These alignment and diameter of penstock is discussed later in detail in the second screening study to seek best configuration.

Thickness of the steel conduct is estimated based on the assumption that allowable strength of steel is 120 MPa and the maximum pressure rise is 25 % of static gross head.

3.3.3 Powerhouse

Surface type powerhouse with two generating units was considered. From the head and plant discharge under study, vertical shaft Francis type turbine was selected. Dimensions of turbine and generator as well as powerhouse dimensions were estimated by empirical data. Fig. VII.3.11 shows typical layout of the powerhouse.

Finished level of ground in the powerhouse yard was set at EL. 215 m which is the same level as the water level estimated for the 10,000-year flood.

3.3.4 Designed Principal Features

Principal dimensions and capacities of major project components designed are listed in Table VII.3.2.

3.4 Energy Generation Study

3.4.1 General

ELETROBRAS's criteria defines that the firm energy of a power plant is the incremental firm energy of the power system to which the plant is connected. Such firm energy can be computed by the generation simulation analysis of the system under two system conditions; with and without the power plants at issue.

The Salto Pilão power plant will be connected to the integrated South/Southeast power system. Capacity of the plant is not so large as it affects significantly to system operation, which is less than 0.5 % of the system's total energy production capacity. Accordingly, it is considered that the energy produced by the Salto Pilão can be absorbed completely by the system without surplus and further that the system analysis can be replaced with an independent generation analysis which discards the effect of system load. In order to evaluate the effect of the system, CELESC carried out the system analysis for the Salto Pilão based on the monthly discharge data using the computer simulation program;

MSUI, which was developed by ELETROBRAS. The analysis was made on 2 cases; one is the Salto Pilão with connection to the system and another is the Salto Pilão without connection to the system. The analysis result on the scheme of dam axis B (FSL=319m) is as follows:

		Salto Pilão with System		Salto Pilão without System		
Max.Plant Discharge (cms)	Installed Capacity (MW)	Firm Energy (MWy)	Second Energy (MWy)	Firm Energy (MWy)	Second Energy (MWy)	Ratio
		A		B		B/A
30	50.7	43	2	43.0	1.0	1.000
60	102.0	78	5	77.4	4.0	0.992
90	154.0	99	7	97.9	7.0	0.989
105	180.3	109	9	109.3	8.0	1.003

As shown in this table, the influence of the Salto Pilão to the system is very small and negligible. Therefore, simulation of power generation in this study is made by the independent generation analysis without taking into account the influence of system's load.

Energy production of the project was computed by the independent simulation using the daily discharge series for 50 years from 1941 to 1990. For the plan formulation, the simulation was made applying the discharge given at an interval of 1 % of the entire time length on the discharge duration curve for each critical and long term period.

3.4.2 Effective Head

Effective head at turbine was calculated by deducting loss of head in waterway during operation from the static gross head. Reservoir level to compute the static head was set at middle of FSL and MOL or at flood surcharge level when spilling out. The loss of head is approximated by :

$$Hs\ max = L1/650 + L2/150 + L3/1000 + Hj$$

$$Hs = (Q/Qmax)^2\ Hs\ max$$

where,
- Hs max ; total loss of head at Qmax
- Hs ; loss of head at Q
- L1 ; length of headrace tunnel
- L2 ; length of penstock
- L3 ; length of tailrace
- Hj ; loss at intake, desanding basin and draft tube outlet (0.1 m) + loss at turbine inlet valve (0.5 m)
- Q max ; maximum plant discharge
- Q ; arbitrary plant discharge

The length of the headrace tunnel between the desanding basin and the surge tank varies with the location of damsite; L1 is 6100 m for the axis B, 5980 m for the axis C and 5620 m for the axis D. Lengths of the penstock and the tailrace are estimated at 450 m and 50 m, respectively.

Tailwater level varies with river flow discharge at the powerhouse site. Its daily discharge in this study is approximated by assuming that run-off originating from the sub-basin between the damsite and the powerhouse equals 60 % of the natural daily discharge of damsite since ratio between catchment areas of the damsite and powerhouse site is approximately 1: 1.6. The discharge at powerhouse site (Qt) is calculated by :

$$Qt = Qs + Qr + Qb + Q_p$$

where, Qs : spillage from dam
Qr : river maintenance flow (=7.2 cms)
Qb : sub-basin run-off
Q_p : turbine discharge

Design tailwater level to decide the maximum plant capacity was obtained by replacing with; Qs = 0, Qb = 0.6Qmax and Qt = Qmax. The stage-discharge rating curve at the tailrace is shown in ANNEX III.

3.4.3 Efficiency of Turbine and Generator

Combined efficiency of turbine and generator varies with unit capacity of plant and peration load coefficient. The efficiency is approximated by the following equation for the unit capacity of 20 to 100 MW of which operation head is 170 to 220 m;

$$F = 0.000283 \cdot (P-60) - 0.5928 \cdot A^2 + 1.0035 \cdot A + 0.482$$
$$A = Q \cdot H/(Qmax \cdot Hd)$$

where, F : combined efficiency
P : installed capacity of one unit (MW)
Q : arbitrary plant discharge (cms)
Qmax : max. plant discharge (cms)
H : effective head at a (m)
Hd : design effective head (m)

3.4.4 Power Output

Output of the power plant is calculated by :

$$P = Q \cdot H \cdot F \cdot g$$

where, P : power output (kW)
Q : turbine discharge (cms)
H : effective head (m)
F : combined efficiency of generator and turbine
g : acceleration of gravity (=9.8m/sec^2)

Installed capacity of the plant (Pmax) is thus given by; Pmax = Qmax • Hd • Fd • g, where Fd is design efficiency at full capacity operation. The installed capacity of each alternative option is shown in Table VII.3.1.

3.4.5 Energy Output

Power generation was simulated on each alternative option utilizing the two daily discharge duration curves; one for the critical period to estimate the firm energy and another for the long term period to estimate the secondary energy. The computed power outputs in respect of two different generation modes; with and without daily regulation, are illustrated in Fig. VII.3.12 in which the case B319 without regulation and the case C315 with regulation are shown as typical output pattern. By this simulation, possibly exploitable energy of the project was obtained.

Effective firm and secondary energies were then computed from the possible energies by deducting ineffective energies not exploitable due to plant stoppage as stated in Section 2.7. The effective firm and secondary energies computed for each case are shown in Table VII.3.3 and summarized below for typical cases.

Case	Max.Plant Discharge (cms)		Installed Capacity (MW)	Firm Energy (MWy)	Second. Energy (MWy)
B319-1	30		50.6	39.99	2.13
3	60		102.0	61.03	6.82
5	90		154.0	74.00	12.12
6	105		180.2	78.33	14.22
B324-1	30	*	52.0	41.13	2.15
3	60	*	104.6	63.35	6.77
5	90	*	158.0	78.29	11.53
6	105	*	185.0	84.01	13.45
C310-1	30		48.3	38.17	2.03
3	60		97.3	58.26	6.50
5	90		146.8	70.62	11.55
6	105		171.8	74.74	13.55
C315-1	30	*	49.6	39.31	2.05
3	60	*	99.9	60.53	6.46
5	90	*	150.9	74.75	11.00
6	105	*	176.6	80.19	12.83
D305-1	30		47.2	37.25	1.98
3	60		94.9	56.82	6.35
5	90		143.2	68.86	11.27
6	105		167.6	72.86	13.22
D310-1	30	*	48.5	38.39	2.01
3	60	*	97.6	59.10	6.31
5	90	*	147.3	73.00	10.74
6	105	*	172.3	78.31	12.53

* : With daily regulation

3.5 Costs of Alternative Options

3.5.1 Investment Cost

Investment cost (project construction cost) of each alternative was estimated in accordance with the ELETROBRAS's standard format of which main categories are as list below :

Item No.	Title	Works
10	Land and Facilities	Land acquisition and relocation
11	Structures and Other Improvement	Powerhouse
12	Reservoir, Dam & Waterway	Reservoir, diversion, dam, spillway, intake, headrace, surge tank, penstock, tailrace and environmental measures
13	Turbines and Generators	Turbine, generator, draft gate
14	Accessory Electrical Equipment	Electric accessory
15	Other Equipment of Powerhouse	OH crane & others
16	Access Road/Railway/Bridge	New road, railway & bridge
17	Indirect Cost	Construction camp, engineering and administration
18	Interest during Construction	

Price basis for this estimation is the prices at December 1992. Exchange rates at that time is 1 US$ = 11,163.33 Cruzeiros, or 1 US$ = 120 Japanese Yen.

Direct costs of works in the items 10 to 16 are estimated based on unit prices and quantities estimated for major work items. The unit prices of major work items are as listed in Table VII.3.4. These prices were based on cost data of recent hydropower projects in the South region of Brazil. Cost for physical, biotic and social environment measures was included in the item 12. Physical contingency; 15 % of the direct cost, was added to every sub-item except for the indirect cost. The indirect cost of the item 17 was estimated by assuming that such cost is proportional to the sum of direct costs. The rate of 29 % was applied to this estimate, which is close to the average of rates used for similar projects in Brazil.

Interest during construction was computed applying annual rate of 10 %. Construction period from beginning of tendering to commissioning of power plant was assumed to be 4 years. Cost disbursement assumed is 20 % in each first and fourth year and 30 % in each second and third year.

Estimated investment cost of every alternative scheme is listed in Table VII.3.5. The costs initially estimated in the interim report were thoroughly reviewed and modified after clarification by ELETROBRAS on the cost estimation criteria. Table VII.3.5 shows the modified costs.

3.5.2 Cost of Operation and Maintenance

Annual cost for operation and maintenance of the Project after commissioning was estimated by the following equation which was derived by ELETROBRAS in 1993 from operation records in Brazil;

$$COM = A \cdot P^{B}$$

Where, COM : annual O & M cost (US$/kW/year)

P : installed capacity (MW)

A and B: coefficients variable with plant capacity as shown below,

P (MW)	A	B
Less than 146.71	124.28	-0.61
146.71 or more	11.43	-0.1281

3.6 Optimization of Dam Site and FSL

3.6.1 Procedure

Based on the ELETROBRAS's criteria as stated in Section 2.7, the dam site and the full supply level (FSL) were optimized so as to maximize the net benefit. The optimization was made in the following three steps:

Step 1 : Tentative selection of optimal plant discharge for each combination of damsite and FSL.

Step 2: Selection of optimal FSL for each damsite applying the selected optimal plant discharge.

Step 3: Selection of optimal damsite

The scale of development or installed capacity of power plant will be optimized in the second screening.

3.6.2 Tentative Selection of Optimal Plant Discharge

In order to seek the most economical scale of each alternative scheme, the six cases of discharge for power generation (30, 45, 60, 75, 90 and 105 cms), are tentatively screened by comparing their benefits and costs. The benefit is estimated from the computed amount of firm energy and the reference cost of firm energy; 51 US$/MWh.

This optimization is made by comparison of the incremental benefit (ΔB) and the incremental cost (ΔC) since the net benefit is maximized at the point where ΔB becomes equal to ΔC. Besides, the optimum discharge is selected at the point where the value of ΔB/ΔC is still higher than and closest to 1.0.

The computations of ΔB and ΔC are shown as Table VII 3.6 and the results are summarized below.

Dam axis B (ΔC and ΔB in US$ million)

Max Plant Disch. (cms)	Case B319			Case B324					
	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC
60	-	-	-	-	-	-	-	-	-
75	26.8	31.1	1.16	28.4	36.0	1.27			
90	26.3	26.3	<u>1.00</u>	27.9	30.1	<u>1.08</u>			
105	24.2	19.1	0.79	27.4	25.3	0.92			

Dam axis C

Max Plant Disch. (cms)	Case C310			Case C315			Case C319		
	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC
60	-	-	-	-	-	-	-	-	-
75	26.5	29.7	1.12	27.1	34.3	1.27	26.9	35.1	1.30
90	24.2	25.1	<u>1.04</u>	24.5	28.7	<u>1.17</u>	24.9	29.3	1.18
105	24.0	18.2	0.76	24.5	24.1	0.99	24.5	24.7	<u>1.01</u>

Dam axis D

Max Plant Disch. (cms)	Case D305			Case D310			Case D315		
	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC	Δ C	Δ B	ΔB/ΔC
60	-	-	-	-	-	-	-	-	-
75	23.7	28.9	1.22	26.0	33.5	1.29	26.0	34.4	1.32
90	23.4	24.4	1.04	24.1	28.0	1.16	23.5	28.8	1.22
105	23.2	17.7	0.76	24.0	23.5	0.98	23.5	24.2	1.03

Since the net benefit reaches the maximum when ΔB/ΔC = 1.0 and the development scale is chosen at the point where ΔB/ΔC is larger than 1.0, the plant discharge of 90 cms is the optimal discharge for almost all cases except for the cases C319 and D315. Accordingly, the discharge of 90 cms were tentatively selected for optimization of damsite and FSL.

3.6.3 Optimal Full Supply Level

In order to seek the optimal full supply level (FSL) at each damsite, reservoir level was varied at 2 or 3 m steps and concurrent variation of cost and benefit were studied. The cost in respect of the intermediate reservoir level other than those selected initially as the alternative options were estimated by interpolation while their benefits were computed from supplemental analysis of power generation. The plant discharge was fixed at 90 cms as selected above.

In selecting the optimal FSL, possibility of daily regulation is one of major concerns. If the FSL is high enough to provide sufficient reservoir volume for daily regulation, plant operation can be continued by using stored water even though the reservoir inflow rate is lower than the restricted minimum plant discharge. The optimal FSL is therefore selected for the respective cases; "with" and "without" regulation.

In the case of "with regulation", sediment deposit in the reservoir has to be removed periodically to keep storage volume required. Volume of the annual average deposit was estimated at about 120,000 m^3 as stated in Section 3.2.5. Unit cost of the dredging including disposal of the dredged silt to spoiling area; 2 to 3 km from the dam, was estimated at 9 US$/$m^3$ on the basis of dredging cost at Blumenan.

Volume of sediment deposition will vary year by year because sediment flow depends mainly on amount of flood discharge and its frequency. If a large flood occurred, its sediment yield may reach several folds of the average annual volume. Accordingly, in the case of daily regulation, storage space below the minimum operation level (MOL) needs sufficient volume larger than those of no regulation. In this study, a dead storage space of at least 1 million m^3 is provided in the reservoir below MOL for the case of regulation. The lowest limit of FSL for regulation was therefore set at 324.0 m for the axis B, 315 m for the axis C and 310 m for the axis D.

Cost and benefit of each FSL option is calculated in Table VII.3.7 and the result is summarized below:

Dam Axis	FSL (m)	Without Regulation Cost C ($M)	Without Regulation Energy Benefit B ($M)	Without Regulation Net Benefit B-C ($M)	With Regulation Cost C ($M)	With Regulation Energy Benefit B ($M)	With Regulation Net Benefit B-C ($M)
	319	227.4	340.3	112.9			
B	322	238.3	345.6	107.3			
	* 324	245.7	349.1	103.4	256.5	358.7	102.2
	310	229.7	324.8	95.1			
	313	239.3	330.0	90.8			
C	315	245.6	332.6	87.1	256.3	342.5	86.2
	317				263.5	346.2	82.8
	319				270.5	349.9	79.4
	305	229.9	316.7	86.8			
	307	235.7	320.1	84.4			
D	310	245.1	325.7	80.7	255.8	334.5	78.6
	313				265.4	339.9	74.4
	315				272.0	343.5	71.5

$M : US$ million * : Max.limit of FSL

As seen in this table, raising of FSL results in decrease of the net benefit in both cases of "with" and "without" regulation. This means that lower dam is more economical. In addition to this economical advantage, the lower dam can minimize impacts to natural environment caused by implementation of the project. The optimal FSL for each damsite was thus selected as follows:

Dam Axis	Optimal FSL (m) Without Regulation	Optimal FSL (m) With Regulation
B	319	324
C	310	315
D	305	310

3.6.4 Optimal Damsite

The three damsite options; axis B, C and D, were evaluated on both technical and economical points of view. Major dimensions and features of each damsite option are shown in Table III.3.8. Technical advantage or disadvantage of each option is as follows:

Reservoir

The dam axis B is located on the uppermost site and its reservoir level is highest among the three options. Natural drop of river level from the beginning point of cascades to powerhouse site is almost fully utilized for power generation in the case of axis B.

Fig. VII.3.13 shows the reservoir submergence area of each option. Reservoir size is quite small and most part of the reservoir is put in the existing river channel.

The resort complex (Paraiso Comping) is located approximately 1 km upstream of the axis B. Submergence problem of this resort area has once been argued in the previous pre-feasibility study (1991) and the reservoir far apart from the resort area has been proposed. In the present study, CELESC carried out leveling survey of low land in the resort area and confinned that the present normal water level at the river margin in the resort area is 319 m in elevation and the FSL lower than 319 m does not affect the resort area.

The reservoir of higher FSL intrudes into the resort area and submerges resort facilities. In case of the highest FSL 324 m, most part of low land in the resort area is submerged. However, by the recent CELESC's investigation, it was revealed that the submergence is permissible to the owner of the resort complex and consequent problem is negotiable. Therefore, as far as the resort complex, there is no restriction to limit the FSL.

Excepting the resort area, houses to be relocated for formation of reservoirm dam and intake are only 3 for axis B, 8 for axis C and zero for axis D. Permanent residents are only 2 families for either axis B or C and the other houses are recreational house. Therefore, there is no significant difference in respect of resettlement problem among three dam axis options.

Topographic and Hydraulic Aspects

Dam axis B is located at downstream end of relatively flat river channel. This is advantageous in hydraulic aspects since the flow velocity in the reservoir is relative slow even in floods and water enters smoothly into the intake. The other axes C and D are located on the way of relatively steep river channel. Flow in their reservoirs at flood time has high velocity and rushes to dam and intake. This may cause damage on dam and intake. Furthermore, slow velocity at the axis B facilitates river diversion works during construction of dam. Therefore, the axis B is considered to be the best site among three sites in respect of topographic and hydraulic aspects.

Geologic Aspect

Hard granite is exposed on river bed at all sites. However, soil overburden is deep on both abutments of all sites except right bank of the dam axis B where granite rock appears within 10m of depth. The overburden is slightly permeable and it is probably susceptible to slide down if saturated by deep reservoir. Shallower reservoir is preferable to minimize seepage through the overburden and avoid sliding. Approximate reservoir depth is only 2 m for the axis B and 7 to 9 m for the other axes C and D. Therefore, the dam axis B is considered to be best also in the geological aspects.

Economical Comparison

From Table of VII.3.7, the project cost and the energy benefit on each damsite option are given as follows:

		Unit	Without Regulation			With Regulation		
			Dam Axis			Dam Axis		
			B	C	D	B	C	D
Optimal FSL		m	319	310	305	324.0	315.0	310.0
Minimum operation level		m	-	-	-	322.4	313.0	309.0
Max. plant discharge		cms	90	90	90	90	90	90
Installed capacity		MW	154	147	143	158	151	147
Firm energy		MWy	74.00	70.62	68.86	78.29	74.75	73.0
Secondary energy		MWy	12.12	11.55	11.27	11.53	11.00	10.74
Capitalized benefit	(B)	US$ mill.	340.3	324.8	316.7	358.7	342.5	334.9
Construction cost		US$ mill.	218.2	221.0	221.3	236.3	236.5	236.2
O & M cost		US$ mill.	9.2	8.7	8.6	20.2	19.8	19.6
Total cost	(C)	US$ mill.	227.4	229.7	229.9	256.5	256.3	255.8
Net benefit	(B-C)	US$ mill.	112.9	95.1	86.8	102.2	86.2	78.7

As seen in this table, the net benefit of the axis B is highest among three dam axis options in either case of "with" or "without" regulation.

Selected Dam Site

Among the three dam site options, the axis B is most advantageous in both technical and economical aspects. The axis B was thus selected as the optimal dam site.

3.6.5 Possibility of Daily Regulation

Peak power generation is preferable for flexible operation of power network especially for CELESC which does not possess any peak generation plant. However, as a whole of the regional network (integrated South/Southeast system) from which CELESC receives most of its required electricity, there are many reservoir type power plants capable of peak generation. Those plants are large in scale and able to supply peak power with relatively cheap price.

If the Salto Pilão project is planned for the peak generation, dam becomes larger than that for non peak generation because reservoir has to have additional storage space for discharge regulation. Such storage space has to be kept by periodical dredging of sediment deposition. These push up the construction cost as well as operation cost and consequently result in high cost of generated energy. The table in the Section 3.6.4 indicates differences of net benefit between both cases of "with" and "without" regulation. The values for the selected axis B are rewritten as follows:

		Without Regulation	With Regulation	Ratio
		(A)	(B)	(B)/(A)
FSL	m	319.0	324.0	-
Firm Energy	MWy	74.0	78.29	1.058
Benefit	US$ mill	340.3	358.7	1.053
Cost	US$ mill	227.4	256.5	1.128
Net benefit	US$ mill	112.9	102.2	0.905

As seen in this table, the peak generation scheme with regulation pond is 12.8 % higher in cost than the non-peak generation scheme while the amount of firm energy increases by 5.8 %. Consequently, the net benefit goes down by 9.5 %. This means that the peak generation by the Salto Pilão project is not economical in comparison with the non-peak generation scheme.

The Salto Pilão project is thus decided to be the pure base-load power station without regulation pond. For this selection, the JICA team discussed with the CELESC's engineers who are responsible for operation of CELESC's power net work. They expressed that system operation is not affected seriously even if the Salto Pilão is not capable of peak generation and the JICA team's conclusion is acceptable.

The finally selected scheme is the combination of the dam axis B with FSL of 319 m which is not capable of daily regulation.

4. SECOND SCREENING FOR SELECTING INSTALLED CAPACITY

4.1 General

In the preceding Chapter 3, the optimal combination of the damsite and the full supply level (FSL) was selected to be the dam axis B with FSL of 319.0 m. Daily regulation capability is not included in this scheme. Development scale or installed capacity of the selected scheme was studied in this second screening.

For the second screening, the design of project components, especially of headrace tunnel and penstock, of the selected scheme was thoroughly reviewed and their optimum layout and dimensions were decided. This design refining is detailed in ANNEX VIII. Based on the refined design, power generation study and cost estimation were carried out on the selected scheme with different installed capacities corresponding to the 6 cases of the maximum plant discharge of 30, 45, 60, 75, 90 and 105 cms. The optimum installed capacity was selected by economic comparison of their costs and energy benefits and further by engineering evaluation.

4.2 Refined Structural Design

Major refining in designs made after the first screening is described below:

Headrace Tunnel and Culvert

In the first screening, the diameter of tunnel and culvert was estimated by empirical limit of maximum velocity of flow in power waterway; i.e., 2.5 m/sec for the shotcrete-lined section and 3.5m/sec for concrete-lined section. These limits are reasonable when the plant utilization factor is 60 to 70 %. The plant utilization factor (= effective energy / installed capacity both in MW) varies with the installed capacity as shown below:

Max. Discharge (cms)	Installed Capacity (MW)	Plant Factor (%) of Case B319	
		Critical Period	Long Term
30	50.6	79	83
60	102.0	60	61
90	154.0	48	56
105	180.2	43	51

The loss of head in waterway varies with its diameter and it relates to amount of energy output of the power plant. Larger diameter results in lower head loss and consequently in higher energy production while the construction cost also increases. The most economical diameter is given at the point where the net benefit (= energy benefit-cost) becomes the maximum. According to the result of economic comparison made in ANNEX VIII, the optimal diameter for different plant discharges are as follows:

Max. Plant Discharge (cms)	Shotcrete-lines Tunnel		Concrete-lined Section*	
	D	V	D	V
30	4.4	2.0	3.6	3.0
60	5.4	2.6	4.3	4.1
90	5.8	3.4	4.8	5.0
105	5.9	3.9	4.9	5.6

Remarks, D=Optimal internal diameter in m

V=Maximum flow velocity in m/sec

*= including culvert and tunnel near surge tank

By this optimization, it was revealed that the tunnel diameter considerably smaller than that assumed in the first screening is economical. These reduced diameters are used in this second screeing.

Penstock

In the first screening, steel pipe penstock embedded in underground inclined shaft was adopted and its diameter was computed by the maximum flow velocity of 6.0 m/sec. For the second screening, the design of penstock was thoroughly reviewed in respect of alignment, diameter and necessity of steel lining. The review study was based on economic comparison and engineering justification as set out in ANNEX VIII. As the result, penstock alignment was changed to a vertical concrete-lined shaft without steel lining for upstream section and a steel-lined horizontal tunnel for downstream part. The optimal diameter of penstock was computed on each plant discharge by economic comparison on cost and benefit. The selected optimal diameters are as follows:

Max. Plant Discharge (cms)	Concrete-lined Penstock		Steel-lined Penstock	
	D	V	D	V
30	3.6	3.0	2.8	4.9
60	4.3	4.1	3.7	5.6
90	4.8	5.0	4.3	6.2
105	4.9	5.6	4.5	6.6

Remarks, D= Optimal internal diameter in m.

V= Maximum flow velocity in m/sec.

4.3 Generation Study

By applying the structural designs refined as above, the firm and secondary energies were computed again for the selected scheme of dam axis B with FSL of 319 m. The method of this generation simulation is the same as stated in Section 3.4. Besides, since the diameter of waterway tunnel and penstock was considerably changed to smaller size in the design refining, effective water head for power generation was computed again for every case of the plant discharges; 30, 45, 60, 75, 90 and 105 cms.

Efficiencies of turbine and generator were also reviewed and modified taking into account the re-estimated effective heads and plant capacities. Two-unit plant configuration was adopted also in this study. The computed effective heads and plant capacities are listed below:

		Max. Plant Discharge (cms)					
		30	45	60	75	90	105
1) Reservoir FSL	(m)	319.0	319.0	319.0	319.0	319.0	319.0
2) Tailwater level *	(m)	110.9	111.1	111.2	111.4	111.5	111.7
3) Gross Static head	(m)	208.1	207.9	207.8	207.6	207.5	207.3
4) Average plant disch.**	(cms)	25.4	34.1	40.6	45.7	49.9	53.9
5) Loss of head , Max.	(m)	15.8	19.7	22.0	26.5	28.2	32.8
, Average	(m)	11.4	11.3	10.1	9.8	8.7	8.6
6) Effective head, Max.	(m)	192.8	188.2	185.8	181.1	179.3	174.5
, Average	(m)	196.7	196.6	197.8	198.2	199.1	199.3
7) Combined efficiency*		0.885	0.888	0.892	0.895	0.898	0.901
8) Installed capacity	(MW)	50.0	73.8	97.4	119.2	142.0	161.8
9) Number of units		2	2	2	2	2	2
(10) Unit Capacity	(MW)		36.9	48.7	59.6	71.0	80.9

*: at full capacity operation

**: in critical period

The generation simulation was made by applying the duration curves of two daily discharge series of critical and long term periods as mentioned in Subsection 3.4.1. The effective firm and secondary energies computed are as follows:

Effective Energy		Max. Plant Discharge (cms) 30	45	60	75	90	105
Firm energy	(MWy)	38.09	50.16	59.07	65.14	70.51	74.35
Secondary energy	(MWy)	2.01	3.94	6.39	8.45	11.03	12.45

4.4 Cost Estimation

Investment cost of each case was re-estimated by applying the same criteria as mentioned in Section 3.5. Besides, unit costs of civil works as well as electrical/mechanical equipment were thoroughly reviewed on the basis of work quantities calculated on the refined design. The estimated investment cost of each case is shown in Table VII.4.1 and summarized below:

Unit: Million US$

		Max. Plant Discharge (cms)					
Item No.	Cost Items	30	45	60	75	90	105
10 to 16	Direct cost	75.3	94.2	111.4	124.9	137.4	149.1
17	Indirect cost	21.8	27.3	32.3	36.2	39.8	43.3
	Sub-total :	97.1	121.5	143.7	161.1	177.2	192.4
18	Interest during construction	20.9	26.2	31.0	34.8	38.3	41.5
	Total	118.1	147.7	174.3	195.9	215.5	233.9

(December 1992 price level)

Annual cost for operation and maintenance after commissioning of the project was estimated by the equation given in Section 3.5.2 and the result is tabulated below:

Unit: Million US$

	Max. Plant Discharge (cms)					
	30	45	60	75	90	105
Amount of O & M cost	0.57	0.67	0.74	0.80	0.86	0.96

4.5 Optimization of Development Scale

<u>Net Benefit</u>

Development scale of power project is expressed by its installed capacity. The optimal installed capacity was selected so as to maximize the net benefit. The cost and energy benefit estimated for every case of the plant discharge are tabulated below:

Max Pant Discharge (cms)	Installed Capacity (MW)	Capitalized Cost (Mill. US$)			Capitalized Energy Benefit (Mill.US$)	Net Benefit (Mill. US$)
		Investment	O & M	Total C	B	B-C
30	50.0	118.1	5.7	123.8	170.8	47.0
45	73.8	147.8	6.6	154.5	226.3	71.9
60	97.4	174.7	7.3	182.0	268.3	86.3
75	119.2	195.9	8.0	203.9	297.3	93.4
90	142.0	215.5	8.5	224.0	323.8	99.8
105	161.8	233.9	9.6	243.5	342.2	98.7

According to an increasing tendency of the above net benefits, the maximum net benefit will be gained at a plant discharge between 90 and 105 cms or at a plant capacity between 142 and 162 MW.

Mean Cost of Generation

The generation cost to evaluate the competitiveness of power project is obtained by the equation shown in (7) of Section 2.7. The generation cost of each case was computed by applying the two reference costs; 51 US$/MWh of firm energy and 11.97 US$/MWh of secondary energy. The result is as follows:

Max. Plant Discharge (cms)	Installed Capacity (MW)	Mean Cost of Generation (US$/MWh)
30	50.0	36.8
45	73.8	34.5
60	97.4	34.2
75	119.2	34.5
90	142.0	34.7
105	161.8	35.7

Optimal Installed Capacity

The above comparison of the net benefit suggests that the optimal installed capacity falls between 142 and 162 MW. However, the lowest generation cost is obtained by the installed capacity of around 100 MW. The generation cost gradually increases with the scale of plant and its increasing rate becomes bigger in the range over 142 MW. This means that larger plant is lower in its competitiveness. The plant capacity of 142 MW was thus adopted as the optimal installed capacity for the Salto Pilao project.

4.6 Optimal Development Scheme

Based on the results of the first and second screening, the following scheme was finally selected as the optimal scheme;

Dam site:		Axis B
Reservoir ,	Full supply level (FSL)	EL.319.0m
,	Volume at FSL	280.000 m^3
,	Area at FSL	16 ha
,	Regulation storage	Non
Dam, Crest level, Concrete section		EL. 325.0 m
Dam, Crest level, Embankment		EL. 326.0 m
Spillway ,	Discharge capacity	5,300 cms
,	Overflow width	200 m
Intake ,	Design max. discharge	90 cms
Headrace ,	Culvert, type:	Circular Concrete
,	Culvert, diameter x length	4.8 m x 404 m, 5.8 m x 50 m
,	Tunnel, type:	Horseshoe shape shotcrete-lined tunnel (partially, concrete-lined circular tunnel)
,	Tunnel, length:	5.637 m
,	Tunnel, diameter:	5.8 m (partially 4.8 m)
Surge tank, Type:		Cylinder type
Penstock, Type:		Vertical concrete-lined shaft in upstream section and horizontal steel-lined tunnel in downstream section
Penstock, diameter x length x nos.		4.8 m x 264.4 m x 1 no. 4.3 m x 292 m x 1 no. 2.5 m x 31.5 m x 2 nos.
Powerhouse, Type:		Ordinary surface type concrete building
Generating Equipment		
• Number of units:		2
• Maximum discharge:		90 cms for 2 units

• Rated head	179.3 m
• Installed capacity	142 MW (= 2 x 71 MW)

Table

Table VII.3.1 List of Alternative Options for First Screening

Case No.	Dam Axis	Maximum Plant Discharge (cms)	Reservoir Level FSL (m)	Reservoir Level MOL (m)	Reservoir Level Draw-Down (m)	Reservoir Volume FSL (TCM)	Reservoir Volume MOL (TCM)	Reservoir Volume Active (TCM)	Tail Water Level (m)	Static Gross Head (m)	Rated Head Loss (m)	Rated Effect. Head (m)	Design Efficiency	Installed Capacity (MW)	Nos. of Unit	Unit Capacity (MW)
B319- 1	B	30	319	319	0	280	280	0	110.91	208.09	12.98	195.11	0.883	50.6	2	25.30
2		45	319	319	0	280	280	0	111.07	207.93	12.98	194.94	0.887	76.2	2	38.10
3		60	319	319	0	280	280	0	111.23	207.77	12.98	194.79	0.890	102.0	2	51.00
4		75	319	319	0	280	280	0	111.38	207.62	12.98	194.63	0.894	127.9	2	63.95
5		90	319	319	0	280	280	0	111.53	207.47	12.98	194.48	0.897	154.0	2	77.00
6		105	319	319	0	280	280	0	111.67	207.33	12.98	194.34	0.901	180.2	2	90.10
B324- 1	B	30	324	323.56	0.44	1,473	1,343	130	110.91	213.09	12.98	200.11	0.883	52.0	2	26.00
2		45	324	323.34	0.66	1,473	1,279	194	111.07	212.93	12.98	199.94	0.887	78.2	2	39.10
3		60	324	323.11	0.89	1,473	1,214	259	111.23	212.77	12.98	199.79	0.891	104.6	2	52.30
4		75	324	322.89	1.11	1,473	1,149	324	111.38	212.62	12.98	199.63	0.894	131.2	2	65.60
5		90	324	322.67	1.33	1,473	1,084	389	111.53	212.47	12.98	199.48	0.898	158.0	2	79.00
6		105	324	322.45	1.55	1,473	1,019	454	111.67	212.33	12.98	199.34	0.902	185.0	2	92.50
C310- 1	C	30	310	310	0	600	600	0	110.91	199.09	12.80	186.29	0.883	48.3	2	24.15
2		45	310	310	0	600	600	0	111.07	198.93	12.80	186.13	0.886	72.7	2	36.35
3		60	310	310	0	600	600	0	111.23	198.77	12.80	185.97	0.889	97.3	2	48.65
4		75	310	310	0	600	600	0	111.38	198.62	12.80	185.82	0.893	122.0	2	61.00
5		90	310	310	0	600	600	0	111.53	198.47	12.80	185.67	0.896	146.8	2	73.40
6		105	310	310	0	600	600	0	111.67	198.33	12.80	185.53	0.900	171.8	2	85.90
C315- 1	C	30	315	314.38	0.62	1,449	1,319	130	110.91	204.09	12.80	191.29	0.883	49.6	2	24.80
2		45	315	314.06	0.94	1,449	1,255	194	111.07	203.93	12.80	191.13	0.886	74.7	2	37.35
3		60	315	313.74	1.26	1,449	1,190	259	111.23	203.77	12.80	190.97	0.890	99.9	2	49.95
4		75	315	313.40	1.60	1,449	1,125	324	111.38	203.62	12.80	190.82	0.893	125.3	2	62.65
5		90	315	313.04	1.96	1,449	1,060	389	111.53	203.47	12.80	190.67	0.897	150.9	2	75.45
6		105	315	312.68	2.32	1,449	995	454	111.67	203.33	12.80	190.53	0.901	176.6	2	88.30
C319- 1	C	30	319	318.55	0.45	2,457	2,327	130	110.91	208.09	12.80	195.29	0.883	50.7	2	25.35
2		45	319	318.32	0.68	2,457	2,263	194	111.07	207.93	12.80	195.13	0.887	76.3	2	38.15
3		60	319	318.09	0.91	2,457	2,198	259	111.23	207.77	12.80	194.97	0.890	102.0	2	51.00
4		75	319	317.85	1.15	2,457	2,133	324	111.38	207.62	12.80	194.82	0.894	128.0	2	64.00
5		90	319	317.60	1.40	2,457	2,068	389	111.53	207.47	12.80	194.67	0.898	154.1	2	77.05
6		105	319	317.35	1.65	2,457	2,003	454	111.67	207.33	12.80	194.53	0.901	180.4	2	90.20
D305- 1	D	30	305	305	0	1,150	1,150	0	110.91	194.09	12.25	181.85	0.882	47.2	2	23.60
2		45	305	305	0	1,150	1,150	0	111.07	193.93	12.25	181.68	0.886	71.0	2	35.50
3		60	305	305	0	1,150	1,150	0	111.23	193.77	12.25	181.52	0.889	94.9	2	47.45
4		75	305	305	0	1,150	1,150	0	111.38	193.62	12.25	181.37	0.893	119.0	2	59.50
5		90	305	305	0	1,150	1,150	0	111.53	193.47	12.25	181.22	0.896	143.2	2	71.60
6		105	305	305	0	1,150	1,150	0	111.67	193.33	12.25	181.08	0.899	167.6	2	83.80
D310- 1	D	30	310	309.69	0.31	2,804	2,674	130	110.91	199.09	12.25	186.85	0.883	48.5	2	24.25
2		45	310	309.53	0.47	2,804	2,610	194	111.07	198.93	12.25	186.68	0.886	72.9	2	36.45
3		60	310	309.37	0.63	2,804	2,545	259	111.23	198.77	12.25	186.52	0.890	97.6	2	48.80
4		75	310	309.21	0.79	2,804	2,480	324	111.38	198.62	12.25	186.37	0.893	122.3	2	61.15
5		90	310	309.04	0.96	2,804	2,415	389	111.53	198.47	12.25	186.22	0.897	147.3	2	73.65
6		105	310	308.87	1.13	2,804	2,350	454	111.67	198.33	12.25	186.08	0.900	172.3	2	86.15
D315- 1	D	30	315	314.75	0.25	5,211	5,081	130	110.91	204.09	12.25	191.85	0.883	49.8	2	24.90
2		45	315	314.62	0.38	5,211	5,017	194	111.07	203.93	12.25	191.68	0.886	74.9	2	37.45
3		60	315	314.49	0.51	5,211	4,952	259	111.23	203.77	12.25	191.52	0.890	100.2	2	50.10
4		75	315	314.36	0.64	5,211	4,887	324	111.38	203.62	12.25	191.37	0.894	125.7	2	62.85
5		90	315	314.23	0.77	5,211	4,822	389	111.53	203.47	12.25	191.22	0.897	151.3	2	75.65
6		105	315	314.11	0.89	5,211	4,757	454	111.67	203.33	12.25	191.08	0.901	177.1	2	88.55

Note: FSL = Full Supply Level
MOL = Minimum Water Level
TCM = Thousand cubic meter

Table VII.3.2 Principal Features of Alternative Scheme (1/8) - Dam Axis B

	Unit	B319-30	B319-45	B319-60	B319-75	B319-90	B319-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	319.0	319.0	319.0	319.0	319.0	319.0
Minimum Operating Level	masl	319.0	319.0	319.0	319.0	319.0	319.0
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	50.7	76.2	102	127.9	154	180.3
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	21.5	21.5	21.5	21.5	21.5	21.5
Active storage	MCM	0	0	0	0	0	0
Total storage	MCM	0.28	0.28	0.28	0.28	0.28	0.28
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	325.2	325.2	325.2	325.2	325.2	325.2
Dam - height x length	m	18 x 248	18 x 248	18 x 248	18 x 248	18 x 248	18 x 248
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	319.0	319.0	319.0	319.0	319.0	319.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	313.0	313.0	313.0	313.0	313.0	313.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.3 x 1.0 x 9	2.5 x 1.0 x 9	1.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 528	4.1 x 518	4.7 x 500	5.2 x 488	5.7 x 483	6.2 x 469
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,030	4.8 x 5,030	5.5 x 5,030	6.2 x 5,030	6.8 x 5,030	7.3 x 5,030
Tunnel with concrete lined - dia. x length	m	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	334.1	334.1	334.1	334.1	334.1	334.1
Lower surging level	masl	306.2	306.2	306.2	306.2	306.2	306.2
Penstock							
Size (dia. x length)	m	2.5 x 503	3.1 x 499	3.6 x 496	4.0 x 494	4.4 x 491	4.7 x 489
Powerhouse							
Size (length x width x height)	m	31 x 17 x 25	36 x 20 x 28	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	53 x 30 x 36
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	63	68	72	76	80	85
Replace of House	nos.	16	16	16	16	16	16

Note ; C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (2/8) - Dam Axis B

	Unit	B324-30	B324-45	B324-60	B324-75	B324-90	B324-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	324.0	324.0	324.0	324.0	324.0	324.0
Minimum Operating Level	masl	323.56	323.34	323.11	322.89	322.67	322.45
Maximum Plant Discharge	m3/sec	30	45	60	75	90	105
Installed Capacity	MW	52	78.2	104.7	131.3	158.1	185.1
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	33	33	33	33	33	33
Active storage	MCM	0.13	0.19	0.26	0.32	0.39	0.45
Total storage	MCM	1.47	1.47	1.47	1.47	1.47	1.47
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	330.2	330.2	330.2	330.2	330.2	330.2
Dam - height x length	m	22 x 263	22 x 263	22 x 263	22 x 263	22 x 263	22 x 263
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	324.0	324.0	324.0	324.0	324.0	324.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/sec	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	320.5	319.3	319.1	318.9	318.7	318.4
Trash rack - size (h x w x nos.)	m	5.0 x 3.5 x 2	5.7 x 4.7 x 2	5.0 x 4.9 x 3	6.3 x 5.1 x 3	5.7 x 5.3 x 4	6.6 x 5.6 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.3 x 1.0 x 9	2.5 x 1.0 x 9	1.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 528	4.1 x 518	4.7 x 500	5.2 x 488	5.7 x 483	6.2 x 469
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,030	4.8 x 5,030	5.5 x 5,030	6.2 x 5,030	6.8 x 5,030	7.3 x 5,030
Tunnel with concrete lined - dia. x length	m	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	339.1	339.1	339.1	339.1	339.1	339.1
Lower surging level	masl	311.2	311.2	311.2	311.2	311.2	311.2
Penstock							
Size (dia. x length)	m	2.5 x 503	3.1 x 499	3.6 x 496	4.0 x 494	4.4 x 491	4.7 x 489
Powerhouse							
Size (length x width x height)	m	31 x 17 x 25	36 x 20 x 28	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	53 x 30 x 36
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	91	96	100	104	108	113
Replace of House	nos.	25	25	25	25	25	25

Note ; C.G. : Concrete Gravity dam

Table VII.3.2 Principal Features of Alternative Scheme (3/8) - Dam Axis C

	Unit	C310-30	C310-45	C310-60	C310-75	C310-90	C310-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	310.0	310.0	310.0	310.0	310.0	310.0
Minimum Operating Level	masl	310.0	310.0	310.0	310.0	310.0	310.0
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	48.3	72.7	97.3	122	146.8	171.8
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	17	17	17	17	17	17
Active storage	MCM	0	0	0	0	0	0
Total storage	MCM	0.6	0.6	0.6	0.6	0.6	0.6
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	316.2	316.2	316.2	316.2	316.2	316.2
Dam - height x length	m	17 x 262	17 x 262	17 x 262	17 x 262	17 x 262	17 x 262
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	310.0	310.0	310.0	310.0	310.0	310.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x7.0 x 1	5.0 x7.0 x 1	5.0 x7.0 x 1	5.0 x7.0 x 1	5.0 x7.0 x 1	5.0 x7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	304.0	304.0	304.0	304.0	304.0	304.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x. 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 85	4.1 x 75	4.7 x 65	5.2 x 75	5.7 x 65	6.2 x 55
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,300	4.8 x 5,300	5.5 x 5,300	6.2 x 5,300	6.8 x 5,300	7.3 x 5,300
Tunnel with concrete lined - dia. x length	m	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	325.3	325.3	325.3	325.3	325.3	325.3
Lower surging level	masl	296.8	296.8	296.8	296.8	296.8	296.8
Penstock							
Size (dia. x length)	m	2.5 x 499	3.1 x 495	3.6 x 492	4.0 x 489	4.4 x 487	4.7 x 485
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	36 x 20 x 27	41 x 23 x 30	45 x25 x 32	49 x 27 x 34	52 x 29 x 35
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	76	80	84	88	93	98
Replace of House	nos.	20	20	20	20	20	20

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (4/8) - Dam Axis C

	Unit	C315-30	C315-45	C315-60	C315-75	C315-90	C315-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	315.0	315.0	315.0	315.0	315.0	315.0
Minimum Operating Level	masl	314.38	314.06	313.74	313.4	313.04	312.68
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	49.7	74.7	99.9	125.3	150.9	176.6
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	19.8	19.8	19.8	19.8	19.8	19.8
Active storage	MCM	0.13	0.19	0.26	0.32	0.39	0.45
Total storage	MCM	1.45	1.45	1.45	1.45	1.45	1.45
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	321.2	321.2	321.2	321.2	321.2	321.2
Dam - height x length	m	23 x 266	23 x 266	23 x 266	23 x 266	23 x 266	23 x 266
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	315.0	315.0	315.0	315.0	315.0	315.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	309.0	309.0	309.0	309.0	309.0	309.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3. 6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 3.4 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3. 6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 85	4.1 x 75	4.7 x 65	5.2 x 75	5.7 x 65	6.2 x 55
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,300	4.8 x 5,300	5.5 x 5,300	6.2 x 5,300	6.8 x 5,300	7.3 x 5,300
Tunnel with concrete lined - dia. x length	m	3.3 x 600	3.3 x 600	3.3 x 600	5.2 x 600	2.5 x 600	3.3 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	330.3	330.3	330.3	330.3	330.3	330.3
Lower surging level	masl	301.2	301.2	301.2	301.2	301.2	301.2
Penstock							
Size (dia. x length)	m	2.5 x 501	3.1 x 497	3.6 x 494	4.0 x 491	4.4 x 489	4.7 x 486
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	36 x 20 x 28	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	52 x 29 x 35
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	84	88	93	97	102	108
Replace of House	nos.	20	20	20	20	20	20

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (5/8) - Dam Axis C

	Unit	C319-30	C319-45	C319-60	C319-75	C319-90	C319-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	319.0	319.0	319.0	319.0	319.0	319.0
Minimum Operating Level	masl	318.55	318.32	318.09	317.85	317.6	317.35
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	50.7	76.3	102.1	128	154.1	180.4
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	43.1	43.1	43.1	43.1	43.1	43.1
Active storage	MCM	0.13	0.19	0.26	0.32	0.39	0.45
Total storage	MCM	2.46	2.46	2.46	2.46	2.46	2.46
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	325.2	325.2	325.2	325.2	325.2	325.2
Dam - height x length	m	27 x 277	27 x 277	27 x 277	27 x 277	27 x 277	27 x 277
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	319.0	319.0	319.0	319.0	319.0	319.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	313.0	313.0	313.0	313.0	313.0	313.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 85	4.1 x 75	4.7 x 65	5.2 x 75	5.7 x 65	6.2 x 55
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,300	4.8 x 5,300	5.5 x 5,300	6.2 x 5,300	6.8 x 5,300	7.3 x 5,300
Tunnel with concrete lined - dia. x length	m	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600	3.3 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	334.3	334.3	334.3	334.3	334.3	334.3
Lower surging level	masl	305.4	305.4	305.4	305.4	305.4	305.4
Penstock							
Size (dia. x length)	m	2.5 x 502	3.1 x 499	3.6 x 495	4.0 x 493	4.4 x 490	4.7 x 488
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	36 x 20 x 28	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	53 x 30 x 36
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	1.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x2.8	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	116	121	125	130	134	140
Replace of House	nos.	26	26	26	26	26	26

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (6/8) - Dam Axis D

	Unit	D305-30	D305-45	D305-60	D305-75	D305-90	D305-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	305.0	305.0	305.0	305.0	305.0	305.0
Minimum Operating Level	masl	305.0	305.0	305.0	305.0	305.0	305.0
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	47.2	71	94.9	119	143.2	167.6
PROJECT COMPONENTS							
Pondage							
Pondage area	ha	28.7	28.7	28.7	28.7	28.7	28.7
Active storage	MCM	0	0	0	0	0	0
Total storage	MCM	1.15	1.15	1.15	1.15	1.15	1.15
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	311.2	311.2	311.2	311.2	311.2	311.2
Dam - height x length	m	18 X 268	18 X 268	18 X 268	18 X 268	18 X 268	18 X 268
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	305.0	305.0	305.0	305.0	305.0	305.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	299.0	299.0	299.0	299.0	299.0	299.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 44 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 83	4.1 x 73	4.7 x 60	5.2 x 48	5.7 x 48	6.2 x 34
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,460	4.8 x 5,460	5.5 x 5,460	6.2 x 5,460	6.8 x 5,460	7.3 x 5,460
Tunnel with concrete lined - dia. x length	m	3.3 x 600	4.1 x 600	4.7 x 600	5.2 x 600	5.7 x 600	6.2 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	320.0	320.0	320.0	320.0	320.0	320.0
Lower surging level	masl	292.4	292.4	292.4	292.4	292.4	292.4
Penstock							
Size (dia. x length)	m	2.5 x 497	3.1 x 493	3.6 x 491	4.0 x 487	4.4 x 485	4.7 x 483
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	35 x 20 x 27	40 x 23 x 30	45 x 25 x 32	48 x 27 x 33	52 x 29 x 35
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	96	100	106	110	117	124
Replace of House	nos.	28	28	28	28	28	28

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (7/8) - Dam Axis D

	Unit	D310-30	D310-45	D310-60	D310-75	D310-90	D310-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	310.0	310.0	310.0	310.0	310.0	310.0
Minimum Operating Level	masl	309.69	309.53	309.37	309.21	309.04	308.87
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	48.5	73	97.6	122.4	147.3	172.4
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	38.8	38.8	38.8	38.8	38.8	38.8
Active storage	MCM	0.13	0.19	0.26	0.32	0.39	0.45
Total storage	MCM	2.8	2.8	2.8	2.8	2.8	2.8
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	316.2	316.2	316.2	316.2	316.2	316.2
Dam - height x length	m	23 x 279	23 x 279	23 x 279	23 x 279	23 x 279	23 x 279
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	310.0	310.0	310.0	310.0	310.0	310.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1	5.0 x 7.0 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	304.0	304.0	304.0	304.0	304.0	304.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.7 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.0 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x 4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 83	4.1 x 73	4.7 x 60	5.2 x 48	5.7 x 48	6.2 x 34
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,460	4.8 x 5,460	5.5 x 5,460	6.2 x 5,460	6.8 x 5,460	7.3 x 5,460
Tunnel with concrete lined - dia. x length	m	3.3 x 600	4.1 x 600	4.7 x 600	5.2 x 600	5.7 x 600	6.2 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	325.0	325.0	325.0	325.0	325.0	325.0
Lower surging level	masl	286.1	286.1	286.1	286.1	286.1	286.1
Penstock							
Size (dia. x length)	m	2.5 x 499	3.1 x 495	3.6 x 492	4.0 x 489	4.4 x 487	4.7 x 484
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	36 x 20 x 27	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	52 x 29 x 35
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	101	104	107	110	111	112
Replace of House	nos.	28	28	28	28	28	28

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.2 Principal Features of Alternative Scheme (8/8) - Dam Axis D

	Unit	D315-30	D315-45	D315-60	D315-75	D315-90	D315-105
HYDROPOWER GENERATION							
Full Supply Level (FSL)	masl	315.0	315.0	315.0	315.0	315.0	315.0
Minimum Operating Level	masl						
Maximum Plant Discharge	m3/se	30	45	60	75	90	105
Installed Capacity	MW	49.8	74.9	100.2	125.7	151.3	177.1
PROJECT COMPONENTS							
Pontage							
Pontage area	ha	45.9	45.9	45.9	45.9	45.9	45.9
Active storage	MCM	0.13	0.19	0.26	0.32	0.39	0.45
Total storage	MCM	5.21	5.21	5.21	5.21	5.21	5.21
Dam & Spillway							
Dam type		C.G.	C.G.	C.G.	C.G.	C.G.	C.G.
Crest elevation of non-overflow section	masl	321.2	321.2	321.2	321.2	321.2	321.2
Dam - height x length	m	29 x 286	29 x 286	29 x 286	29 x 286	29 x 286	29 x 286
Spillway type		N.G.	N.G.	N.G.	N.G.	N.G.	N.G.
Crest elevation of weir	masl	315.0	315.0	315.0	315.0	315.0	315.0
Length of overflow section	m	200	200	200	200	200	200
Design discharge	m3/se	3,700	3,700	3,700	3,700	3,700	3,700
Sand flush gate - size (h x w x nos.)	m	5.0 x 70 x 1	5.0 x 70 x 1	5.0 x 70 x 1	5.0 x 70 x 1	5.0 x 70 x 1	5.0 x 70 x 1
Intake & Desanding Basin							
Intake Sill Elevation	masl	309.0	309.0	309.0	309.0	309.0	309.0
Trash rack - size (h x w x nos.)	m	5.0 x 3.0 x 2	5.1 x 4.0 x 2	5.0 x 4.0 x 3	6.3 x 4.0 x 3	5.7 x 4.0 x 4	6.6 x 4.0 x 4
Inlet gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x4
Desanding basin - size (l x w x h)	m	140 x 28 x 5.0	150 x 34 x 5.8	160 x 49 x 5.2	150 x 49 x 6.4	160 x 65 x 5.7	170 x 65 x 6.7
Intake gate - size (h x w x nos.)	m	3.1 x 3.0 x 2	3.3 x 4.0 x 2	3.0 x 4.0 x 3	3.5 x 4.0 x 3	3.3 x 4.0 x 4	3.6 x 4.0 x4
Sanddrain gate - size (h x w x nos.)	m	1.5 x 1.0 x 6	1.5 x 1.0 x 6	2.5 x 1.0 x 9	2.5 x 1.0 x 9	2.5 x 1.0 x 9	1.5 x 1.0 x 12
Headrace							
Culvert channel - dia x length	m	3.3 x 83	4.1 x 73	4.7 x 60	5.2 x 48	5.7 x 48	6.2 x 34
Tunnel with shotcrete lined - dia. x length	m	3.9 x 5,460	4.8 x 5,460	5.5 x 5,460	6.2 x 5,460	6.8 x 5,460	7.3 x 5,460
Tunnel with concrete lined - dia. x length	m	3.3 x 600	4.1 x 600	4.7 x 600	5.2 x 600	5.7 x 600	6.2 x 600
Surge Tank							
Diameter	m	11.7	14.4	16.5	18.6	20.4	21.9
Upper surging level	masl	330.0	330.0	330.0	330.0	330.0	330.0
Lower surging level	masl	302.1	302.1	302.1	302.1	302.1	302.1
Penstock							
Size (dia. x length)	m	2.5 x 489	3.1 x 498	3.6 x 494	4.0 x 492	4.4 x 489	4.7 x 487
Powerhouse							
Size (length x width x height)	m	30 x 17 x 25	36 x 20 x 28	41 x 23 x 30	45 x 25 x 32	49 x 27 x 34	52 x 29 x 35
No. of units	nos.	2	2	2	2	2	2
Type of turbine		Francis	Francis	Francis	Francis	Francis	Francis
Draft tube gate - size (h x w x nos.)	m	2.0 x 1.8 x 2	2.4 x 2.2 x 2	2.8 x 2.5 x 2	3.1 x 2.8 x 2	3.4 x 3.1 x 2	3.7 x 3.3 x 2
Access road							
Access Road - new construction	km	2	2	2	2	2	2
LAND ACQUISITION AND COMPENSATION							
Acquisition of Land	ha	110	115	119	124	128	134
Replace of House	nos.	28	28	28	28	28	28

Note : C.G. : Concrete Gravity dam
N.G.: Non Gated Spillway

Table VII.3.3 Plant Capacity and Energy Output

Case	Dam Axis	Reserv. FSL (m)	Max. Plant Discharge (cms)	Installed Capacity (MW)	Energy Producible Firm (MWy)	Energy Producible Secondary (MWy)	Effective Energy Firm (MWy)	Effective Energy Secondary (MWy)	Remarks
B319- 1	B	319	30	50.6	42.86	2.28	39.99	2.13	
2			45	76.2	57.33	4.61	51.48	4.14	
3			60	102.0	67.96	7.60	61.03	6.82	
4			75	127.9	75.79	10.77	68.06	9.68	
5			90	154.0	82.41	13.49	74.00	12.12	
6			105	180.2	87.22	15.83	78.33	14.22	
B324- 1	B	324	30	52.0	44.09	2.31	41.13	2.15	With Daily Regulation
2			45	78.2	59.23	4.61	53.19	4.14	- do -
3			60	104.6	70.55	7.54	63.35	6.77	- do -
4			75	131.2	79.60	10.36	71.48	9.30	- do -
5			90	158.0	87.18	12.84	78.29	11.53	- do -
6			105	185.0	93.55	14.98	84.01	13.45	- do -
C310- 1	C	310	30	48.3	40.91	2.18	38.17	2.03	
2			45	72.7	54.72	4.39	49.14	3.95	
3			60	97.3	64.87	7.24	58.26	6.50	
4			75	122.0	72.33	10.27	64.96	9.22	
5			90	146.8	78.64	12.86	70.62	11.55	
6			105	171.8	83.22	15.09	74.74	13.55	
C315- 1	C	315	30	49.6	42.14	2.20	39.31	2.05	With Daily Regulation
2			45	74.7	56.60	4.40	50.83	3.95	- do -
3			60	99.9	67.40	7.19	60.53	6.46	- do -
4			75	125.3	76.03	9.89	68.28	8.88	- do -
5			90	150.9	83.24	12.25	74.75	11.00	- do -
6			105	176.6	89.30	14.29	80.19	12.83	- do -
C319- 1	C	319	30	50.7	43.02	2.25	40.14	2.10	- do -
2			45	76.3	57.79	4.49	51.90	4.03	- do -
3			60	102.0	68.83	7.35	61.81	6.60	- do -
4			75	128.0	77.66	10.10	69.73	9.07	- do -
5			90	154.1	85.03	12.52	76.36	11.24	- do -
6			105	180.4	91.24	14.60	81.93	13.11	- do -
D305- 1	D	305	30	47.2	39.92	2.12	37.25	1.98	
2			45	71.0	53.39	4.29	47.94	3.85	
3			60	94.9	63.27	7.07	56.82	6.35	
4			75	119.0	70.54	10.02	63.35	9.00	
5			90	143.2	76.68	12.55	68.86	11.27	
6			105	167.6	81.13	14.72	72.86	13.22	
D310- 1	D	310	30	48.5	41.15	2.15	38.39	2.01	With Daily Regulation
2			45	72.9	55.27	4.29	49.63	3.86	- do -
3			60	97.6	65.81	7.02	59.10	6.31	- do -
4			75	122.3	74.24	9.65	66.67	8.67	- do -
5			90	147.3	81.29	11.96	73.00	10.74	- do -
6			105	172.3	87.21	13.95	78.31	12.53	- do -
D315- 1	D	315	30	49.8	42.25	2.21	39.42	2.06	- do -
2			45	74.9	56.75	4.41	50.96	3.96	- do -
3			60	100.2	67.58	7.22	60.69	6.48	- do -
4			75	125.7	76.24	9.92	68.47	8.91	- do -
5			90	151.3	83.49	12.29	74.97	11.04	- do -
6			105	177.1	89.58	14.34	80.44	12.88	- do -

Table. VII.3.4 Unit Costs of Major Construction Items

WORK ITEMS	UNIT	UNIT PRICE (US$)
DIRECT COST		
(1) LAND ACQUISITION & COMPENSATION		
Land Acquisition		
Dam site		
Left bank - flat area	ha	1,500
- hillside	ha	1,000
- resort complex	LS	50,000
Right bank - flat area	ha	1,000
- hillside	ha	800
Surge tank, Penstock & Powerhouse site		
Adjacent to municipal roads	lm	100
Other area	ha	1,000
Compensation		
House	nos.	6,000
(2) CIVIL WORK		
Earth Works		
Open excavation in common	cu.m	5.8
Open excavation in rock	cu.m	12.8
Underground excavation for headrace tunnel	cu.m	75.0
Underground excavation for surge tank	cu.m	99.5
Underground excavation for penstock shaft	cu.m	105.2
Embankment Works		
Embankment of rocks	cu.m	16.0
Embankment impervious soils	cu.m	11.0
Concrete Work		
Mass concrete	cu.m	99.0
Structure concrete	cu.m	136.2
Lining concrete for headrace tunnel	cu.m	165.0
Lining concrete for surge tank	cu.m	149.3
Lining concrete for penstock	cu.m	135.9
Shotcrete concrete	cu.m	212.0
Cement	ton	190.0
Reinforcement bar	ton	1,400.0
Foundation Treatment		
Preparation of foundation	sq.m	4.4
Consolidation grouting	lin-m	72.5
Access Road		
For construction purpose - new construction	km	300,000
Permanent use - new construction	km	500,000
(3) HYDRO-MECHANICAL EQUIPMENT		
Penstock steel pipe (FOB+Transportation+Erection/test)	ton	3,322
Gate (FOB+Transportation+Erection/test)	ton	6,344
Mechanical rake (FOB+Transportation+Erection/test)	nos.	244,000
(4) ELECTRICAL EQUIPMENT		
Estimated based on international price		
(5) ENVIRONMENT	: Estimated based on environmental study	
(6) CONTINGENCY	: 15 % for civil works and land & facilities (account no. 10) : 10% for electrical & hydro-mechanical equipment	
INDIRECT COST	: 29% of direct cost	

Table VII.3.5 Estimated Cost for 1st Screening (1/3)

Axis B and FSL 319 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec) 30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	258	258	258	258	258	258
11.	Powerhouse	3,132	4,525	6,151	7,987	9,847	11,892
12.	Reservoir, Dam & Waterway	46,297	58,768	70,425	81,471	91,944	100,852
13.	Turbine & Generator	8,761	12,899	16,562	19,726	22,998	26,311
14.	Accessary Electrical Equipm	5,136	5,938	6,632	7,219	7,749	8,286
15.	Other Equipment	2,430	3,219	3,827	4,249	4,809	5,397
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	67,358	87,014	105,311	122,414	139,158	154,598
INDIRECT COST		19,534	25,234	30,540	35,500	40,356	44,833
TOTAL COST WITHOUT INTEREST		86,891	112,248	135,851	157,914	179,514	199,431
INTEREST DURING CONSTRUCTION		18,749	24,221	29,314	34,075	38,735	43,033
TOTAL COST WITH INTEREST		105,641	136,469	165,165	191,989	218,250	242,464

Axis B and FSL 324 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec) 30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	306	306	306	306	306	306
11.	Powerhouse	3,132	4,602	6,187	8,023	9,918	12,306
12.	Reservoir, Dam & Waterway	54,512	67,259	79,379	91,373	102,656	113,115
13.	Turbine & Generator	8,982	13,193	16,901	20,112	23,502	26,902
14.	Accessary Electrical Equipm	5,182	5,994	6,698	7,288	7,833	8,384
15.	Other Equipment	2,476	3,273	3,878	4,301	4,901	5,505
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	75,934	96,036	114,806	132,908	150,669	168,119
INDIRECT COST		22,021	27,850	33,294	38,543	43,694	48,755
TOTAL COST WITHOUT INTEREST		97,955	123,886	148,100	171,451	194,363	216,874
INTEREST DURING CONSTRUCTION		21,137	26,732	31,957	36,996	41,940	46,797
TOTAL COST WITH INTEREST		119,091	150,618	180,057	208,447	236,303	263,671

Note : Each work item in direct cost contains each contingency, 15% for civil works and 10 % for electrical and hydro-mechanical works.

Table VII.3.5 Estimated Cost for 1st Screening (2/3)

Axis C and FSL 310 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	241	246	250	255	259	265
11.	Powerhouse	3,094	4,490	6,074	7,838	9,731	11,653
12.	Reservoir, Dam & Waterway	50,641	63,064	74,616	85,551	94,989	104,073
13.	Turbine & Generator	8,368	12,375	15,948	19,067	22,122	25,279
14.	Accessary Electrical Equipme	5,060	5,835	6,514	7,094	7,603	8,115
15.	Other Equipment	2,350	3,122	3,731	4,168	4,650	5,208
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	71,098	90,539	108,590	125,477	140,907	156,195
INDIRECT COST		20,619	26,256	31,491	36,388	40,863	45,296
TOTAL COST WITHOUT INTEREST		91,717	116,796	140,081	161,866	181,770	201,491
INTEREST DURING CONSTRUCTION		19,791	25,202	30,227	34,927	39,222	43,478
TOTAL COST WITH INTEREST		111,507	141,998	170,308	196,793	220,992	244,968

Axis C and FSL 315 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	257	262	267	272	277	283
11.	Powerhouse	3,088	4,527	6,112	7,913	9,807	11,818
12.	Reservoir, Dam & Waterway	59,554	72,135	83,825	94,680	104,129	113,374
13.	Turbine & Generator	8,589	12,674	15,895	19,444	22,617	25,860
14.	Accessary Electrical Equipme	5,105	5,894	6,581	7,166	7,687	8,212
15.	Other Equipment	2,397	3,176	3,786	4,217	4,741	5,314
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	80,334	100,076	117,923	135,196	150,811	166,462
INDIRECT COST		23,297	29,022	34,198	39,207	43,735	48,274
TOTAL COST WITHOUT INTEREST		103,630	129,098	152,121	174,403	194,546	214,736
INTEREST DURING CONSTRUCTION		22,361	27,857	32,824	37,633	41,979	46,336
TOTAL COST WITH INTEREST		125,992	156,955	184,945	212,036	236,525	261,072

Axis D and FSL 319 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	332	337	342	346	358	128
11.	Powerhouse	3,121	4,518	6,051	7,786	9,605	11,561
12.	Reservoir, Dam & Waterway	64,986	77,459	89,100	99,893	109,178	118,239
13.	Turbine & Generator	8,766	12,910	16,571	19,742	23,016	26,331
14.	Accessary Electrical Equipme	5,140	5,941	6,635	7,220	7,754	8,291
15.	Other Equipment	2,434	3,222	3,829	4,250	4,815	5,401
16.	Access Road/Railway & Brid	1,285	1,346	1,393	1,439	1,485	1,531
	TOTAL	86,063	105,735	123,921	140,676	156,211	171,481
INDIRECT COST		27,454	33,729	39,531	44,876	49,831	54,702
TOTAL COST WITHOUT INTEREST		113,517	139,464	163,452	185,552	206,042	226,183
INTEREST DURING CONSTRUCTION		24,495	30,094	35,270	40,038	44,460	48,806
TOTAL COST WITH INTEREST		138,012	169,558	198,722	225,590	250,501	274,989

Note : Each work item in direct cost contains each contingency, 15% for civil works and 10 % for electrical and hydro-mechanical works.

Table VII.3.5 Estimated Cost for 1st Screening (3/3)

Axis D and FSL 305 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	322	327	333	338	345	353
11.	Powerhouse	3,021	4,451	6,036	7,798	9,621	11,631
12.	Reservoir, Dam & Waterway	52,892	65,098	77,464	86,671	95,788	104,384
13.	Turbine & Generator	8,171	12,107	15,630	18,728	21,676	24,757
14.	Accessary Electrical Equipme	5,020	5,784	6,453	7,029	7,531	8,028
15.	Other Equipment	2,312	3,072	3,680	4,125	4,570	5,115
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	73,082	92,247	111,051	126,193	141,083	155,870
INDIRECT COST		21,194	26,752	32,205	36,596	40,914	45,202
TOTAL COST WITHOUT INTEREST		94,275	118,998	143,256	162,789	181,997	201,072
INTEREST DURING CONSTRUCTION		20,343	25,677	30,912	35,126	39,271	43,387
TOTAL COST WITH INTEREST		114,618	144,676	174,168	197,916	221,268	244,459

Axis D and FSL 310 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	334	338	341	344	346	348
11.	Powerhouse	3,094	4,490	6,074	7,839	9,731	11,705
12.	Reservoir, Dam & Waterway	61,752	73,898	84,549	95,161	104,061	112,632
13.	Turbine & Generator	8,394	12,406	15,983	19,110	22,177	25,343
14.	Accessary Electrical Equipme	5,064	5,842	6,522	7,102	7,614	8,125
15.	Other Equipment	2,359	3,130	3,736	4,174	4,660	5,105
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	82,339	101,511	118,662	135,235	150,142	164,859
INDIRECT COST		23,878	29,438	34,412	39,218	43,541	47,809
TOTAL COST WITHOUT INTEREST		106,218	130,949	153,074	174,453	193,683	212,668
INTEREST DURING CONSTRUCTION		22,920	28,256	33,030	37,643	41,793	45,889
TOTAL COST WITH INTEREST		129,138	159,206	186,104	212,096	235,476	258,557

Axis D and FSL 315 masl Unit : 1000US$

Account No	WORK ITEM	Maximum Plant Discharge (m3/sec)					
		30	45	60	75	90	105
DIRECT COST							
10.	Land and Facilities	346	352	356	361	366	372
11.	Powerhouse	3,095	4,527	6,112	7,913	9,807	11,818
12.	Reservoir, Dam & Waterway	71,725	83,872	94,558	105,144	113,928	122,502
13.	Turbine & Generator	8,613	12,707	16,334	19,484	22,673	25,926
14.	Accessary Electrical Equipme	5,109	5,901	6,589	7,173	7,696	8,222
15.	Other Equipment	2,401	3,184	3,793	4,222	4,752	5,328
16.	Access Road/Railway & Brid	1,343	1,408	1,456	1,504	1,553	1,601
	TOTAL	92,632	111,952	129,199	145,801	160,774	175,769
INDIRECT COST		26,863	32,466	37,468	42,282	46,624	50,973
TOTAL COST WITHOUT INTEREST		119,496	144,418	166,667	188,083	207,398	226,742
INTEREST DURING CONSTRUCTION		25,785	31,162	35,963	40,584	44,752	48,926
TOTAL COST WITH INTEREST		145,281	175,580	202,630	228,668	252,150	275,668

Note : Each work item in direct cost contains each contingency, 15% for civil works and 10 % for electrical and hydro-mechanical works.

Table VII. 3. 6 Selection of Optimum Plant Discharge of Each Alternative Scheme in 1st Screening

B-319

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	50.7	105.6		39.99		
45	76.2	136.5	30.8	51.48	11.49	50.89
60	102.0	165.2	28.7	61.03	9.55	42.29
75	127.9	192.0	26.8	68.06	7.03	31.13
90	154.0	218.2	26.3	74.00	5.94	26.31
105	180.3	242.5	24.2	78.33	4.32	19.15

B-324

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	52.0	119.1		41.13		
45	78.2	150.6	31.5	53.19	12.06	53.38
60	104.7	180.1	29.4	63.35	10.16	45.00
75	131.3	208.4	28.4	71.48	8.13	36.00
90	158.1	236.3	27.9	78.29	6.80	30.11
105	185.1	263.7	27.4	84.01	5.72	25.34

C-310

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	48.3	111.5		38.17		
45	72.7	142.0	30.5	49.14	10.97	48.57
60	97.3	170.3	28.3	58.26	9.11	40.35
75	122.0	196.8	26.5	64.96	6.70	29.67
90	146.8	221.0	24.2	70.62	5.66	25.08
105	171.8	245.0	24.0	74.74	4.12	18.22

C-315

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	49.7	126.0		39.31		
45	74.7	157.0	31.0	50.83	11.51	50.98
60	99.9	184.9	27.9	60.53	9.70	42.95
75	125.3	212.0	27.1	68.28	7.75	34.32
90	150.9	236.5	24.5	74.75	6.48	28.67
105	176.6	261.1	24.5	80.19	5.44	24.09

C-319

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	50.7	138.0		40.14		
45	76.3	169.6	31.5	51.90	11.76	52.07
60	102.1	198.7	29.2	61.81	9.91	43.89
75	128.0	225.6	26.9	69.73	7.93	35.09
90	154.1	250.5	24.9	76.36	6.63	29.34
105	180.4	275.0	24.5	81.93	5.57	24.67

D-305

Max. plant Discharge (m3/sec)	Installed Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	47.2	114.6		37.25		
45	71.0	144.7	30.1	47.94	10.69	47.35
60	94.9	174.2	29.5	56.82	8.88	39.31
75	119.0	197.9	23.7	63.35	6.53	28.90
90	143.2	221.3	23.4	68.86	5.51	24.40
105	167.6	244.5	23.2	72.86	4.00	17.72

D-310

Max. plant Discharge (m3/sec)	Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	48.5	129.1		38.39		
45	73.0	159.2	30.1	49.63	11.24	49.77
60	97.6	186.1	26.9	59.10	9.47	41.94
75	122.4	212.1	26.0	66.67	7.57	33.52
90	147.3	236.2	24.1	73.00	6.33	28.01
105	172.4	260.1	24.0	78.31	5.32	23.55

D-315

Max. plant Discharge (m3/sec)	Capacity (MW)	Cost (Mil US$)	Incremental Cost (Mil US$)	Gain of Energy (MW)	Incremental Energy (MW)	Incremental Benefit (Mil US$)
30	49.8	145.3		39.42		
45	74.9	175.6	30.3	50.96	11.54	51.11
60	100.2	202.6	27.0	60.69	9.73	43.07
75	125.7	228.7	26.0	68.47	7.78	34.44
90	151.3	252.2	23.5	74.97	6.50	28.79
105	177.1	275.7	23.5	80.44	5.47	24.22

Table VII.3.7 Economic Comparison of Full Supply Levels

Dam Axis	FSL	Installed Capacity	Const. Cost	Without Regulation						With Regulation					
				O & M Cost	Total Cost C	Firm Energy	Second. Energy	Energy Benefit B	Net Benefit B-C	O & M Cost	Total Cost C	Firm Energy	Second. Energy	Energy Benefit B	Net Benefit B-C
	(m)	(MW)	($M)	($M)	($M)	(MWy)	(MWy)	($M)	($M)	($M)	($M)	(MWy)	(MWy)	($M)	($M)
	319	154	218.2	9.2	227.4	74.00	12.12	340.3	112.9						
B	322	156	229.0	9.3	238.3	75.14	12.31	345.6	107.3						
	324	158	236.3	9.4	245.7	75.91	12.43	349.1	103.4	20.2	256.5	78.29	11.53	358.7	102.2
	310	147	221.0	8.7	229.7	70.62	11.55	324.8	95.1						
	313	149	230.3	9.0	239.3	71.76	11.75	330.0	90.8						
C	315	151	236.5	9.1	245.6	72.52	11.00	332.6	87.1	19.8	256.3	74.75	11.00	342.5	86.2
	317	153	243.5							20.0	263.5	75.56	11.13	346.2	82.8
	319	154	250.5							20.0	270.5	76.36	11.24	349.9	79.4
	305	143	221.3	8.6	229.9	68.86	11.27	316.7	86.8						
	307	145	227.1	8.6	235.7	69.61	11.40	320.1	84.4						
D	310	147	236.2	8.9	245.1	70.75	11.91	325.7	80.7	19.6	255.8	73.00	10.74	334.5	78.6
	313	149	245.7							19.7	265.4	74.18	10.92	339.9	74.4
	315	151	252.2							19.8	272.0	74.97	11.04	343.5	71.5

Remarks, $M : US$ million
Unit benefit : 51 US$/MWh for firm energy and 11.92 US$/MWh for secondary energy
O & M cost for "with Regulation" includes cost for annual reservoir dredging (1.08 million US$/year).
Costs and benefits shown are total present values.

Table VII.3.8 Features of Each Damsite Alternative

Description	Unit	Without Regulation Dam Axis B	Without Regulation Dam Axis C	Without Regulation Dam Axis D	With Regulation Dam Axis B	With Regulation Dam Axis C	With Regulation Dam Axis D
Reservoir							
Flood water level	m	324.2	315.2	310.2	329.2	320.2	315.2
Optimal FSL	m	319.0	310.0	305.0	324.0	315.0	310.0
Minimum operation level	m	-	-	-	322.4	313.0	309.0
Volume at FSL	MCM	0.28	0.60	1.15	1.47	1.45	2.80
Area at FSL	ha	16.0	14.9	27.3	32.5	18.0	41.6
Length at FSL	km	0.9	0.7	1.1	1.7	1.0	1.3
Land submerged	ha	4	9	15	20	14	28
Houses to be relocated	nos.	3	7	3	15	7	3
Dam							
Foundation level, Lowest	m	307	300	293	307	300	293
Highest	m	314	300	294	314	300	294
Dam crest level	m	326.2	317.2	312.2	331.2	322.2	317.2
Spillway crest level	m	319.0	310.0	305.0	324.0	315.0	310.0
Max. dam height	m	19.2	17.2	19.2	24.2	22.2	24.2
Dam length	m	248	262	268	256	266	279
Dam volume (concrete)	m3	35,400	43,600	48,700	54,900	68,300	75,400
Intake							
Max. discharge	cms	90	90	90	90	90	90
Sill level	m	315.0	306.0	301.0	318.4	309.0	305.0
Houses to be relocated	nos.	0	2	6	0	2	6
Headrace							
Length of culvert	m	454	65	50	454	65	50
Length of tunnel	m	5,637	5,915	5,570	5,637	5,915	5,570
Powerhouse							
Installed capacity	MW	154	147	143	158	151	147

Table VII.4.1 Estimated Costs for 2nd Screening (Axis B, FSL = 319masl)

Unit ; 1,000US$

ACCOUNT No.	WORK ITEM	MAXIMUM PLANT DISCHARGE (M3/SEC) 30	45	60	75	90	105
10	LAND AND FACILITIES	407	407	407	407	407	407
11	STRUCTURES & OTHER IMPROVEMENT	3,888	5,399	7,122	9,031	11,069	12,941
12	RESERVOIR, DAM & WATERWAYS	49,757	60,893	70,893	77,903	83,702	88,535
13	TURBINES & GENERATORS	9,013	13,228	16,969	20,238	23,406	26,978
14	ACCESSORY ELECTRICAL EQUIPMENT	6,978	8,066	9,010	9,805	10,535	11,269
15	OTHER EQUIPMENT	3,125	4,116	4,888	5,417	6,148	6,895
16	ACCESS ROAD/RAILWAY & BRIDGES	2,110	2,110	2,110	2,110	2,110	2,110
	Total of 10. to 16.	75,278	94,220	111,399	124,911	137,377	149,134
17	INDIRECT COST	21,830	27,324	32,306	36,224	39,839	43,249
	TOTAL COST WITHOUT INTEREST	97,108	121,543	143,705	161,135	177,216	192,382
	INTEREST DURING CONSTRUCTION	20,954	26,227	31,009	34,770	38,239	41,512
	TOTAL COST WITH INTEREST	118,062	147,770	174,713	195,905	215,455	233,894

Figure

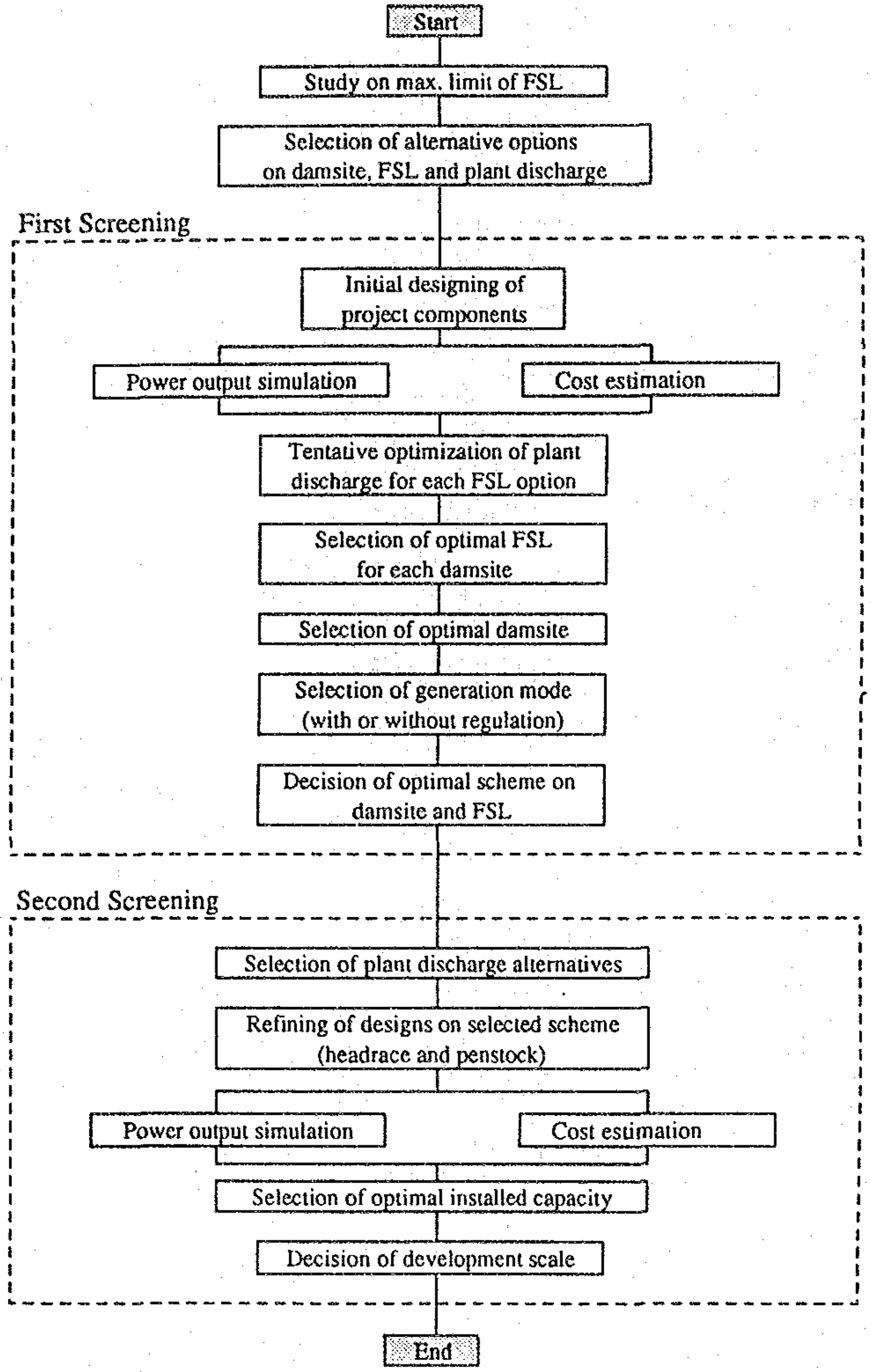

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Fig. VII.3.1
Flow of Plan Formulation Study

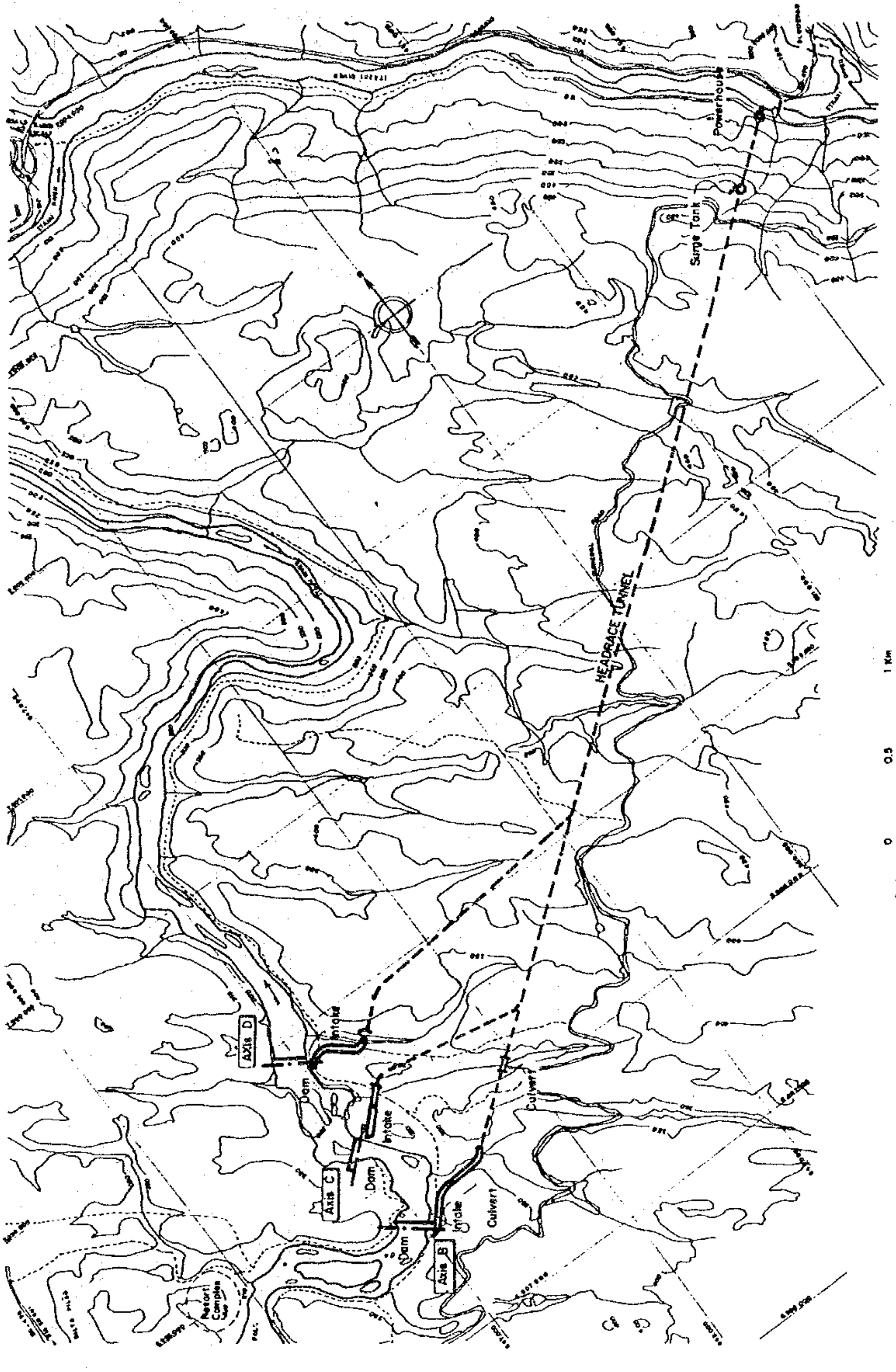

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Fig. VII.3.2
Overall Alignment of Alternatives

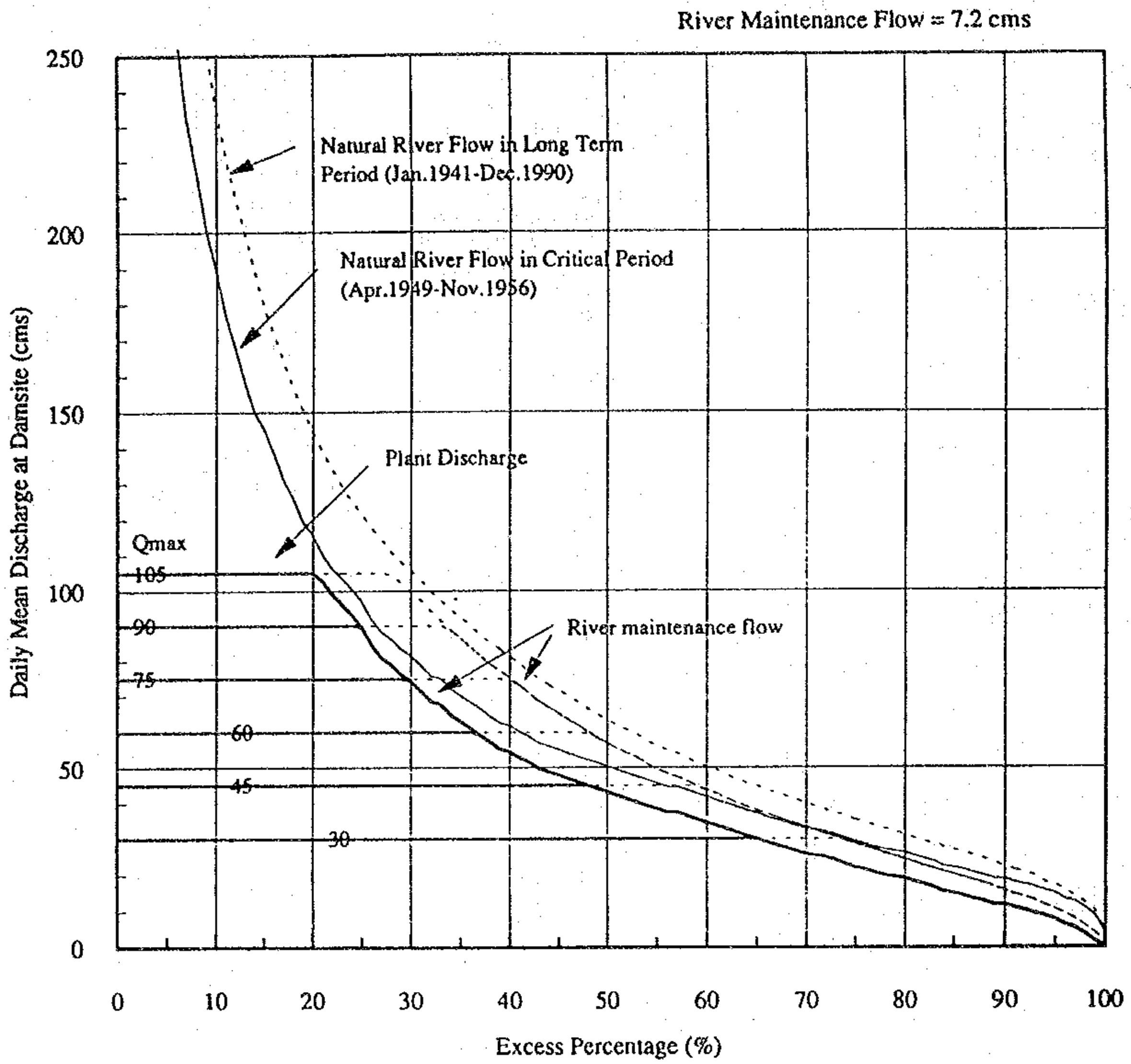

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Fig. VII.3.3

Duration Curves of Plant Discharge

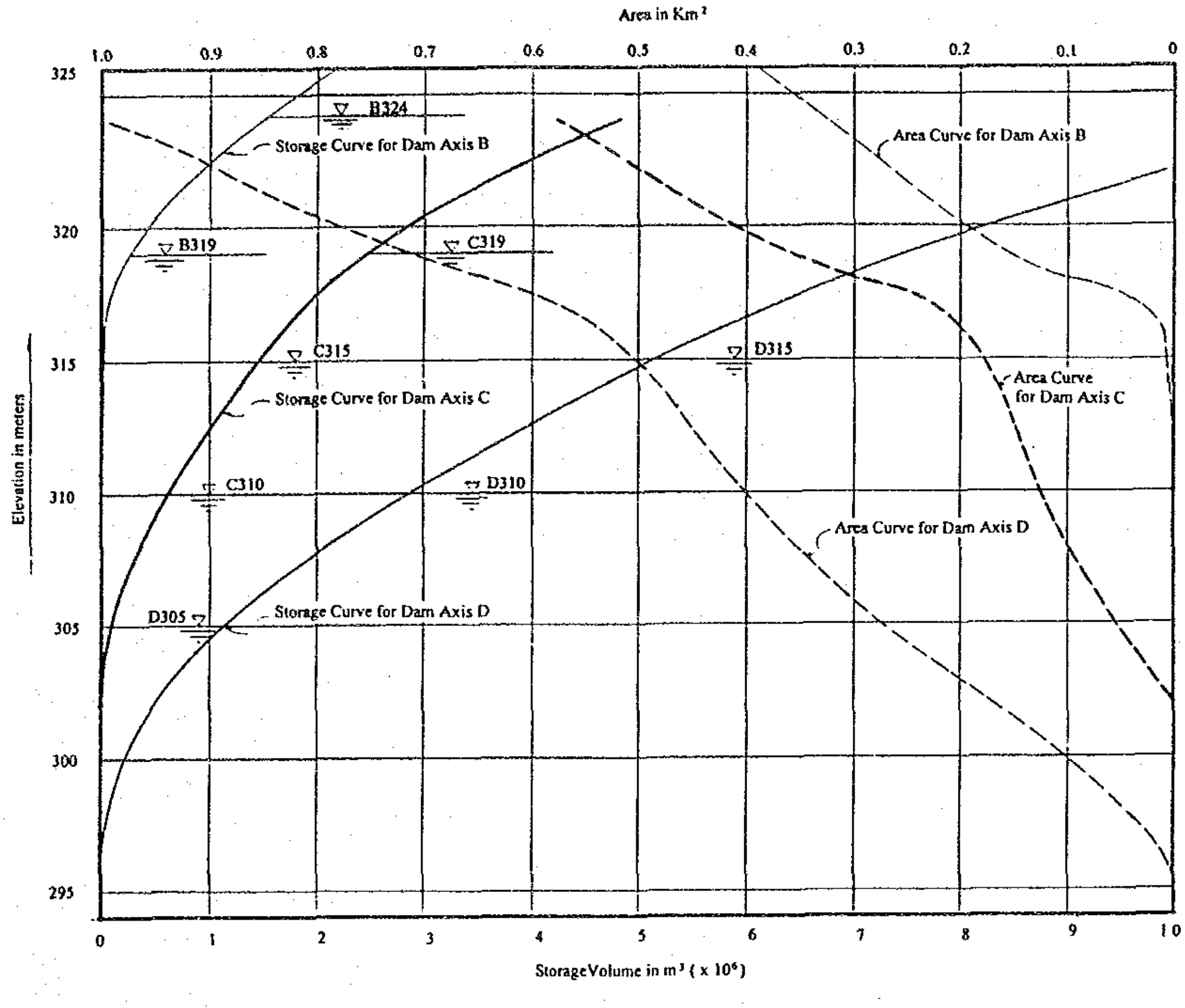

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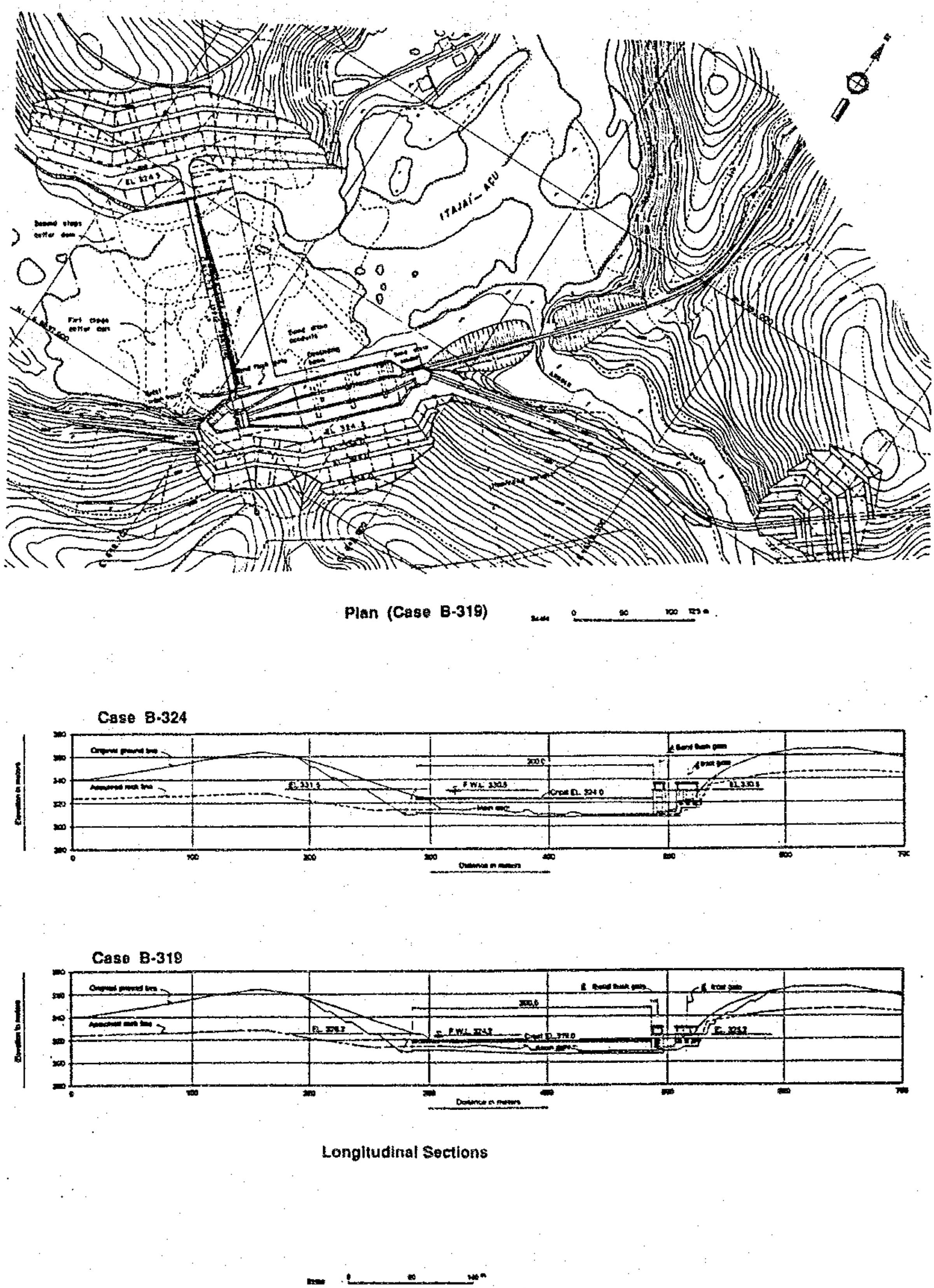
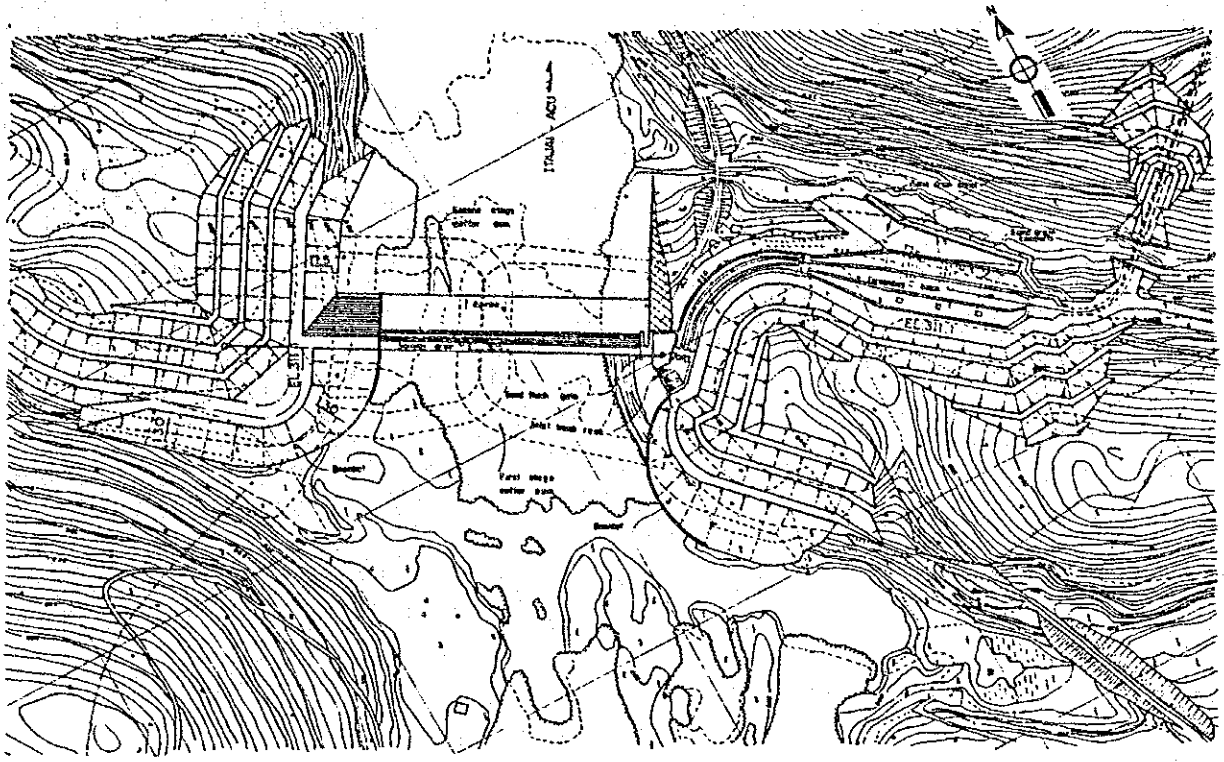
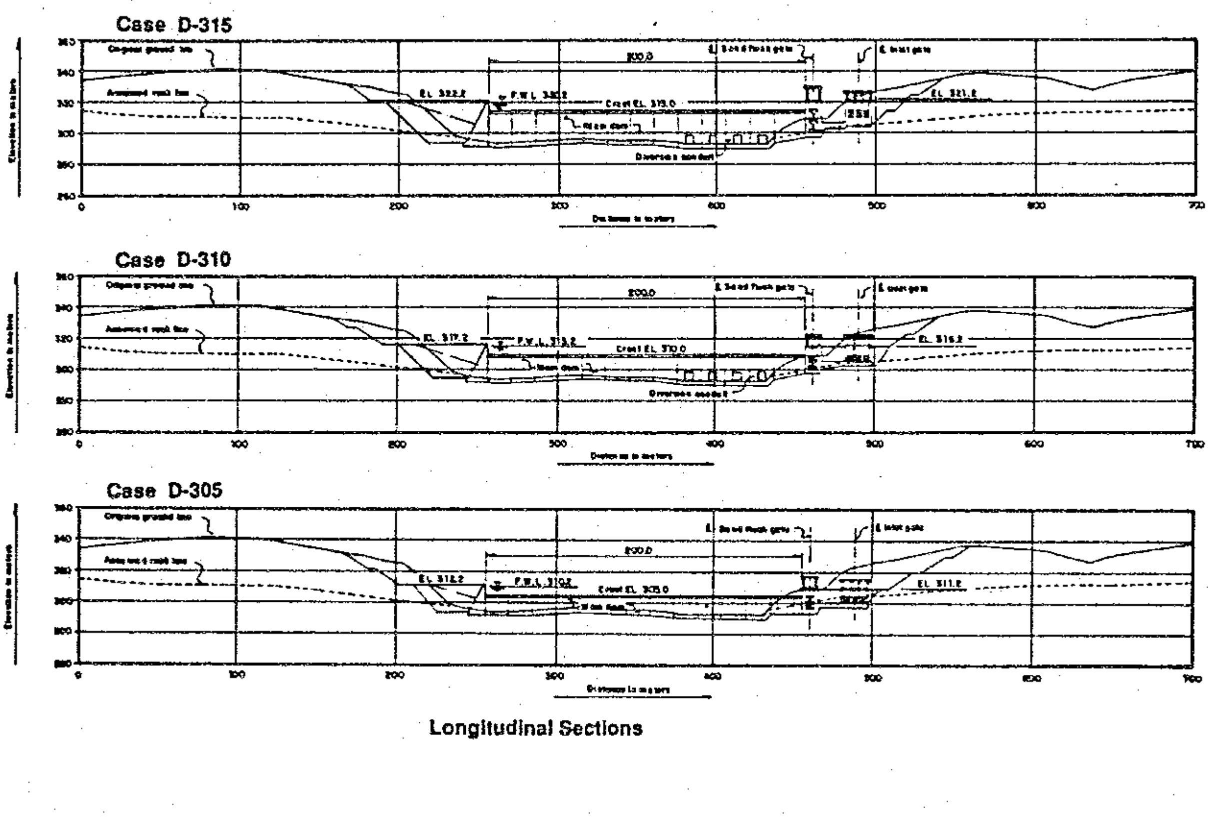
Fig. VII.3.4
Reservoir Area and Storage Curves



JAPAN INTERNATIONAL COOPERATION AGENCY CENTRAIS ELÉTRICAS DE SANTA CATARINA S.A.,BRAZIL SALTO PILÃO HYDROELECTRIC POWER DEVELOPMENT PROJECT	**Fig. VII.3.5** **Dam Layout of Axis B**

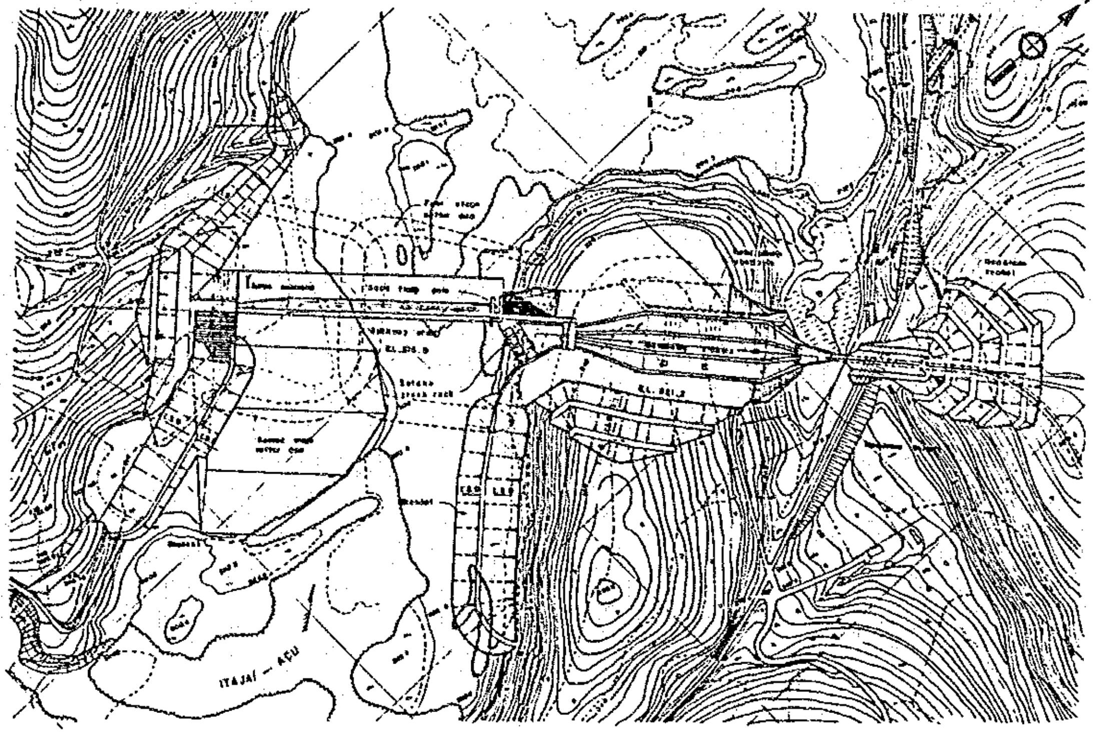


Plan (Case D-305)

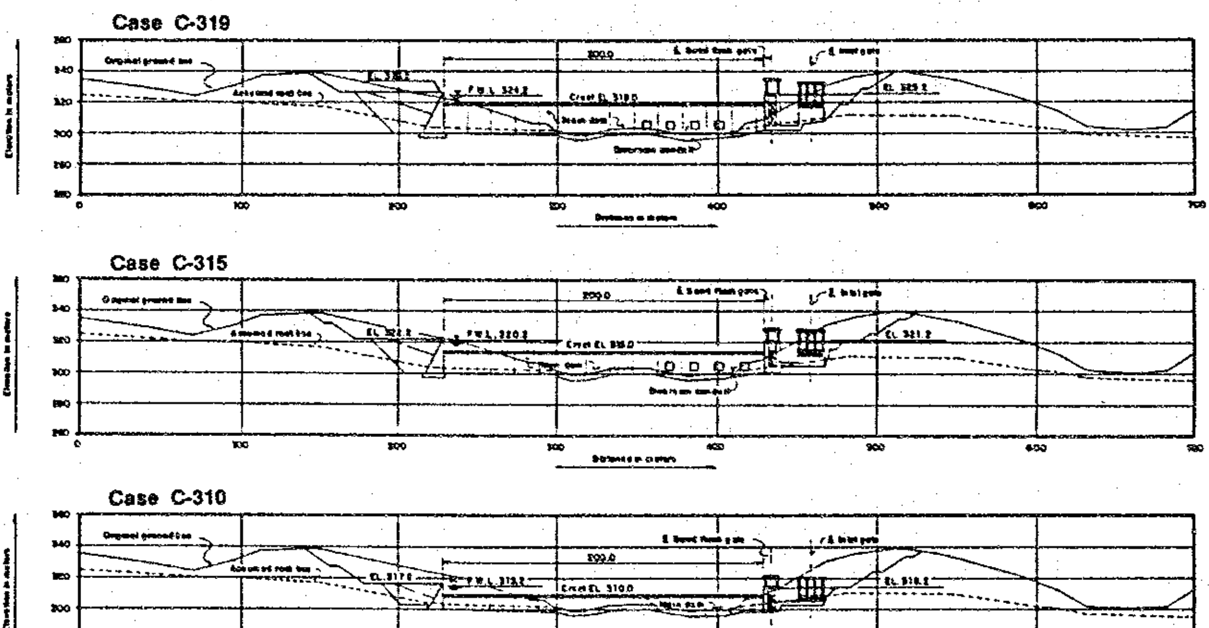


Longitudinal Sections

JAPAN INTERNATIONAL COOPERATION AGENCY CENTRAIS ELÉTRICAS DE SANTA CATARINA S.A.,BRAZIL SALTO PILÃO HYDROELECTRIC POWER DEVELOPMENT PROJECT	**Fig. VII.3.6** **Dam Layout of Axis C**

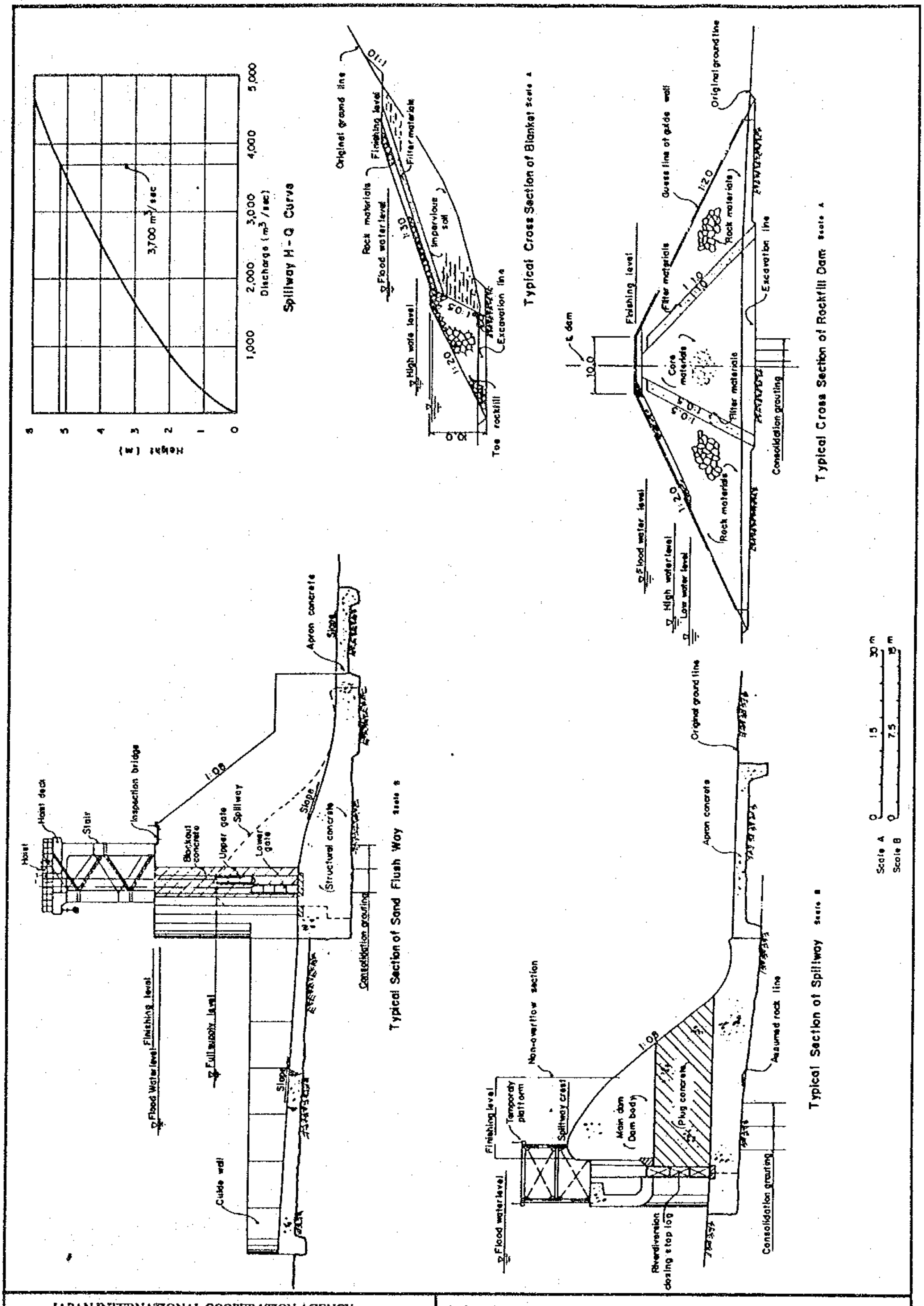


Plan (Case C-315)



Longitudinal Sections

JAPAN INTERNATIONAL COOPERATION AGENCY CENTRAIS ELÉTRICAS DE SANTA CATARINA S.A.,BRAZIL SALTO PILÃO HYDROELECTRIC POWER DEVELOPMENT PROJECT	Fig. VII.3.7 Dam Layout of Axis D



JAPAN INTERNATIONAL COOPERATION AGENCY CENTRAIS ELÉTRICAS DE SANTA CATARINA S.A.,BRAZIL SALTO PILÃO HYDROELECTRIC POWER DEVELOPMENT PROJECT	**Fig.VII.3.8** **Typical Sections of Dam and Spillway**

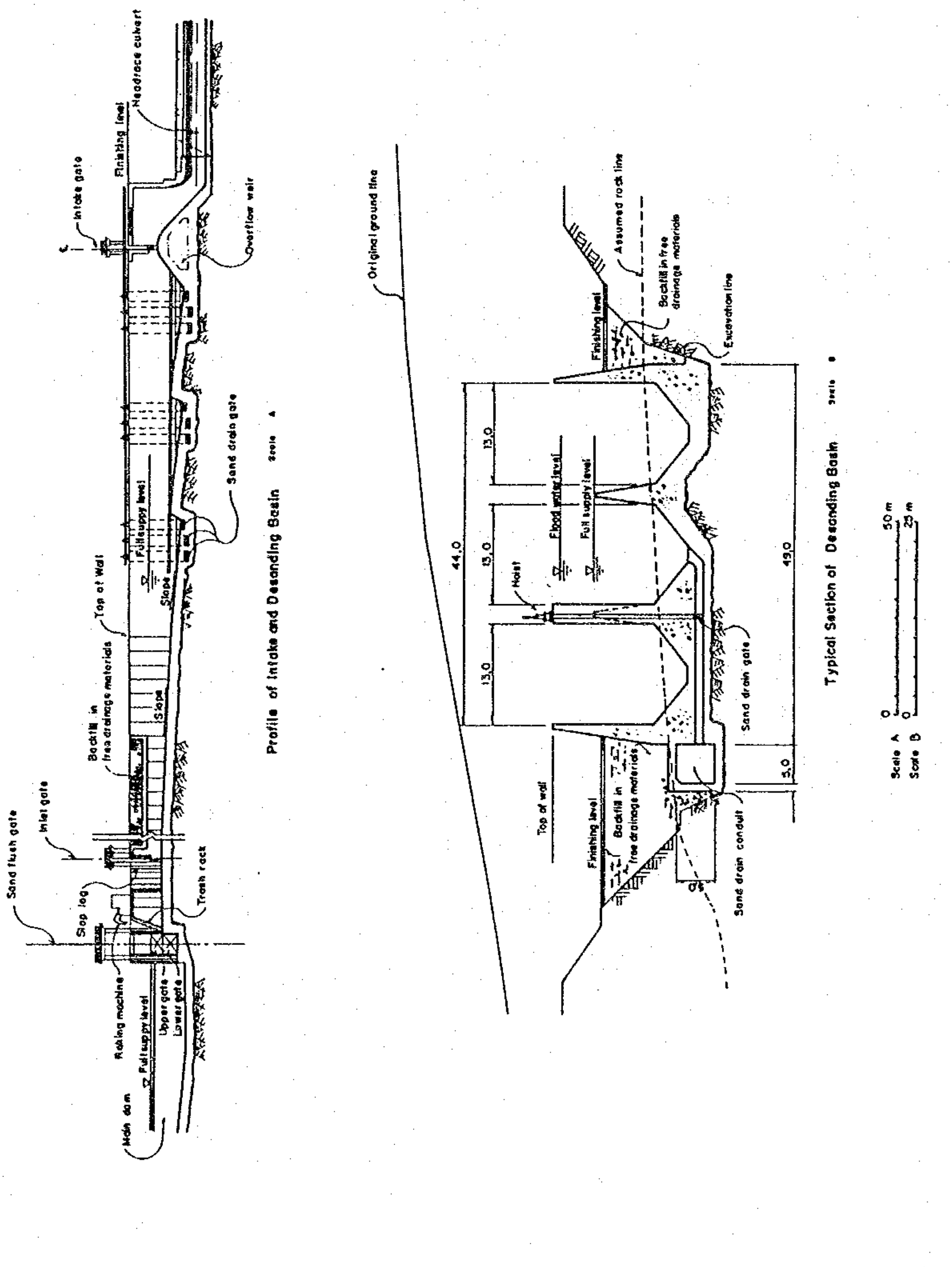


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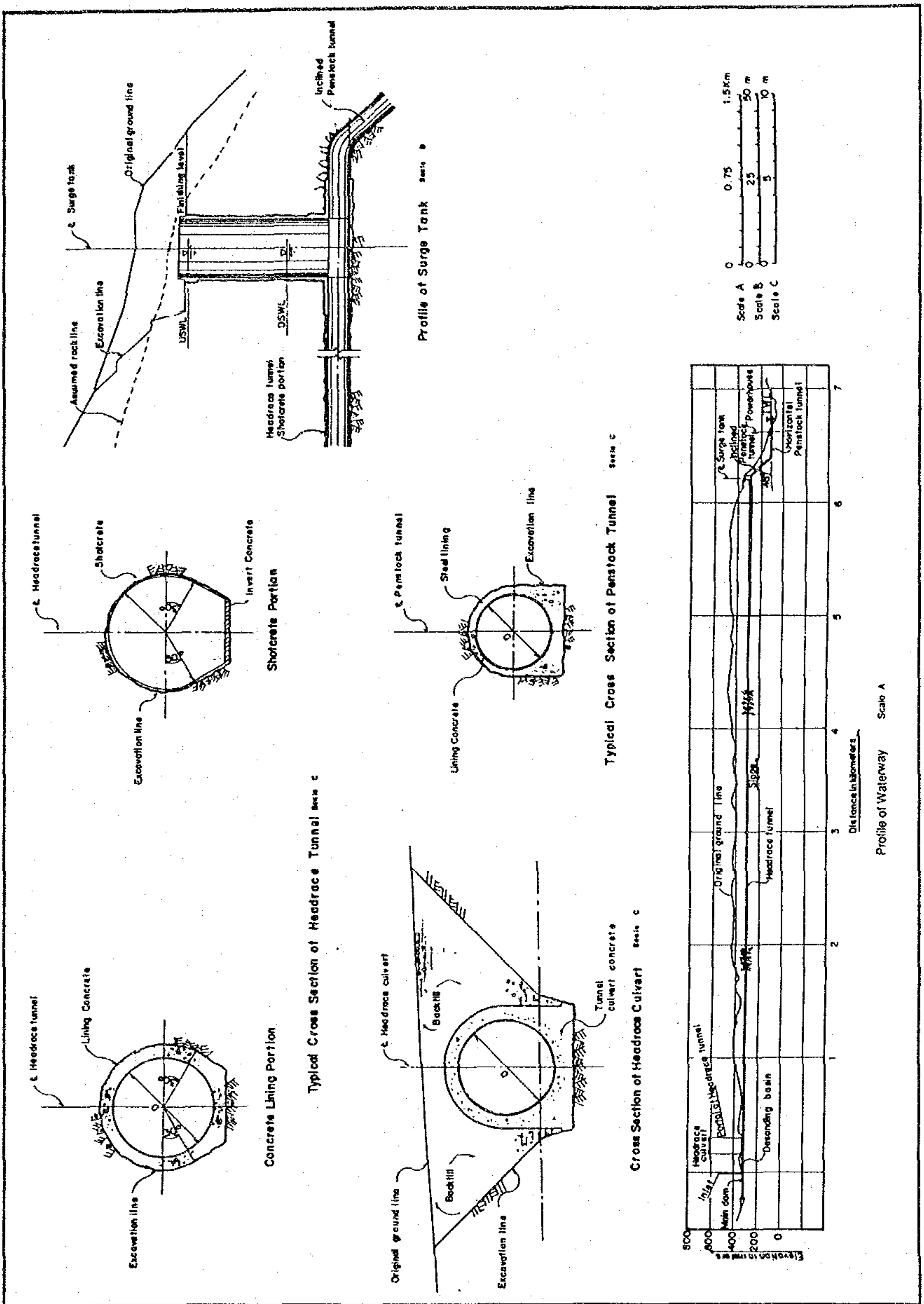
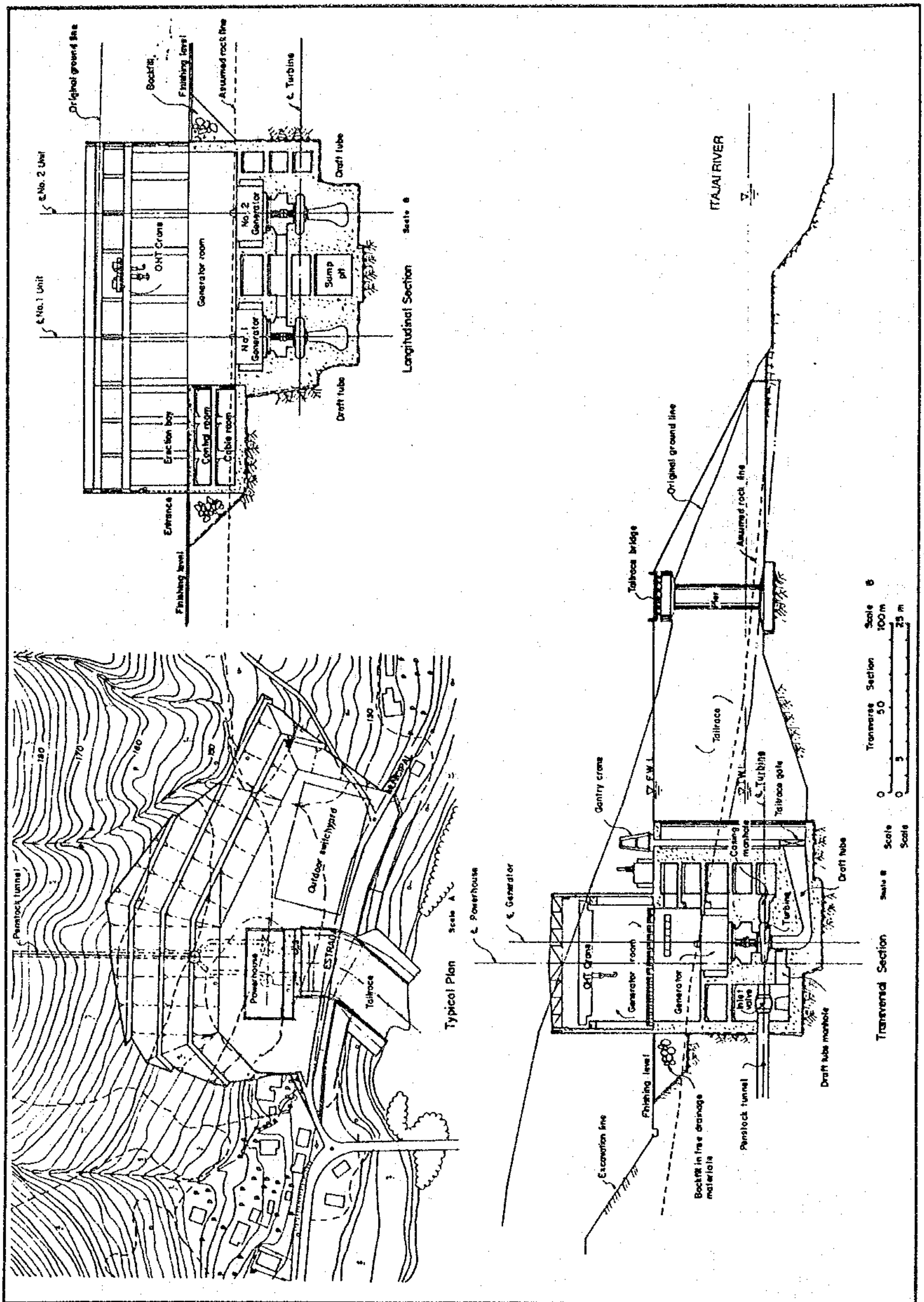
Fig.VII.3.9
Typical Sections of Intake and Desanding Basin



JAPAN INTERNATIONAL COOPERATION AGENCY CENTRAIS ELÉTRICAS DE SANTA CATARINA S.A.,BRAZIL SALTO PILÃO HYDROELECTRIC POWER DEVELOPMENT PROJECT	**Fig.VII.3.10** **Profile and Typical Sections of Waterway**



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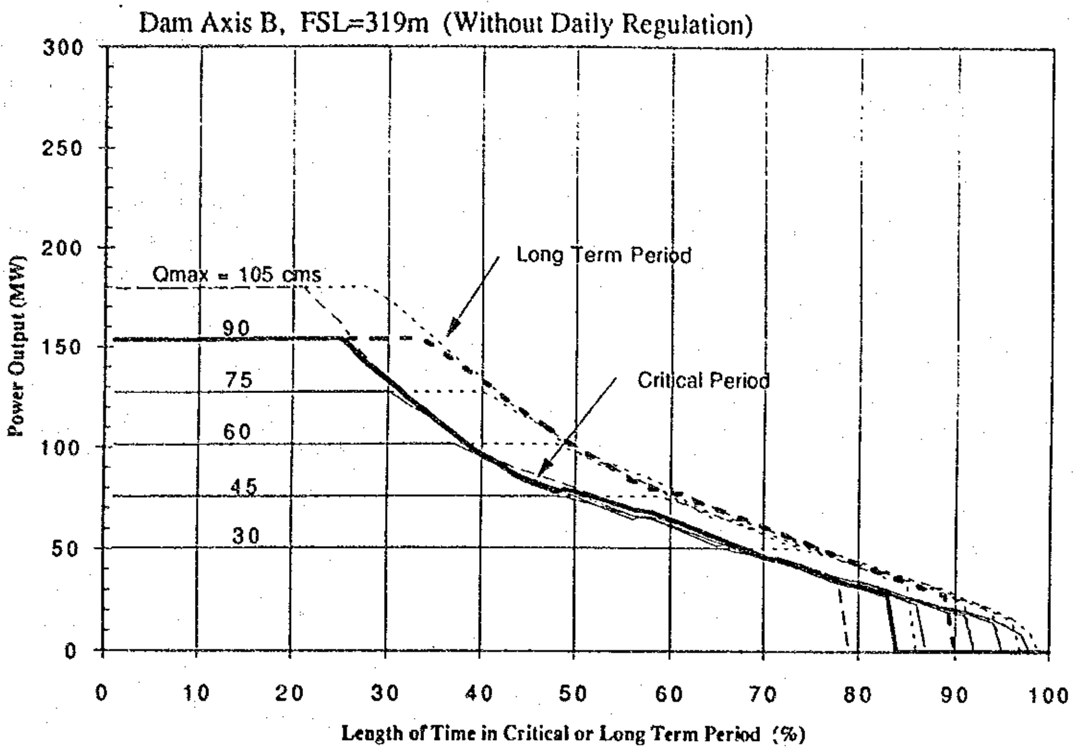

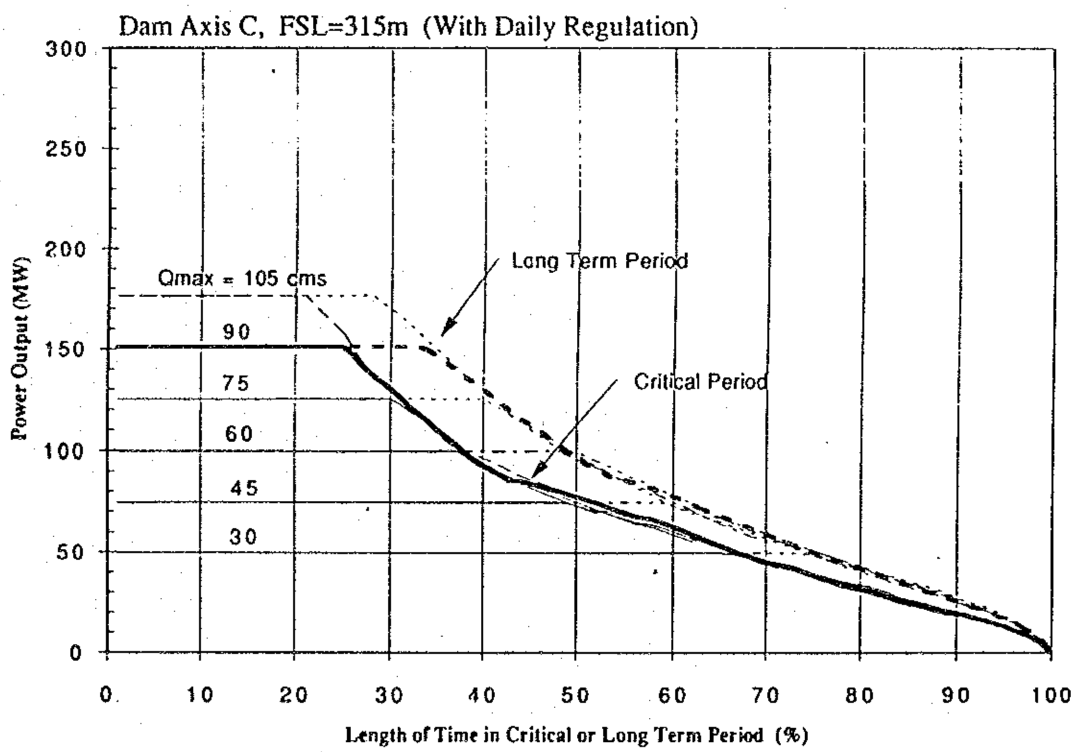

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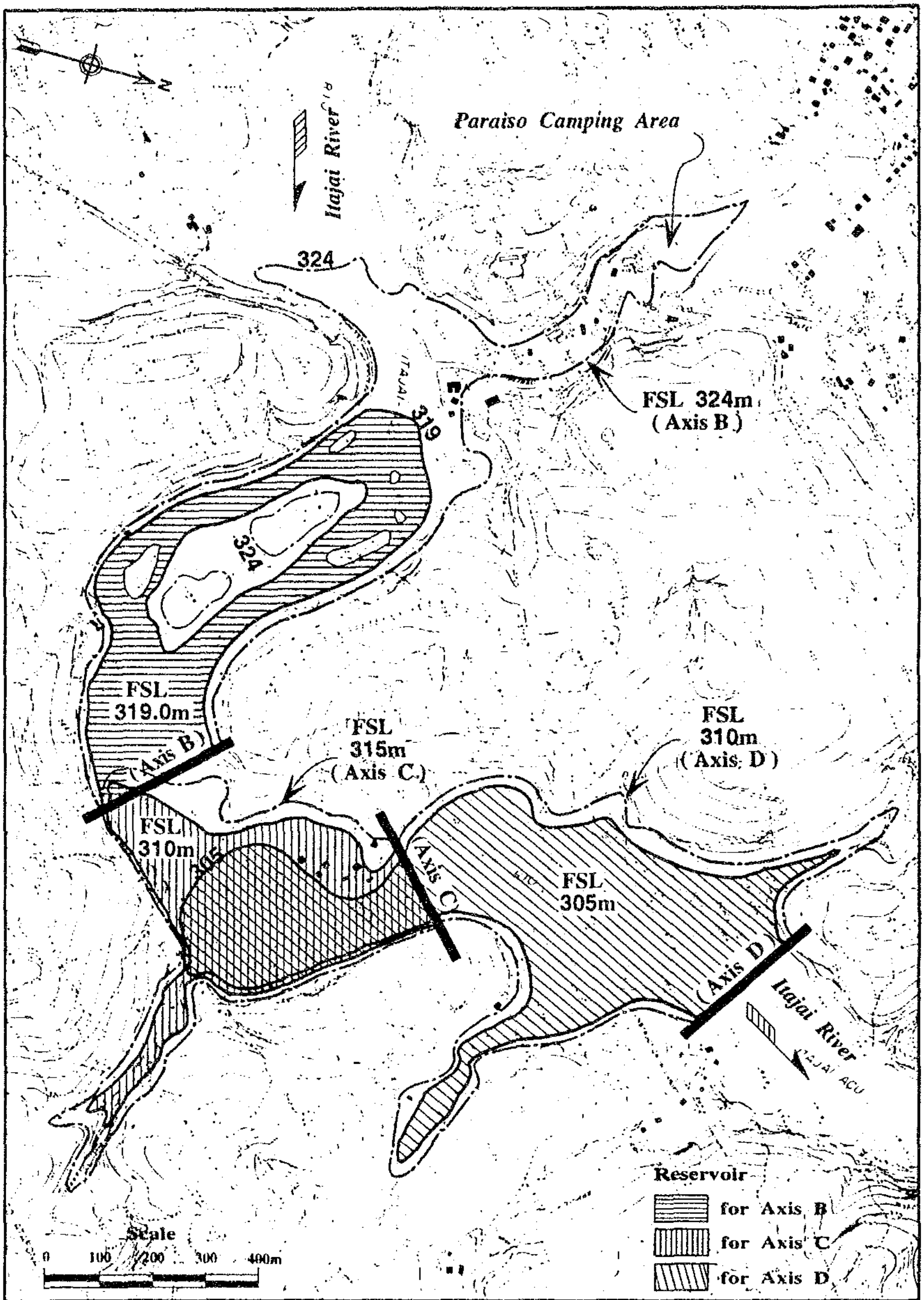

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Fig. VII.3.13
Reservoir Area of Each Damsite Option

ANNEX VIII

OPTIMIZATION AND DESIGNS OF PROJECT COMPONENTS

ANNEX VIII OPTIMIZATION AND DESIGN OF PROJECT COMPONENTS

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1. INTRODUCTION

Based on the result of plan formulation, optimization study and consequent feasibility design of the project components were carried out for civil works including river diversion, dam and spillway, intake/desanding basin, headrace tunnel surge tank, penstock and powerhouse, hydromechanical equipment and generating equipment. These studies and feasibility designs were performed incorporating the result of the typographic survey geological investigation and environmental study.

2. CIVIL WORKS

2.1 River Diversion

2.1.1 Alternatives on Diversion Method

Width of the river at dam axis is approximately 200 m and river flow changes from sub-critical to supercritical around the dam axis because the dam axis is located immediately upstream of water falls. Design discharge has been set at 1,100 cms of 2-year return period.

From the topographical view and amount of design flood, two alternative diversion methods are conceivable. One is a conventional tunnel diversion method and another is a multi-stage diversion method with use of an open channel. Design flood is 1,100 cms of 2-year probable flood.

The former method is composed of a horse-shoe shaped tunnel, 12m in diameter and 300m in length, and two cofferdams with total length of 260 m. The latter will be carried out by two stages and be composed of a 50m wide open channel at the left bank and a cofferdam in each stage of which total length will be 660m. Conceptual design of the both methods are shown in Figure VIII.2.1. Advantage of the former method is that construction of the dam and intake facilities can be carried out in one stage, while this method will be more costly due to the construction of tunnel compared with the latter method.

Estimated direct construction cost including 15 % of contingency of the tunnel diversion method is US$ 8.6 million which is much higher than the multi-stage diversion method by US$ 6.0 million. Cost breakdowns are presented in Table VIII.2.1.

For this economical reason the multi-stage diversion method has been selected.

2.1.2 Feasibility Design

Feasibility design of river diversion by applying the multi-stage diversion method (two stage) are as described below:

The first stage diversion, of which purpose is for the construction of the intake, desanding basin and part of dam located right side from the middle of the river, is composed of an open channel with 50 m bottom width at the dam axis and fill type cofferdam. Crest elevation of the cofferdam upstream of the waterfall was set at 320 m above sea level (asl) and varies according to change of riverbed elevation, which elevation is so determined that the design flood can be discharged with freeboard of 1m. The cofferdam consists of impervious soil zone and rock riprap zone. Excavated soils and rocks for the construction of desanding basin will be used for these materials.

In the second stage the first stage cofferdam will be removed and a new cofferdam will be made for the construction of the remaining part of dam on the left bank. The crest of cofferdam was set at 324 masl upstream of the dam so that the design flood can be discharged by overflowing the top of dam constructed in the first stage with the freeboard.. The sandflush gate will be kept fully open in this stage. Materials obtained from excavation of the dam, intake facilities and waterway, which may have to be stocked, will be used for this coffering.

2.2 Dam and Spillway

2.2.1 Alternative Study on Dam Type

River cross section at the dam site is a flat trapezoidal section. General width of the river flow is around 200 m. The river flows in the river channel with a series of rapids and small scale of waterfalls. Hard granite is exposed along the river bed but both banks, especially left bank is covered with deep overburden. Design of flood discharge applied to the dam site is 5,300 cms corresponding to 1,000-year return flood.

The following three (3) combination of dam type and spillway type were studied for selection of the optimum configuration.

Type 1 : Concrete dam with non-gated spillway
Type 2 : Concrete dam with non-gated spillway
Type 3 : Rockfill dam with gated spillway

For the Type 1, the width of the river channel is fully utilized for a concrete weir of non-gated spillway of which crest level is set at the same elevation as the full supply level (FSL) of the pondage. In the Type 2 and 3, a half of the river channel width is utilized for spillway and non-overflow dam is located in the remaining part. General layouts of these alternatives are as shown in Figure VIII.2.2.

As the result, Type 1 was estimated the most economical as shown in Table VIII.2.2. Besides the non-gated spillway is much superior in operational view point. Since the pondage volume is negligible small for the volume of flood, water level rises very quickly and varied sensitively by change of inflow. To avoid overtopping or excessive opening of gate, the spillway gate has to be operated very precisely without any delay, always giving careful attention to the movement of water level, which will be almost impractical. Due to the economical reason and operational view point, concrete dam with non-gated spillway has been selected as the optimum dam configuration.

2.2.2 Feasibility Design

Based on the alternative study result, concrete dam with ungated spillway has been designed at feasibility level as presented below:

The dam axis is located obliquely crossing a waterfall; right abutment is located upstream of the waterfall and the left abutment is at its downstream portion. This layout has been determined mainly for the geological reason; one boring made in the Power Study of South Brazil is located about 50m upstream of the present dam axis of which result indicates that rock was not found. As the result intake is located immediately downstream of the waterfall. This layout allow lower dam to take intake design discharge by utilizing natural depth, while this layout may cause concern on sedimentation in front of intake unless sand flushing facilities function well. Regarding removal of sedimentation is further discussed in the following chapter.

The dam is concrete gravity type dam with non-gated spillway. The total dam width is 301m and spillway width is 200m. Due to very deep rock foundation at the left abutment, a part of non-overflow section at the abutment is designed as fill type dam. River bed elevation varies from 309 masl to 316 masl. FSL has been set at 319 masl. Since cracks develops near the surface of riverbeds, 2 m depth foundation excavation is considered. Accordingly height of overflow section varies from 5 m to 12 m and maximum height of non-overflow section is 18 m.

Maximum flood water level for 1,000 year flood of 5,300 cms was computed at 325 msasl. Free board for concrete non-overflow sections is none and 1.0 m for the fill section according to the criteria agreed with ELETROBRAS.

Fill dam section comprises of four zones, impervious soil zone, filter zone, shell zone and rock riprap on the upstream face. Excavated soil for the desanding basin will be used for the impervious zone. Mixture of concrete coarse and fine aggregates produced at a crushing plant will be used for the filter zone. Blasted rock in the designated quarry will be placed in the shell zone and riprap.

Consolidation grouting has been designed for entire area of concrete dam and fill dam in this stage. This design will have to be reviewed based on further geological investigation study. High pressure grout curtain is not needed, except for the foundation of fill dam which should be treated by curtain grout if rock condition is poor.

The layout of dam and spillway and typical sections are as shown in Figures VIII.5.2 and VIII.5.3.

2.3 Intake and Desanding Basin

2.3.1 General

No alternative study was made because change of type will not remarkably affect construction cost. Feasibility design of intake facilities are as follows:

The intake facilities comprises a sandflushway, desanding basin inlet, desanding basin, and power intake. General layout, profile and sections of these structures are shown Figures VIII.5.1, VIII.5.3 and VIII.5.4.

The sandflushway is located on the right abutment of the dam immediately downstream of the inlet to the desanding basin. A double-leaves gate, 5.0 m wide by 7.3 m high, will be provided for operation of the sandflushway.

The inlet to the desanding basin is located on the right abutment of the dam, immediately upstream of the dam axis. The inlet has net opening with size of 22.8 m width by 9.8 m height.

A desanding basin, comprising three chambers with sand flushing facilities, has been provided to flush out the bedload and suspended sediments and prevent them from entering into the power waterway. The settling basin, 52.9 m wide and 190 m long, will be located on the right abutment immediately downstream of the dam axis.

The power intake is located immediately downstream of the desanding basin. Regulation of the flow into the intake will be undertaken with 3.8 m wide by 4.8 m high roller gates.

2.3.2 Sandflushway

A sandflushway has been incorporated into the right abutment of the dam. The flushway, the sill of inlet and the submerged sill have been oriented in order to maintain high velocity flow through the sandflushway. Final dimensions and orientation will be based on hydraulic model tests.

The invert of the sandflushway inlet has been set at 312 masl, 3 m lower than inlet sill. This invest has a function to prevent sediment, which will have accumulated in front of the sandflushway, from flowing into the inlet as much as possible.

Regulation of the sandflushway will be achieved by operation of a double leaf gate. The sundflush gate will be operated by mechanical equipment installed on a suspended slab located above the sandflush gate. The gate will be also regulated to discharge river maintenance flow and to flow out debris floating in front of the inlet. For maintenance purpose a stoplog slot is provided for in the sandflushway piers.

2.3.3 Inlet

The inlet was designed based on the intake design discharge of 90.0 cms and allowable velocity of 1.0 m/sec at the trash racks, which is the maximum velocity appropriate for operation of trash-rack rates and gates. As the result, the inlet sill is set at 315.0m and the opening below FSL has a cross-sections of 4.0 m high and 22.8 m wide. In front of the trash rack concrete curtain wall was provided for prevention of timbers and floating debris from approaching to the trashrack. Four sets of trash-rack will be required. Two mechanical trash-rack rake will provided for the trash-racks to clean.

As described in Section 8.2.2, the inlet is located immediately downstream of waterfall. This layout may increase sedimentation accumulation in front of the inlet. It is considered to remove it by flushing operation at this stage. In case of more sedimentation is anticipated in hydraulic model tests in the next design stage, it is recommended to study sluicing operation which sluices out bed loads downstream without allowing sedimentation in front of inlet by keep partial opening of gate for the period of rainy season when bed loads are transported. At this time type of the gate should be re-studied to meet this operational requirement.

2.3.5 Desanding Basin

One of the essential design features of the intake facilities of a height-head power plant is prevention of suspended sediments and bedload intrusion into the headrace tunnel. The sediment investigation results made in this stage indicate that concentration of sediment with particle size larger than 0.3 mm which will significantly affect progress of abrasion of hydro-mechanical equipment is very low, though most of the sample were taken during dry season. However, due to the following reasons, an installation of the desanding basin has been proposed;.

(i) This project has a high-head power plant with about 6 km long headrace. It is vulnerable to scouring due to high velocity of sediment flow and once the scouring takes place in the headrace tunnel, much difficulties will be encountered for its repairing and maintenance.

(ii) The penstock steel pipe is embedded in under ground and power operation is made using muddy water in the rainy season. It may occur abraison for the embedded penstock steel pipe and turbine.

The desanding basin comprises three inlet gates, three settling chambers and three sets of flushing system composed of channels. At the upstream end of each desanding chamber a roller gate, 3.7m wide by 4.4m high, will be installed. It will be operated when sediments deposited in the chambers is flushed out. Transition channels extend immediately after the gate to the desanding chamber. The width of settling chamber is 15.3m and depth of water will vary from 4.3 to 6.5 m. The side walks extends to elevation 319.0 masl so that surplus inflow can de spilled out. This side spillway will function to spill out the water push back from the headrace waterway during surging. Dimension of the chamber has been decided based on particle size of 0.3 mm and a flow velocity of 0.3 m/sec in the desanding chambers.

The flushing channels are located at the invert of chamber. Regulation of flushing will be achieved by operation of 2.5m wide and 1.0m high slide gates. Each chamber will be used sequentially in order to avoid suspension of power generation. In the design, three set of flushing facilities consisted of a flush channels and slide gates are installed in a chamber and flushing channels are oriented perpendicular to the flow water in the chambers. Theses design should be defined based on hydraulic model test to be carried out in the following stage.

2.3.6 Power Intake

The invert of the headrace tunnel at the intake has been set at 306.0 masl. This elevation has been determined to provide sufficient depth at the intake below the full supply water level, to avoid vortex formation and consequently air entrapment. Regulation of flow into headrace waterway will be made by three roller gates which will be operated from mechanical equipment installed on the hoist deck.

2.4 Heardace Waterway

2.4.1 Alternatives of headrace waterway

Alternatives on type of water conveyance method, alignment, liming type and diameter of headrace waterway were studied.

(1) Water conveyance method

There are two water conveyance system, open channel waterway system and tunnel waterway system. In this scheme, for topographical reason, tunnel waterway system has been selected. For tunnel conveyance system, two water conveyance methods, free flow tunnel and pressure tunnel are conceivable. Free flow tunnel system comprises free flow tunnel, head tank at the end of tunnel and spillway at the head tank. Pressure tunnel is composed of pressure tunnel and surge tank. Free flow tunnel has an economical merit because water can be conveyed with less head loss in comparison with pressure tunnel with similar construction cost when plant discharge is small.

In this scheme, however, head tank will have to be made underground and conveyance of 90 cms discharge will need large size tunnel, which will be more costly than pressure tunnel system. Accordingly pressure tunnel has been selected.

(2) Alignment

There are two deep valleys downstream of the desanding basin, one is approximately 100m and another is 800m, which will affect the design of tunnel alignment. Location of these valleys are as shown in Figure VIII.2.3.

To cross the first valley two methods were considered, one is supporting a culvert type waterway by bridge structure and another method is driving the culvert type waterway along the slope of upstream side of the valley up to the location where the culvert can cross the valley on ground. The first method will shorten the waterway culvert by approximately 100m in comparison with the latter method, while a 200 m long bridge will be required. The construction cost will be significantly costly compared with the latter method. For this reason the latter method has been selected. The length of culvert will be 404 m and at the end of the culvert tunnel waterway starts. For water tightness steel liner will be embedded.

Two alternative alignments, Route I and Route II, were considered to cross the second valley. The alignment of route I is straight from alignment the portal of tunnel to the surge tank that makes the length of waterway shortest. The total length of waterway from the end of desanding basin to the surge tank is 6,091 m. This alignment, however, will not allow the waterway to pass the valley by tunneling because of insufficient thickness of cover under the valley. A culvert waterway lined with steel is, consequently, required to cross the valley, which construction cost will be higher in comparison with that of tunneling. The alignment of Route II was set by sifting the tunnel downstream by 500 m so that the tunnel can pass under the valley, resulting that the waterway length is 6,330 m, longer by 239 m than that of Route I. This alignment saves the construction cost of culvert, while the construction cost of tunnel and head loss is increased by the increased length. The above alignment of each route is also shown in Figure VIII.2.3. Shotcrete lining will be applied to the entire length of each tunnel except the first 20m at each portal and 540m for rhyolite rock zone and weak strength rock zones, which total length is assumed around 540m as stipulated in the plan formulation. Length of each rote are as shown below.

	Route I	Route II
Culvert waterway (m)	454	404
Tunnel waterway		
Concrete lining (m)	600	560
Shotcrete lining	5,037	5,366
Total length (m)	6,091	6,330

Incremental construction cost and loss of energy benefit due to head loss of Route II for these of Route I were estimated. Rate of contingency, indirect cost and interest are as used in the plan formulation studies. Assuming three year construction period for these structure rate of annual disbursement is set at 0.35, 0.35 and 0.30.

As the result, the construction cost of Route II was estimated cheaper by US$ 171,000 but loss of benefit due to incremental head loss is US$ 415,000 in 50 years operation, of which breakdowns are presented in Table VIII.2.3. In total incremental cost of Route II is US$ 244,000, which indicates no remarkable differences in economic superiority to each other. Route I, however, has possibility to cross the valley by tunneling if the geological condition allow tunneling with thin cover, which should be studied based on drilling investigation in the next stage. Based on this Route I has been adopted.

(3) Lining type

Result of the geological investigation shows that most of rock along waterway tunnel is hard and massive granite which is considered to selfstand without supporting in long term. Serious erosion due to flow of water will not cause and leakage through the tunnel is considered very little. From these rock condition, non-lining low pressure tunnel is considered technically feasible.

Non-lining tunnel is cheaper in construction cost but reduces generation energy due to high head loss compared with concrete lined tunnel having the same diameter. In order to select an appropriate type of tunnel lining, at first an optimum diameter of concrete lined tunnel was determined to minimize total of construction cost and loss of energy benefit. Secondly, the diameter of different types of tunnel lining, unlined, shotcrete lined and TBM, was first obtained which yields the same amount of head loss in the concrete lined tunnel. Construction cost of these were then estimated.

Roughness coefficient (Manning's coefficient) by which head loss is estimated was estimated 0.013 for concrete surface, 0.024 for shotcrete surface, 0.035 for blasted rock surface and 0.016 for rock surface excavated by a tunnel boring machine (TBM). Further 0.019 and 0.029 were used for shotcrete surface and 0.03 and 0.04 for blasted rock surface to observe the sensitivity due to varies of roughens coefficient since head loss is affected by the roughness coefficient. The results of study are as follows:

	Roughens coef.	Dia. (m)	Unit const. cost (thousand us$/m)	Construction cost (mill. us$)	Incremental cost to concrete lining (mill.us$)
Concrete lining	0.013	4.8	5.76	32.5	-
Shotcrete lining	0.019	5.5	4.43	25.0	-7.5
	0.024	6.0	5.04	28.4	-4.1
	0.029	6.3	5.63	31.7	-0.8
Unlined	0.030	6.3	4.53	25.5	-6.9
	0.035	6.7	5.04	28.4	-4.1
	0.040	7.0	6.74	38.0	5.5
TBM	0.016	5.3	5.24	29.5	-2.9

As seen the above, shotcrete lining is superior to concrete lining over the range of roughness coefficient from 0.019 to 0.029 and almost same as unlined except the case that roughness coefficient of blasted rock is 0.04 where unlined tunnel becomes costly. Based on these figures and considering limited geological data available at present, shotcrete lining has been selected.

(4) Optimum tunnel diameter

Larger tunnel diameter increases construction cost and at the same time decreases head loss resulting in incremental energy benefit. Optimum diameter was, hence, determined to minimize the total of construction cost and loss of energy benefit in 50 year operation. Roughness coefficient applied to concrete and shotcrete are 0.013 and 0.024.

Percentage of contingency, indirect cost and interest rate are the same as those applied to the waterway alignment studies.

The result indicates that 4.8 m is an optimum diameter for concrete lining tunnel and 5.8 m for shotcere lining tunnel, as seen in Figure VIII.2.4.

2.4.2 Feasibility Design

Based on the above alternative study results the feature of headrace waterway designed are as described below:

Headrace waterway from the end of desanding basin to the surge tank is composed of two culvert type waterways and two tunnel type waterways, of which total length is 6,165m and with longitudinal slope of 1:309.

The first culvert waterway, named No.1 culvert, starts from the end of power intake to the first tunnel portal with length of 403.8 m. The second culvert, named No.2 culvert, is located at the cross point of the waterway and the valley about 800m downstream from the end of desanding basin, which length is 50 m. Diameter of No.1 culvert was set at 4.8 m and no.2 at 5.8 m. In general it is not preferable to change the diameter from smaller size to larger size in pressure tunnel because this change creates gaps at the crown of tunnel which will trap air when the tunnel is filled with water. Major section of the headrace tunnel was designed as the shotcrete lining with 5.8 m in diameter. However, to save the construction cost of No.1 culvert smaller diameter has been adopted to lower the invert of tunnel connecting from culvert in order to keep the crown of tunnel at the same level. For prevention of water leakage through construction joints and cracks of concrete, installation of steel liner is recommended. The steel liner is extended into the tunnel by 20 m from portal aiming at improving the water tightness between the tunnel and the culvert. To minimize differential settlement rock foundation is recommended for the culvert. The portion where deep excavation is required to reach to rock bed is filled up with free drain materials after completion of the construction of culvert.

The upstream tunnel, named No.1 tunnel is located between the culverts of which length is 492 m. The another tunnel, named No.2 tunnel, extends from the end of No.2 culvert to the surge tank with length of 5,145.3 m.

According to the past and present exploratory drilling and geological survey along the tunnel alignment (8 holes in total for tunnel and penstock), rock along the majority of the tunnel alignment is hard and massive granite from the portal of No.1 tunnel to around 120 m upstream from the surge tank. Rock in the vicinity of the surge tank is hard rhyolite and lots of joints are developed. Most of the granite is considered strong enough to selfstand in long term. Such rock portion is assumed to be about 90 % of total tunnel length. The remaining portions; about 600 m in total length are assumed to be weak granite and cracky rhyolite.

The tunnel where rock is hard granite will be lined with shotcrete and the remaining is protected by concrete. The diameter of shotcrete lining section is 5.8 m and 10 cm thickness of shotcrete will be sprayed. Diameter of about 120 m long concrete lined tunnel upstream of surge tank was also set at 5.8 m to keep the same tunnel diameter of shotrcrete lined tunnel. Downstream from there to the end of headrace tunnel the entire section was lined with concrete. In this section diameter was set at 4.8 m. Thickness of concrete lining depends on rock condition and 40 cm thick concrete for the portals portion and poor rock condition, and 25 cm plain concrete for fair rock were adopted. These diameters and lining methods should be reviewed based on further geological information in the next design stage as well as the construction stage because these sensitively depend on blasted rock condition.

Plan and Profile of waterway and sections are shown in Figure VIII.5.1 and VIII.5.5.

The tunnel will be excavated by conventional drilling and blast methods. No construction adit will not be provided for the both tunnel considering that tunneling will be carried out from both upstream and downstream portals without encountering large amount of spring water during its excavation.

2.5 Surge Tank

For regulation of the turbines as well as for control of water hammer a surge tank is, in general, provided at the end of the headrace tunnel when the headrace pressure tunnel is relatively long. In this design stage a cylindrical shaft type with orifice, has been designed.

The dimensions of surge tank was determined based on the results of a hydraulic computation on upsurge and downsurge. In up-surge analysis, rapid closure of governer at 100% load at maximum water level at the power intake, 322.5 masl, was considered. In downsurge rapid increases of load from 50 to 100% was considered at full operation level of 319 masl. The results indicates upsurge of 329.8 masl and downsurge of 297 masl for the diameter of 17 m.

According to the geological investigation, rock is hard rhyolite while joints are developed. The shaft will be constructed being temporally supported by shotcrete and spot rock bolts and lined with 80 cm thick concrete.

Plan and sections of surge tank is as shown in Figure VIII.5.6.

2.6 Penstock

2.6.1 Alternatives of penstock

Alternatives on type and alignment, number of lanes and diameter of penstock were studied.

(1) Type and alignment

Since the powerhouse will be an open air type powerhouse, both type of penstock, open air penstock and high pressure tunnel penstock, are conceivable in this scheme. Advantage and disadvantage of each type are summarized as follows;

Open air type : Construction cost of open excavation and concrete work is much cheaper than that of underground construction, while steel penstock pipe is required for the entire length from the surge tank to the powerhouse. In addition a penctock valve will be necessary at the beginning of the open steel penstock for prevention of fatal damage to the powerhouse in the event of accident such as collapse of the pipe.

High pressure tunnel type : Construction cost of underground structures is costly while steel liner will be partly eliminated if rock condition is good enough to satisfy structural stability and water tightness of high pressure tunnel, and no penstock valve will be required.

In this study, following one open type penstock and two high pressure tunnel type penstocks were contemplated;

Type I :, Open penstock type which comprises 60 m horizontal penstock tunnel extending from surge tank, 433m long steel pipe, a penstock valve, and concrete supports.

Type II : High pressure tunnel penstock with combination of a 174m deep vertical shaft and two horizontal tunnels, 20m long upper tunnel and 394 m long lower tunnel. The horizontal tunnel is lined by steel pipe over the length of

324 m from the powerhouse toward upstream. Remaining section and the vertical shaft is lined only by concrete.

Type III : High pressure tunnel penstock with combination of 250m inclined shaft and two horizontal tunnels, 20 m long upper tunnel and 324m long lower tunnel. This alignment was determined to allow the length of inclined tunnel and upper tunnel unlined with steel liner, resulting that the surge tank was shifted by 100m upstream. As huge amount of excavation will be required to make open end at the top of surge tank, the surge is designed as a cavern shape.

Sketches of the above alternatives are shown in Figure VIII.2.5 and main dimensions are presented in Table VIII.2.4. The section unlined with steel liner was determined based on the criteria so called rule-of-thumb criteria introduced by Bergh-Christese and Dannevig in 1971 and the criteria used by ELETROSU of which contents are as described in Chapter 3.9. Diameter of each component was obtained from an optimum diameter analysis, which is described at the end part of this section.

Economic comparison was carried out by estimating construction cost including contingency, indirect cost and interest and loss of energy benefit due to head loss in 50 year operation. The result indicates that Type II is more economical, by US$ 3.0 million and US$ 1.3 million than Type I and Type III, respectively. A summary of economic comparison is presented as below and details are shown in Table VIII.2.5.

	Type I	Type II	Type III
Construction cost (mill.US$)	21.1	17.8	19.3
Loss of energy benefit (mill.US$)	1.5	1.8	1.6
Total	22.6	19.6	20.9

(2) Number of lanes

As two units will be installed in the powerhouse, the following two alternatives were considered.

Alternative I : One lane from the surge tank to some ten meters upstream of the powerhouse and from there the two lanes by a bifurcation.

Alternative II: Two lanes from the surge tank to the units. At the beginning of each pressure tunnel a penstock valve is installed.

Alternative II is evidently much more costly than that of Alternative I, while this has an advantage to be able to keep operation of one unit in the event of maintenance and repair of penstock.

No fatal shortage in the system where this powerhouse will be connected is foreseeable due to stoppage of operation because capacity of power system where the power house will be connected is very large in comparison with that of the powerhouse. Alternative I has been selected accordingly.

(3) Diameter

Incremental diameter increases construction cost, while at the same time decreases head loss and water hammer and loses stability in generating. Incremental head loss directly affect energy benefit. Increased water hammer will require heavier steel liner and turbine. Loss of stability will need heavier generator interia ($GD^{2)}$ generator.

Water hammer is also controlled by closing time. Accordingly two parameters, diameter and closing time were considered.

The alignment of penstock as shown in Figure VIII.5.6 was used. Length of penstock tunnel to be lined with steel was based on the above criteria. The optimized diameter of headrace tunnel lined with concrete has been applied to the section with steel liner. The diameter after the bifurcation was determined from the design of turbine. Based on the estimation of total of construction cost and loss of energy benefit for 50 year operation, a combination of 4.8 m diameter of steel liner and 10 second closing time is found the most economical, as seen in Figure VIII.2.6. Details of the study results are presented in Table VIII.2.6.

2.6.2 Feasibility Design

The penstock extends from the center of surge tank to the powerhouse. Horizontal length from the surge tank to the center of unit is 425m. Difference in elevation between the spring line of tunnel at surge tank and the turbine center is 179 m. The penstock is composed of a 20 m long low pressure tunnel driving from the surge tank, a 174.4 m long vertical pressure shaft and a 393.5 m long pressure horizontal tunnel connecting to the power house. Length of the tunnel lined with steel liner is 323.5 m from the powerhouse. A bifurcation will be set at 43m upstream from the center of unit. Diameter of the section with concrete lining is 4.8m and 40cm thick reinforced concrete lining will be applied to this section Diameter of section with steel liner before the bifurcation is 4.3 m, and 2.5 m in its downwards. Sixty (60) cm thick concrete lining has been designed which is considered minimum working space between steel liner and tunnel.

Two boring investigations were made along the penstock route in the Power Study of South Brazil and three borings were drilled in this stage at about 500 m downstream away from the penstock route. These investigation results show that rock along the penstock route is hard rhyolite with axial compressive strength of 191 to 207 Mp and dynamic elastic modulus of 66.9 to 69.9 Gpa. Though the cores recovered were fractured along joints, most of these joints are considered tightly closed insitu in general because all the Luzion values except one result are recorded less than 1.0. It should be noted that water escape during drilling of the hole made in this stage was observed, which indicates high permeable zones existing locally.

Because of considerable favorable geological condition in Brazil design of high pressure tunnel unlined with steel liner has been common approach in feasibility design except that geological investigation indicates clear concerns. Although amount of boring investigation made so far is insufficient to make a reliable whole geological picture for definitive design, the pressure tunnel without steel lining has been adopted considering the available investigation results of high physical strength and low permeability of rock. In the design, the criteria so called rule-of-thumb criteria introduced by Bergh-Christese and Dannevig in 1971 and the criteria which have been used by ELETROSUL were used. To minimize amount of leakage from the unlined section consolidation grouting will be carried out. Installation of drainage system composed of two horizontal drain adits, one will be at 160 masl and another at 220 masl and drainage curtain was recommended mainly for prevention of buckling of steel liner due to external water pressure in case of dewatering of the penstock. In addition this drainage system will be effective for prevention of slope failure in front of powerhouse due to rise of ground water level after commencement of operation. At the upstream end of steel liner high pressure grout curtain will be made for seepage control.

Plan, profile and sections of high pressure tunnel are shown in Figure VIII.5.7.

Tunnel and shaft will be constructed by conventional drill and blast method. Shotcrete and spots rock bolts will be required for temporally tunneling support. To be able to construct in parallel with powerhouse construction an construction adit of which diameter is 5.5m will be provided from the switch yard. After installing the steel liner concrete will be placed for filling the spaces between steel liner and tunnel.

2.7 Powerhouse

2.7.1 Alternatives of powerhouse

Alternatives on number of generating units, location of powerhouse site and type of powerhouse were studied.

(1) Number of generating units

If the more number of generating unit is installed, the flexibility in operation will be higher but the construction cost will increase. From this view point, one unit is not usually adapted except for a very small scale plant compared with the system scale since the generating capacity of the powerhouse will be totally lost during its scheduled maintenance and forced stoppage. Two unit installation for this scale of generating capacity is considered the most economical solution in general. However, in the case of run-of-river type development where daily regulating pondage is not provided, further consideration should be made to an operation mode during low inflows and the minimum discharge. In this view, more units installation than two could be more feasible.

Accordingly two, three and four units installation were studied. The following factors which will substantially affect economic comparison studies have been considered as follows.

Minimum plant discharge

Efficiency of Francis turbines of recent production may allow partial load operation down to about 30 % load, however minimum load of 40% has been adopted as a common practice. The minimum turbine discharge will be 18cms for two units installation, 12cms for 3 units installation and 9cms for 4 units installation.

Combined efficiency

Combined efficiency of each number installation is as shown in Figure VIII.2.7. The principle features of generating equipment for each case are as shown below:

	2 units	3 units	4 units
Unit output (MW)	70	46.6	34.8
Max. unit discharge (cms)	45.0	30.0	22.5
Min. unit discharge (cms)	18.0	12.0	9.0
Unit efficiency at full load	0.894	0.892	0.889
Rated speed (rpm)	327.3	400	450
Specific speed (m-kw)	135.0	133.1	131.1

Power loss due to planed stoppage

Two kinds of scheduled maintenance are foreseen, minor maintenance to be made yearly for 7 days and major overall to be made every 12 years for 30 days for each unit. However, as seen in Figure VIII.2.8 stoppage of generating due to low inflow are well foreseeable. Low flow records are observed especially in May and June. Based on the daily runoff records during the critical period the number of days in May and June which are lower than 63 cms (=45 cms+18 cms) for 2 unit installation, 72 cms (=60 cms+12 cms) for 3 unit installation and 78 cms (=68 cms+9 cms) for 4 unit installation are 44 days, 46 days and 50 days in average. This indicates that power loss can be avoided if the minor maintenance is scheduled in May and June. Hence power loss only due to the major maintenance was considered. Rates of power loss due to the planed stoppage for each case is as follows:

No.of units	2 units	3 units	4 units
Rate of power loss	0.125	0.194	0.271

Power losses due to unplanned stoppage

Percentage of unplanned stoppage to be used for feasibility study has been specified as 7.7 % by ELETROBRAS, which seems to be relatively high. With this, probabilities of various unplanned stoppage can be derived as shown below:

	2 units	3 units	4 units
Probability of one unit shut down	0.14807	0.21276	0.27102
Probability of one unit shut down	0.00593	0.00178	0.00355
Probability of one unit shut down	-	0.00046	0.00137
Probability of one unit shut down		-	0.00004
Overall probability of shut down	0.15400	0.23100	0.30800

The potential loss of water during the unplanned stoppages are shown in Figure VIII.2.8. The expected loss of water can be estimated by multiplying the potential loss of water with the probability of stoppages. As the result the loss of energy generation in percentage are estimated as follows:

	2 units	3 units	4 units
Loss of water in year (mill.cum)	79.7	74.4	75.6
Loss of energy generation (%)	5.38	4.88	4.92

Construction cost

For each alternative construction cost of turbine and generator, civil works and penstock bifurcation and penstock liner downstream of it are considered.

Based on the above consideration, an economic comparison study has been made. Estimated net benefit, namely energy benefit in 50 years operation minus construction cost are as summarized below, and details are shown in Table VIII.2.7.

	2 units	3 units	4 units
Construction cost (mill.US$)	73.1	81.8	85.0
Energy benefit (mill. US$)	327.2	334.3	334.5
Net benefit (mill. US$)	254.1	252.5	249.5

As shown above two unit installation is the most economical, namely by 1.6 million US$ and 4.6 million US$ in comparison with three unit installation and four unit installation. Besides, duration of non-operation period was examined for two units and three units installation. Result of the examination shows that the duration of non-operating period is 15% of the critical period for two units case and 10% for three units case. Based on this economical comparison and duration period of non-operation two units installation was selected. It is, however, recommended in the next deign stage, to further study on this matter from the operational point of view because economic superiority of two units installation is not so large as to discard three unit installation.

(2) Location of site

At around 400m downstream of the powerhouse site selected in the pre-feasibility study, the river sharply bends to the left and there is relatively flat area at the beginning of the bend. This area had been considered more attractive than that of original site due to the following reasons:

- Amount of excavation will be smaller by 100,000 cum which corresponds to US$ 0.6 million. in direct cost, and

- No serious damages to the opposite river bank by erosions will not be caused by tailrace water flow because direction of tailrace is oriented to the middle of river flow.

Sifting of the powerhouse downstream increases the construction cost of waterway by the increased length, but does not effectively contribute to gain available head because tailwater level around these sites is fully controlled by the natural sill at riverbed around 400m downstream of the original powerhouse site. As the result economic superiority will be almost same as the original site. However, number of houses to be removed will increase from 12 to 18.

Locations of the original powerhouse site, named Location I, and the alternative site, Location II, are shown in Figure VIII.2.9.

Three boring investigations were carried out at the surge tank, penstock and powerhouse of Location II. The results indicate no remarkable difference of geological condition between these two sites.

Mainly for the environmental reason, the alternative location was discarded and the location proposed in the pre-feasibility study has been employed.

(3) Type of powerhouse

In addition to the open air type powerhouse considered in the plan formulation, an underground type powerhouse was studied. Because of flatter topography at the site and low land compensation cost, construction cost of underground type is more costly in comparison with open air type in general, however underground type has an advantage to minimize environmental impact mainly for appearance and erosions of excavated slopes.

General plan, profile and sections of underground powerhouse designed are shown in Figures VIII.2.10 to VIII.2.12.

It was, however, found that excavated area would not reduce as expected because deep open excavation would be required for construction of the access tunnel portal due to thick deposit on the slope. Construction cost of the underground powerhouse was estimated at US$ 16.8 million as shown in Table VIII.2.8. Reduction of penstock steel liner length in comparison with the layout of open air type powerhouse as shown in Figure

VIII.5.6 will reduce the construction cost by about US$ 2.0 million. As the result, the total incremental cost to the open air type will be about US$ 3.8 million.

Based on the above, open air type has been selected.

2.7.2 Feasibility design

The proposed powerhouse site is located on the right bank of the Itajai river. There are small rapids at around 100 m upstream and at 400 m downstream which controls water level at the powerhouse site. Tailwater level at mean annual runoff is approximately 111.6 masl. Ten thousand year flood of 12,000 cms with corresponding water level of 215 masl. has been adopted for design of substructure of the powerhouse.

The powerhouse is an open air type powerhouse with two units. Type of turbine will be vertical Fransis and generator is three-phase, vertical shaft and seem-umbrella type. Main dimension of the powerhouse structure is 58.5 in length, 31.5m in width and 43.5m in height. To discharge used water for power generation smoothly into river, the powerhouse was shifted by 15 degree clockwise to the penstock line.

The powerhouse will be separated by six floors; namely, inlet valve floor (El. 104.0 masl), cooling water floor (El. 105.7 masl), turbine floor (El. 109.7 masl), generator floor (El. 114.0 masl), main machine floor (El. 120.0 masl) and erection bay (El. 125.2 masl). The machine bay will need 19 m in width, considering size of the turbine spiral case, size of the inlet valve and space necessary for connection to the penstock. Span between the crane rail centers will be 16 m. The two units will be spaced at 18 m centers to secure the necessary space for the turbine installation and for arrangement of the turbine auxiliaries and electrical equipment cubicles. The length of the machine bay will be 40 m. The erection bay for unloading and assembly of the machinery will be provided at El. 125.2 masl to carry packages easily into the erection bay from the ground level at El. 125.0 masl. The erection bay will need an area of 19.0 m x 18.5 m considering the possible approach of the powerhouse crane, and will need 13 m in height for lifting the generator stators and rotors with the powerhouse crane. The control room will be located at El. 120.0 masl floor in the powerhouse at the downstream side of the machine bay. An elevator will be provided in the powerhouse for the operator's convenience.

The dimension of tailrace channel will be 60 m in length and 28 m in bottom width. Since the existing road will be discontinued by the tailrace channel, a concrete bridge with a clear span of 36m will be provided. An concrete pier will be installed at the middle portion.

The powerhouse arrangement is shown on Figures VIII.5.6 to VIII.5.8.

The transformer yard for installation of the main transformers will be located at the ground level behind (at the penstock side of) the powerhouse. The outdoor switchyard will be located next to the powerhouse at the opposite side of the erection bay and will have an area of 80 m x 55 m for 138 kV conventional switch gear.

3 HYDROMECHANICAL EQUIPMENT

3.1 General

The Hydromechanical equipment including gate for sandflushway, inlet trash racks, raking equipment and disposal system, inlet gate and stoplog, sand drain gates, draft tube gate, steel conduits and steel liner were designed in accordance with the standards applied in Japan.

3.2. Gate for Sandflushway

It was contemplated for design of the gate for sandflushway to satisfy the following functions capable of;

(i) Keeping the closed condition usually.

(ii) Fully opening the sandflushway for releasing flood discharge more than 135 cu.m/s (1.5 times of maximum intake inflow), which will be automatically operated in response to change of water level measured by a float-mounted type water level detector.

(iii) Fushing out opening against silt load of sediments which will be accumulated in front of the gate.

(iv) Releasing river maintenance flow of 7.2 cu.m/s with about 1.0 m water depth.

(v) Allowing overflow of debris and timbers.

To satisfy the required functions, the gate and stoplog for the sandflushway were designed under the following concepts;

(i) A freeboard of 0.3 m is adopted for wave by wind.

(ii) A fixed-wheel type with double-leaves is required. A fixed-wheel type should be strong for impact such as by timbers and stones.

(iii) Deflecting plates of about 1.3 m in height on the both end of the top of upper leaf are provided to regulate overflow.

(iv) The guide frame is extended up to of the hoist deck for gate maintenance of the gate on the deck at EL. 324.8 m.

(v) Reservoir water level detector is provided to monitor water level.

(vi) Design head of the gate should be calculated based on the maximum flood water level to cope with the operation during flood.

(vii) One set of stoplog is provided in front of gate facilities. Each stoplog will be handled by an electrical monorail crane. A steel wagon for storage is provided at a space between the sand flush way and the power intake. This monorail crane will also be utilized for handling inlet stoplog.

The features of the gate design based on the above concept are as follows;

- One (1) set of double leaves-fixed wheel gate, with guide frame and electrically driven hoist (one motor - two drums type), having net opening of 5.0 m wide by 7.3 m high.

- One (1) set of four pieces of stoplog with one (1) set of frame for each stoplog slot, a lifting beam, a monorail crane and all necessary steel structures, having net opening of 5.0 m wide by 7.2 m (4 @ 1.8 m) high.

3.3 Inlet Trash Racks

To prevent flowing foreign material into the waterway, the trash racks were designed under the following concepts;

(i) Trash racks are provided for the surface type intake covering 9.8 m in height between the sill at EL. 315.0 m and the deck at EL. 324.8 m. Approach velocity should be limited to be less than 1.0 m/s, for prevention of unfavorable vibration of the trash racks due to Karman vortex and for the raking operation.

(ii) Trash racks should be of the fixed type, but removable when repair, and should keep a water head difference of about 3.0 m between their up-and downstream sides considering raking operation.

(iii) Space of bars in the trash racks less than one-fortieth of diameter of turbine (Dt) is recommended. Accordingly the space will be around 75 mm.

Feature of the trash rack thus designed is as follows;

- Four (4) sets of fixed trash rack, covering each net area of 5.7 m wide by 10.146 m slant high.