission line parameters of this plan are as presented below.

Transmission voltage	:	230 kV
Number of circuits	:	2
Total length	:	Approximately 44 km
Conductor type/size	:	ACSR, 2 x 636 MCM

The major advantages of this power transmission plan are as follows:

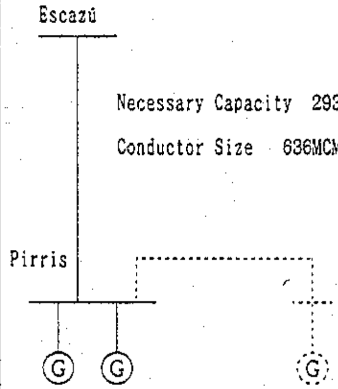
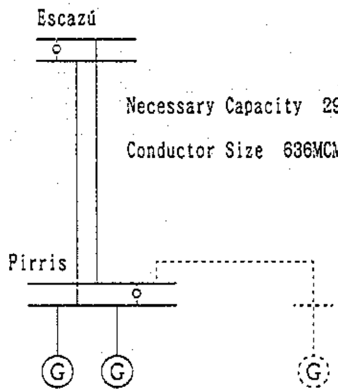
- Although the initial investment is high, the plan is economical in a long run.

- The power transmission reliability is high, as it has two circuits. (ICE regards Pirris Project as an important power plant in its power system.)

- The output of hydroelectric power plants in the vicinity of Pirris Project, which are planned to be constructed following Pirris (i.e., Savegre Power Plant), can be transmitted by this line.

- The voltage drop at the receiving end substation is small.

Table 10-2 Economic Comparison of Transmission Plan for Pirris Project

Plan (pattern)	Pattern 1 (230kV 1-circuit)	Pattern 2 (230kV 2-circuits)	Note
Power System Diagram Economic Evaluation Items (10³US$)	 Necessary Capacity 293MW *1 Conductor Size 636MCM×2	 Necessary Capacity 293MW *1 Conductor Size 636MCM×2	
Total Constraction Cost (10³US$)	(5,843) 7,383	(7,688) *2 9,714	
Annual Cost (10³US$) (Annual factor = 10.81%)	(632) 798	(831) 1,050	
Transmission Losses (10³US$) Peak Power Loss Annual Energy Loss Annual Cost	 (1.76MW) 210 (10.26×10⁶ kWh) 312 522	 (0.88MW) 105 (5.13×10⁶ kWh) 156 261	*1 Pirris (128MW) +Savegre(165MW) = 293MW *2 ( ) value. = Data of ICE
Total Annual Cost (10³US$)	(1,154) 1,320	(1,092) 1,311	

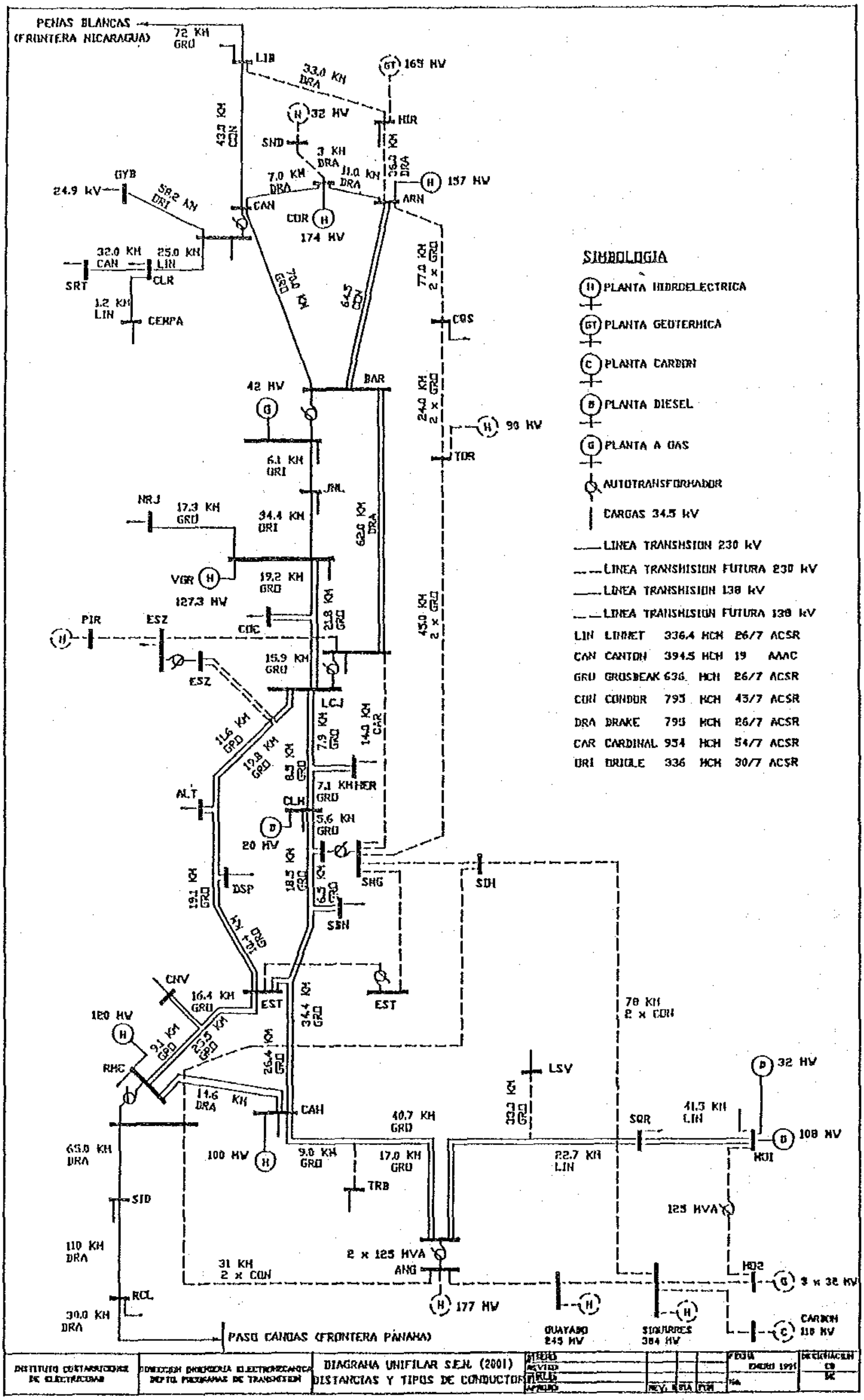


Fig. 10-1 System of National Transmission Line

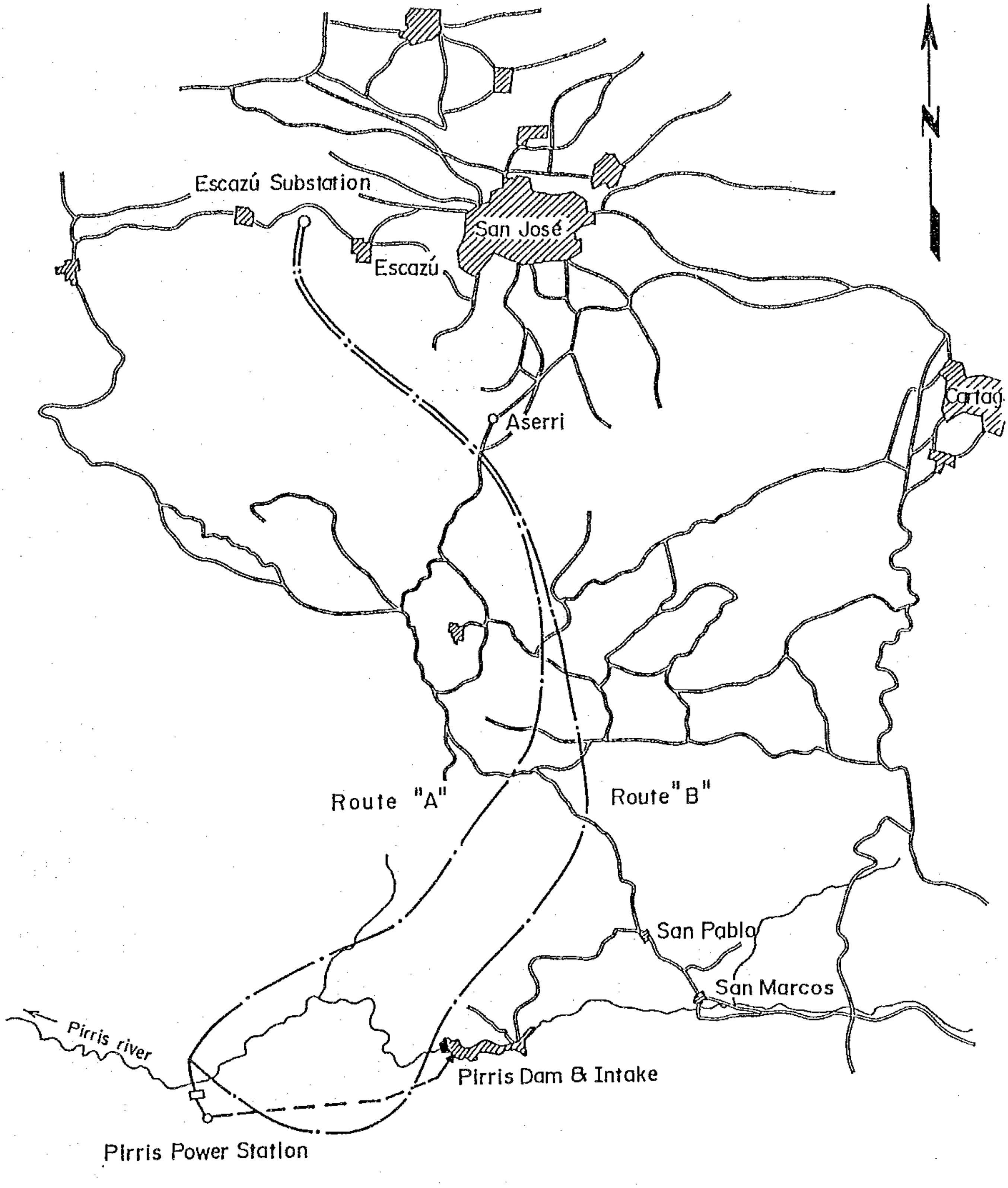


Fig. 10-2 Alternatives of Transmission Line Route