Properties of the fresh granite are same as that in the axis B. The slightly weathered granite has the compressive strength of 40MPa and dynamic elasticity of 15GPa, and its permeability is 20 Lugeons. Any zone seriously fractured was not found at this site.

Axis D

Hill on the left bank is thick and wide but right abutment is formed by a narrow promontory between two valleys. Although hard granite is exposed over the valley floor of main stream, no rock outcrop appears in the slopes of both banks. Rock can be seen in abandoned headings for a railway tunnel (El. 310 m) on the right bank and it is a highly decomposed granite, soft and loose. Consequently, surface of hard bed rock is supposed to lie gently ascending towards inside of the hill or the promentory. The rock surface on both banks is therefore assumed to be 310 m in elevation at 100 m inside in horizontal distance from river side. Geological section along the dam axis is shown in Fig. 6.7.

While no exploratory drilling was made at this site, properties of the base rock and overburden are assumed to be the same as those in the axes B and C.

(3) Headrace Tunnel and Culvert

Culvert

Cut-and-cover type culvert is planned for upstream part of headrace. Its length is about 350 m in the scheme of dam axis B and about 100 m in the scheme of dam axes C and D. In the all three routes of culvert, overburden consisting of clayey soil and decomposed granite is deep. Based on the geological reconnaissance, it is assumed that the base granite lies at the same level as the valley river bed of main stream at the area close to river bank and the rock level gradually ascends in parallel with stream bed of tributary flowing nearby the culvert route.

The overburden is likely susceptible to erosion and massive sliding if it is cut deeply. Cut slope of the culvert needs to be protected against such erosion and sliding.

Tunnel

Length of tunnel is 5.5 km to 5.9 km. Most part of the tunnel passes the granite mass zone except a downstream end section of about 100 m which passes rhyolite zone.

The granite is hard and massive, and classified as the class A or the index H1F1A1S1. Its compressive strength and dynamic elastic modulus are estimated to be 150 MPa and 62 GPa, respectively.

The rhyolite is also hard and has the compressive strength of 170 MPa and the dynamic elastic modulus of 69 GPa in tested samples. However, many joints and cracks develop in the rock mass. In view of the whole rock mass, the rock classification becomes one rank lower than the granite, i.e., the class B to CH or the index H2F3A2S2. In-situ permeability is less than 2 Lugeons in the granite and 5 to 10 Lugeons in the rhyolite.

While exact border between two rock masses of granite and rhyolite could not be identified in the geological mapping, its approximate location could be classified considering topographic difference. Although geological condition of the interface needs to be checked at the future design stage, it is presumably firm contact as can be seen in contact zone of usual Pre-Cambrian rock masses.

Six geological lineaments cross the tunnel route. Along those lineaments, v-shape valleys are formed. The lineaments GL-2 to GL-7 cross at nearly right angle and the GL-5 crosses diagonally as shown in Fig. 6.4. According to the log of drilling B-2 (in 1969) which was located on the GL-6, a fractured zone supposedly 50 m in length was found. This fractured zone is classified as the class CL or the index H4F4A4S4. However, it is uncertain whether or not the other lineaments are accompanied with fractured zone. As far as the result of past drilling and present geological reconnaissance is concerned, any sign of fault was not found along the lineaments.

Where the culvert from the dam axis B enters into tunnel, the depth of sound rock is about 11 m according to the past drilling. However, the depth is locally variable. The tunnel from the dam axes B and C crosses small valley located about 500 m downstream from the tunnel portal. To cross the valley, the tunnel is replaced by cut-and-cover type culvert since rock covering is too thin to make tunnelling. Foundation rock at this valley is poor as classified as CL or H4F4A4S4. To locate the tunnel portal and the culvert accurately, further investigations are required.

Concrete lining for the purpose of rock support may be omitted in the granite zone except in the portal and some portions crossing below the small valleys as mentioned above. As consequence, length of the concrete lining structurally required in the granite is estimated at about 150 m. On the other hand, in the rhyolite zone (approx. 100 m in length), concrete lining is required because joints and cracks develop.

(4) Powerhouse Area

Two alternative sites for powerhouse were selected for the present study. One is the upstream site as proposed in the Hydro Inventory Study - 1991, the other is the downstream site located 200m downstream of the above. Site of surge tank and route of penstock were also selected independently for each of powerhouse alternatives.

<u>Upstream Site</u>

Rhyolite is exposed on rock cliff just below the surge tank area and also on river side edge. Area between base of the rock cliff and the river side is covered with coluvial soil and weathered rhyolite classified as CM-CL or H3F4A4S4. Depth of the soil overburden is about 25 m at the maximum and gradually becomes thinner towards the river side. The weathered rhyolite is about 10 m in its maximum thickness and about 6 m at the powerhouse site. Base rock underlying the weathered rock is rhyolite which is hard but many vertical or high dip joints appear in the rock mass at intervals of 5 to 10 cm. The rhyolite is classified as CH or H2F4A2S3. Below the rhyolite, hornfels appears at the depth of 27 m in the drilling BI-1. Contact of both rocks is tight. The hornfels is hard but cracky, and is classified as CH-B or H2F3A2SVS2. Spring water and sign of fractured zone were not found in the present investigation.

<u>Downstream Site</u>

Penstock route is aligned in a ridge between two small ravines. Along the ridge, clayey soil and weathered rhyolite appear up to the depth of about 20 m. The weathered rock in the surge tank area is heavily decomposed state like soil while in the middle part of penstock route it still shows state of moderately hard rocks with many cracks as classified as CL-CM or H3F4A3S4. Fresh rhyolite underlies 20 m below the ground surface. The rock is same as in the upstream site and classified as B-CH or H2F3A2S3.

River side flat area for powerhouse site is covered with alluvial deposits of clay and boulders. Rock is not exposed even on the river side. Base rock underlies the alluvial overburden which has the thickness of 12 m. Type of the rock is diabase intruded into rhyolite mass. The rock is classified as CH or H2A2F3F3.

6.5. Rock and Soil Materials for Construction Use

6.5.1 Concrete Aggregate

Natural deposit of sand and gravel mixture is scarce in and near the project area. Any deposits containing sand do not appear in river bottom in a 85 km long strech down to Blumenau. Most practicable way to obtain materials for concrete aggregate will be utilization of the proposed quarry near the construction site.

Quarry site-A located on right bank hill opposite to the existing Paraíso resort camp area; 1 km upstream of the dam axis B was initially investigated. While outcrop of hard granite appears on a 150 m long scarp of the hill, its available volume estimated is 150,000 m^3. Since it is considered difficult to extract it from environmental aspect especially from scenic aspect, alternative quarry area was saught in the vicinity of damsite and a right bank hill located 3 km downstrem of the dam axis B was selected as the quarry site-B. Granite outcrop appears also on scarp along the river side in the length of about 150 m and in the height of about 10 m. Overburden in the hill top area is deeper than 15 m. Available quantity of rock was estimated at 120,000 m^3.

According to laboratory test on rock samples taken at the quarry-A, compressive strength of the granite is more than 150 MPa. Crushing loss is however relatively large; 32% in the Los Angeles abrasion test.

Rock excavated in the headrace tunnel is also usable as material for concrete aggregate.

6.5.2 Embankment Material

For constructin of cofferdams and embankment portion of main dam in the left bank including abutment blankets, rock material and impervious clay material are required. Overburden on the hills in the dam area is clayey soil and thick in depth everywhere. The overburden is usable as the impervious material. Rock obtained in the quarry sites or excavated in foundations of structures can be utilized as the rock material for embankment. For construction of left bank cofferdam, an existing big quarry site located on left bank hill about 2 km downstream of the dam axis C is usable. Rock in this quarry (named as Quarry-C) is hard and good in quality and is obviously sufficient in quantity.

Chapter 7 POWER DEMAND AND SUPPLY STATUS

7.1 Power Supply Network and Organization

DNAEE is responsible for framing the electric power policy, approving the implementation program for construction of power facilities, deciding the electric power tariffs, etc. and generally controlling Brazilian power industries as the competent authority.

The nation wide electric power supply is entrusted to ELETROBRAS, a partly state-owned corporation under the jurisdiction of the Ministry of Mines and Energy. ELETROBRAS is responsible for implementing the Brazilian electric power policy including planning, financing, and supervising the program for the power generation, transmission and distribution systems. ELETROBRAS is the principal funding agency of the power sector, both for federal and state utilities and operates the power system in whole Brazil via four regional subsidiaries: ELETRONORTE in the north and middle west, CHESF in the northeast, FURNAS in the southeast and middle west, and ELETROSUL in the south. In addition, ELETROBRAS has two state subsidiaries, LIGHT in the state of Rio de Janeiro, and ESCELSA in the state of Espirito Santo. ELETROBRAS is also a partner in state electric utilities and holds 50% of the stock of ITAIPU Binational.

The major state governments also have their own electric power enterprises other than ELETROBRAS group and have also the right to develop the power generating plants by themselves within their territories with the DNAEE's approval.

ELETROSUL controls the electric power supply to all the states in south region and also Mato Grosso do Sul in middle - west region. Main electric power enterprises under ELETROSUL are COPEL; CELESC, CEEE and ENERSUL.

CELESC is a Santa Catarina state government owned utility and is responsible for supplying electrical power to the state of Santa Catarina. CELESC was established in 1956 by the merger of power companies. In 1992, CELESC had its owned generating facilities of 72.87 MW in total effective capacity which corresponds to about 5 % of the state demand. The rest has been purchased from others, mainly from ELETROSUL and ITAIPU Binational through ELETROSUL.

7.2 Existing Power Supply System

7.2.1 Whole Brazil.

ELETROBRAS has divided the whole country of Brazil into four regional areas by category of power system and established the subsidiary companies, ELETRONORTE, CHESF, FURNAS and ELETROSUL. These subsidiary companies have their own power transmission network. They are also interconnected in two major power systems, namely, north/northeast and south/southeast systems. These two systems are operated separately each other and will not be interconnected until 2000. Most major load centers and major power plants in each system have been interconnected by the trunk transmission lines of ultra high voltage of AC 230/345/440/500/750 kV and DC ± 600 kV. The existing power supply systems of Brazil in 1992 is summarized as follows:

Installed capacity (MW)	Hydro	Thermal	Total
ELETROBRAS	22,299	3,090	25,389
Systems;			
FURNAS	(6,800)	(1,323)	(8,123)
CHESF	(7,251)	(453)	(7,704)
ELETROSUL	(2,602)	(620)	(3,222)
ELETRONORTE	(4,685)	(694)	(5,379)
ESCELSA	(160)	-	(160)
LIGHT	(801)	-	(801)
Other Companies	18,486	1,665	20,151
ITAIPU Binational	12,600	-	12,600
TOTAL	53,385	4,755	58,140
	(91.9%)	(8.2%)	(100%)

Energy Production (GWh)			
ELETROBRAS	98,837	1,244	106,081
ITAIPU	50,156	-	50,156
Others	95,954	880	96,834
TOTAL	244,947	8,124	253,071
	(96,8%)	(3.2%)	(100%)

This table shows that hydro power provides 92% of the total installed capacity and 97% in total energy production respectively.

7.2.2 South and Southeast Power System

South and southeast power systems including ITAIPU Binational which are interconnected each other are the biggest one in Brazil. Breakdown of power supply conditions in this network is summarized as follows:

Installed capacity (MW)	Hydro	Thermal	Nuclear	Total
South	6297.9	1,133.0	-	7,430.9
Southeast	23,039.0	1,317.2	657.0	25,013.2
ITAIPU Binational	12,600.0	-	-	12,600.0
Total	41,936.9	2,450.2	657.0	45,044.1
	(93.1%)	(5.4%)	(1.5%)	(100%)
Energy Production (GWh)				
South *	25,521	3,223	-	28,744
Southeast	117,835	382	1,759	119,926
ITAIPU Binational	50,156	-	-	50,156
Total	193,512	3,605	1,759	198,826
	(97.3%)	(1.8%)	(0.9%)	(100%)

Note: * including ENERSUL (Mato Grosso do Sul)

7.2.3 CELESC Power System

CELESC's transmission and distribution lines have been linked with the south/southeast of the transmission system through ELETROSUL's substations in the state. CELESC takes care 100% of power demand in the state of Santa Catarina territories with an area of 95,483 km^2. The existing power supply facilities owned and operated by CELESC itself in 1992 comprise 12 nos of run-of-river type hydro power plants with 72,32 MW in total effective capacity, transmission lines of 3,454.2 km in total length and substation transformers of 3,157.4 MVA in total installed capacity. The annual report prepared by CELESC in 1992 states that CELESC generated the power energy of 376,160 MWh by 12 hydropower plants in 1992 and annual mean capacity factor for 1982 - 1992 period was 58% for the annual power output energy. The total annual energy required and power energy supplied in the state in 1992 was as follows:

Description		Energy (MWh)	Ratio
Generated by CELESC's owned plants		376,160	(4.8%)
Purchased from	- ELETROSUL	4,575,203	(58.7%)
	- ITAIPU Binational	2,806,292	(36.0%)
	- Others	40,503	(0.5%)
Total		7,798,158	(100%)

The above table shows that 95.2% of the total required energy is purchased from ELETROSUL, ITAIPU and others. CELESC's own power generation is only 4.8%. The power trading between CELESC and ELETROSUL has been made at 9 points of substations of CELESC and/or ELETROSUL in 69 kV and /or 138 kV including ITAIPU's power.

7.3 Power Market

7.3.1 Present Power Demands

(1) South and Southeast System

Integrated Network

The integrated south/southeast system is the biggest power network in Brazil, and involves Brazil's major power consumption centers such as São Paulo and Rio de Janeiro. Energy and peak power demands in the south/southeast system in 1992 are summarized as follows:

		South System	Southeast System*	Total
• Energy Consumed	(GWh)	32,684	137,200	169,884
• Peak Power	(MWh/h)	6,650	26,268	32,923
• Installed Capacity	(MW)	7,431	37,613	45,044
• Load Factor	(%)	65	69	68

* : Including Itaipu Binational

South System

The south system in which the CELESC system is involved is one-fifth of the integrated south/southeast system in power market scale. Overall operation of the south system is commanded by ELETROSUL.

Distribution of electric power to consumers in the south system is handled by four state power companies ; COPEL (Paraná), CELESC (Santa Catarina), CEEE (Rio Grande do Sul) and ENERSUL (Mato Grosso do Sul). Energy and peak power handled in 1990 - 1992 by these four state companies are summarized for 1992 as follows:

Description	Energy (GWh)	Ratio (%)
• Sold Energy		
Residential	8,572	26.9
Industrial	12,770	40.1
Commercial	4,145	13.0
Rural	2,879	9.1
Public & Others	3,479	10.9
Sub-total	31,845	100
• Bulk supply to local distribution companies	573	
• Losses and difference	3,242	
• Total energy required :	35,660	
- own generation	(14,296)	
- received from ITAIPU and others	(21,364)	
• Annual peak demand	4,260 MWh/h	
• Average load factor	62.8%	

(2) CELESC System

Total energy requirement recorded in the CELESC system in 1992 was 7,798 GWh. Except own use and losses, 7,234 GWh (93% of the total) was sold to consumers and supplied to the several local power distribution companies in the state, which are to be merged into CELESC in the future.

The CELESC's energy supply in the 1992 as follows:

Description	Energy (GWh)	Ratio (%)
• Sold energy		
Residential	1,708	24.0
Industrial	3,454	48.6
Commercial	723	10.2
Rural	670	9.4
Public & Others	556	7.8
Sub-total	7,111	100
• Bulk supply to local distribution companies	123	
• Losses and difference	565	
• Total energy required :	7,798	
- own generation by CELESC	(376)	
- received from ELETROSUL and others	(7,422)	
• Annual peak demand	1,358 MWh/h	
• Average load factor	65.6%	

In the recent 10 years, the number of consumers in the system has rapidly increased especially in residential and industrial sectors; to 1.8 times and 2.8 times, respectively. Sectoral share of energy consumption increased in the residential sector from 19% level to 23% level for the 10 years. The share of the industrial sector, however, dropped from 55% level to 50% level due to faster growth of the other sectors.

7.3.2 Load Curve

Taking into account a development scale of the Salto Pilão Project anticipated to the 100 to 150MW, load curves of two power networks are discussed herein.One is the whole CELESC network currently having 1,300 MW load scale and another is a CELESC's sub-network for the Itajaí river basin which has 300 MW load scale. The Itajaí sub-network covers cities of Blumenau, Rio do Sul and Itajaí and their surroundings.

Hourly power demands in various days in the CELESC network and the Itajaí sub-network, monthly peak power demands for the recent 10 years in the CELESC system, yearly growth of the monthly peak demands for recent 13 years and variation of annual average load factors for the recent 22 years were investigated based on the available records. From these records, the followings are made clear;

(1) There are two load-peaks every day; high peak in night time and low peak in day time. Ratios between high and low peaks are 1.05 to 1.35 on weekdays

(2) Daily load factors varies between 0.65 and 0.81 in the CELESC network and between 0.65 and 0.89 in the Itajaí sub-network. Usual load factor on weekdays is 0.71 to 0.84 but the load factors in Sunday become rather low as 0.64 to 0.67.

(3) In the recent 10 years, monthly peak demand gradually increased on linear tendency. Annual peak appeared almost in 3 months from May to July, though difference between loads of peak and off-peak months was relatively small. The annual load factor was improved and raised from a 55% level in early 1970s to a 65% level in early 1990s while decrease was observed 3 times; 1973, 1981 - 83 and 1989 - 91 due to economic recession.

7.3.3 Trend of Power Market Growth in CELESC System

Based on historical records of energy consumption by various sectors in the CELESC system for the recent 10 years from 1983 to 1992, annual rate of consumption growth was studied. Result of study clarified that overall energy demand has steadily grown in the recent 10 years and average annual growth rate was 7.1%. In 1990 the demand growth has slightly retarded mainly due to recession of the industrial sector. Average rate of demand growth in the industrial sector was rather low but this sector still consumes approximately a half of total energy of the CELESC system. Residential and rural sectors have grown at relatively high rates in energy consumption.

Meanwhile, in the period of 6 years from 1986 to 1991, growth rate of overall demand in the CELESC system was 5.6% on an annual average while the gross regional domestic product (GRDP) of the state of Santa Catarina was as low as 2.2%. The power market of CELESC has expanded at much higher rate than that of GRDP mainly due to steady increase in economic activities and rapid rise of living standard in the state.

7.3.4 Electric Tariffs

Tariff rates of electric power and energy are subject to approval of DNAEE. Change of the tariff rates are proposed by either federal or state power enterprises (concessionaires) to DNAEE. Approved tariff rates are applied to power supply to consumers and to power trading between the concessionaires. When the tariff rates have been changed, it is immediately published on the official gazetteers. In order to cope with continuous price escalation in Brazil, the tariff rates are raised 2 or 3 times every year. In this report, the tariff rates decided on March 11, 1993 are considered. The tariffs expressed in Cr$ are converted to US$ at the rate of Cr$ 20,062 to US$ 1 on March 1, 1993.

The tariff structures consist of tariff for consumers, tariff for concessionnaires and tariff for electricity from Itaipu Binational.

The tariff for consumers comprises tariff for large consumer (500 KW load or more) and small consumers. The tariff rate for large consumers are different between peak and off-peak hours and also between dry and wet seasons. The peak hour is from 17:00 to 20:00 and the dry season is from May to November. The tariff rates vary from 1 US$/KW to 7 US$/KW for power demand and from about 26 US$/MWh to 47 US$/MWh for energy consumption in peak time, while those for small consumers consists of only energy consumption and it varies from about 17 US$/MWh to 88 US$/MWh.

Tariffs rate for power trading between the regional and state power enterprises (concessionaires) is derived from system expansion marginal cost. It is composed of three different classes in terms of steadiness of energy consumption, these are T1 (long term contracted power (P1) and energy (E1)), T2 (Short term contracted power (P2) and energy (E2) exceeding P1 and/or E1) and T3 (actually consumed power (P3) and energy (E3) exceeding (P1 + P2) and/or (E1 + E2))

P1 and E1 are computed every year by the Electric System Planning Coordination Group (GCPS). P2 and E2 are decided based on short term operation plan of the state power enterprises.

Breakdown of the tariffs for T1, T2 and T3 to be applied to CELESC for purchasing energy from ELETROSUL are as follows:

		Tariff Class		
		T1	T2	T3
Energy	(US$/MWh)	19.33	5.46	2.22
Power, Peak	(US$/kW)	1.36	1.36	1.36
Off-peak	(US$/kW)	1.35	1.35	1.35

In the present contract, as the peak and off-peak classification have been deleted, only the peak power tariffs shown above are applied for all electric power.

In tariffs for electricity from Itaipu Binational, CELESC receives electric power from the Itaipu Binational via ELETROSUL's transmission system under the contract between them. The contracted power demand is 422 MW as of March 1993. Tariff rate for electricity from the Itaipu to CELESC is composed of the following two costs.

- Generation cost to be paid to the Itaipu : 17.017 US$/MW
- Transmission cost to be paid to ELETROSUL : 2.688 US$/MWh

7.4 Power Demand Forecast

7.4.1 Demand Forecast for Regional Power System

The latest official forecast of nation-wide and regional power demand is given in the 10-Year Expansion Plan (1992/2003) prepared by GCPS and approved by DNAEE in 1992. The Plan is based on the ELETROBRAS's National Electric Energy Plan - 1987/2010 (Plano 2010).

On the basis of state-wide demand projections indicated in the 10-Year Plan, annual energy demands for each of the four regional power systems and annual peak demands for the south/southeast systems were calculated. The projected demands for 1992, 1995, 2000 and 2003 are summarized as follows:

Power System	1992	1995	2000	2003
Energy (GWh)				
North	18,256	20,966	29,685	37,282
Northeast	27,767	32,734	44,271	52,715
Southeast	137,200	155,961	195,317	223,725
South	32,684	38,338	49,778	57,901
Whole of Brazil	215,907	247,999	319,051	371,623
		(4.5)	(5.3)	(5.2)
Peak Power (MW)				
Southeast	26,268	30,083	38,369	42,134 *
South	6,655	7,666	9,525	10,406 *
Total	32,923	37,749	47,894	52,540 *
		(4.8)	(4.9)	(4.7)

* : Peak power in 2002

( ) : Annual growth rate in %

7.4.2 Demand Forecast for CELESC System

Power demand projection for the CELESC system has been made by CELESC and revised every year analyzing past trend of energy consumption and forecast of future economic activities in the state of Santa Catarina. The CELESC's latest demand forecast for the period up to 2003 for each consumer sector was made and summarized as follows;

	Sectors	1992	1995	2000	2003
Energy (GWh)					
	Consumption	7,111	8,226	10,578	12,207
	Bulk supply	123	137	168	189
	Losses, others	564	625	803	926
	Total required	7,798	8,988	11,549	13,322
			(4.8)	(5.1)	(4.9)
Peak Load (MW)		1,358	1,569	1,957	2,215
			(4.9)	(4.5)	(4.2)

( ) : Annual growth rate in %

This CELESC's forecast was made based on ELETROBRAS's method, which is applied to all the power companies in Brazil in order to coincide nation-wide power demand forecasting.

The forecast was reviewed in this study with an economic parameter method in which the power consumption is devided into several categories or sectors, i.e. residential, industrial, commercial, rural, public and others, bulk supply, loss and difference. Growth of the power demand could be correlated to the growth of GRDP. Referring to the past records of energy consumption and future socio-economic forecast, the parameter for each sector to assess the power demand was estimated as follows:

- Annual growth rate of energy consumption (1992-2003)

Residential	7.0 - 5.0 %
Industrial	5.0 - 5.0 %
Commercial	5.5 - 4.5 %
Rural	5.5 - 4.5 %
Public and Others	5.5 - 3.5 %

- Ratio to total consumption

Bulk supply	1.6 %
Loss and difference	7.5 %

The above estimation was based on the following asumptions.

1) As a world-wide economic activity is retarded recently and Santa Catarina State is fairly developed area, annual growth rate of each sector will rather decrease slightly in the coming 10 years than the past.

2) Annual growth rates of population in Santa Catarina from 1992 to 2003 are 1.6 - 1.3 %.

3) Though the growth rate of industrial sector is much affected by world and domestic economies, it will keep the same rate as past average.

4) As the rate of urbanization will increase from 74.2% to 80.1% in the coming 10 years, share of rural sector will decrease.

5) Ratio of bulk supply and loss to total consumption will not change from the past average, i.e. 1.6% and 7.5% respectively.

The demand forecast was calculated applying the above parameters. Future energy demands obtained by this independent forecast approximately coincide with those of the CELESC's forecast. It is therefore concluded that the forecast made by CELESC is reasonable.

7.5 Power Balance

7.5.1 Power Expansion Program

In compliance with the demand forecast set out in the previous Section 7.4, ELETROBRAS prepared the National Generation Expansion Program for 1993/2002 and it was revised by GCPS in 1992.

The projected chronological expansion of power generation in the south/southeast system and in the state is summarized as follows:

(1) Power expansion program in south/southeast system

Year	Installed Capacity Increment (MW)						Firm Energy Increment* (GWh/year)	
	Hydro		Thermal		Total			
Existing plants in 1992								
South/Southeast Sys.	41,416	(5,983)	3,115	(1,133)	44,531	(7,116)	150,001	(28,531)
Itaipu	12,600		-		12,600		50,519	
Ongoing & new plants								
1993	1,702	(630)	-	(-)	1,702	(630)	6,824	(2,526)
1994	2,314	(315)	350	(350)	2,664	(665)	10,681	(2,666)
1995	119	(-)	425	(425)	544	(425)	2,181	(1,704)
1996	861	(63)	375	(375)	1,236	(438)	4,956	(1,756)
1997	1,406	(63)	350	(-)	1,756	(63)	7,041	(253)
1998	3,286	(1,310)	350	(350)	3,636	(1,660)	14,577	(6,656)
1999	2,139	(1,310)	875	(50)	3,014	(1,360)	12,085	(5,453)
2000	1,808	(826)	400	(400)	2,208	(1,226)	8,853	(4,916)
2001	2,731	(962)	50	(50)	2,781	(1,012)	11,503	(4,058)
2002	4,696	(3,032)	175	(175)	4,871	(3,207)	19,530	(12,858)

Remark; ( ): values of south system
* : calculated using the assumed capacity factor of 0.4577 which is 90% of capacity factor of south/southeast system in 1992

(2) Power expansion program in the state of Santa Catarina

Year	Installed capacity increment(MW)	
	CELESC	ELETROSUL
1992	3	-
1993	-	-
1994	-	350
1995	45	-
1996	-	-
1997	-	-
1998	-	810
1999	-	810
2000	-	440
2001	-	1,640
2002	-	992

7.5.2 Balance of Power

Future balance of power and energy in the south/southeast system, which is expressed in difference between demand and supply is shown in Figs. V.7.1 and 7.2, respectively. Supply capacities of both peak power and energy seem to have sufficient reserve against demands for the future 10 years if all the planned projects are implemented on schedule. Due to recent fund raising and environmental difficulties, many of the on-going power projects in the south/southeast system are delaying in their completion. Many of the other planned projects are also anticipated to be postponed due to the same reasons.

Even if the CELESC's present expansion plan is achieved on schedule and a total of 48 MW new plants is put into service by the end of 1995, the CELESC's own generation will still be as low as 7.8% of its total energy demand in that year. This CELESC's self supply ratio is very low compared with those of the other state power utilities in the south system; 100 % of COPEL and 50 % of CEEE. CELESC is continuously vulnerable to power shortage because its own generation capacity is small and most of its required electricity has to be purchased from ELETROSUL.

7.6 Need of Hydropower Development

It has been estimated that the share of hydropower plants to the total power installation in south/southeast power supply system was 93% in 1992. This share of hydropower plants is remarkably higher than that of general power system in other countries and such high share will last for at least up to 2010.

It is general practice of power system operation to operate nuclear and coal thermal power plants for base power supply and hydropower plants and gas turbine and diesel power plants for peak power supply. In the south/southeast power system, run-of-river type hydropower plants have been mainly operated for base power supply together with nuclear and coal fired power plants and reservoir type hydropower plants have been operated for peak power supply.

The relationship curves between the demand forecast and power supply under the power expansion program of the south/southeast system for the power and energy show that both power output and energy between demand and supply are in balance with a reasonable reserve of power. However, development of the planned power schemes is still retarded due to financial problem and it seem to be difficult to implement these power schemes on schedule, besides, share of CELESC's own power generation is too small compared with cases of other electric companies or utilities in Brazil.

In this circumstances, it is considered that the new hydropower plants to generate cheaper electric energy and to supply base power to CELESC power system together with the existing CELESC's hydropower plants should be planned in the Itajaí river basin to avoid effect of retardation of power projects planned by the federal utilities and to stabilize CELESC's power supply capacity.

Chapter 8 PLAN FORMULATION

8.1. Introduction

The plan formulation for selecting the optimal development scale was studied by combinations of several alternative options in respect of such items as selection of damsite, dam scale including full supply level (FSL), and installed capacity of power plant.

Due to the topographic limitation of the damsite, idea of reservoir type power plant capable of monthly or seasonal peak generation is discarded and only a run-of-river type scheme was studied. However, small pondage is inevitably formed by intake dam and it is useful for daily regulation of flow. Effect of the daily regulation by the pondage is therefore analyzed in this study.

For selection of alternative options of FSL, the highest limit of FSL was examined. The limitation was so determined that backwater of the dam does not affect to the wide flat land above the waterfall located at 1 to 2 km upstream of prospective damsites. For dam scale, two cases were contemplated, i.e., without and with daily regulation pondage to make daily peak operation of the power plant.

After designing the components of the alternatives, construction cost and energy output of each combination of alternatives are calculated. From these cost and energy, unit cost of generated energy and net benefit are obtained for each combination. Referring the costs and benefits, and considering environmental constraint as well as possibility of daily regulation, the optimal combination of damsite and FSL was selected. The installed capacity was optimized on the selected combination of damsite and FSL based on the ELETROBRAS's optimization method so as to maximize its net benefit. After the optimal development scale has been determined, optimal design was studied for such project items as tunnel alignment, necessity of concrete lining of tunnel, diameter of tunnel, type of penstock, type of surge tank and number of generating units.

8.2 Assumptions and Conditions for the Study

8.2.1 Discharge for Power Generation

Daily discharge series for 50 years from 1941 to 1990 was used for estimation of energy output. Hydrologically the most critical period as a whole of the integrated system was defined by ELETROBRAS, i.e, 92 months from April 1949 to November 1956.

Estimation of energy output was carried out on the basis of the following two discharge series;

- Critical period series : Apr. 1949 to Nov. 1956 (92 months)
- Long term series : Jan. 1941 to Dec. 1990 (600 months)

Duration curves of those two discharge series at the damsite are illustrated in Fig. 5.2. Average of the daily discharges is 86.3 m^3/sec in the critical period and 108.2 m^3/sec in the long term period.

8.2.2 River Maintenance Flow

In order to ensure moving of fishes in the downstream river stretch, river water has to be always released from the dam as the river maintenance flow. The rate of discharge to be released was decided to be at least 7.2 m^3/sec which corresponds to 80 % of the minimum monthly discharge ever recorded. Discharge to be used for power generation was thus determined by deducting 7.2 m^3/sec from the natural discharge at the damsite.

8.2.3 Design Floods

(1) Design flood for dam and spillway

Magnitude of flood for designing the dam and spillway varies with scale of dam and its reservoir as well as with scale of hazard risk resulted from failure of dam. ELETROBRAS, in its guideline, stipulates the flood magnitude to be applied to different scales of dam and hazard risk which are classified into several categories. The dam in this study was classified into the category of "medium dimensions, low risk" in the guideline. Design flood to be applied for this category is 100-year probable flood or 1/2 of the probable maximum flood (PMF). The PMF approximately coincides with the 10,000 year flood. Discharges of the 100-year flood and the 10,000-year flood are 3,600 m^3/sec and 7,400 m^3/sec, respectively. The discharge of 3,700 m^3/sec was applied as the design flood for the dam and spillway in the initial stage of plan formulation study.

The magnitude of design flood was, however, increased to the 1,000-year flood (5,300 m^3/sec) for further safety of the dam after discussion with ELETROBRAS.

(2) Design flood for powerhouse

Based on the ELETROBRAS's standard, the 10,000-year probable flood, 12,000 m^3/sec at the powerhouse site, was applied as the design flood for powerhouse.

(3) Design flood for construction

Major part of the dam was planned as concrete with relatively small scale. Anticipated construction period of the dam is two dry seasons. Dam structures under construction will not be damaged seriously even if overtopping of flood occurs. The 2-year probable flood (1,100 m^3/sec) was thus applied as the design flood for river diversion works.

8.2.4 Free Board

Based on the ELETROBRAS's practice, free board against the design flood water level determined at 1 m for embankment section of the dam and nil for concrete section of the dam (1000-year flood) and also nil for powerhouse (10,000-year flood)..

82.5 Maximum Limit of Full Supply Level

There exists a resort complex on left bank river side at 1 to 2 km upstream of the prospective damsites. The resort complex is located at scenic site just downstream of a waterfall. Major resort facilities are located on flatter area between EL.321 m and EL.326 m. In order to avoid any influence of the Salto Pilão reservoir to the other wide plain on the upstream streach of the first waterfall, the maximum limit of the reservoir's full supply level (FSL) was set at 324 m in elevation.

8.2.6 Power Plant Operation Mode

The Salto Pilão reservoir under study is relatively small in capacity and will be filled up with sediment in early stage of the project life. However, regular removal of the sediment deposit enables to keep necessary storage capacity for the daily regulation. Taking into account these conditions, semi-base load operation was applied only in the case that storage capacity for the daily regulation can be provided in the reservoir. This semi-base load operation mode consists, in principle, of 3-hour peak operation and 21-hour off-peak operation. Daily combined load factor under this operation mode was determined at 60 % taking into account of daily load variation expected in the future. The case without daily regulation was also studied as pure base load plant.

8.2.7 Criteria for Economic Analysis

ELETROBRAS's standard specifies that economic analysis of power projects in Brazil is made in accordance with ELETROBRAS's criteria. The criteria on economic analysis are as follows :

(1) Firm energy is defined as the average energy generated in the hydrologically critical period of the power system in reference; i.e., integrated South/Southeast system.

(2) Secondary energy is defined as the energy generated in excess of the firm energy and calculated as the difference between the long term average energy and the firm energy.

(3) Plant operation is stopped for planned maintenance or unplanned troubles. ELETROBRAS's standard figures with regard to period of stoppage in percentage of total time span under study are as follows:

Plant Capacity: P (MW)	Length of Stoppage: ß (%)		
	Planned	Unplanned	Total
$30 \leq P < 60$	1.6	5.1	6.7
$60 \leq P < 200$	2.5	7.7	10.2

Effective energy suppliable to the system is therefore (100-ß) % of the energy producible without stoppage.

(4) Guaranteed peak power is defined as the peak power generatable at 95 % of permanency in the system.

(5) Construction cost is estimated in accordance with cost estimation criteria and format of ELETROBRAS. Cost of transmission line is excluded from the project cost. Interest during construction is computed by applying the rate of 10 % per annum and included in the project cost.

(6) Economic comparison for dimensioning of the project is made applying the project life of 50 years and the annual discount rate of 10 %.

(7) The mean cost of generation (unit cost of firm energy) is one of basic factors to evaluate competitiveness of a power project and is compared with the marginal cost of expansion of power system. Mean cost of generation is defined by the following equation.

$$CMG = \frac{CAI + COM - 8760 \cdot CRES \cdot ES - 1000 \cdot CMP \cdot PG}{8760 \cdot EF}$$

where, CMG : Mean cost of generation (US\$/MWh)

CAI : Annualized investment cost (US\$/year)

COM : Annual cost of operation and maintenance (US\$/year)

CRES : Reference cost of secondary energy (US\$/MWh)

CMP : Reference cost of peak power (US\$/kW/year)

ES	:	Secondary energy (MW year)
PG	:	Guaranteed peak power (MW)
EF	:	Firm energy (MW year)

(8) The reference costs in the above expression are closely linked to the marginal costs of expansion of system. Values of the reference cost are defined by ELETROBRAS based on its study of long term program for generation expansion of each regional power system. Those values are reviewed and revised every year by ELETROBRAS.

The reference costs of firm energy and peak power defined in March 1993 for the South System in which the Salto Pilão is involved are as follows :

	Reference Cost	
Year of Commissioning	Firm Energy (US$/MWh)	Peak Power (US$/kW/year)
up to 2000	51	0
2001 - 2005	60	0
2006 - 2010	62	0
2011 - 2015	64	0
2016 - 2020	69	0
2021 - 2029	70	0

(Note : These values are applicable for dimensioning of projects.)

The reference cost of secondary energy corresponds to fuel cost for thermal plant, which is defined to be 11.92 US$/MWh.

The reference cost of peak power is defined as nil because the system is composed of majority of hydropower plants and most of them are reservoir type plants capable of easily generating daily peak power required in the system.

(9) Dimensioning of major project components including installed capacity is made so that the net benefit is maximized; i.e., the cost necessary for a certain incremental variation of the component is close to but does not exceed the benefit attributable to such incremental variation. This is expressed by :

$$\Delta B > \Delta C$$

$$\Delta B = [(\Delta EF) \cdot CRE + (\Delta ES) \cdot CRES \cdot 8760 + (\Delta PG) \cdot CMP \cdot 1000] \cdot a$$

Where, ΔB : incremental capitalized benefit (US$)

ΔC : incremental investment cost (US$)

CRE : reference cost of firm energy (US$/MWh)

a : annuity cost factor (50 years, 10 % rate), a = 9.9148

Δ : expression of increment

The others are defined in (7) above.

(10) Unitary cost of installation (CUI) for a simple comparison between two or more projects is defined as below :

$$CUI = \frac{\text{Investment Cost}}{\text{Installed Capacity}} \text{ (US\$/kW)}$$

8.3. First Screening for Selection of Damsite and FSL

8.3.1 Study Flow and Study Cases

Dam sites and full supply levels (FSL) of the reservoir are first screened to select the optimum combination of their alternative options. This optimum combination is subjected to the succeeding second screening to select the optimum scale of power plant capacity. For the first screening, the following alternative options were selected;

- Dam location : Axes B, C and D
- Full supply level (FSL),

For axis B : El. 319 m, 324 m
For axis C : El. 310 m, 315 m, 319 m
For axis D : El. 305 m, 310 m, 315 m

Other intermediate FSLs are supplemented in optimization of FSL.

- Maximum plant discharge for each combination of dam axis and FSL:
30, 45, 60, 75, 90 and 105 m^3/sec.

Fig. 8.1 shows general alignment of major project components considered for the plan formulation.

8.3.2 Alternative Options

(1) Damsite alternatives

The dam axis C is located at the same location as selected in the pre-feasibility stage. The axis B is located on top of a rapid located at 500 m upstream of the axis C. Scheme of the axis B having FSL of El. 330 m has been studied previously in 1991 using map of 1:10,000 scale. That scheme has been abandoned because it required large flooding area for reservoir. However, the axis B was chosen again for the present study since the site is topographically and geologically attractive and reservoir flooding area can be minimized by lowering FSL below El.324 m. River bed elevation at the axis B is El.309 m which is 7 m higher than that at the axis C. The dam axis D is located at 500m downstream of the axis C. This site has relatively steep abutments on both left and right banks. The axis D is 8 m lower than the axis C in river bed elevation. This disadvantage in head for generation is expected to be offset by reduction of head loss owing to shorter length of headrace tunnel.

(2) Full supply level

Full supply level (FSL) of reservoir is restricted below El.324 m due to the limitation of reservoir level to avoid flooding of the upstream flat land. At the axis B, reservoir level lower than El.319 m is not practicable due to topographic limitation and ensuring the intaking head. The river bed level of axis C is E1.302 m and the lowest limit of FSL was decided to be E1.310m. Two other FSLs; E1.315 m and E1.319 m were selected to obtain the head and to examine function of regulation pond. The lowest limit of FSL for the axis D was decided to be El.305 m since the river bed level is El.294 m. The highest FSL was set at El.315 m. Another intermediate FSL; El.310 m, were also selected for comparison.

(3) Plant discharge

Six discharges; 30, 45, 60, 75, 90 and 105 m^3/sec were selected as the maximum plant discharges for the present study. Duration curves of plant discharge usable for generation after deducting the river maintenance flow of 7.2 m^3/sec are illustrated in Fig. 8.2.

(4) Reservoir storage capacity

Volume of the storage space required for the daily regulation was determined as follows:

$V = (Qp - q) \cdot 3h \cdot 3600$ sec

where, V : required daily regulation storage volume (m3)

Qp : plant discharge in peak operation (m^3/sec)

q : daily mean plant discharge (m^3/sec)

In case of the 60 % load factor, V is maximized when peak operation is made with the maximum plant discharge (Qmax) and q becomes 60 % of Qmax. Accordingly, the required maximum storage (Vmax) is given by Vmax = 4320 Qmax. Reservoir volume, full supply level (FSL) and minimum operation level (MOL) of each alternative are as shown in Table.8.1.

(5) Reservoir sedimentation

Sediment volume flowing into the reservoir has been determined at 246,300 m^3 /year on an average. It is assumed that 50% of the sediment occupies by wash load which consists of very small particles floating in water and it will flow out from reservoir without any deposit. Sediment deposit in the reservoir was thus estimated at 50 % of the total sediment flow into the reservoir, which is approximately 123,000 m^3 per year.

8.3.3 Design of Project Components

(1) Dam and Spillway

(i) Type

River channel at the damsites is a flat trapezoidal section with width of about 200m. Height of dam under study is 25 m at the maximum. The types of dam conceivable for these sites are concrete gravity type and rockfill type. Necessary spillway is rather large compared with the size of dam since flood flow to be handled is as large as 3,700 m^3/sec. The spillway types conceivable are non-gated type and gated type.

Economic comparison to select the best configuration of dam and spillway was made. The result showed that concrete dam with non-gated spillway is most economical. In the case that reservoir volume is extremely small as in this project in comparison with flood inflow volume, the non-gated spillway is much safer than the gated spillway which needs delicate gate operation for releasing flood flow. Accordingly, concrete dam with non-gated spillway was adopted in the plan formulation study.

(ii) Foundation geology and seepage cut-off measures

Hard granite is exposed in river bed in all dam sites, However, overburden consisting of soil and soft decomposed granite is considerably deep; 20 to 30 m, on both left and right abutments except at the right abutment of the axis B. The overburden is slightly permeable. A short embankment section is provided between concrete dam and abutment soil body left unexcavated. In order to avoid excessive seepage through the abutment overburden, a clay blanket covering reservoir shore slope is provided.

(iii) Spillway

Concrete ogee weir spillway without gate is provided across the river channel. Overflow width was determined to be 200 m for all cases so that it approximately equals to the width of downstream river channel. Crest elevation of the ogee weir was set at the same level of FSL. Overflow capacity of the ogee weir is computed by :

$$Q = C \cdot B \cdot H^{1.5}$$

where,	Q	:	overflow discharge (m^3/sec)
	B	:	length of ogee weir (m)
	H	:	overflow depth (energy head) on crest (m)
	C	:	coefficient of overflow
			C = 1.6 (after sediment deposit in reservoir)

The overflow depth of the design flood (3700 m^3/sec) was estimated at 5.2 m. Downstream foot of the ogee weir is protected by concrete apron with thickness of 1.0 m extending up to the length equal to dam height.

(iv) Dam crest level

Top of dam in non-overflow section is set at 1.0 m and 2.0 m above the flood level at discharge of 3,700 m^3/sec for concrete section and embankment section, respectively.

(v) Sand flush way and river outlet

A sand flush way to remove sand accumulated in front of power intake is provided in the dam adjoining the intake. Discharge capacity of the sand flush way was determined to be 1.5 times the maximum intake discharge so that sand deposit in the area around the intake can be flushed out completely. Sill elevation of the sand flush way was so determined that sand flushing slope of 5 % is kept below the intake.

A double-leaf roller gate was equipped in the sand flush way. Its upper leaf is an overflow type and is utilized as a river outlet facility to release the river maintenance flow of 7.2 m^3/sec from the reservoir.

(vi) River diversion

For river diversion during dam construction, two options are conceivable. One is by bypass tunnel and the other is by bypass channel. Design flood for the river diversion is 1,100 m3/sec of 2-year probable flood. Among two options, the channel diversion was adopted in this study due to topographic and geological conditions.

(2) Power Intake and Waterway

(i) Power intake

Intake for a run-of-river scheme is located at right angle to the river flow and immediately adjacent to a sand flushing channel in order to minimize the intrusion of sediments into the intake. The intake is divided into 2 to 4 channels, each leading to a bay of desanding basin. A steel roller gate is equipped at upstream end of each channel to enable closure of the intake. The maximum velocity in the channel was set at 2.0m/sec so as to avoid sediment accumulation in the channel.

(ii) Desanding basin

In order to remove harmful large particles of sand contained in power water, a desanding basin is provided immediately downstream of the intake. The desanding basin is composed of 2 to 4 trapezoidal shape basin. Sand deposited in bottom of the desanding basin is periodically drained to outside of the basin from sand drain gates equipped at the bottom of each bay. Top elevation of the desanding basin was set at the same level of dam crest so as not to spill out from the desanding basin. Downstream end of the desanding basin is connected to headrace tunnel or culvert.

(iii) Headrace tunnel

Headrace tunnel to lead water from the intake to penstock was aligned so as to maintain external water pressure higher than internal pressure in order to avoid excessive water leakage from the tunnel and to enable application of unreinforced concrete lining in the portion where rock needs structural support. Most part of the tunnel route comprises a granite rock and it is considered strong enough to selfstand in long term. Such good rock portion is assumed to occupy about 90 % of total tunnel length. The remaining portions; 600 m in their total length are assumed to be weak granite and cracky rhyolite.

In order to select an appropriate type of tunnel section for the plan formulation study, a tentative economic comparison was made on the following four types of tunnel:

Type 1: Fully concrete-lined circular section, driven by blasting

Type 2: Shotcrete-lined horseshoe section with concrete invert, driven by blasting

Type 3: Unlined horseshoe section with concrete invert, driven by blasting

Type 4: Unlined circular section with concrete invert, driven by tunnel boring machine (TBM)

As the result of cost comparison, the Type 2 was selected as the most economical tunnel type. The shotcrete-lined section was thus applied to most part of the tunnel except weak rock parts; 600 m in total, which are fully lined with concrete.

Based on the above result, diameter of the tunnel was so determined that the maximum velocity of flow in the tunnel becomes 2.5m/sec in the shotcrete-lined section or 3.5m/sec in the concrete-lined section, while the minimum diameter is set at 1.8 m. Thickness of the shotcrete lining is 10 cm. Thickness of plain concrete lining is 25 cm which is considered to be the minimum for easiness of construction.

(iv) Surge tank

Type of surge tank selected is simple cylinder type. In up-surging analysis, rapid closure of governor at 100 % load is considered. In down-surging, rapid increase of load from 50 to 100 % is considered. Up-surge and down-surge were analyzed with the use of the Calame-Gadan monogram.

(v) Penstock

Single steel penstock embedded in an inclined underground shaft and lower horizontal tunnel was adopted for this study. Technical discussion on whether a vertical shaft can be applied or not and also on necessity of steel conduit will be made in the following chapter. The mean inside diameter of steel conduit was determined so that the maximum velocity becomes 6.0 m/sec. The inclination of the penstock tunnel was set at 48°.

(3) Powerhouse

Surface type powerhouse with two generating units was considered. Based on the available head and plant discharge, vertical shaft Francis type turbine was selected. Dimensions of turbine and generator as well as powerhouse dimensions were estimated by empirical data. 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.

8.3.4 Energy Generation Study

(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. 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; system with and without Salto Pilão project. The result of the system analysis shows that the influence of the Salto Pilão to the system is very small and negligible namely, ratio of firm energy of system with and without Salto Pilão project for several alternative scales varies only from 0.992 to 1.003. Therefore, simulation of power generation in this study was made by the independent generation analysis using the daily discharge series of 50 years from 1941 to 1990.

(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.

Tailwater level varies with river flow discharge at the powerhouse site. The daily discharge is approximated by assuming that run-off originating from the sub-basin between the damsite and the powerhouse equals to 60 % of the natural daily discharge of damsitthe.

(3) Efficiency of turbine and generator

Combined efficiency of turbine and generator varies with unit capacity of plant and operation 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 (m^3/sec)
Qmax	:	max. plant discharge (m^3/sec)
H	:	arbitrary effective head (m)
Hd	:	design effective head (m)

(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 (m^3/sec)
H	:	effective head (m)
F	:	combined efficiency
g	:	acceleration of gravity (=9.8 m/sec^2)

Installed capacity of the plant (Pmax) is thus given by; $Pmax = Qmax \cdot Hd \cdot Fd \cdot g$, where Fd is design efficiency at full capacity operation. The installed capacity of each alternative option is shown in Table 8.1.

(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 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. 8.3 in which the case B319 without regulation and the case C315 with regulation are shown as typical output pattern.

Effective firm and secondary energies were then computed by deducting ineffective energies owing to plant stoppage. The effective firm and secondary energies computed for each case are shown in Table 8.2.

8.3.5 Costs of Alternative Options

(1) Investment cost

Investment cost (project construction cost) of each alternative was estimated in accordance with the ELETROBRAS's standard format. Price basis for this estimation is the prices at December 1992. Exchange rates is 1 US$ = 11,163.33 Cruzeiros or 1 US$ = 120 Japanese Yen.

(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 its records of power plant operation;

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

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

P : installed capacity (MW)

A and B : coefficients variable with plant capacity as follows:

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

8.3.6 Optimization of Damsite and FSL

(1) Procedure

Based on the ELETROBRAS's criteria, the dam site and the full supply level (FSL) were optimized in this first screening 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 damsite and FSL
Step 2: Selection of optimal FSL for each damsite
Step 3: Selection of optimal damsite

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

(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 m^3/sec), were 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.

Since the net benefit reaches the maximum when $\Delta B/\Delta C = 1.0$ and the development scale is chosen at the point where $\Delta B/\Delta C$ is larger than 1.0, the plant discharge of 90 m^3/sec is the optimal discharge for almost all cases except for the cases C319 and D315. Accordingly, the discharge of 90 m^3/sec were selected for optimization of damsite and FSL.

(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 m^3/sec 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 was therefore selected for the respective cases; "with" and "without" regulation.

Cost and benefit of each FSL option is calculated in Table 8.3 and the result is summarized as follows;

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	90.7			
C	315	245.6	332.6	87.1	256.3	342.5	86.2
	317				263.5	346.2	82.7
	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	80.6	255.8	334.5	78.7
	313				265.4	339.9	74.5
	315				272.0	343.5	71.5

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

This table shows that 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

(4) Optimal damsite

The three damsite options; axis B, C and D, were evaluated from the economical view point. Economic comparison of full supply levels is shown in Tabe 8.3. This table shows that the net benefit of the axis B is the highest among three dam axis options in either case of "with" or "without" regulation.

(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. Table 8.3 shows the differences of net benefit between both cases of "with" and "without" regulation. Major estimated factors for the selected axis B are as follows:

Item	Unit	Without Regulation	With Regulation	Ratio
		(A)	(B)	(C)
FSL	m	319.0	324.0	-
Firm energy	MWh	74.0	78.29	1.058
Benefit	US$ mill	340.3	358.7	1.054
Cost	US$ mill	227.4	256.5	1.128
Net benefit	US$ mill	112.9	101.2	0.896

This table shows that 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 about 10%. 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 was decided to be the pure base-load power station without regulation pond. The finally selected scheme is the combination of the dam axis B with FSL of 319.0 m which is not capable of daily regulation.

8.4 Second Screening for Selection of Installed Capacity

8.4.1 General

For the second screening, the design of project components, especially of headrace tunnel and penstock, of the selected scheme (Dam axis B, FSL=319 m) was thoroughly reviewed and their optimum layout and dimensions were decided.

Based on the refined design, power generation study and cost estimation were carried out on the selected scheme (axis B, FSL-319 m) with different installed capacities corresponding to the 6 cases of the maximum plat discharge of 30, 45, 60, 75, 90 and 105 m^3/sec. The optimum installed capacity was selected by economic comparison of their costs and energy benefits and further by engineering evaluation.

8.4.2 Refined Structural Design

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 (m^3/sec)	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, the optimal diameter for different plant discharges are as follows:

Max. Plant Discharge (m³/sec)	Shotcrete-Lined Tunnel		Concrete-lined Tunnel	
	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.

By this optimization, it was revealed that the tunnel diameter smaller than that assumed in the first screening is economical.

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 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. The selected optimal diameters are as follows:

Max. Plant Discharge (m³/sec)	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.

8.4.3 Power 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. 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 m^3/sec.

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:

Item	Unit	Max. Plant Discharge (m^3/sec)					
		30	45	60	75	90	105
Effective head	(m)	192.8	188.2	185.8	181.1	179.3	174.5
Installed capacity	(MW)	50.0	73.8	97.4	119.2	142.0	161.8

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

Effective Energy		Max. Plant Discharge (m^3/sec)					
		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

8.4.4 Cost Estimation

Investment cost of each case was re-estimated. 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 8.4 and summarized below:

(Unit: Million US$)

Cost Items	Max. Plant Discharge (m^3/sec)					
	30	45	60	75	90	105
(1) Direct cost	75.3	94.2	111.4	124.9	137.4	149.1
(2) 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
(3) Interest during construction	20.1	26.2	31.0	34.8	38.3	41.5
Total	118.1	147.7	174.3	195.9	215.5	233.9

Annual cost for operation and maintenance after commissioning of the project was estimated as follows:

(Unit: Million US$)

	Max. Plant Discharge (m^3/sec)					
	30	45	60	75	90	105
Annual O & M cost	0.57	0.67	0.74	0.80	0.86	0.96

8.4.5 Optimization of Development Scale

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. Plant Discharge (m^3/sec)	Installed Capacity (MW)	Capitalized Cost (Mill. US$) Investment	O & M	Total C	Capitalized Energy Benefit (Mill. US$) B	Net Benefit (Mill. US$) 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	71.8
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

The maximum net benefit is gained at a plant discharge between 90 and 105 m^3/sec or at a plant capacity between 142 and 162 MW.

The generation cost to evaluate the competitiveness of power project 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 (m^3/sec)	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

The 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 75 MW. The generation cost gradually increases with the scale of plant and its increasing rate becomes bigger in the range over 120 MW. This means that larger plant is lower in its competitiveness for the Salto Pilao project. Accordingly, the plant capacity of 142 MW was adopted as the optimal installed capacity.

8.4.6 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
Spillway ,	Discharge capacity	5,300 m^3/sec
,	Overflow width	200 m
Intake ,	Design max. discharge	90 m^3/sec
Headrace ,	Culvert, type:	Circular concrete culvert
,	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:	4.8 m to 2.5 m
Powerhouse, Type:	Ordinary surface type concrete building
Generating Equipment	
• Number of units:	2
• Maximum discharge:	90 m^3/sec for 2 units
• Rated head	179.3 m
• Installed capacity	142 MW (= 2 x 71 MW)

Chapter 9 OPTIMIZATION AND DESIGN OF PROJECT COMPONENTS

9.1 Introduction

Based on the result of plan formulation, optimization study and feasibility design of the project components were carried out for civil works including dam and spillway, intake/desanding basin, headrace waterway, surge tank, penstock and powerhouse, hydromechanical equipment and generating equipment.

9.2 Civil Work

9.2.1 Dam and Spillway

River cross section at the dam site is a flat trapezoidal section with 200 m in width. Hard granite is exposed along the river bed but both banks, especially left bank, are covered with deep overburden. Design flood discharge applied to the dam site is 5,300 m^3/sec corresponding to 1,000-year return flood.

The following three 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: Rock fill 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. For the Types 2 and 3, a half of the river channel width is utilized for spillway and non-overflow dam is located in the remaining part.

The result of the comparative study shows that Type 1 is the most economical. Besides, the non-gated spillway is much superior in operational viewpoint, since timely gate operation is difficult without telemetering system for flood control in the upstream stretches.

The proposed dam is concrete gravity type dam with 200 m wide non-gated spillway. Total dam width is 301 m and the maximum height is 18 m. 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. Full supply water level (FSL) has been set at 319 masl. Maximum flood water level for flood of 5,300 m^3/sec was computed at 325 masl. Free-board for concrete non-overflow sections is zero and 1.0 m for the fill section according to the criteria agreed with ELETROBRAS. Fill dam section comprises four zones, i.e. impervious soil zone, filter zone, shell zone and rock riprap on the upstream face. Two m deep foundation excavation is considered and consolidation grouting has been designed for entire area of concrete dam and fill dam in this stage. The layout of dam and spillway and typical sections are as shown in Figs 9.2 to 9.4.

9.2.2 Intake and Desanding Basin

No alternative study was made because change of type will not remarkably affect to construction cost . The intake facilities comprises a sandflushway, desanding basin inlet, desanding basin, and power intake. General layout, profile and sections of these structures are shown in Figs 9.2, 9.3 and 9.5.

The sandflushway is located on the right abutment of the dam immediately downstream of the inlet of 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 invert of the sandflushway inlet has been set at 312 masl, 3 m lower than the sill of inlet in order to prevent the sediment that will have accumulated in front of the sandflushway from flowing into the inlet as much as possible.

The inlet of the desanding basin was located on the right abutment of the dam, immediately upstream of the dam axis. The inlet was designed based on the inlet design discharge of 90.0 m^3/sec and allowable maximum velocity of 1.0 m/sec at the trash racks, which is considered maximum velocity appropriate for operation of trash rack rakes. As the result, the inlet sill was set at 315 masl and the opening below FSL has a cross-section of 4.0 m high and 22.8 m wide. In front of the trash racks, concrete curtain wall was provided to prevent timbers and floating debris from approaching to the trash racks. Four sets of trash racks will be installed. Two mechanical trash-rack rakes will be provided for the trash rack to clean.

The desanding basin was proposed to be provided at the inlet portion of the headrace tunnel due to the following reasons;

(i) This project has high-head power plant with a long headrace (6 km). It is vulnerable to scouring due to high velocity of the 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 abrasion for the embedded penstock steel pipe and turbine.

The desanding basin with 52.9 m wide and 190 m long will be located on the right abutment immediately downstream of the dam axis. The desanding basin comprises three inlet gates, there settling chambers and three sets of flushing system composed of channels and sand drain gates At the upstream end of each desanding chamber, a roller gate with 3.7 m wide by 4.4 m high will be installed. Transition channels extend immediately downstream of the gates to the desanding chambers of which width is 15.3m and depth of water will vary from 4.3 to 6.5 m. The top of side walls was set at 319.0 masl so that surplus inflow can be spilled out. The flushing channels to flush out sediments are located at the invert of chamber. Regulation of flushing will be achieved by operation of 2.5 m wide and 1.0m high slide gates installed at each channel. Operation of cleaning will be made for each chamber sequentially in order to avoid suspension of power generation.

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

9.2.3 Headrace Waterway

Two water conveyance methods, i.e. free flow tunnel and pressure tunnel are conceivable. Among them, pressure tunnel was employed from economic view point.

There are two deep valleys downstream of the desanding basin, one is approximately 100m and the other is 800m, which will affect the design of tunnel alignment. Location of these valleys are as shown in Fig. 9.1. To cross the first valley, two methods were considered, one is by supporting a culvert type waterway by bridge structure and the other method is by driving the culvert type waterway. As the result of the cost comparative study, the latter method was adopted .

Two alternative alignments, Route I and Route II, were contemplated to cross the second valley as shown in the figure. The alignment of Route I is straight alignment from the portal of tunnel to the surge tank by applying culvert waterway lined with steel liner. The alignment of Route II was set by sifting the tunnel downstream by 500m so that the tunnel can pass under the valley, resulting that the length of waterway is longer by 239m than that of Route I. Shotcrete lining will be applied to the entire length of each tunnel except for the first 20m at each portal and 540m for rhyolite rock zone and weak strength granite rock zones.

Incremental construction cost and loss of energy benefit due to head loss of Route II to those of Route I were estimated. Result of study shows that no remarkable differences in economic superiority between Routes I and II. Route I, however, has possibility to cross the valley by tunneling if the geological condition allow tunneling with thin cover. For these reasons, Route I has been adopted.

From the view point of result of the geological survey that most of rock along waterway tunnel is hard and massive granite, non-lining low pressure tunnel was 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, an optimum diameter of concrete lined tunnel was determined to minimize total of construction cost and loss of energy benefit at first, and then the diameter of different types of tunnel lining, unlined, shotcrete lined and TBM, was obtained, which yields the same amount of head loss in the concrete lined tunnel. Construction cost for these alternatives were then estimated. Based on this cost comparison study, shotcrete lining was selected.

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. The result indicates that 4.8 m is an optimum diameter for concrete lining tunnel and 5.8m for shotcrete lining tunnel.

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 at about 800 m downstream from the end of desanding basin, which length is 50 m. The upstream tunnel, named No.1 tunnel, is located between the culverts of which length is 492 m. The other tunnel, named No.2 tunnel, extends from the end of No.2 culvert to the surge tank with length of 5,145.3 m. Shotcrete lining was applied to 90 % of the total length of the tunnel and the remaining portions; about 600 m in total length are assumed to be weak granite and cracky rhyolite extending from about 100 m upstream of the surge tank to the downstream site. The diameter of shotcrete lining section is 5.8 m and 10 cm thickness of shotcrete. That of concrete lined tunnel to be arranged from 120 m upstream of the surge tank to cover the rhyolite rock zone is 4.8 m. Except for this portion where concrete lining is arranged in spots, the diameter was set at 5.8 m to keep the same tunnel diameter of shotcrete lined tunnel. Thickness of concrete lining depends on rock condition, and 40 cm thick concrete for the portals portions and poor rock portions and 25 cm concrete for fair rock portion. Plan and Profile of water way and sections are shown in Figs. 9.2 and 9.5.

9.2.4 Surge tank

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 upsurge analysis, rapid closure of governer at 100% load at maximum water level in the desanding basin, 322.5 masl, is considered. In downsurge, rapid increases of load from 50 to 100% is considered at the water supply level of 319 masl. The results indicates upsurge of 329.8 masl and downsurge of 297 masl for the diameter of 17 m. Plan and sections of surge tank is as shown in Fig. 9.7.

9.2.5 Penstock

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 study, 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, 433 m long steel pipe, a penstock valve, and concrete supports.

Type II : High pressure tunnel penstock with combination of a 174 m 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 250 m inclined shaft and two horizontal tunnels, 20 m long upper tunnel and 324 m long lower tunnel. This alignment was determined to allow the entire length of inclined tunnel and upper tunnel lined without 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.

The unlined sections were determined based on the criteria so called rule-of-thumb criteria introduced by Bergh-Christese and Dannevig in 1971 and the criteria used by

ELETROSUL. Result of comparative study shows that Type II is the most economical among them.

The following two alternatives were contemplated on the number of lanes of the penstock;

Alternative I: One lane from the surge tank to some ten meters upstream of the powerhouse and from there the two lanes via 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 keep operation of one unit in the event of maintenance and repair of penstock. However no total shortage due to stoppage of operation is foreseenable since the power capacity of powerhouse is very small in comparison with that of net work where the power system of this project is to be connected. Accordingly Alternative I has been selected. Diameter of the penstock steel lines was determined at 4.8 m considering affect of closing time in addition to construction cost and loss of head due to change of diameter.

The penstock is composed of a 20 m length low pressure tunnel driving from the surge tank, 174 m long vertical pressure shaft and 393.5 m long pressure horizontal tunnel connecting to the power house. Length of tunnel lined with steel is 323.5 m from the powerhouse. A bifurcation will be set at 43 m upstream from the center of unit. Diameter of section unlined with steel liner is 4.8m and 40 cm 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 considering the a minimum working space between steel lines and rock surface of the tunnel. Plan, profile and sections of the penstock are shown in Fig.9.7.

9.2.6 Powerhouse

For determination of the optimum number of generating units, three cases, i.e, two unit installation, three unit installation and four unit installation, were studied considering duration of non-operation period, combined efficiency, power loss due to planned stoppage, power loss due to unexpected stoppage, and construction cost of each case. Results of the study shows that two units installation is the most economical. The duration of non-operation period is 15% in the critical period for two unit installation and 10% for three unit installation, which effect to energy benefit were counted in the study. Based on this economic study, two unit installation has been employed.

Two alternative powerhouse sites are conceivable. One is the site proposed in pre-feasibility study and the other is site at about 400 m in its downstream. Economic superiority of the downstream site is almost same but the number of house to be relocated will increase in comparisom with the original site. Due to this environmental reason the original site was employed.

Two types of the powerhouse, open air type and underground type are conceivable. Comparative cost study was made for both types and consequently open air type was selected.

The proposed powerhouse site is located on the right bank of the Itajaí river. Tailwater level at mean annual runoff is approximately 111.6 masl. Ten thousand year flood of 12,000 m^3/sec with corresponding water level of 215 masl has been adopted. 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 smoothly discharge water used for power generation into the river, the powerhouse is shifted by 15 degree clockwise to the penstock line. 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 36 m will be provided. The powerhouse arrangement including the tailrace channel is shown in Figs. 9.7 to 9.9. The transformer yard for installation of the main transformers will be located at the ground level behind 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. Plan and profile are as shown in Figs 9.10 and 9.11.

9.3 Hydromechanical Equipment

A series of the hydromechanical equipment including gate for sandflushway, inlet trash racks, raking equipment and disposal system, inlet gates and stoplog, sand drain gates, intake gates, draft tube gate, steel conduits and steel liner were designed in accordance with the general standard and considering site conditions. The followings show a list of designed equipment and their dimensions;

Name of equipment	Dimensions
(1) Gate for sandflushway	One set of double-leaves-fixed wheel gate width 5m and 7.3m high One-set of four pieces of sloplog with 5m wide and 7.2 m high

(2) Inlet trash racks	Four sets of fixed trash rack, each 5.7 m wide by 10.146 m slant high Two sets of traveling type mechanical raking equipment
(3) Inlet gates and stoplog	One set of fixed-wheel gate with guide frame having 3.7 m wide and 4.4 m high One set of three pieces - divided stoplog with 3.7 m wide and 4.5 m high
(4) Sand drain gates	Nine sets of fixed-wheel gate with 2.5m wide and 1.0 m high
(5) Intake gates	Three sets of fixed-wheel gates with 3.8 m wide and 4.8 m high
(6) Draft tube gates	Two sets of slide gate with 3.5 wide and in 4.4 m high
(7) Steel conduits	Two thinner steel lined conduit pipes with 4.8 m in diameter and 394 m in length and 5.8 m in diameter and 70 m in length
(8) Steel liner	One line of steel liner with 4.3 m diameter to a spherical type bifurcation in 6.4 m in diameter and two lines of steel liner with 2.5m diameter after the bifurcation In total 365 m in length

9.4 Generating Equipment

A series of the generating equipment including hydraulic turbine, generator and main transformer were designed in accordance with the general standard and result of the optimization study. The followings show the equipment list and then dimensions;

Name of equipment	Dimensions
(1) Hydraulic turbines	Two units of hydraulic turbine of vertical shaft, single runner, single flow, Francis type, rated outlet of 72,600 kw
(2) Generator	Two units of generator of three-phase, vertical shaft, semi-umbrella type, synchronous alternator, rated at 78,900 KVA, 60 Hz, 0.9 power factor, 13.8 KV and 327.3 rpm.

(3) Main transformer	Three-phase, two-windings, oil-immersed, OF AF cooling and outdoor use type with an off-circuit tap changer

Two circuits of the 138 kV transmission lines will be introduced to this power station by means of T-branch of the double circuit transmission line between Blumenau and Rio do Sul Substations. A part of the existing Blumensu-Rio do Sul line is still single circuit line as of 1993 but is scheduled by CELESC to be revised to double circuit line by end of 1994.

9.5 Energy Output of Proposed Scheme

Based on the fixed dimensions of the proposed scheme as shown in Figs.9.1 to 9.8, energy production of the project was simulated on the daily basis for 50 years from 1941 to 1990 utilizing the daily discharge series of damsite and powerhouse site. The salient features of the proposed scheme are as follows:

- Dam axis:	B
- Reservoir full supply level:	319.0 m
- Design tailwater level:	111.5 m
- Design static head:	207.5 m
- Max. loss of head:	28.2 m
- Headrace, length:	6,091 m
- Penstock, length:	588 m
- Generating equipment,	
Number of units:	2
Installed capacity:	2 x 71,0 MW = 142.0 MW
- Max. plant discharge:	90 m^3/sec
- Rated head:	179.3 m

Daily power output was computed by:

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

Where,
- P: power output (kW)
- Q: turbine discharge: daily average discharge determined by available river flow at damsite (m^3/sec)
- H: effective head at turbine (m)
- F: combined efficiency of turbine and generator
- g: acceleration of gravity (= 9.8 m/sec^2)

Q was calculated by deducting the river maintenance flow of 7.2 m^3/sec from the natural river flow at the damsite. The effective head (H) was calculated by deducting loss of head in the power waterway from the static head. The static head varies with fluctuation of the tailwater level which relates to river discharge at the powerhouse site. The daily discharge at the powerhouse site was computed from the actual discharge records at Apiúna by applying the catchment area ratio between both sites. The effective head was computed by:

$$H = Hg - Hs$$

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

Where,
- Hg: gross static head (m)
- Hs: loss of head at Q (m)
- Hs max: max. loss of head at Q max (m)
- Q max: max. plant discharge (= 90 m^3/sec)

The combined efficiency of turbine and generator was estimated against rates of operation load to full load. When the rate drops below 40 % per unit, the turbine is not operable. The efficiency estimated is as follows:

Load	Combined efficiency
100 %	0.898
85 %	0.912
70 %	0.899
40 %	0.794

The average monthly energy outputs calculated from the result of daily generation simulation are shown in Table 9.1 and Fig.9.9. The yearly average energy outputs are also graphed in Fig.9.10. with the yearly maximum and minimum monthly outputs. These simulated output means the energy producible by available water without operational outage.

The effective firm and secondary energies were computed taking into account the outage rate of 10.2 %. The result is as follows:

Effective firm energy:	70.46 MWy
Effective secondary energy:	11.08 MWy

Chapter 10 CONSTRUCTION PLAN AND COST ESTIMATE

10.1 General

This chapter describes construction plan and cost estimates of the project which includes civil works such as cofferdams, dam and spillway, intake and desanding basin, headrace culverts, headrace tunnels, surge tank, penstock tunnel, powerhouse, tailrace channel and outdoor switchyard, hydromechanical equipment and generating equipment.

The project cost was estimated on local cost basis in accordance with the standard of ELETROBRAS. Cost allocation for foreign and local currency portions was made for reference for loan request.

10.2 Construction Plan and Method

10.2.1 Basic Condition

Construction plan was formulated based on the following conditions;

(1) Workable day

For river diversion	:	220 days
For earth and rock excavation works	:	264 days
For concrete work	:	288 days
For tunnel work	:	300 days
Daily working hours except for tunnel	:	8 hrs
Daily working hours for tunnel works	:	2 shifts (10 hrs for one shift)

(2) Swell factor: 0.95 for compaction of common material and 1.40 for rock material

(3) Construction equipment and plant : Majority of the construction equipment and plant are available at local market in Brazil.

(4) Hourly production rate of construction equipment and plant : The hourly production rate of the construction equipment and plant employed by ELETROBRAS and ELETROSUL was adapted.

(5) Construction materials : Most of the construction materials are available in local markets in Brazil. It is assumed that blasted rock obtained from the proposed quarries and excavation for the dam, intake facilities and tunnel will be used as raw material for concrete aggregate and filter material production. Forty percent of total requirement will be supplied from the excavation for structures. As for fine aggregate, it is assumed that 55 % of the

required amount will be produced at the crushing plant and the remaining 45 % is obtained from local market at Blumenau. The location of proposed quarry sites are as shown in Fig. 10.1.

(6) Labor source : Sufficient number of common labor will be recruited in the vicinity of the project area. Skilled and semi-skilled labor will be employed from Florianopolis, Itajai and Blumenau.

(7) The cost required for promotion of environment management and monitoring program was included in the project cost.

10.2.2 Construction Plan

(1) Preparation works

Prior to the construction works, construction of access road, preparation of spoil banks, construction of camps, installation of crushing and concrete plants and power and water supply system will be carried out. Locations of these facilities are as shown in Fig. 10.1.

One crushing plant with capacity of 100 ton/hr will be installed near the upstream quarry site and two concrete plants, capacity of 60 m^3/hr and 45 m^3/hr, at the dam site and powerhouse site.

(2) Main civil works

River diversion work, dam and spillway, intake and desanding basin

Construction works for the dam, intake and desanding basin will require common excavation of 296,000 m^3, rock excavation of 188,200 m^3, concrete of 98,000 m^3. The works will be conducted in 2 stages. In order to provide dry construction areas, river flows will be diverted alternately by multi-stage diversion method. Procedure of the river diversion is as shown in Fig. 10.2.

The right bank part of the dam (125 m) and intake and a part of desanding basin will be constructed in the 1st stage. The left bank part of the concrete dam (75 m), rockfill dam and other works at the left bank will be constructed in the 2nd stage. The center part of 2nd stage cofferdam will be constructed prior to the removal of the 1st stage cofferdam.

The common excavation will be executed by combination of backhoes and bulldozers. Rock will be excavated by bulldozers with ripper and bench cut blasting method using crawler drills and leg drills. The dam concrete placement will be carried out by concrete buckets and movable cranes. One lift is assumed 1.5 m height and block width 15 m. The structural concrete placement will be carried out by concrete buckets, chute or concrete pumps. Consolidation grouting for the dam and spillway foundation will be carried out using crowler drills and grout pumps. Since the geological conditions of the dam foundation are expected to be good, no grout curtain works are planned.

No.1 and No.2 headrace culverts

Length of headrace culvert lined with steel lining is 454 m (404 m for No.1 culvert and 50 m for No.2 culvert). Construction for No. 1 headrace culvert will require common excavation of 40,000 m^3, rock excavation of 6,900 m^3, concrete of 8,700 m^3 and steel pipe of 501 tons. Construction work for No.2 headrace culvert will require common excavation of 65,000 m^3, rock excavation of 5,000 m^3, concrete of 1300 m^3 and steel pipe of 90 tons. Excavation and concrete placing method will be similar to the manner as described above. After concrete placement, filling work will be executed. No. 1 headrace culvert will be executed after completion of excavation of the power intake at the end of desanding basin and No. 1 headrace tunnel. Construction of No. 2 headrace culvert will commence after completion of excavation of No. 1 and No. 2 headrace tunnels.

No.1 and No.2 headrace tunnels

Headrace tunnels of about 5,637 m in total length (492 m for No.1 tunnel and 5,145 m for No.2 tunnel) with an inner diameter of 5.8 m for shotcrete lined and 4.8 m for concrete lined tunnel were planned to be driven along the right bank of the Itajai river, of which shape is horse shoe and circular, respectively. The construction of No. 1 headrace tunnel will be carried out by rubber-tired tunneling method or railway system tunneling method. No. 2 headrace tunnel will be carried out by railway system tunneling method. Excavation of No. 1 headrace tunnel works will be proceeded from the No. 2 headrace culvert side. Excavation of No. 2 headrace tunnel works will be conducted from the both ends. The excavation works will be made by full face blasted method by 2 shifts operation. The shotcrete lining works and invert concrete works will be carried out in parallel with the tunnel excavation. The concrete lining works will start after the completion of tunnel excavation.

For excavation of the tunnel, a combination of 0.35 m^3 muck loader, train loader, 8 ton battery locomotives and 6 m^3 muck cars will be employed. Spoil banks (SB-8, SB-12) will be provided within 1 km from each portal.

Majority of tunnel excavation will be carried out without any tunnel support. The length and thickness of shotcrete lining will be 5,037 m and 10 cm. The shotcrete lining works will be carried out in parallel with tunnel excavation with keeping distance of about 200 m - 500 m away from the heading. The shotcrete lining concrete will be placed by the combination of shotcrete pump, shotcrete machine, agitator cars, and battery locomotives. After completion of the tunnel excavation, 25 cm or 40 cm thick concrete lining concrete lining including tunnel portal will be around 600 m. The lining concrete will be poured by the combination of a rail mounted type concrete pump, agitator cars, and battery locomotives. Concrete will be transported by agitator trucks from the batching plants to the portals and reloaded into the agitator cars for traveling in the tunnel. Invert concrete with 25 cm thickness will be placed in the entire length of shotcrete lined tunnel, of which concrete volume is estimated at around 5,200 m^3. The invert concrete work will be also carried out in parallel with the tunnel excavation work.

Following to the lining concrete works, backfill and consolidation grouting will be carried out. Total length of consolidation grouting is estimated to be about 3,900 m. The works will be made by leg hammers, grout mixers and grout pumps.

Surge tank

The cylinder type surge tank will be constructed between the headrace tunnel and the penstock line. The diameter of the surge tank are 17 m, and the height between the roof and bottom is 42.9 m. The excavation works will be made by two stages, pilot shaft excavation and enlargement works. A pilot heading, 2.0 m by 2.0 m will be drilled by 2 leg drills equipped with a raise climber. The pilot heading will start from the headrace tunnel level and proceed upwards. Enlargement of the surge tank will be performed by drilling and blasting method from the top to the bottom. Excavated materials will be removed through the pilot shaft.

Concrete lining for the surge tank will proceed upwards from the bottom using a slip form method. Concrete will be transported from the top of surge tank by two sets of cranes and concrete buckets.

Penstock

Underground type penstock which comprises vertical shaft and horizontal tunnels is planned to be constructed. The penstocks will have a total length of 587.5 m including a 174 m vertical shaft. The inner diameter of vertical shaft is 4.8 m and 0.35 m thick reinforced concrete lining will be placed. In the horizontal penstock tunnel connecting to the powerhouse, 335m long steel liner will be installed. The inner diameter of the steel liner varies from 4.3 m to 2.5 m via the bifurcation. In order to effectively carry out the tunneling works, a work adit will be constructed from the switch yard. The length of work adit will be about 350 m. The vertical shaft will be excavated by the similar manner as described in the surge tank, by using a raise climber operated upwards from the bottom. After making a hole by the raise climber the hole will be enlarged by conventional drilling and blasting method.

Installation of the steel liner and backfill concrete will be undertaken after completion of excavation of the tunnel. Concrete placement method in the tunnel will be almost the same as that of the headrace tunnel. Concrete in the vertical shaft will be placed by a slip form method. Concrete pumps, concrete buckets and chute with bumpers will be used for transportation of concrete.

Powerhouse, tailrace channel and switchyard

A surface type powerhouse is planned to be constructed on the right bank of the Itajai river near Subida. The dimensions of the powerhouse are approximately 28.0 m in width, 58.5 m in length, and 43.5 m in height.

Excavation of common for the powerhouse, tailrace channel and switchyard will be carried out by bulldozers. Rock excavation will be made by the bench cut blasting method and bulldozers with ripper. A temporary cofferdam will be provided at the tailrace channel end for the protection of working area from river flow. The crest of cofferdam will be used as temporary road. Concrete works will be commenced following the excavation works. Concrete placed for the powerhouse will be hauled by using 5 m^3 agitator trucks from the concrete plant to be located near the powerhouse. Concrete for the substructure will be poured by steel chutes and by concrete pumps. Superstructure works will commence following the completion of substructure concrete work. Second stage concrete around the turbine units, gate flames, blockouts, etc. will be required. The concrete works for the switchyard will commence before the installation of switchyard equipment. The building works for internal accommodation, finishing and others will be undertaken following the substructure and superstructure concrete works and erection of the roof. Utilities works will be carried out in parallel with the installation of generating equipment.

(3) Hydromechanical works

The hydromechanical works comprises a series of gates such as sand flush gate, stoplogs for sand flush gate, inlet trash racks, raking equipment, inlet gates, inlet stoplogs, sand drain gates, intake gate and draft tube gates, steel penstocks and steel conduits.

All gates, stoplogs, trash racks and other steel facilities will be fabricated at a local manufacturer's factory and transported to the project site.

Due to relatively large diameter of steel penstocks and in consideration of transport limit, the steel plates will be cut and rolled into the semi-circular sections at a manufacturer's factory and be transported to the field shop to be provided in the project area. Penstock pipes in each 3.0 m length will be fabricated from the semicircular sections in the field shop. Unit length for installation will be 6.0 m, coupling with 2 nos. of 3.0 m pipe. Unit segments of bifurcation will be transported from the work adit and erected and welded at the site. X-ray examination and hydrostatic pressure test to be made. After the erection of all the penstocks, backfill concrete will be placed. Contact grouting works between the pipe shells from grout holes provided on the pipe shells. After grouting, the holes will be plugged and sealed by welding.

The steel conduits will be installed in No.1 and No.2 culverts. Length of the steel conduits is 394 m and 70 m respectively. Since conduit shell thickness will be relatively thin, a lot of inner spiders will be required for fabrication and handling. Unit pipe, 6 m in length, will be directly placed at the designated place and welded. X-ray examination to be made. Coverage concrete will be placed after pipe installation.

(4) Generating equipment

Major components to be installed in the field will be the turbine draft tubes that will be installed after the primary concrete work of the lower part of the powerhouse substructure has been completed. The installation of the turbines will start after the powerhouse crane is available. Some parts of the turbines will be joined by field welding. The field welded parts will be subject to non-destructive inspection and pressure test or both of them. The first generating equipment will be installed in the concrete housing that will be made by the secondary concrete being placed following the installation of the turbine spiral cases and the turbine pits. The generator stators and rotors will be assembled on the erection bay and then be installed in their positions. Unloading, assembling and handling of the equipment in the powerhouse will be done by the powerhouse crane.

That second generating unit is scheduled to be completed one month behind the first unit. The last two months before the completion will be used as a period for the commissioning test to verify the guaranteed and specified performance of the equipment.

The outdoor switchyard area will not be useful until the penstock work is completed because the work adit for the penstock work will be provided in this area. The installation of the 138 kV switchgear will follow their concrete foundation work that will start after the land formation of the switchyard area.

10.3 Estimate of Project Cost

10.3.1 General

The project cost was estimated on a feasibility study level in accordance with the "Standard Budget Form" of ELETROBRAS.

10.3.2 Conditions for Cost Estimate

The following conditions were applied to the project cost estimate:

1) Price level of materials, labor and equipment: December 1992
2) Exchange rate: US$ 1 = Cr$ 11,163.33
3) Standard budget form of ELETROBRAS is used.
4) Local currency, Cruzeiro is used as shown above.

10.3.3 Constitution of Project Cost

The project cost comprises direct cost, indirect cost and interest during construction. The direct cost consists of acquisition of land and improvement, environmental management cost, civil works, hydromechanical works, electrical equipment and physical contingency. The indirect cost comprises construction site and camping cost, administration cost and engineering cost.

10.3.4 Direct Construction Cost

(1) Acquisition of land & improvement

The following figures were applied to the cost of acquisition of land and improvement;

Powerhouse area	
Urban	US$ 100/lot
Hill	US$ 1,000/ha
Dam right bank area	
Flat area	US$ 1,000/ha
Not Flat area	US$ 800/ha
Dam left bank area	
Flat area	US$ 1,500/ha
Not Flat area	US$ 1,000/ha

The estimated land acquisition cost of the project is shown in Table 10.1.

(2) Environmental management cost

Environmental management cost including cost for reservoir cleaning program, physical-biotic program and socio-cultural program was estimated at US$ 1.843 million as shown in Table 10.2.

(3) Civil works

The civil works comprises the work items of structure for river diversion, dam, spillway, intake, desanding basin, headrace culvert and tunnel, surge tank, penstock tunnel, tailrace channel and access road and bridges.

The works which do not coincide with the work items specified in the form were counted under the item of other cost. Breakdown of the other cost is as shown in Table 10.3.

The units cost of each works consists of process cost, BDI (overhead charge, site expense) and transportation cost. The process cost comprises, equipment cost, labour cost, and material cost.

(4) Hydromechanical works

Major equipment to be installed in the project works will be available in local market. The bases of estimates were, at first, developed based on standard design and statistical data, and then the cost has been refined considering past bidding records, recent cost estimate of similar projects and market price in Brazil. The cost includes designing, supplying materials, manufacturing, painting, testing, packing, delivering up to the project site with insurance premium, erection, installation, and completion test.

(5) Electrical equipment

The costs of the electrical equipment were estimated on the basis of the statistical price data that were obtained from the actual contract prices of the past international bid projects. All costs estimated are the lump sum basis. The estimated costs of the electrical equipment were summarized in three items specified, a) Turbines and generators, b) Accessory electrical equipment and c) Other equipment of powerhouse in the standard budget form of ELETROBRAS.

(6) Physical contingency

Contingencies are provided to cover unforeseenable change of physical conditions as the physical contingency. The rate of physical contingency is assumed at 15 % of each amount of acquisition of land & improvement, environmental management cost and civil works, while 10 % for hydromechanical works and electrical equipment.

10.3.5 Indirect Cost

(1) Construction site & camping

This cost is for temporary buildings and camping facilities required for construction work and supervision, which was estimated at 7 % of the total direct construction cost.

(2) Engineering service cost

The cost of engineering services for the detailed design including the preparation of tender documents was estimated at 7 % referring the past results in similar type and scale of projects.

(3) Administration cost

The administration expense was estimated on the lump sum basis, applying rate of 15 % to the total direct cost. This percentage was determined in the discussions among CELESC, ELETROBRAS and the Study Team.

10.3.6 Interest during Construction

Interest rate during the construction period is 10 %, which is a specified value used by ELETROBRAS.

10.3.7 Project Cost and Annual Disbursement Schedule

Based on the conditions stated, the project cost was estimated in US$ as shown in Table 10.4 and a summary is as follows;

(unit: thousand US$)

Account No.		Amount
10-16.	Direct cost	
10.	Land and Facilities	407
11.	Structures & Other Improvement	11,069
12.	Reservoir, Dam & Waterways	83,702
13.	Turbines & Generators	23,406
14.	Accessory Electrical Equipment	10,535
15.	Other Equipment of Powerhouse	6,148
16.	Access Road/Railway & Bridges	2,110
	Sub-Total	137,377
17.	Indirect cost	
17.21	Construction site & Camping	9,616
17.22.40.36.	Basic Engineering	9,616
17.22.41.	Administration of Properties	20,607
	Sub-Total	39,839
	Total without Interest	177,216
18.	Interest during construction	38,239
Total		215,455

The project cost estimated was further divided into foreign currency portion and local currency portion for reference for loan request using the ratio of foreign and local currency of major materials and equipment as listed in Table 10.5. The result is as follows;

(unit: thousand US$)

Account No.		Foreign	Local	Total
10-16.	Direct cost			
10.	Land and Facilities	0	407	407
11.	Structures & Other Improvement	6,259	4,810	11,069
12.	Reservoir, Dam & Waterways	45,895	37,807	83,702
13.	Turbines & Generators	15,403	8,003	23,406
14.	Accessory Electrical Equipment	6,933	3,602	10,535
15.	Other Equipment of Powerhouse	4,046	2,102	6,148
16.	Access Road/Railway & Bridges	1,061	1,049	2,110
	Sub-Total	79,597	57,780	137,377
17.	Indirect cost			
17.21	Construction site & Camping	5,569	4,047	9,618
17.22.40.36.	Basic Engineering	1443	8,173	9,616
17.22.41.	Administration of Properties	0	20,607	20,607
	Sub-Total	7,012	32,827	39,839
	Total without Interest	86,609	90,607	177,216
18.	Interest during construction	18,680	19,559	38,239
	Total	105,250	110,205	215,455

The annual disbursement schedule was prepared based on the construction time schedule in Fig. 13.3. The annual disbursement estimated is as follows:

(unit: thousand US$)

	Foreign	Local	Total
1st Year	18,159	19,014	37,113
2nd Year	29,055	30,423	59,478
3rd Year	31,960	33,464	65,424
4th Year	26,076	27,304	53,380
Total	105,250	110,205	215,455

10.4 Project Operation and Maintenance Costs

Annual operations and maintenance cost (O & M cost) was estimated using the empirical formula employed by ELETROBRAS as shown below;

O & M Cost = 124.28 x P^-0.61 US$/kW/Year (P<146.71 MW)

= 11.43 x P^-0.1281 US$/kW/Year (P>146.71 MW)

where, P : Installed capacity (MW)

For the installed capacity of 142 MW, the O & M cost was accordingly estimated at 6.1 US$/kW/year.

Chapter 11 ENVIRONMENTAL IMPACT ASSESSMENT

11.1 General

Comparative study on selection of dam axis was made for three alternative sites, namely dam axes B, C and D from the technical and financial aspects and it was determined to adopt the dam axis B. Based on this decision, the related damsite, headrace tunnel route and powerhouse site were determined.

Objectives of this environmental assessment impact study (EIA) is to identify and predict the impacts of the defined project features from viewpoints of natural and social environments and to prepare mitigative measures for the effect to the project.

EIA was made referring to the environmental impact study (EIS) performed by the local consultant, based on direct observation and reconnaissance and incorporating the related information such as photo interpretation, and map plotting performed by the environmental expert of the JICA team.

11.2 Natural and Social Conditions

11.2.1 Physical Environment

(1) Climate

The Itajaí river basin is characterized by a subtropical rainy climate. The annual rainfall is between 1,300 and 1,500 mm in the basin's central area, and 1,600 to 1,800 mm in the mountain regions situated northwest and southwestwards. The average annual rainfall in the basin ranges between 1,500 and 1,600 mm. In the surroundings of the Salto Pilão Hydropower site, the average annual rainfall is 1,530 mm.

The annual mean temperature oscillates between 18-21 °C. The monthly maximum of 25 °C occurs in January - February, and the minimal of 15 °C in July.

The annual mean relative humidity reaches its peaks of 85.7% in Itajaí, and the lowest values of 77% in Indaial. The highest values of relative moisture occur during the months of June to August.

(2) Soils

The proposed reservoir area in general is classified as red-yellow latosolic podsol type soils. They are mineral soils with fading waxy characteristics with low natural fertility, with low contents of calcium and magnesium, moderate content of organic matter, and very low potassium content. These are insufficient for the normal development of majority of crop cultivation.

The river stretch downstream of the reservoir site extends towards northeast and the Itajaí do Norte river joins at about 8 km downstream of the damsite. Afterwards the Itajaí river changes its direction southeast wards. Both banks of the river stretch between the damsite and the Itajaí do Norte is very steep with presence of rocks and the soils are considered improper for any kind of cultivation.

(3) Topography

The Itajaí river valley is rather carved and have sharp declivity. The declivity increases to 1:60 between the village of Subida and the Salto Pilão, allowing rapids and waterfalls. Between Subida and Blumenau the mean declivity is 1:500, whereas downstream from this city is decreased to less than 1:10,000, allowing the possibility of alluvial deposits. The topography of the Itajaí river basin differs from that of the northern part of Santa Catarina. Occurrence of several sparse coastal ranges of mountains is a typical feature. Topography of the damsite is formed by a series of low hilly zones with a height of 50 to 100m .

In the upstream area of the Salto Pilão up to Rio do Sul city, there exists a wide valley of low declivity and the town of Lontras lies on the right bank downstream of the Rio do Sul city.

(4) Agricultural soil value

The reservoir area is almost entirely composed of deep soil layer. Most of this soil lies on hilly topography and restricts to annual crops. In the powerhouse area, the rocks present and 56% of the soils which are impropoer for annual crops have regular aptitude for fruit growing and good aptitude for pasture and reforestation. There is no farming activity steep along the river stretch between the damsite and Itajaí do Norte confluence because of the steep slopes of the river banks.

(5) Water quality

The analysis of the water quality test so far carried out concludes that:

- The Biological Oxygen Demand (BOD) was always lower than 3.0 ppm. It belongs to class 1 according to CONAMA standard.

- Levels of Dissolved Oxygen (DO) sampled were always above the minimal value required for the maintenance of aquatic life, characterizing good water quality.
- The values of ammonia nitrogen and total phosphates appear at a higher concentration than those established by CONAMA.
- Fecal and total coliforms in high contents reveal the existence of sanitary sewage from towns.

With a low BOD and a normal DO concentration, the levels of ammonia nitrogen degrade to non toxic nitrates, especially when the conditions of the river offer rapid flows with high oxigenation in several stretches of the river. It forms slightly high phosphate concentration. The phosphorus concentration is probably related to the use of inorganic fertilizers in agricultural crops and the associated agricultural runoff from this areas.

(6) Pollutant sources

The total organic load estimated for the Itajaí river basin was 880 tons BOD/day. This load is equivalent to a population of 16,000,000 inhabitants, which is 23 times the urban population of the basin. This may be caused by industrial activity especially textile, starch, metal mechanic and frigorific among others and rural activity especially swine activity and use of inorganic fertilizers.

(7) Scenic resources

In the upstream reaches of the town of Apiúna, outcropped rock walls and steep hillsides covered with secondary growth give room to the excavated canyon of the Itajaí river, where small tributaries flow into the Itajai river through small waterfalls.

The highway upstream of Subida lies along the left bank of the river for about 8 km, and climbing about 100m it crosses the steep hillsides, and many of them exploited for the extraction of masonry stone. In a following road stretch of about 6 km, the road passes through a stretch of easy topography and round shape hills, which are rich in granite areas, with crops and grasslands, with a background of mountain ranges of altitudes of about 800 m.

The landscape and topographic characters of the banks and proximity of the river offer steep gradients. In many cases a sort of canyon has some small tributaries which flow into the main river by way of small waterfalls. Along the steep river banks igneous rocks are exposed with plenty granite boulders popping out throughout the hillside, many of theirs exploited or under exploitation for masonry stone. In these areas clearing by fire is common to expose the rock and find the quarries.

(8) Landscape aspects

In the future reservoir area, the morphology is relatively gentle, consisting of round like hills with convex and no very steep hillsides.

In the powerhouse site difference of altitude between the riverside and hillside is more than 300 m. The hillside is steep in the upper part. This stretch probably corresponds to post sliding accumulated at the foot of the hillside.

The proposed place for the damsite does not compose a typical scenery when compared with the traditional landscape of the Itajaí valley. There are several quarries over the hillside which is located in parallel with the river, and have seriously degraded the slopes. The degradation of the landscape is caused mainly by deforestation and exploitation of granite.

(9) Sediments

The specific sediment transportation has been estimated at 91.4 ton/km^2 per year, which is considered low.

(10) Water use

Most of the households in the upstream from the damsite depend on water from water wells and water treatment is considered substandard.

In the directly affected area composed of areas belonging to the rural zone of the municipalities of Ibirama, Lontras and Apiúna, the water use is incipient, and there is some uses of water for irrigation in the upstream stretch of the damsite.

11.2.2 Biological Environment

(1) Flora

There is no aquatic flora of economic or scientific interest in the area between the damsite and the Itajaí do Norte confluence.

The exuberant vegetation in the Itajaí valley practically does not exist any more at present. The original primary forest gives place to the secondary vegetation.

There are no endangered or protected species recorded or found in the reservoir site, or the area between the damsite and the Itajaí do Norte river and no species of commercial or scientific interest has been reported.

(2) Fauna

River fish was more abundant in the past according to local inhabitants, but as soon as the manioc starch production started during second World War, the population of fishes has been decreased due to pollution such as cyanic acid and solid residues from the lumber industry.

The Itajaí river is a habitat for an important population of birds. Some of them are progressively less abundant, non the less, the region of Salto Pilão for its most part and following the river upstream of the municipality of Lontras is completely altered. This fact together with the low impact due to small scale of the project offers no significant threat to the remaining bird populations of the region.

Since the project is located in a degraded area and outside of the buffer and nucleus zones for the remaining vegetation of importance, the project does not entail threat to the remaining population of birds.

Although there is no statistical data on the forest fauna in the municipalities studied previously, the region is characterized by a considerable human occupation, preventing to a great extent the occurrence of forests. At the same time, fauna has been hunted historically, associated with the livestock rearing, and in an effort to prevent the risk of loosing stock. The destruction of the forest habitat due to lumber activities and the expansion of the agricultural activity have eliminated the natural habitat for their subsistence.

There is no evidence of mastofauna such as herbivorous animals and carnivores in this region according to the result of field surveys or information of local residents specially in the project area.

The majority of the Itajaí valley has its forest cleared, or is covered with altered forests, where the light penetration has increased. Many species of amphibians and reptiles have elements of their habitats affected, and therefore disappear, depleting the environment of its original biological diversity.

The remaining forest are discontinuous, and therefore do not allow the free traffic of organisms from one forest patch to another. With this fragmentation, the area is often insufficient to convey a population, and the species disappear locally.

The discontinuity of the forest habitat throughout the region has contributed to the isolation and eventual disappearance of several species of interest once found in the area.

The project area is located at outside the buffer zones or any other category reflecting the natural forest regeneration process at shown in Fig. 11.1. Due to this reason, there are no protected or endangered species which are found or have been reported in the area of interest. Since the project area is not covered with forest, it does not represent a sensitive or important habitat for species considered endangered or protected.

11.2.3 Socio - Cultural Environment

(1) Land use

There is no farming activity in the reservoir area and along the river stretch between the damsite and the Itajaí do Norte confluence because of the steep slopes of the river banks. In the region of the surge tank, the soil is covered with a secondary forest. No agricultural activities are detected in this area. At the powerhouse area, the soil is covered with a field like vegetation, and there no agricultural activities in spite of its potential.

Some cattle raising is done in the upper reaches of the slopes in the river stretch between the damsite and the Itajaí do Norte confluence. The activity does not dependent on the river water since the pronounced slopes present an obstacle for the animals to reach the water. There is no pasturage activity in the area defined for the powerhouse.

On the upstream margin of the reservoir (axis B), a resort complex (Paraíso camping) operated by a landowner is located on left bank. This area extends from the river side into hill along small stream. This complex is equipped with hotel, restaurant, swimming pool, play lots, and other recreational facilities. The complex is located at scenic site where a waterfall on the Itajaí mainstream is visible.

The use of the land for recreation in the left bank, 500 m downstream of the damsite (axis B) needs relocation of 4 recreation houses and 3 pig houses of independent and private owners who spend weekends and holidays at their estate. These estates will not directly affected by the dam.

In the reservoir area including 100 m wide protection zone around the reservoir, three dwellings are located; one (L-1) on left bank near the Paraíso camping and another two on right bank 550 m upstream (R-1) of the dam axis B, respectively, as illustrated in Fig. 11.2.

There is no population in the vicinity of the river banks along the river stretch between the damsite and the Itajaí do Norte confluence. Few isolated houses in the upper portion of the slopes do not use the river water for their subsistence, and the potable water is extracted from water wells. At the powerhouse site, as part of the village of Subida, 12 estates are located in the influence area of the powerhouse site and a buffer area left on the east and west sides of the project area.

The mineral resources found in the region such as clay for ceramics and kaolin for masonry stone, have low economic value. There are no mineral deposits in the project areas except for the hillsides of the powerhouse site where granite blocks have been exploited for production of masonry stone.

(2) Water use

Sport fishing is popular in the upstream river stretch of the proposed damsite, and it is also the support activity for the camp site located at about 800 m in its upstream. The fish species sampled and recorded are very plastic and located upstream as well downstream of the damsite. According to the information available, it seems that these species do not depend on a migration pattern to reproduce, feed, or complete a part of their life cycle. It is also believed that the creation of the reservoir will enhance the fish population in this area. The river stretch between the damsite and the Itajaí do Norte confluence is characterized by rapid currents and high velocity and murky river water throughout the year. The high velocity of the water current offers no chance for sport or commercial fishing, and the area is considered poor by local fishermen.

The directly affected area is composed of areas belonging to the rural zone of the municipalities of Ibirama, Lontras and Apiúna. The water use is incipient. There is some uses of water for irrigation in the upstream sector of the dm site, and along the surroundings of the left bank, although it is inconspicuous quantities. There are no uses of the water for irrigation in the sector between the damsite and the Itajaí do Norte confluence because agricultural activities are absent.

Most of the households in the upstream of the damsite use water from water wells, and water treatment is considered substandard. There is no water use for municipal or industrial activities in the area directly related or affected by the project development.

In the upstream of the damsite of the project area, sport fishing activity is regular. There is a camping site located at about 800 m upstream of the damsite, which is regularly visited by sport fishermen and their families. Since some areas of the river in this sector are suitable for swimming and water sports, this area is considered as valuable in terms of its recreation potential. It is believed that the formation of the reservoir downstream of the camping site will enhance the tourist attractions of the area.

The river stretch between the damsite and the Itajaí do Norte confluence does not offer swimming places due to the high velocity of the water, and access is difficult due to steep slopes. Sports fishing is considered poor due to the strong currents. There are no tourist facilities along the river stretch, except for the Ilha Cotia (Cotia island), which is a tourist area located at about 8 km downstream of the damsite. This island has two houses for weekend rental and it is related to water sport activities such as swimming and fishing.

(3) Population - human settlements

The directly affected areas of the municipalities of Lontras and Ibirama is composed of 10 estates. Four of them in the municipality of Lontras (right bank) will be affected by the construction of the desanding basin and the spoil bank and 6 in Ibirama (left bank) will be affected by the formation of the reservoir. In the case of Subida, the rural estates in and around the powerhouse area will be affected by the provision of the powerhouse, tailrace/substation, surge tank and other construction sites.

The most significant impacts would be distributed over a small part of the municipalities of Ibirama and Lontras, which will have a part of their land submerged due to the reservoir as well as the municipalities of Apiúna caused by displacements necessary to for construction of the powerhouse. These are considered as first degree area of influence. Blumenau and Rio do Sul are considered to be second degree area of influence due to their political influence and their available infrastructure in commerce and services. No direct effect of the project actions is foreseen in these communities.

The economy in Ibirama is based on subsistence farming, cattle raising and wood extraction. Lontras is located in the upper part of the Itajaí river valley. Lontras presents a strong tendency to farming and cattle raising. Apiúna is located in the Itajaí middle valley. Apiúna is experiencing a retraction of the agricultural activities, and a strong trend towards urbanization.

Ibirama, Lontras and Apiúna show a transition economy with farming and cattle raising activities in decline with the exodus of the rural population. The reformation of the industrial activities is being directed towards the textile activities; and there is also a significant underground economy based on the extraction of granite. There is a gradual trend to use the rural estates as leisure areas for people who live in the urban areas. Rio do Sul and Blumenau are considered to be second degree area of influence due to the proximity and availability of services and commerce that can be activated during the constructions