3.2.3 Spillway

(1) Type and function of spillway

In view of the topographic condition, a overflow type spillway with gates was planned to be provided in the dam body. This gates spillway will have two functions, namely, one is to secure the safety of dam against flood and the other is to secure storage volume required for regulation capacity for power generation.

(2) Spillway system

The spillway structure comprises overflow weir, chuteway and energy dissipate portion.

The flood peak discharge of 5,700 m^3/sec with 200-year probability was adopted for design of the overflow weir. Width of the overflow weir was determined considering the flow area of the river channel in the upstream and downstream of the damsite and also the size of gates (ratio of the height to the width). In this study, the width of the overflow weir was decided at 66 m assuming that 4 units of 16.5 m wide roller gates are installed. Total width of the overflow weir is 78 m.

The crest elevation was calculated by the following equation;

$$H = \left(\frac{Q}{B \cdot C}\right)^{2/3}$$

where;

B	;	Effective overflow width (m)
Q	;	Design flood (m^3/sec)
C	;	Coefficient of overflow discharge; 1.9
H	;	Overflow depth at crest (m)

The crest elevation of the overflow weir determined using this equation is EL 306.1 m. The crest elevation of hoist deck was decided at EL 338 m.

The width of chuteway is 75 m and its slope is 1:1. In order to release safely the outflowed flood discharge by changing into steady flow, a horizontal stilling basin of 55 m in length was planned.

General plan and profile of the designed spillway facility are shown in Figs. III.3.3 and IV.3.4.

(3) Flood operation rule

Since the objective of the scheme is only hydropower generation, the flood flow will be released in accordance with inflow = outflow rule in principle, or the gates will be operated to keep the full supply level for power generation.

3.2.4 Intake

Two tributaries join the Itajai river in the right bank of the damsite. If the intake structure is provided directly toward the powerhouse, a sufficient overburden necessary for excavation of the headrace tunnel cannot be ensured. Then, the intake structure was planned to be provided toward the reservoir area to align the route of the headrace tunnel in the mountain ridge with thick overburden.

In order to avoid the flowing of sediment into the headrace tunnel, sand trap basin was planned. Bottom elevation and dimension of the sand trap basin were determined based on the flow velocity at full supply level and minimum operation level and referring to the topographic condition at the basin site. Consequently two units of the basin with sill elevation of 309 m, and 48 m long and 36 m wide were planned. Besides, orifice type sediment scouring gate with 4 m high and 15 m wide was installed at the front of the intake structure. For sake of maintenance of the sand trap basin and also of the headrace tunnel, two roller gates were planned in front of the sand trap basins. General plan and profile of the intake structure are shown in Figs. III.3.3 and III.3.4.

3.2.5 Headrace tunnel

(1) Route of headrace tunnel

The proposed powerhouse is located at the right bank of the Itajai river near Subida. The waterway route connecting the damsite and the powerhouse was decided considering the following conditions;

(i) The distance connecting the damsite and powerhouse is the shortest.

(ii) To secure a sufficient overburden necessary for tunnel excavation and to avoid geological faults, route connecting the mountain ridges is selected.

To meet these conditions, the route south to southeastwards was aligned and at about 1.5 km from the intake site, it was bent eastwards. Location of the headrace tunnel thus decided is shown in Fig. III.3.5. Although many tributaries cross almost right angle against the tunnel route, geological soundness was already confirmed by the results of core boring and geo-surface inspection. Total length of the headrace tunnel is 6,305 m.

(2) Diameter of headrace tunnel

The diameter of the headrace tunnel was determined from two aspects, namely, economic comparison and allowable flow velocity in view of operation and maintenance of the tunnel.

The headrace tunnel was designed to be of pressure type with circular section. The economical comparison was studied by such a way that sum of the annual cost required for tunnel construction and annual power tariff equivalent to loss head is estimated for different diameters and the diameter with least value of the above sum is selected. The study was made based on the following conditions;

(i) Lining of the tunnel is 8 % of the inside diameter of the tunnel for ordinary case and the minimum lining is 30 cm. The lining for fault zone is increased by 50 % for that of the ordinary case. 20 cm of extra excavation is considered.

(ii) Consolidation grouting will be performed for fault zone and insufficient overburden portion.

(iii) Annual cost for tunnel construction is estimated based on the project life time of 50 years and discount rate of 10 %.

(iv) Loss head was assessed against firm discharge, 50.3 m^3/sec.

(v) Unit cost of guaranteed energy, US$18.8/MWh was used as the unit cost to estimate the power tariff.

The result of the economic comparison is illustrated in Fig. III.3.6. It shows that the case of 4.5 m in inside diameter is the least cost.

While, the allowable flow velocity in the headrace tunnel in view of operation and maintenance has been decided at 2.5 m/sec to 3.5 m/sec. Maximum plant discharge, 71.9 m^3/sec flows in the tunnel when sufficient river flow is available. The flow velocity in such case is 4.5 m/sec, while the flow velocity for the firm discharge is 3.2 m/sec. In order to reduce the flow velocity within the allowable one in case of the maximum plant discharge, inside diameter should be enlarged to 5.2 m. In such case, flow velocity is 3.4 m/sec. Thus, inside diameter of the headrace tunnel was decided at 5.2 m.

3.2.6 Surge tank

For design of the surge tank, four types, namely, simple type, restricted orifice type, differential type and chamber type are conceivable. The differential and chamber types are not so suitable because the structures are complicated and their cost would be high due to increased quantity of the concrete works. The simple type needs rather large size of the surge tank compared with that for the restricted orifice type, but it can surely absorb the water hammer and variation of water level due to variation of load is rather slow. Considering these advantages, the simple type surge tank was adopted in this study.

The dimensions of the surge tank were determined so as to satisfy dynamic stability conditions, namely, rising of water level for instantaneous full load rejection and lowering of water level for instantaneous load increase from half of full load were checked. As a result, the simple type surge tank with inside diameter of 20 m, 283.5 m in bottom elevation and 338.5 m in top elevation was planned. In view of the connection of the underground inclined pressure shaft type penstock, underground embedded type concrete structure as shown in Fig. III.3.7 was designed. The lining of the surge tank was decided at 1.0 m considering the geological condition that the surge tank structure is provided in rhyolite zone which is very hard rock.

3.2.7 Penstock line

The proposed penstock is of underground inclined pressure shaft type. Its route passes rhyolite zone which has hard and massive characteristics corresponding to B class of rock classification. Total length of the penstock line was estimated to be 505 m, consisting of 20 m in upper horizontal part, 233 m in inclined part and 252 m in lower horizontal part. Open air conduit type penstock line is conceivable as an alternative plan. However, about 10 m thick heavily weathered layer of decomposed layer soil overlies along the penstock line route, and this layer is apt to be easily collapsed. It is necessary to locate the foot of the penstock line on the firm rock in case of the open air conduit type penstock line. Thus it was judged that the open air steel conduit type plan is not economically suitable due to huge excavation work and slope protection work.

Two cases of the penstock line, single line and double line are conceivable. In this study, one line was adopted due to the economical reason. The average diameter of the penstock was determined by the economic comparison and allowable flow velocity as stated for the determination of the diameter for the headrace tunnel. The conditions applied for the economic comparison are as follows;

(i) Space between the inside diameter of the penstock tunnel and excavated surface of the tunnel is set at 40 cm, considering the working clearance for installation of the penstock pipes. 10 cm of extra excavation is considered.

(ii) Consolidation grouting with 3 m in interval is executed.

(iii) Annual cost for construction works of the penstock is calculated assuming the project life time of 50 years and discount rate of 10 %.

(iv) Loss head is calculated against firm discharge.

(v) Annual power tariff equivalent to loss head is calculated based on US$18.8MWh which is the cost of the guaranteed energy.

The result of the economic comparison shows that the average inside diameter of 3.8 m is the least value as shown in Fig. III.3.8. While, according to the regulation issued recently, the allowable flow velocity is restricted less than 7 m/sec. The flow velocity in case of 71.9 m^3/sec of the maximum plant discharge is 6.3 m/sec. Thus the average inside diameter of 3.8 m was adopted. It was planned that one lane steel lined circular tunnel is branched into two lanes at immediately upstream of the powerhouse. General plan and profile of the penstock line are shown in Fig. III.3.7.

3.2.8 Powerhouse and tailrace

The open air type powerhouse was planned to be provided at the right bank of the Itajai river at about 0.7 km upstream from Subida. Result of core boring shows that weathered rhyolite with fractured zone distributes up to above 12 m from ground surface.

The turbine center was set at an elevation of 107.2 m which is 3.3 m lower than the normal tailwater level. The lowest elevation below the draft tube was set at an elevation of 100 m. The ground formation height of the power station was set at an elevation 120 m which is 1.75 m higher than the water level against 100-year probable flood. The determined dimensions of the powerhouse are 33.2 m high, 34.0 m wide and 50.0 m long. About 40 m long open channel type tailrace to the Itajai river was planned. General plan and profile of the powerhouse are shown in Figs. III.3.7 and III.3.9.

3.2.9 Generating facilities

Two sets of hydro turbine generators and their auxiliary equipment will be installed in the powerhouse. Considering working head and rated output, the hydraulic turbine will be of vertical shaft Francis type and their particulars are as follows;

(1) Hydraulic conditions

- Reservoir water level		
Full supply level	;	319 m
Minimum operation level	;	317 m
Rated	;	319 m
- Tail water level	;	110.5 m
- Gross head		
Maximum	;	208.5 m
Minimum	;	206.5 m
- Rated head	;	191.9 m
- Maximum discharge	;	71.9 m^3/sec

(2) Hydraulic turbines

- Type	;	Vertical shaft Francis
- Rated head	;	191.9 m
- Number of unit	;	2
- Rated output	;	58.8 MW
- Speed	;	360 rpm

The generator will be vertical shaft alternator directly coupled with the hydraulic turbine with particulars as follows;

- Type ; Vertical shaft, semi-umbrella, synchronous generator
- Number of unit ; 2
- Rated output ; 56.8 MW
- Rated capacity ; 66.8 MVA
- Rated voltage ; 13.8 kV

Major auxiliary equipment of the power station would include the following:

- Two (2) sets of throughout flow (bi-plane) type inlet valve of 2.1 m in diameter,
- Two (2) sets of proportional-integral-derivate (PID) type electro-hydraulic governor,
- One (1) 130 ton overhead travelling crane with an 30-ton auxiliary hoist for handling miscellaneous equipment, and
- One (1) set of diesel engine generator for emergency power supply to the auxiliary equipment.

The outdoor switchyard of this power plant would be located near the power house and its particulars are as follows;

- Main transformer
  (1) Type : 3-phase, 2-winding, oil-immersed, forced-oil-circulation and forced-air-cooled and outdoor use type
  (2) Number of banks : 2
  (3) Rated Power : 66.8 MVA
  (4) Rated Voltage : 13.8/138 kV

- One (1) 138 kV single bus
- Two (2) 138 kV main transformer bays
- Two (2) 138 kV transmission line bays

3.2.10 Transmission line and substation

An electric power at the Salto Pilão (1) hydropower station would be supplied to the Rio do Sul and Blumenau areas through the existing CELESC's 138 kV power system.

For this purpose, the existing 138 kV transmission line between the Rio do Sul II Substation and the Blumenau Substation would be divided in the middle point and connected to the Salto Pilão (1) hydropower station by a 138 kV double-circuit line of about 7 km in length which would be constructed newly by this scheme.

No extension of substation equipment in the Rio do Sul II and the Blumenau substations would be required for the Salto Pilão (1) hydropower station.

3.3 Assessment of Power Output and Energy

Based on the determined dimensions of the project components, power output and energy were assessed based on the following conditions;

- Normal operation level ; 319 m
- Tailwater level ; 110.5 m
- Firm discharge (Qf) ; 50.3 m^3/sec
- Maximum plant discharge (Qp) ; 71.9 m^3/sec
- Long term average discharge excluding the parts of discharge exceeding maximum plant discharge (Qav) ; 55.1 m^3/sec
- Effective head (He)
  He = 200.4 m for Qf (50.3 m^3/sec)
  He = 191.9 m for Qp (71.9 m^3/sec)
  He = 198.8 m for Qav (55.1 m^3/sec)
- Overall efficiency of generating equipment ; 0.84

The installed capacity, firm energy, average energy, secondary energy and guaranteed energy assessed based on these conditions and determined dimensions of the project components are as follows;

- Installed capacity ; 113.6 MW
- Firm energy ; 726,901 MWh
- Average energy ; 789,910 MWh
- Secondary energy ; 63,009 MWh
- Guaranteed energy ; 654,211 MWh

4. STUDY ON DALBERGIA HYDROPOWER SCHEME

4.1 Site and Type of dam

Comparative study to select the most appropriate dam axis among three alternative dam axes was carried out in the same manner as explained in Section 3.1. Location of three alternative dam axes is shown in Fig. III.4.1.

Since the damsites for three dam axes forms V-shape with hard rock in its river bed and there is topographically no space to provide a spillway beside the damsite, the concrete gravity type dam with gated spillway was conceived for three damsites.

Cross sections of three dam axes are given in Fig. II.4.1 in Annex II. The dam axis-A is located at the upmost of the conceivable river stretch. The river width is about 310 m. The result of geo-surface inspection clarifies that gneiss outcrops in the river bed forming small rapid, and about 50 m wide river terrace deposit spreads in the left bank. Gneiss is relatively hard rock corresponding to CH class of rock classification. The excavation depth of foundation rock was estimated at about 2 m in the river bed, 15 m in the left bank and 10 m in the right bank. Consolidation and curtain groutings are needed to remedy vertical joints associated with open cracks developing in several places in the river bed.

The dam axis-B is located at about 800 m downstream from the dam axis-A. The river width of the dam axis-B is about 240 m. Along the dam axis, gneiss distributes almost all part of the river bed. Right river bank of the damsite is covered with talus deposit and left river bank is heavily weathered. The excavation depth of the rock foundation was estimated to be about 2 m in the river bed 15 m in the left bank and 10 m in the right bank side. Both the consolidation and curtain groutings are needed to remedy vertically and seriously dipped faults developing in left and right sides of the rive bed.

The dam axis-C is located at about 1000 m downstream from the dam axis-B. The river width is about 250 m. The result of core boring and geo-surface inspection shows that gneiss develops along the dam axis and outcrops in the right half part of the river bed. In both banks, gneiss is weathered and decomposed into soil. Open cracks develop in the river bed. The excavation depth of foundation rock was estimated to be about 2 m in the river bed, 8 m in the left bank and 5 m in the right bank. Considering crackly and high permeable conditions, both the consolidation and curtain grouting are needed.

The full supply level (F.S.L) of reservoir to keep the daily regulation capacity required for power generation was set for the respective dam axes as follows;

Dam axis	F.S.L (EL.m)
A	232
B	227
C	215

The same criteria for dam section as applied for Salto Pilão (1) scheme were adopted for concrete gravity type dam for respective dam axes.

Location of the headrace tunnel from intake site to the surge tank is shown in Fig. IV.4.2. Total length of the headrace tunnel is as follows;

Dam axis	Length of headrace tunnel (m)
A	9,800
B	9,000
C	9,000

The result of the core boring and geo-surface inspection shows that gneiss distributes in whole route of the headrace tunnel though shale is found and gneiss is weathered and loosened at the beginning part of the tunnel. The gneiss has very hard and tight characteristics corresponding to A to B class of rock classification.

The inside diameter of the headrace tunnel was set at 3.6 m which is the same dimension as applied for the master plan but tunnel lining was decided at 0.3 m considering the rock characteristics.

Location of the proposed surge tank site is shown in Fig. III.4.2. It was presumed that the surge tank is provided in gneiss layer which is hard and massive rock corresponding to B class of rock classification. In this study, a simple type surge tank was assumed and its inside diameter was assumed to be 4 times that of the headrace tunnel.

An underground inclined pressure shaft type was adopted for penstock line route. It was confirmed by the result of core boring that the penstock line passes gneiss layer which is hard and massive rock corresponding to B class of rock classification. The total length of the penstock line was estimated to be 350 m. The penstock line with one lane and diameter of 2.9 m was adopted in this study.

The result of core boring performed at the proposed powerhouse site showed that fresh gneiss with hard and massive properties distributes below 11 m from the ground surface. A open-air type powerhouse of 22.5 m wide and 41.5 m long was adopted in this study and its foundation was set at 11 m in depth. A Francis type power generating equipment was applied. About 2 km long and 23 kV transmission line to Ibirama substation was planned.

Based on the foregoing dimensions of the project components, work quantities for the cases of three dam axes were estimated. Using the same unit prices as applied for the master plan, the construction cost was estimated as shown in Tables III.4.1 to III.4.3.

The power energy to be generated for the cases of three dam axes was assessed assuming the following tail water level (T.W.L.);

Discharge	T.W.L. (EL. m)
Firm discharge, 19.3 m3/sec	133.5
Max. plant discharge, 27.3 m3/sec	133.6

The assessed firm energy, guaranteed energy and secondary energy are as follows;

(Unit: MWh)

Dam axis	Firm energy	Guaranteed energy	Secondary energy
A	124.2	111.8	13.4
B	118.2	106.4	12.9
C	101.8	91.6	10.6

Based on the estimated construction cost and power energy, unit cost of the guaranteed energy was estimated as shown in Table III.3.4. It shows that the unit cost of the guaranteed energy for the case of dam axis-B, US$64.2/MWh is the smallest value among three cases.

According to the result of the environmental impact study, number of household and acreage of land to be submerged by the scheme were estimated as follows;

Dam axis	Number of household	Acreage of land (km2)
A	17	0.19
B	17	0.25
C	20	0.16

It shows that the effect to the riparian area due to implementation of the scheme is almost the same for three cases. In consideration of the foregoing results, the dam axis-B was selected for further study.

4.2 Optimization Study and Pre-feasibility Design of Project Components

4.2.1 River diversion

Since the proposed damsite is located at an intermediate portion of the river channel bent sharply toward the left side, the diversion tunnel was planned to be located at the left bank connecting the v-shaped river channel directly. The planned length of the diversion tunnel is 155 m. In view of the concrete gravity dam type, design peak flood for river diversion was decided to be 770 m^3/sec with 2-year probability.

In order to lower the height of upstream cofferdam, a free flow type diversion tunnel was adopted in this study. The result of hydraulic calculation showed that diameter of the diversion tunnel is 6.8 m and maximum level to discharge the design peak flood of 770 m^3/sec is EL 222.2 m. A concrete gravity type cofferdam was planned for the upstream cofferdam. Crest elevation of the upstream cofferdam was decided at EL 223 m considering freeboard of 0.8m. Maximum height of the upstream cofferdam is 13 m. General plan of the river diversion is shown in Fig. III.4.3.

4.2.2 Dam

The proposed damsite is situated at an intermediate portion of v-shaped river channel. The both river banks form hills with a relative height of 50 to 60 m. The river width is about 240 m. Gneiss distributes almost all parts of the river bed. The right bank of the damsite is covered with talus deposit and left river bank is heavily weathered. Considering these geological conditions, excavation depth of the foundation rock was estimated at about 2 m in the river bed, 15 m in the left bank and 10 m in the right bank sides.

In order to coincide the spillway structure with center line of the river channel, the dam axis was set by bending its axis in an obtuse angle. Full supply of the reservoir was set at EL 227 m to keep the daily regulation capacity of 239,000 m^3 for power generation. A concrete gravity type dam was adopted in this study due to the same reason as stated in paragraph 3.2.2. The dam section was determined based on stability analysis by the same manner as stated in paragraph 3.2.2 and the following dam section was adopted;

- Upstream slope ; Vertical
- Downstream slope ; 1:0.85
- Crest width ; 4.5 m
- Freeboard above full supply level ; 1.5 m

The planned dam is 392 m in crest length, EL 228.5 m in crest elevation, and 22.5 m in maximum height. General plan and profile of the dam are shown in Figs. 4.3 and III.4.4.

4.2.3 Spillway

In view of the topographic condition, an overflow type spillway with gates was planned to be provided in the dam body. This gate spillway will have two functions as stated in paragraph 3.2.3.

The flood peak discharge of 4,100 m^3/sec with 200-year probability was adopted for design of the overflow weir. Width of the overflow weir was determined considering the flow area of the river channel in the upstream and downstream of the damsite and also the size of gates (ratio of the height to the width). The width of the overflow weir was decided at 87.5 m assuming that 7 units of 12.5 m wide roller gates are installed. Total width of the overflow weir is 111.5 m. The crest elevation was calculated by the equation as mentioned in paragraph 3.2.3. The determined crest elevation of the overflow weir is EL 218.5 m. The crest elevation of hoist deck was decided at EL 241.5 m. In order to release safely the outflowed flood discharge by changing to a steady flow, a horizontal stilling basin of 50 m in length was planned.

General plan and profile of the designed spillway facility are shown in Figs. III.4.3 and III.4.4.

4.2.4 Intake

The intake structure was planned to be provided at almost right angle against the river channel in the right river bank.

In order to avoid the flowing of sediment into the headrace tunnel, sand trap basin was planned. Bottom elevation and dimension of the sand trap basin were determined based on the flow velocity at full supply level and minimum operation level and referring to the topographic condition at the basin site. Consequently two units of the basin with sill elevation of 220.5 m, and 26 m long and 25 m wide were planned. Besides, orifice type sediment scouring gate of

11.5 m high and 10 m wide was installed at the front of intake structure. For sake of maintenance of the sand trap basin and the headrace tunnel, two roller gates were planned in front of the sand trap basins. General plan and profile of the intake structure are shown in Figs. III.4.3 and III.4.4.

4.2.5 Headrace tunnel

(1) Route of headrace tunnel

The proposed powerhouse is located at the right bank of the Itajai do Norte river in the downstream from Ibirama. The waterway route connecting the damsite and the powerhouse was decided considering the conditions as stated in paragraph 3.2.5.

To meet these conditions, the route toward almost right angle against the river channel was aligned and at about 1.5 km from the intake site, it was bent southeastwards and at about 3 km from that bending point, it was further bent eastwards. Location of the headrace tunnel thus decided is shown in Fig. III.4.5. Total length of the headrace tunnel is 8,720 m.

(2) Diameter of headrace tunnel

The diameter of the headrace tunnel was determined from two aspects, namely, economic comparison and allowable flow velocity in view of operation and maintenance of the tunnel.

The headrace tunnel was designed to be of pressure type with circular section. The economical comparison was made in the same manner as stated in paragraph 3.2.5. The study was made based on the following conditions;

(i) Lining of the tunnel is 8 % of the inside diameter of the tunnel for ordinary case and the minimum lining is 30 cm. The lining for fault zone is increased by 50 % for that of the ordinary case. 20 cm of extra excavation is considered.

(ii) Consolidation grouting will be performed for about 410 m long insufficient overburden portion.

(iii) Annual cost for tunnel construction is estimated based on the project life time of 50 years and discount rate of 10 %.

(iv) Loss head was assessed against firm discharge, 19.3 m^3/sec.

(v) Unit cost of guaranteed energy, US$64.2/MWh was used as the unit cost to estimate the power tariff.

The result of the economic comparison is illustrated in Fig. III.3.6. It shows that the case of 3.6 m in inside diameter is the least cost.

The allowable flow velocity in the headrace tunnel in view of operation and maintenance has been decided at 2.5 m/sec to 3.5 m/sec. The flow velocity for the case of the maximum plant discharge, 27.6 m^3/sec was estimated at 2.7 m/s. Thus, the inside diameter of the headrace tunnel was decided at 3.6 m.

4.2.6 Surge tank

A simple type surge tank was applied in this study due to the same reason as stated in paragraph 3.2.6.

The dimension of the surge tank was determined so as to satisfy dynamic stability conditions by rising of water level for instantaneous full load rejection and lowering of water level for instantaneous load increase from half to full load. As a result, surge tank with inside diameter of 14 m, 182.6 m in bottom elevation and 243.5 m in top elevation was planned. In view of the connection of the underground inclined pressure shaft type penstock, underground embedded type concrete structure as shown in Fig. III.4.6 was designed. The lining of the surge tank was decided at 1.0 m considering the geological condition that the surge tank structure is provided in gneiss zone which is hard and massive rock.

4.2.7 Penstock line

The proposed penstock is of underground inclined pressure shaft type. Its route passes gneiss layer which is hard and massive rock. Total length of the penstock was estimated to be 524 m consisting of 23 m in upper horizontal part, 71 m in inclined part and 430 m in lower horizontal part. Open air conduit type penstock line is conceivable as an alternative plan. But about 10 m thick weathered and decomposed loose soil overlies along the penstock line route. Since this layer has to be removed in case of open air conduit type penstock line due to its soil characteristics with sliding, it was judged that this alternative plan is not appropriate due to huge excavation works and slope protection works.

In this study, single lane penstock line was adopted. The average diameter of the penstock was determined by the economic comparison and allowable flow velocity as stated for the determination of the diameter for the headrace tunnel. The conditions applied to the economic comparison are the same as stated in paragraph 3.2.7 except for annual power tariff equivalent to loss head. The annual power tariff was estimated based on US$64.2/MWh which is the unit cost of the guaranteed energy.

The result of the economic comparison in Fig. III.3.8 shows that the average inside diameter of 2.9 m is the least cost. While, allowable flow velocity is limited within 7 m/sec. The flow velocity for the maximum plant discharge, 27.6 m^3/sec is 4.1 m/sec. Thus, the average inside diameter of 2.9 m was adopted. It was planned that one lane steel lined circular tunnel is branched into two lanes at immediately upstream of the powerhouse. General plan and profile of the penstock line are shown in Fig. III.4.6.

4.2.8 Powerhouse and tailrace

The open air type powerhouse was planned to be provided at the right bank of the Itajai do Norte river at about 2 km downstream of Ibirama. Result of the geological investigation shows that fresh gneiss with hard and massive properties distributes below 11 m from the ground surface.

The turbine center was set at an elevation of 129.9 m which is 3.6 m lower than the normal tailwater level. The lowest elevation below the draft tube was set at an elevation of 125.4 m. The ground formation height of the power station was set at an elevation 145.0 m which is 2.1 m higher than the water level against 100-year probable flood. The determined dimensions of the powerhouse are 30.4 m high, 23.6 m wide and 35.0 m long. About 25 m long open channel type tailrace to the Itajai do Norte river was planned. General plan and profile of the powerhouse are shown in Figs. III.4.6 and III.4.7.

4.2.9 Generating facilities

Two sets of hydro turbine generators and their auxiliary equipment will be installed in the powerhouse. Considering working head and rated output, the hydraulic turbine will be of vertical shaft Francis type and their particulars are as follows;

(1) Hydraulic conditions

- Reservoir water level		
Full supply level	;	227 m
Minimum operation level	;	226.2 m
Rated	;	227 m
- Tail water level	;	133.5 m
- Gross head		
Maximum	;	93.5 m
Minimum	;	92.7 m
- Rated head	;	74.1 m
- Maximum discharge	;	27.6 m^3/sec

(2) Hydraulic turbines

- Type	;	Vertical shaft Francis
- Rated head	;	74.1 m
- Number of unit	;	2
- Rated output	;	8.7 MW
- Speed	;	514 rpm

The generator will be vertical shaft alternator directly coupled with the hydraulic turbine with particulars as follows;

- Type	;	Vertical shaft, suspension type, synchronous generator
- Number of unit	;	2
- Rated output	;	8.4 MW
- Rated capacity	;	9.9 MVA
- Rated voltage	;	6.6 kV

Major auxiliary equipment of the power station would include the following:

- Two (2) sets of butterfly type inlet valve of 1.4 m in diameter,
- Two (2) sets of proportional-integral-derivate (PID) type electro-hydraulic governor,
- One (1) 30 ton overhead travelling crane with an 5-ton auxiliary hoist for handling miscellaneous equipment, and

- One (1) set of diesel engine generator for emergency power supply to the auxiliary equipment.

The outdoor switchyard of this power plant would be located near the powerhouse and its particulars are as follows;

- Main transformer

(1)	Type	:	3-phase, 2-winding, oil-immersed natural cooled and outdoor use type
(2)	Number of banks	:	2
(3)	Rated Power	:	9.9 MVA
(4)	Rated Voltage	:	6.6/23 kV

- One (1) 23 kV single bus
- Two (2) 23 kV main transformer bays
- Two (2) 23 kV transmission line bays

4.2.10 Transmission line and substation

An electric power at the Dalbergia hydropower station would be supplied to the Ibirama, Rio do Sul and Indaial areas through the existing CELESC's 23 kV power system.

For this purpose, a 23 kV double-circuit transmission line of about 2 km in length would be constructed newly between the Dalbergia hydropower station and the existing Ibirama 23 kV substation.

To connect the 23 kV Dalbergia - Ibirama transmission line, extension of two 23 kV transmission line bays with necessary control and auxiliary facilities would be required at the existing Ibirama substation.

4.3 Assessment of Power Output and Energy

Based on the determined dimensions of the project components, power output and energy were assessed based on the following conditions;

- Normal operation level	;	227 m
- Tailwater level	;	133.5 m
- Firm discharge (Qf)	;	19.3 m^3/sec

- Maximum plant discharge (Qp) ; 27.6 m^3/sec
- Long term average discharge excluding the parts of discharge exceeding maximum plant discharge (Qav) ; 22.1 m^3/sec
- Effective head (He)
  He = 84.1 m for Qf (19.3 m^3/sec)
  He = 74.1 m for Qp (27.6 m^3/sec)
  He = 81.1 m for Qav (22.1 m^3/sec)
- Overall efficiency of generating equipment ; 0.84

The installed capacity, firm energy, average energy, secondary energy and guaranteed energy assessed based on these conditions and determined dimensions of the project components are as follows;

- Installed capacity ; 16.8 MW
- Firm energy ; 117,048 MWh
- Average energy ; 129,248 MWh
- Secondary energy ; 12,200 MWh
- Guaranteed energy ; 105,343 MWh

5. STUDY ON BENEDITO NOVO HYDROPOWER SCHEME

5.1 Site and Type of Dam

Comparison study to select the most appropriate dam axis among three alternative dam axes was performed in the same manner as explained in Section 3.1. Location of three alternative dam axes is shown in Fig. III.5.1.

In this study, a concrete gravity type dam was adopted due to the same reasons as stated in Section 3.1.

Cross sections of three dam axes are shown in Fig. III.5.1 in Annex II. The dam axis-A is located at the upmost of the conceivable river stretch. The river width is about 130 m. The foundation rock consists of gneiss which is hard and corresponds to CH to B class of rock classification. It is predicted that vertically and highly dipped joints develop in the river bed. The excavation depth of the foundation rock was estimated to be about 5 m in the river bed and 10 m in both banks. Considering existence of joints and permeable condition of the rock, both the consolidation and curtain groutings are needed.

The dam axis-B is situated at about 250 m downstream of the dam axis-A, at about intermediate portion of about 25 m high rapid. The river width is about 170 m. Outcrop of gneiss appears from place to place forming small step like falls. Gneiss is heavily weathered and decomposed into soil in the left bank. In the right bank, about 100 m wide terrace deposit and talus deposit distribute. The foundation rock, gneiss is fairly hard and corresponds to CH to B class of rock classification. The excavation depth was estimated to be about 2 m in the river bed, 10 m in the left bank and 5 to 10 m in the right bank. Both the consolidation and curtain groutings are required considering the existence of joints crossing the dam axis.

The dam axis-C is located at about 250 m downstream from the dam axis-B, at just downstream of about 25 m high rapid. The river width is about 130 m. The foundation rock is composed of gneiss associated partly with granite, which is hard and corresponds to CH to B class of rock classification. The left bank is covered with weathered gneiss. In the right bank, talus deposit covers the ground surface. The excavated depth of foundation rock was estimated to be about 2 m in the river bed, 5 m in the left bank and 3 to 5 m in the right bank. Considering the existence of joints crossing the dam axis, both the consolidation and curtain groutings are required.

The full supply level (F.S.L.) of reservoir to keep the daily regulation capacity required for power generation was set for the respective dam axes as follows;

Dam axis	F.S.L (EL.m)
A	290
B	287
C	277

The same criteria for dam section as applied for Salto Pilão (1) scheme were adopted for the concrete gravity dam.

Location of the headrace tunnel from intake site to surge tank site is shown in Fig. III.5.2. Total length of the headrace tunnel is as follows;

Dam axis	Length of headrace tunnel (m)
A	2,800
B	2,560
C	2,000

It was confirmed by the result of core boring that majority of the headrace tunnel route passes hard and massive gneiss which corresponds to CH to B class of rock classification. Talus deposit covers the ground surface of the intake portions for three alternative dam axes. The inside diameter of the headrace tunnel was set at 2.8 m which is the same figure as applied for the master plan but concrete lining was decided at 0.3 m considering the rock property.

Location of the surge tank site is shown in Fig. III.5.2. It was presumed that the surge tank is provided in gneiss layer which is hard and massive rock corresponding to B class of rock classification. As for type of surge tank, simple type was assumed and its inside diameter was assumed to be 4 times that of the headrace tunnel.

An underground inclined pressure shaft type was adopted for penstock line considering geological condition of the penstock line route. It was presumed that the penstock line route passes gneiss layer. The total length of the penstock line was estimated to be 390 m. The penstock line with one lane and its diameter of 2.2 m was adopted.

The proposed powerhouse site is located at a flat space of river deposit with thickness of 5 m. The result of core boring showed that hard gneiss layer distributes at 5 to 14 m deep from the ground surface. An open air type powerhouse of 21 m wide and 38.5 m long was adopted and its foundation was set at 9 m in depth. A Francis type power generating equipment was adopted. About 17 km long and 69 kV transmission line to Timbo substation was planned.

Based on the dimensions of the project components, work quantities for the case of three dam axes were estimated. Using the same unit prices as applied for the master plan, the construction cost was estimated as shown in Tables III.5.1 to III.5.3.

The power energy to be generated for the cases of three dam axes was assessed assuming the following tail water level (T.W.L);

Discharge	T.W.L (EL.m)
Firm discharge, 8.4 m3/sec	154.2
Max. plant discharge, 13.9 m3/sec	154.3

The assessed firm energy, guaranteed energy and secondary energy are as follows;

(Unit; MWh)

Dam axis	Firm energy	Guaranteed energy	Secondary energy
A	80.6	72.6	12.8
B	78.9	71.0	12.5
C	73.1	65.7	11.7

Based on the estimated construction cost and power energy, unit cost of the guaranteed energy was estimated as shown in Table IV.3.4. It shows that the unit cost of the guaranteed energy for the case of dam axis-A, US$43/MWh is the smallest value among three cases.

According to the result of environmental impact study, number of household and acreage of land to be submerged by the scheme were estimated as follows;

Dam axis	Number of household	Acreage of land (km2)
A	112	0.31
B	28	0.17
C	23	0.03

It shows that large effect exerts to the riparian area if the dam axis-A is adopted and the effect on the dam axis-C is the minimum.

The second smallest unit cost of the guaranteed energy is US$43/MWh for the case of dam axis-A. The unit cost of the guaranteed energy for dam axis-C is US$45.9/MWh. Difference of unit cost of guaranteed energy between the cases of dam axes-A and -C is US$2.9/MWh, and ratio of decrease in the guaranteed energy in case of dam axis-C against that for the dam axis-A is about 9 %. In consideration of both the economic comparison and environmental aspects, the dam axis-C was selected for further study.

5.2 Optimization Study and Pre-feasibility Design of Project Components

5.2.1 River diversion

Since the proposed damsite is located at intermediate portion of the river channel bent sharply toward the left side, the diversion tunnel was planned to be located at the left bank connecting the V-shaped river channel directly. The planned length of the diversion tunnel is 155 m. In view of the concrete gravity dam type, design peak flood for river diversion was decided to be 280 m^3/sec with 2-year probability.

In order to lower the height of the upstream cofferdam, a free flow type diversion tunnel was adopted in this study. The result of hydraulic calculation showed that diameter of the diversion tunnel is 4.5 m and maximum water level to discharge the design peak flood of 280 m^3/sec is EL 269.7 m. A concrete gravity type cofferdam was planned for the upstream cofferdam. Crest elevation of the upstream cofferdam was decided at EL 270.5 m considering freeboard of 0.8m. Maximum height of the upstream cofferdam is 10.5 m. General plan of the river diversion is shown in Fig. III.5.3.

5.2.2 Dam

The proposed damsite is located at V-shaped river stretch. Both river banks form a mountainous area with a relative height of 40 m in the left bank and 160 m in the right bank. The river width is about 130 m. Longitudinally, the damsite is situated at just downstream of about 25 m high rapid. The foundation rock is composed of gneiss associated partly with granite, which is hard and corresponds to CH to B class of rock classification. The left bank is covered with weathered gneiss. In the right bank, talus deposit covers the ground surface. Considering these geological conditions, the excavated depth of foundation rock was estimated to be about 2 m in the river bed, 5 m in the left bank and 3 to 5 m in the right bank.

Full supply level of the reservoir was set at EL 277 m to keep the daily regulation capacity of 158,000 m^3 for power generation. A concrete gravity type dam was adopted due to the same reasons as stated in paragraph 3.2.2. The dam section was determined based on the stability analysis by the same manner as stated in paragraph 3.2.2 and the following section was adopted;

- Upstream slope ; Vertical
- Downstream slope ; 1:0.9
- Crest width ; 4.5 m
- Freeboard above full supply level ; 1.5 m

The planned dam is 130 m in crest length, EL 278.5 m in crest elevation and 24.5 m in maximum height. General plan and profile of the dam are shown in Figs. III.5.3 and III.5.4.

5.2.3 Spillway

In view of the topographic condition, a overflow type spillway with gates was planned to be provided in the dam body. This gates spillway will have two functions as stated in paragraph 3.2.3.

The flood peak discharge of 1,500 m^3/sec with 200-year probability was adopted for design of the overflow weir. Width of the overflow weir was determined considering the flow area of the river channel in the upstream and downstream of the damsite and also the size of gates (ratio of the height to the width). The width of the overflow weir was decided at 34 m assuming that 2 units of 17 m wide roller gate are installed. Total width of the overflow weir is 43 m. The crest elevation was calculated by the equation as mentioned in paragraph 3.2.3.

The determined crest elevation of the overflow weir is EL 263.9 m. The crest elevation of hoist deck was decided at EL 296 m. In order to release safely the outflowed flood discharge by changing to a steady flow, a horizontal stilling basin of 50 m in length was planned.

General plan and profile of the designed spillway facility are shown in Figs.III.5.3 and III.5.4.

5.2.4 Intake

The intake structure was planned to be provided at right angle against the dam axis in the right river bank.

In order to avoid the flowing of sediment into the headrace tunnel, sand trap basin was planned. Bottom elevation and dimension of the sand trap basin were determined based on the flow velocity at full supply level and minimum operation level and referring to the topographic condition at the basin site. Consequently two units of the basin with sill elevation of 267 m and 40 m long and 24 m wide were planned. Besides, orifice type sediment scouring gate with 8 m high and 4 m wide will be installed at the front of the intake structure. For sake of maintenance of the sand trap basin and the headrace tunnel, two roller gates were planned in front of the sand trap basins. General plan and profile of the intake structure are shown in Figs. III.5.3 and III.5.4.

5.2.5 Headrace tunnel

(1) Route of headrace tunnel

The proposed powerhouse is located at the right bank of the Benedito river, at just upstream of Benedito Novo. The waterway route connecting the damsite and the powerhouse was decided considering the conditions as stated in paragraph 3.2.5.

To meet these conditions, the route toward almost right angle against the river channel and bending southeastwards was aligned and at about 600 m from the intake site, it was bent eastwards. Location of the headrace tunnel thus decided is shown in Fig. III.5.5. Total length of the headrace tunnel is 1,815 m.

(2) Diameter of headrace tunnel

The diameter of the headrace tunnel was determined from two aspects, namely, economic comparison and allowable flow velocity in view of operation and maintenance of the tunnel.

The headrace tunnel was designed to be of pressure type with circular section. The economical comparison was made by the same manner as stated in paragraph 3.2.5. The study was made based on the following conditions;

(i) Lining of the tunnel is 8 % of the inside diameter of the tunnel for ordinary case and the minimum lining is 30 cm. The lining for fault zone is increased by 50 % for that of the ordinary case. 20 cm of extra excavation is considered.

(ii) Consolidation grouting will be performed for about 230 m long insufficient overburden portion.

(iii) Annual cost for tunnel construction is estimated based on the project life time of 50 years and discount rate of 10 %.

(iv) Loss head was assessed against firm discharge, 8.4 m^3/sec.

(v) Unit cost of guaranteed energy, US$45.9/MWh was used as the unit cost to estimate the power tariff.

The result of the economic comparison is illustrated in Fig. III.3.6. It shows that the case of 2.8 m in inside diameter is the least cost.

The allowable flow velocity in the headrace tunnel in view of operation and maintenance has been decided at 2.5 m/sec to 3.5 m/sec. The flow velocity for the case of the maximum plant discharge, 13.9 m^3/sec was estimated at 2.3 m/s. Thus inside diameter of the headrace tunnel was decided at 2.8 m.

5.2.6 Surge tank

A simple type surge tank was applied in this study due to the same reason as stated in paragraph 3.2.6.

The dimension of the surge tank was determined so as to satisfy dynamic stability conditions by rising of water level for instantaneous full load rejection and lowering of water level for instantaneous load increase from half to full load. As a result, the simple type surge tank with inside diameter of 10 m, 255.1 m in bottom elevation and 286.4 m in top elevation was planned. In view of the connection of the underground inclined pressure shaft type penstock, underground embedded type concrete structure as shown in Fig. III.5.6 was designed. The lining of the surge tank was decided at 1.0 m considering the geological condition that the surge tank structure is provided in gneiss zone which is hard and massive rock.

5.2.7 Penstock line

The proposed penstock is of underground inclined pressure shaft type. Its route passes gneiss layer which is hard and massive rock. Total length of the penstock was estimated to be

455 m consisting of 10 m in upper horizontal part, 135 m in inclined part and 310 m in lower horizontal part. Open air steel conduit type penstock line is conceivable as an alternative plan. However, the ground surface along the penstock line is covered with about 10 m thick weathered layer and talus deposit consisting of boulder and they have a tendency of sliding. It was therefore judged that open air steel conduit type penstock line is unsuitable due to huge excavation works and slope protection works.

In this study, single lane penstock line was adapted. The average diameter of the penstock was determined by the economic comparison and allowable flow velocity as stated for the determination of the diameter for the headrace tunnel. The conditions applied to the economic comparison are the same as stated in paragraph 3.2.7 except for annual power tariff equivalent to loss head. It was estimated based on US$45.9/MWh which is the unit cost of the guaranteed energy.

The result of the economic comparison in Fig. III.3.8 shows that the average inside diameter of 2.2 m is the least value. While, allowable flow velocity is limited within 7 m/sec. The flow velocity for the maximum plant discharge, 13.9 m^3/sec is 3.7 m/sec. Thus, the average inside diameter of 2.2 m was adopted. It was planned that one lane steel lined circular tunnel is branched into two lanes at immediately upstream of the powerhouse. General plan and profile of the penstock line are shown in Fig. III.5.6.

5.2.8 Powerhouse and tailrace

The open air type powerhouse was planned to be provided at the right bank of the Benedito river, at just upstream from Benedito Novo. Result of geological investigation shows that the hard gneiss layer distributes at 5 to 14 m from the ground surface.

The turbine center was set at an elevation of 152.1 m which is 2.1 m lower than the normal tailwater level. The lowest elevation below the draft tube was set at an elevation of 147.9 m. The ground formation height of the power station was set at an elevation 159.0 m which is 1.85 m higher than the water level against 100-year probable flood. The determined dimensions of the powerhouse are 21.6 m high, 21.1 m wide and 30.8 m long. About 45 m long open channel type tailrace to the Benedito river was planned. General plan and profile of the powerhouse are shown in Figs. III.5.6 and III.5.7.

5.2.9 Generating facilities

Two sets of hydro turbine generators and their auxiliary equipment will be installed in the powerhouse. Considering working head and rated output, the hydraulic turbine will be of vertical shaft Francis type and their particulars are as follows;

(1) Hydraulic conditions

- Reservoir water level		
Full supply level	;	277 m
Minimum operation level	;	270 m
Rated	;	277 m
- Tail water level	;	154.2 m
- Gross head		
Maximum	;	122.8 m
Minimum	;	115.8 m
- Rated head	;	115 m
- Maximum discharge	;	13.9 m^3/sec

(2) Hydraulic turbines

- Type	;	Vertical shaft Francis
- Rated head	;	115 m
- Number of unit	;	2
- Rated output	;	6.8 MW
- Speed	;	720 rpm

The generator will be of vertical shaft alternator to be directly coupled with the hydraulic turbine with particulars as follows;

- Type	;	Vertical shaft, suspension type, synchronous generator
- Number of unit	;	2
- Rated output	;	6.6 MW
- Rated capacity	;	7.8 MVA
- Rated voltage	;	6.6 kV

Major auxiliary equipment of the power station would include the following:

- Two (2) sets of butterfly type inlet valve of 1.0 m in diameter,
- Two (2) sets of proportional-integral-derivate (PID) type electro-hydraulic governor,
- One (1) 20 ton overhead travelling crane with an 5-ton auxiliary hoist for handling miscellaneous equipment, and
- One (1) set of diesel engine generator for emergency power supply to the auxiliary equipment.

The outdoor switchyard of this power plant would be located near the power house and its particulars are as follows;

- One (1) 69 kV single bus
- Two (2) 69 kV main transformer bays
- One (1) 69 kV transmission line bays

Two (2) 7.8 MVA main transformers step-up the voltage from the generator rated voltage of 6.6 kV to the transmission line voltage of 69 kV. Particulars of main transformers are;

- Main transformer

(1)	Type	:	3-phase, 2-winding, oil-immersed, natural cooled and outdoor use type
(2)	Number of banks	:	2
(3)	Rated Power	:	7.8 MVA
(4)	Rated Voltage	:	6.6/69 kV

5.2.10 Transmission line and substation

An electric power at the Benedito Novo hydropower station would be supplied to the Timbó and Indaial areas through the existing CELESC's 69 kV system.

For this purpose, a 69 kV single-circuit transmission line of about 17 km in length would be constructed newly between the Benedito Novo hydropower station and the existing Timbó 69 kV substation.

To connect the 69 kV Benedito Novo and Timbó transmission line, extension of one 69 kV transmission line bay with necessary control and auxiliary facilities would be required at the existing Timbó substation.

5.3 Assessment of Power Output and Energy

Based on the determined dimensions of the project components, power output and energy were assessed based on the following conditions;

- Normal operation level ; 277 m
- Tailwater level ; 154.2 m
- Firm discharge (Qf) ; 8.4 m^3/sec
- Maximum plant discharge (Qp) ; 13.9 m^3/sec
- Long term average discharge excluding the parts of discharge exceeding maximum plant discharge (Qav) ; 9.8 m^3/sec
- Effective head (He)
  He = 120.0 m for Qf (8.4 m^3/sec)
  He = 115.0 m for Qp (13.9 m^3/sec)
  He = 119.0 m for Qav (9.8 m^3/sec)
- Overall efficiency of generating equipment ; 0.84

The installed capacity, firm energy, average energy, secondary energy and guaranteed energy assessed based on these conditions and determined dimensions of the project components are as follows;

- Installed capacity ; 13.2 MW
- Firm energy ; 72,689 MWh
- Average energy ; 84,097 MWh
- Secondary energy ; 11,408 MWh
- Guaranteed energy ; 65,420 MWh

6. CONSTRUCTION PLAN AND COST ESTIMATE

6.1 Construction Plan and Cost Estimate for Salto Pilão (1) Hydropower Scheme

6.1.1 Conditions for construction

(1) Site conditions

Site conditions affecting the execution of the construction works in the project area are as follows;

(i) Topography

The project area is located along the Itajai river in the middle part of the Itajai river basin. Vicinity of the damsite is composed of hills whose relative heights are ranging from 30 to 100 m. The low slope of these hills has been utilized for pasture and upland crops. River bed slope is generally steep forming a small scale rapid and velocity of river flow is rather fast. No water fall features the project area. Hard rock outcrops in whole river bed.

(ii) Meteoro-hydrology

The project area belongs to subtropical zone, and climatically there is no distinct dry and wet seasons throughout a year. The annual mean rainfall in the area is recorded at around 1,500 mm at Rio do Sul. The annual mean temperature is 20.1 °C.

(iii) Geology

Geomorphological feature of the project areas is a series of outcrop of rocks in the river bed. Granite mainly distributes in the areas from damsite to headrace tunnel route and rhyolite spread in the sites of the proposed surge tank, penstock and powerhouse. In the surrounding area of damsite, recent river deposit and/or terrace deposit distribute.

(iv) Access to the site

There are existing two national highway in the Itajai river basin. One is a national highway, BR-101, which is a trunk route for inland transportation connecting the northern and southern regions of Brazil and crossing the Itajai river at about 18 km upstream from its river mouth. The other is a BR-470 national highway with effective width of 7.2 m, which originates from Itajai city and is located along the Itajai river. It is possible to access to Salto Pilão (1) project site through BR-470 highway and state road which branches from BR-470 highway near Lontras town.

(v) Construction materials

Major construction materials required for the construction works will be cement, steel bar, formed steel, rock and aggregates, wooden material, concrete piles, explosives, fuel and lubricants. A sufficient quantity and quality of these materials are obtainable from the domestic market in Brazil. Rock material for concrete aggregates and other use will be obtained from the proposed quarry site which is situated at about one km upstream of the damsite. The rock material is sufficient in quantity and quality according to the results of the geotechnical investigation. Of the construction materials mentioned above, steel materials and explosive will be procured outside the state.

(vi) Construction plant and equipment

The equipment for the construction works will be mainly of earth moving and concreting equipment with standard or average capacity. Most of these equipment are available in the domestic market in Brazil. Some of the equipment manufacturers have branch stores of dealers in the state.

(vii) Labor source

Sufficient number of common laborers will be recruited in the vicinity of the project area. Skilled and semi-skilled laborers would be employed from Florianópolis, Itajai and Blumenau.

(2) Mode of construction

Considering the extent of scale of the construction works, it is herein assumed to execute the project works by contractor or supplier who will be selected through international competitive bidding.

6.1.2 Construction time schedule

Fig.III.6.1 shows a proposed implementation schedule of the project. The construction works of the project are scheduled to be executed during 4 years including 6 months for the preparatory works. Fig.III.6.2 shows the proposed construction time schedule including the preparatory works.

6.1.3 Construction plan

(1) Conditions for construction planning

For study on the construction plan and schedule, the following conditions and assumptions were applied in consideration of the topography, meteoro-hydrology, geology and site conditions as described in the preceding paragraphs;

(i) Based on the daily rainfall records at Rio do Sul gauge as well as annual number of Sunday and national holidays in the country, annual workable days for construction works were set at 250 days in which rainfall intensity is less than 10 mm per day. The daily working hour is set at 8 hours excluding for the tunnel work for which 2 shifts operation with 15 hours will be applied.

(ii) Conventional method and type of equipment are principally applied.

(2) Preparatory works

(i) Access and construction roads

Existing road network (national, state and rural roads) is available mostly for the access and construction roads. It will be required to construct the access and construction roads and to improve the existing road at the project facility sites. A total length of the new roads and the improvement of existing road were estimated at about 3.5 km and 1.5 km respectively.

(ii) Temporary building

Temporary building are planned to be constructed in the project site for the smooth execution of the construction works. Those are site offices, quarters, material warehouses, repair shop, laboratory and others.

(iii) Communication system

Communication facilities will be required for smooth operation of the construction site. TELESC's public telephone line would be extended to the site offices for the contractors.

(iv) Arrangement of spoil banks

The spoil banks to accommodate a huge amount of the excavated materials of the dam and headrace tunnel will be provided at the several places along the headrace tunnel route.

(3) River diversion works

(i) Diversion tunnel

A diversion tunnel to be constructed in the left bank of the damsites is 560 m long and horse shoe-shaped concrete lined tunnel of 9.8 m in inside diameter. The diversion tunnel works are planned to be executed in a period of 6 months. The estimated work quantities are 160,600 m^3 of excavation and 26,600 m^3 of concrete. The full face excavation method will be applied for the tunnel excavation using 6-boom drill jumbo, side dump tractor shovel and dump truck. H-shaped steel supports with 1.2 m in interval will be installed over the whole length of the tunnel. The concrete lining work will be executed using a sliding form, concrete pump and agitator truck.

(ii) Cofferdams

Primary upstream and downstream cofferdams will be constructed by earth embankment immediately after the diversion tunnel is completed. After river diversion, 17 m high upstream cofferdam by concrete gravity type will be constructed. The concrete volume was estimated at 32,300 m^3. The concrete work will be executed by chuteway method using concrete pump and agitator truck.

(iii) Closure of diversion tunnel

The diversion tunnel will be closed by a roller gate upon completion of all of the works. After closing the gate, the diversion tunnel will be plugged with concrete. Cooling of the plug concrete will be done by circulating natural river water through cooling pipes embedded in the plug concrete. A combination of agitator truck, concrete pump and grout pump will be used for the plugging works.

(4) Main works

(i) Dam and spillway

A concrete gravity type dam having 70,600 m^3 in total volume and 20.5 m in maximum height is constructed at the proposed damsite.

Excavation of dam foundation is commenced at both abutments before river diversion. Excavation of river bed portion will be carried out currently with the construction of the primary cofferdam. The required excavation volume is 110,000 m^3. A combination of 32 ton class bulldozer with ripper, 10 m^3/min class crawler drill, 2.2 m^3 class tractor shovel and 11 ton class dump truck are planned to be used. The excavated bed rock will be transported to the proposed spoil bank. Foundation treatment for the dam bed rock will be carried out by consolidation and curtain groutings. The estimated grout length is 6,400 m in total. Crawler drill, boring machine, grout pump and concrete mixer will be used for this work. Immediately after completion of foundation treatment and clearing of bed rock, concrete placing is executed by chuteway method mainly using concrete pump, vibrator and other concreting equipment. Concrete mixing plant having 90 m^3/h in capacity will be installed near the damsite. Concrete placing for the spillway structure will be executed succeeding to the dam concreting works. Installation work of the spillway gates will be carried out after all of the civil works for the dam are completed. The required period for dam construction works is 41 months.

(ii) Intake and headrace tunnel

The intake structure with gates and sand trap basins will be constructed at the right bank of the damsite. The excavation volume of the intake site was estimated at 317,000 m^3. Excavation of the intake site will be carried out by combination of bulldozer, crawler drill, crawler loader and dump truck. The estimated volume of concrete for the intake is 40,800 m^3. Chuteway method using concrete pump will be applied for placing the intake concrete. Since the intake structure is provided toward the reservoir area, rim grouting will be executed along the foot of the intake structure. Installation of the gates will be performed after all of civil works for the intake structure are completed.

The construction works of the circular type headrace tunnel with 5.2 m in inside diameter and 6,305 m in total length, which are the critical work for this project, will be commenced immediately after completion of a series of temporary works. The work quantity of the tunnel was estimated to be 217,000 m^3 of excavation and 57,300 m^3 of concrete. In order to reduce the construction period, the construction works will be executed by dividing into two stretches by providing two adit tunnels. The length of the adits is 900 m for No.1 and 150 m for No. 2. It is planned to execute the construction of the headrace tunnel within a period of 41 months.

Standard excavation progress is 170 m/month. Excavation of the tunnel will be carried out by full face excavation method using 6-boom drill jumbo. About 60 % of the excavated rock will be transported to the stock pile for concrete use and the remaining to the spoil bank. Average hauling distance of transportation is about 2 km. A combination of crawler loader and dump truck is planned to be used for this work. Steel support will be used for a about 110 m long fault zone and a 500 m long inlet portion where there is no sufficient over burden. Concrete lining work will be executed in two stages; arch lining and inverted lining by combination of concrete pump and agitator truck. Standard progress required for concrete lining is 150 m/month. Consolidation grouting with 3 m in interval will be executed for the stretch with fault zone and about 500 m long inlet portion using groutings pump and concrete mixer.

(vii) Surge tank

A surge tank, 20 m in inside diameter and 55 m in height will be constructed at the end portion of the headrace tunnel at EL 283.5 m. The estimated work quantity is 183,200 m^3 of excavation and 4,150 m^3 of concrete lining. The construction works are planned to be executed in 15 months.

Excavation of the surge tank will be carried out by pilot heading - enlargement method using a raise climber, leg drill and crawler loader. It will be initiated from the headrace tunnel side and excavated rock will be transported to the spoil bank through No. 2 adit tunnel using dump truck. Concrete lining will be placed by concrete pump and agitator truck from the lower portion upward. Consolidation grouting with 3 m interval will be executed after completion of the lining work.

(iv) Penstock

The underground type penstock with an average diameter of 3.8 m consists of 20 m long horizontal tunnel in upper part, 233 m long inclined shaft portion and 252 m long horizontal tunnel in lower part. The estimated work quantities are 19,000 m^3 of excavation and 4,100 m^3 of concrete and 1,710 ton of penstock pipe. The work execution is planned to be 19 months.

Excavation work of the penstock tunnel will be initiated from upper and lower horizontal parts in the same way as applied for the headrace tunnel through No.2 adit tunnel near the surge tank site and No.3 adit tunnel with 250 m in length near the powerhouse respectively. Excavation of the inclined shaft will be executed by pilot heading - enlargement method. The excavated rock will be transported to the spoil banks by dump truck through No. 2 and No. 3 adit tunnels. After completion of the excavation work, steel liners will be installed and concrete is placed between the steel liner and excavated rock surface by concrete pump and agitator truck. Following this concrete filling work, consolidation grouting with 3 m in interval will be carried out.

(v) Powerhouse and tailrace

An open air type powerhouse and open channel type tailrace will be constructed at the right bank of the Itajai river near Subida. The estimated work quantities are 282,400 m^3 of excavation and 25,000 m^3 of concrete for substructure of powerhouse and tailrace. The work execution period is planned to be 38 months.

Excavation for the foundation of substructure will be carried out by combination of crawler drill, bulldozer, loader and dump truck. Concrete work for substructure and superstructure and tailrace will be executed by concrete pump and chuteway methods. A concrete plant having 50 m^3/h in mixing capacity will be installed close to the powerhouse site. Installation work of the power generating equipment will be carried out during the period of 18 months after the completion of the civil works.

General plan of the project facilities is shown in Fig.III.6.3. Major construction plant and equipment to be used for this project are shown in Table III.6.1.

(vi) Construction materials

Major construction materials were estimated as follows;

Cement	;	73,600 t
Reinforcement bar	;	3,979 t
Steel penstocks	;	1,710 t
Gates	;	1,529 t
Aggregates	;	296,200 m^3

6.1.4 Cost estimate

(1) Basic conditions

The construction cost was estimated based on the following basic conditions;

(i) Unit costs for major work items are estimated at price level of May, 1991 referring to the similar projects executed recently by CELESC and other agencies.

(ii) Costs of miscellaneous work in each major work item are estimated at around 5 % of the cost of major works.

(iii) Regarding costs for compensation for the submerged and construction areas, the survey results performed by CELESC are referred.

(iv) Costs of engineering service is estimated to cover remuneration for foreign and local consultants, direct expenses etc.

(v) Administration cost is estimated at 5 % of total direct construction cost.

(vi) Physical contingency accounting for 15 % of total direct construction cost plus engineering service, administration and compensation cost is added.

(2) Total construction cost

The total construction cost including physical contingency was estimated at US$178.2 million as shown in Table III.6.2.

(3) Disbursement schedule

Based on the construction time schedule, the construction fund to be disbursed in each construction year was estimated as shown in Table III.6.3 assuming that the project works are executed by contractor or supplier who will be selected through international competitive tendering and 20% of the construction cost is disbursed as an advance payment.

6.2 Construction Plan and Cost Estimate for Dalbergia Hydropower Scheme

6.2.1 Conditions for construction

(1) Site conditions

(i) Topography

The project area is located along the Itajai do Norte river in the middle part of the Itajai river basin. Vicinity of the damsite forms hilly areas with relative height of 80 to 100 m. They are covered with natural or secondary forests. River bed is generally flat and velocity of river flow is slow. No water fall or cataract features the project area and hard rock outcrops in whole river bed.

(ii) Meteoro-hydrology

The annual rainfall in the project area is around 1,500 mm. The monthly rainfall amount ranges from 100 mm in March to August to 150 mm in September to February. Throughout a year, much rainfall occurs in July.

(iii) Geology

Geological layer is composed of gneiss through whole project area. Deep weathering along the fault is a remarkable geologic feature in this project area which is especially conspicuous in the left bank of the damsite.

(iv) Access to the site

It is possible to access to the project area through BR-470 national highway which originates from Itajai city and through SC-490 state road which branches off from BR-470 at junction of the Itajai river and Itajai do Norte river.

(v) Construction materials and equipment

Major construction materials and equipment are obtainable from domestic market in Brazil. Rock materials for concrete aggregates and other uses are obtained from the proposed quarry site which is located at 0.5 km upstream of the damsite.

(2) Mode of construction

It is assumed to execute the construction works by an international contract system.

6.2.2 Construction time schedule

Fig.III.6.4 shows a proposed implementation schedule of the project. The construction works of the project are scheduled to be executed during 3 years and 7 months including 6 months for the preparatory works. Fig. III.6.5 shows the proposed construction time schedule.

6.2.3 Construction plan

The condition for construction planning are the same as those applied for Salto Pilão (1) hydropower scheme.

(1) Preparatory works

(i) Access and construction roads

Existing roads are available for the access and construction roads. Besides, new access roads and improvement of the existing road are needed for construction of dam, headrace tunnel, surge tank and penstock line. Total length of the new access roads and improvement of the existing road were estimated to be 1.85 km and 0.15 km, respectively.

(ii) Temporary building and communication system

The temporary buildings and communication facilities will be constructed in the project area.

(iii) Arrangement of spoil bank

The spoil banks to accommodate large amount of the excavated materials from the dam and headrace tunnel will be provided at several places along the headrace tunnel route.

(2) River diversion works

(i) Diversion tunnel

A 155 m long and horse shoe-shaped concrete lined diversion tunnel with 6.8 m in inside diameter will be constructed in the left bank of the damsite. The tunnel works are planned to be executed within 5 months. The estimated work quantities are 92,000 m^3 of excavation and 6,400 m^3 of concrete. The full face excavation method will be applied for the tunnel excavation using 4-boom drill jumbo and dump truck. Concrete lining work will be executed using sliding form, concrete pump and agitator truck.

(ii) Cofferdams

Primary upstream and downstream cofferdams will be constructed by earth embankment immediately after the diversion tunnel is completed. After river diversion, 13 m high upstream cofferdam of concrete gravity type will be constructed. The concrete volume was estimated at 18,700 m^3. The works will be executed by chuteway method using concrete pump and agitator truck.

(iii) Closure of diversion tunnel

The diversion tunnel will be closed by roller gate upon completion of all of the works. After closing the gate, diversion tunnel will be plugged with concrete. Concrete pump, grout pump and agitator truck will be used for this work.

(3) Main works

(i) Dam and spillway

A concrete gravity type dam having 22.5 m in maximum height and 113,500 m^3 in total concrete volume will be constructed at the proposed damsite. The required excavation volume is 368,700 m^3. The construction works of the dam are planned to be executed in a period of 37 months.

Excavation of dam foundation will be commenced at both abutments before river diversion. Excavation of river bed portion will be carried out currently with the construction of the primary cofferdam. A combination of bulldozer with ripper, crawler drill, crawler loader and dump truck is planned to be used. The excavated bed rock will be transported to the spoil banks. Foundation treatment for dam bed rock will be carried out by consolidation and curtain groutings. The estimated grout length is 8,460 m in total. Crawler drill, boring machine, grout pump and concrete mixer will be used for this work. Immediately after completion of the foundation treatment and clearing of river bed, concrete placing will be executed by chuteway method mainly using concrete pump, vibrator, and other concreting equipment. A concrete mixing plant with a capacity of 90 m^3/h will be installed near the damsite. Concrete placing for spillway structure will be executed succeeding to the dam concrete work. Installation work of the spillway gates will be carried out after all of the civil works for the dam are completed.

(ii) Intake and headrace tunnel

The intake structure with gates and sand trap basins will be constructed at the right bank of the damsite. The estimated work quantities are 90,400 m^3 of excavation and 9,200 m^3 of concrete. Excavation of the intake will be carried out by combination of bulldozer, crawler drill, crawler loader and dump truck. Concrete placing works will be carried out by chuteway method using concrete pump and agitator truck. Installation work of the gates will be carried out after completion of the civil works.

The construction works of the circular type headrace tunnel with 3.6 m in inside diameter and 8,720 m in total length, which are the critical work for this project, will be commenced immediately after the completion of a series of the temporary works. The estimated work quantities are 149,300 m^3 of excavation and 35,800 m^3 of concrete. The construction works of the headrace tunnel are planned to be executed within 37 months. In order to meet the required construction period, the construction works will be executed by dividing into three stretches by providing three adit tunnels. Length of the adit tunnel is 650 m for No. 1, 550 m for No. 2 and 100 m for No. 3 as shown in Fig. III.6.6.

Standard excavation progress is 200 m/month. Excavation of the tunnel will be carried out by full face excavation method using 4-boom drill jumbo. About 50 % of the excavated rock will be transported to the stock pile for concrete use and the remaining to the spoil bank. Average hauling distance is about 0.8 km. Excavated materials will be transported by battery locomotive and mucking car on rail. A train loader will be provided behind the rocker shovel. Steel support will be used for 110 m long fault zone and 300 m long inlet portion. Concrete lining works will be executed in two stages; arch lining and inverted lining by combination of concrete placer and agitator car. Consolidation grouting with 3 m in interval will be executed for the fault zone and about 300 m long inlet portion.

(iii) Surge tank

A surge tank, 14 m in inside diameter and 61 m in height will be constructed at the end of the headrace tunnel. The estimated work quantities are 54,200 m^3 of excavation and 3,400 m^3 of concrete. The construction works are planned to be executed within 14 months.

Excavation of the surge tank will be carried out by pilot heading-enlargement method using a raise climber, leg drill and rocker shovel. Construction method is the same as that applied for the headrace tunnel of Salto Pilão (1) hydropower scheme. Excavated bed rock will be transported to the spoil bank through No. 3 adit tunnel. Concrete lining will be executed by concrete pump and agitator truck from lower portion upwards. Consolidation grouting with 3 m in interval will be executed succeeding the concrete lining works.

(iv) Penstock

The underground type penstock with an average diameter of 2.9 m consists of 23 m long horizontal tunnel in upper part, 71 m long inclined shaft portion and 430 m long horizontal tunnel in lower portion. The estimated work quantities are 41,900 m^3 of excavation and 3,350 m^3 of concrete and 645 tons of penstock pipe. The work execution period is planned to be 20 months.

Excavation work of the penstock tunnel will be initiated from upper and lower horizontal parts in the same way as applied to the headrace tunnel. The excavated rock will be transported to the spoil bank through No. 3 adit tunnel near the surge tank and No. 4 adit tunnel with 150 m in length near the powerhouse respectively. Excavation of the inclined shaft will be executed by pilot heading - enlargement method. After completion of the excavation, steel liners are installed and concrete is placed between the steel liner and excavated rock surface. Consolidation grouting with 3 m in interval will be executed succeeding to the concrete filling work.

(v) Powerhouse and tailrace

An open air type powerhouse and open channel type tailrace will be constructed at the right bank of the Itajai do Norte river near Ibirama. The estimated work quantities are 30,000 m^3 of excavation and 15,300 m^3 of concrete for substructure of powerhouse and tailrace. The work execution period is planned to be 31 months.

Excavation of the foundation of substructure will be carried out by combination of crawler drills, bulldozer, loader and dump truck. Concrete work for substructure and superstructure and tailrace will be executed by concrete pump and chuteway method. A concrete plant with a capacity of 50 m^3/h will be installed close to the powerhouse site. Installation work of the power generating equipment will be carried out during the period of 17 months after the completion of the civil works.

General plan of the project facilities is shown in Fig. III.6.6. Major construction and equipment to be used for this project are shown in Table III.6.4.

(iv) Construction materials

Major construction materials were estimated as follows;

Cement	;	56,600 t
Reinforcement bar	;	2,206 t
Steel penstock	;	645 t
Gates	;	922.5 t
Aggregate	;	236,000 m^3

6.2.4 Cost estimate

The construction cost of the project was estimated based the same conditions as applied for Salto Pilão (1) hydropower scheme. The estimated construction cost is US$102.2 million as shown in Table III.6.5. Based on the construction time schedule, the construction fund to be disbursed in each construction year was estimated as shown in Table III.6.3.

6.3 Construction Plan and Cost Estimate for Benedito Novo Hydropower Scheme

6.3.1 Conditions for construction

(1) Site conditions

(i) Topography

The project area is located along the Benedito river in the middle part of the Itajai river basin. Vicinity of the damsite forms hilly and mountainous ares with a relative height of 80 to 200 m. Both sides of the damsite are very steep slope. River bed is generally ragged and velocity of river flow is rather fast. There are a series of water fall with a few meters high. Hard rock outcrops in whole river bed.

(ii) Meteoro-hydrology

The annual rainfall in the project area is around 1500 mm. The monthly rainfall amount ranges from 100 mm, in March to August to 150 mm in September to February. Much rainfall occurs in July throughout a year.

(iii) Geology

The geological layer in the project area is composed of gneiss. Relatively thick talus deposit distributes in the intake site and along the penstock line route.

(iv) Access to the site

It is possible to access to the project area through BR-470 national road and SC-417 & 416 state road which branches off from BR-470 national road at junction of the Itajai river and Benedito river.

(v) Construction materials and equipment

Major construction materials and equipment are obtainable from domestic market in Brazil. Rock materials for concrete and other uses are obtained from the proposed quarry site at about 3 km upstream from the damsite.

(2) Mode of construction

It is assumed to execute the construction works by an international contract system.

6.3.2 Construction time schedule

Fig.III.6.7 shows a proposed implementation schedule of the project. The construction works of the project are scheduled to be executed during 3 years including 6 months for the preparatory works. Fig.III.6.8 shows the proposed construction time schedule.

6.3.3 Construction plan

The conditions for construction planning are the same as those applied for Salto Pilão (1) hydropower scheme.

(1) Preparatory works

(i) Access and construction roads

Existing roads are available for the access and construction roads. Besides, new access road are needed for construction of the project works. Total length of the new access road was estimated to be 1.4 km.

(ii) Temporary building and communication system

The temporary buildings and communication facilities will be constructed in the project area.

(iii) Arrangement of spoil bank

The spoil banks to accommodate large amount of the excavated material from the dam and headrace tunnel will be provided at several places along the headrace tunnel route.

(2) River diversion works

(i) Diversion tunnels

A diversion tunnel, 155 m long and horse shoe-shaped concrete lined type with a 4.5 m in inside diameter will be constructed in the left bank of the damsite. The tunnel works are planned to be executed in a period of 5 months. The estimated work quantities are 15,700 m^3 of excavation and 3,100 m^3 of concrete. The full face excavation method will be applied for the tunnel excavation using 2-boom drill jumbo and dump truck. Concrete lining work will be executed using sliding form and concrete pump and agitator truck.

(ii) Cofferdams

Primary upstream and downstream cofferdams will be constructed by earth embankment immediately after the diversion tunnel is completed. After river diversion, 10.5 m high upstream cofferdam of concrete gravity type will be constructed. The concrete volume was estimated to be 3,930 m^3. The works will be executed by chuteway method using concrete pump and agitator truck.

(iii) Closure of diversion tunnel

The diversion tunnel will be closed by roller gate upon completion of all of the works. After closing the gate, diversion tunnel will be plugged with concrete. Concrete pump, grout pump and agitator truck will be used for this work.

(3) Main works

(i) Dam and spillway

A concrete gravity type dam having 24.5 m in maximum height and 53,100 m^3 in total volume will be constructed at the proposed damsite. The required excavation volume is 68,500 m^3. The construction works of the dam are planned to be executed in a period of 24 months.

Excavation of dam foundation will be commenced at both abutments before river diversion. Excavation of river bed portion will be carried out currently with the construction of the primary cofferdam. A combination of bulldozer with ripper, crawler drill, crawler loader and dump truck are planned to be used. The excavated bed rock will be transported to the spoil banks. Foundation treatment for dam bed rock will be carried out by consolidation and curtain groutings. The estimated grout length is 1,900 m in total. Crawler drill, boring machine, grout pump and concrete mixer will be used for this work. Immediately after completion of foundation treatment and clearing of river bed, concrete placing will be executed by chuteway method mainly using concrete pump, vibrator, and other concreting equipment. A concrete mixing plant with a capacity of 70 m^3/h will be installed near the damsite. Concrete placing for spillway structure will be executed succeeding to dam concrete work. Installation of the spillway gates will be carried out after all of the civil works for the dam are completed.

(ii) Intake and headrace tunnel

The intake structure with gates and sand trap basins will be constructed at the right bank of the damsite. The estimated work quantities are 114,300 m^3 of excavation and 21,300 m^3 of concrete. Excavation of the intake will be carried out by combination of bulldozer, crawler drill, crawler loader and dump truck. Concrete placing works will be carried out by chuteway method using concrete pump and agitator truck. Installation work of the gates will be carried out after completion of the civil works.

The construction works of a circular type headrace tunnel with 2.8 m in inside diameter and 1,815 m in total length, which are the critical work for this project , will be commenced immediately after completion of the temporary works. The estimated work quantities are 21,500 m^3 of excavation and 6,100 m^3 of concrete. The construction works of the headrace tunnel are planned to be executed in a period of 30 months. In order to meet the required construction period, the construction works will be executed using 130 m long adit tunnel to be constructed near the surge tank site.

Standard excavation progress is 120 m/month. Excavation of the tunnel will be carried out by full face excavation method using 2-boom drill jumbo. Average hauling distance is about 1.5 km. Excavated materials will be transported by battery locomotive and mucking car on rail. A train loader will be provided behind the crawler jumbo. Steel support will be used for 110 m long fault zone and 300 m long inlet portion. Concrete lining works will be executed in two stages; arch lining and inverted lining by a combination of concrete pump and agitator car. Consolidation grouting with 3 m in interval will be executed for the fault zone and about 300 m long inlet portion.

(iii) Surge tank

A surge tank, 10 m in inside diameter and 31 m in height will be constructed at the end of the headrace tunnel. The estimated work quantities are 29,600 m^3 of excavation and 1,070 m^3 of concrete. The construction works are planned to be executed in 9 months.

Excavation of the surge tank will be carried out by pilot heading-enlargement method using a raise climber, leg drill and rocker shovel. Construction method is the same as that applied for the headrace tunnel of Salto Pilão (1) hydropower scheme. Excavated bed rock will be transported to the spoil bank through the adit tunnel. Concrete lining will be placed by concrete pump and agitator truck from lower portion upward. Consolidation grouting with 3 m in interval will be executed succeeding the concrete lining works.

(iv) Penstock

The underground type penstock with an average diameter of 2.2 m consists of 10 m long horizontal tunnel in upper part, 135 m long inclined shaft portion and 310 m long horizontal tunnel in lower portion. The estimated work quantities are 16,000 m^3 of excavation and 2,340 m^3 of concrete and 350 tons of penstock pipe. The construction works are planned to be executed in a period of 18 months.

Excavation work of the penstock tunnel will be initiated from upper and lower horizontal parts in the same way as applied to the headrace tunnel. The excavated rock will be transported to the spoil bank through the adit tunnel near the surge tank and another adit tunnel with 200 m in length near the powerhouse. Excavation of the inclined shaft will be executed by pilot heading - enlargement method. After completion of the excavation, steel liners will be installed and concrete will be placed between the steel liner and excavated rock surface. Consolidation grouting with 3 m in interval will be executed succeeding to the concrete filling work.

(v) Powerhouse and tailrace

An open-air type powerhouse and an open channel type tailrace will be constructed at the right bank of the Benedito river in the upstream of Benedito Novo. The estimated work quantities are 13,600 m^3 of excavation and 10,300 m^3 of concrete for substructure of power house and tailrace. The work period is planned to be 30 months.

Excavation of the foundation of substructure will be carried out by combination of crawler drill, bulldozer, loader and dump truck. Concrete work for substructure and superstructure and tailrace will be executed by concrete pump and chuteway method. A concrete plant with capacity of 50 m^3/h will be installed close to the powerhouse site. Installation works of the power generating equipment will be carried out during a period of 19 months after completion of the civil works.

General plan of the project facilities is shown in Fig. III.6.9. Major construction and equipment to be used for this project are shown in Table III.6.6.

(vi) Construction materials

Major construction materials were estimated as follows;

Cement	;	27,800 t
Reinforcement bar	;	815 t
Steel penstock	;	350 t
Gates	;	853.5 t
Aggregate	;	116,000 m^3

6.3.4 Cost estimate

The construction cost of the project was estimated based on the same conditions as applied for Salto Pilão (1) hydropower scheme. The estimated construction cost is US$ 56.5 million as shown in Table III.6.7. Based on the construction time schedule, the construction fund to be disbursed in each construction year was estimated as shown in Table III.6.3.

TABLES

Table III.3.1 CONSTRUCTION COST OF SALTO PILÃO (1) HYDROPOWER SCHEME FOR DAM AXIS-A

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	193,400	1,354
(2) Concrete(Mass)	cu.m	80	55,900	4,472
(3) Concrete(Structure)	cu.m	140	2,970	416
(4) Reinforcing bar	ton	1,100	90	99
(5) Bridge for maintenance	L.S	-		34
(6) Curtain grouting	L.S	-		38
(7) Consolidation grouting	L.S	-		4
(8) Spillway gate	ton	4,800	930	4,464
(9) River diversion works	L.S	-		14,913
(10) Miscellaneous work	L.S	-		321
Sub-total				26,114
2. Intake				
(1) Excavation	cu.m	7	4,930	35
(2) Concrete	cu.m	140	1,780	249
(3) Reinforcing bar	ton	1,100	72	79
(4) Intake gate	ton	4,800	78	374
(5) Trashrack	ton	2,600	43	112
(6) Miscellaneous work	L.S	-		18
Sub-total				867
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	80	269,260	21,541
(2) Lining concrete	cu.m	140	65,150	9,121
(3) Reinforcing bar	ton	1,100	2,480	2,728
(4) Steel support	ton	1,220	53	65
(5) Consolidation grouting	L.S	-		34
(6) Work adit	L.S	-		1,843
(7) Miscellaneous work	L.S	-		1,674
Sub-total				37,006
4. Surge Tank				
(1) Shaft excavation	cu.m	100	19,100	1,910
(2) Lining concrete	cu.m	160	3,200	512
(3) Reinforcing bar	ton	1,100	160	176
(4) Consolidation grouting	L.S	-		22
(5) Surge tank gate	ton	4,800	73	350
(6) Miscellaneous work	L.S	-		131
Sub-total				3,101
5. Penstock				
(1) Shaft excavation	cu.m	125	13,280	1,660
(2) Backfill concrete	cu.m	160	5,180	829
(3) Consolidation grouting	L.S	-		132
(4) Steel liner	ton	2,100	2,170	4,557
(5) Work adit	L.S	-		531
(6) Miscellaneous work	L.S	-		131
Sub-total				7,840
6. Power Station				
(1) Excavation	cu.m	7	73,200	512
(2) Concrete	cu.m	140	27,880	3,903
(3) Reinforcing bar	ton	1,100	1,450	1,595
(4) Superstructure	cu.m	180	19,850	3,573
(5) Generating equipment	L.S	-		25,500
(6) T/L&S/S	L.S	-		4,741
(7) Miscellaneous work	L.S	-		301
Sub-total				40,125
7. Access Road				
(1) New construction road	Km	200,000	1.9	380
(2) Improvement of existing road	Km	90,000	2.1	189
(3) Bridge	m	5,000	10	50
(4) Miscellaneous work	L.S	-		31
Sub-total				650
Total of Item I				115,703
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	270,000	1.9	513
(2) Bridge	m	5,000	40	200
Sub-total				713
2. Land and house				
(1) Land	sq.km	115,000	1.81	208
(2) House	nos.	7,350	74	544
Sub-total				752
Total of Item II				1,465
III. Administration Cost				5,785
IV. Engineering Service Cost				3,500
V. Physical Contingency				17,356
VI. Grand Total				143,809

Table III.3.2 CONSTRUCTION COST OF SALTO PILÃO (1) HYDROPOWER SCHEME FOR DAM AXIS-B

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	149,300	1,045
(2) Concrete(Mass)	cu.m	80	55,900	4,472
(3) Concrete(Structure)	cu.m	140	3,030	424
(4) Reinforcing bar	ton	1,100	91	100
(5) Bridge for maintenance	L.S	-		34
(6) Curtain grouting	L.S	-		0
(7) Consolidation grouting	L.S	-		0
(8) Spillway gate	ton	4,800	930	4,464
(9) River diversion works	L.S	-		4,993
(10) Miscellaneous work	L.S	-		304
Sub-total				15,836
2. Intake				
(1) Excavation	cu.m	7	4,930	35
(2) Concrete	cu.m	140	1,780	249
(3) Reinforcing bar	ton	1,100	72	79
(4) Intake gate	ton	4,800	78	374
(5) Trashrack	ton	2,600	43	112
(6) Miscellaneous work	L.S	-		18
Sub-total				867
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	80	230,760	18,461
(2) Lining concrete	cu.m	140	55,880	7,823
(3) Reinforcing bar	ton	1,100	2,070	2,277
(4) Steel support	ton	1,220	53	65
(5) Consolidation grouting	L.S			34
(6) Work adit	L.S	-		1,375
(7) Miscellaneous work	L.S	-		1,433
Sub-total				31,468
4. Surge Tank				
(1) Shaft excavation	cu.m	100	17,900	1,790
(2) Lining concrete	cu.m	160	3,000	480
(3) Reinforcing bar	ton	1,100	150	165
(4) Consolidation grouting	L.S			21
(5) Surge tank gate	ton	4,800	69	331
(6) Miscellaneous work	L.S	-		123
Sub-total				2,910
5. Penstock				
(1) Shaft excavation	cu.m	125	13,170	1,646
(2) Backfill concrete	cu.m	160	5,140	822
(3) Consolidation grouting	L.S			132
(4) Steel liner	ton	2,100	2,190	4,599
(5) Work adit	L.S	-		531
(6) Miscellaneous work	L.S	-		130
Sub-total				7,861
6. Power Station				
(1) Excavation	cu.m	7	73,640	515
(2) Concrete	cu.m	140	28,060	3,928
(3) Reinforcing bar	ton	1,100	1,460	1,606
(4) Superstructure	cu.m	180	19,850	3,573
(5) Generating equipment	L.S	-		26,000
(6) T/L&S/S	L.S	-		4,741
(7) Miscellaneous work	L.S	-		302
Sub-total				40,666
7. Access Road				
(1) New construction road	Km	200,000	2.05	410
(2) Improvement of existing road	Km	90,000	2.35	212
(3) Bridge	m	5,000	10	50
(4) Miscellaneous work	L.S	-		34
Sub-total				705
Total of Item I				100,313
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	270,000	1.9	513
(2) Bridge	m	5,000	40	200
Sub-total				713
2. Land and house				
(1) Land	sq.km	115,000	2.09	240
(2) House	nos.	7,350	74	544
Sub-total				784
Total of Item II				1,497
III. Administration Cost				5,016
IV. Engineering Service Cost				4,200
V. Physical Contingency				15,047
VI. Grand Total				126,072

Table III.3.3 CONSTRUCTION COST OF SALTO PILÃO (1) HYDROPOWER SCHEME FOR DAM AXIS-C

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	120,800	846
(2) Concrete(Mass)	cu.m	80	50,600	4,048
(3) Concrete(Structure)	cu.m	140	2,830	396
(4) Reinforcing bar	ton	1,100	85	94
(5) Bridge for maintenance	L.S	-		27
(6) Curtain grouting	L.S	-		376
(7) Consolidation grouting	L.S	-		45
(8) Spillway gate	ton	4,800	940	4,512
(9) River diversion works	L.S	-		10,893
(10) Miscellaneous work	L.S	-		292
Sub-total				21,528
2. Intake				
(1) Excavation	cu.m	7	5,690	40
(2) Concrete	cu.m	140	2,050	287
(3) Reinforcing bar	ton	1,100	82	90
(4) Intake gate	ton	4,800	80	384
(5) Trashrack	ton	2,600	45	117
(6) Miscellaneous work	L.S	-		21
Sub-total				939
3. Headrace tunnel				
(1) Tunnel Excavation	cu.m	80	219,920	17,594
(2) Lining concrete	cu.m	140	54,740	7,664
(3) Reinforcing bar	ton	1,100	2,080	2,288
(4) Steel support	ton	1,220	293	357
(5) Consolidation grouting	L.S	-		155
(6) Work adit	L.S	-		1,843
(7) Miscellaneous work	L.S	-		1,403
Sub-total				31,304
4. Surge Tank				
(1) Shaft excavation	cu.m	100	18,200	1,820
(2) Lining concrete	cu.m	160	3,050	488
(3) Reinforcing bar	ton	1,100	155	171
(4) Consolidtion grouting	L.S	-		21
(5) Surge Tank gate	ton	4,800	71	341
(6) Miscellaneous work	L.S	-		125
Sub-total				2,965
5. Penstock				
(1) Shaft excavation	cu.m	125	12,000	1,500
(2) Backfill concrete	cu.m	160	4,690	750
(3) Consolidation grouting	L.S	-		121
(4) Steel liner	ton	2,100	1,880	3,948
(5) Work adit	L.S	-		531
(6) Miscellaneous work	L.S	-		119
Sub-total				6,969
6. Power Station				
(1) Excavation	cu.m	7	71,150	498
(2) Concrete	cu.m	140	27,110	3,795
(3) Reinforcing bar	ton	1,100	1,410	1,551
(4) Superstructure	cu.m	180	19,850	3,573
(6) Generating equipment	L.S	-		24,500
(7) T/L&S/S	L.S	-		4,741
(8) Miscellaneous work	L.S	-		292
Sub-total				38,951
7. Access Road				
(1) New construction road	Km	200,000	1.75	350
(2) Improvement of existing road	Km	90,000	2.45	221
(3) Bridge	m	5,000	30	150
(4) Miscellaneous work	L.S	-		36
Sub-total				757
Total of Item I				103,412
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	270,000	0	0
(2) Bridge	m	5,000	0	0
Sub-total				0
2. Land and house				
(1) Land	sq.km	115,000	0.28	32
(2) House	nos.	7,350	7	51
Sub-total				84
Total of Item II				84
III. Administration Cost				5,171
IV. Engineering Service Cost				4,100
V. Physical Contingency				15,512
VI. Grand Total				128,278

Table III.3.4 UNIT COST OF GUARANTEED ENERGY FOR COMPARATIVE STUDY

1. Salto Pilão (1) Hydropower Scheme

Dam axis	Construction cost (x1000 US$)	Secondary energy (MWh)	Guaranteed energy (MWh)	CUEG (US$/MWh)
Axis-A	143,809	65,425	686,366	20.2
Axis-B	126,072	66,350	689,467	17.5
Axis-C	128,278	63,078	654,863	18.8

Note:CUEG; Unit Cost of Guaranteed Energy

2. Dalbergia Hydropower Scheme

Dam axis	Construction cost (x1000 US$)	Secondary energy (MWh)	Guaranteed energy (MWh)	CUEG (US$/MWh)
Axis-A	85,309	13,399	111,793	75.8
Axis-B	68,995	12,850	106,407	64.2
Axis-C	67,794	10,626	91,627	73.5

3. Benedito Novo Hydropower Scheme

Dam axis	Construction cost (x1000 US$)	Secondary energy (MWh)	Guaranteed energy (MWh)	CUEG (US$/MWh)
Axis-A	32,196	12,802	72,562	43.0
Axis-B	33,343	12,549	71,008	45.6
Axis-C	31,071	11,681	65,747	45.9

Table III.4.1 CONSTRUCTION COST OF DALBERGIA HYDROPOWER SCHEME FOR DAM AXIS-A

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	209,100	1,464
(2) Concrete(Mass)	cu.m	80	84,100	6,728
(3) Concrete(Structure)	cu.m	140	2,830	396
(4) Reinforcing bar	ton	1,100	85	94
(5) Bridge for maintenance	L.S	-		36
(6) Curtain grouting	L.S	-		465
(7) Consolidation grouting	L.S	-		149
(8) Spillway gate	ton	4,800	630	3,024
(9) River diversion works	L.S	-		14,510
(10) Miscellaneous work	L.S	-		467
Sub-total				27,331
2. Intake				
(1) Excavation	cu.m	7	2,760	19
(2) Concrete	cu.m	140	1,000	140
(3) Reinforcing bar	ton	1,100	40	44
(4) Intake gate	ton	4,800	30	144
(5) Trashrack	ton	2,600	17	44
(6) Miscellaneous work	L.S	-		10
Sub-total				402
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	84	163,120	13,702
(2) Lining concrete	cu.m	148	36,240	5,364
(3) Reinforcing bar	ton	1,100	930	1,023
(4) Steel support	ton	1,220	28	34
(5) Consolidation grouting	L.S	-		34
(6) Work adit	L.S	-		1,757
(7) Miscellaneous work	L.S	-		1,008
Sub-total				22,922
4. Surge Tank				
(1) Shaft excavation	cu.m	100	9,250	925
(2) Lining concrete	cu.m	160	2,150	344
(3) Reinforcing bar	ton	1,100	110	121
(4) Consolidation grouting	L.S	-		21
(5) Surge tank gate	ton	4,800	35	168
(6) Miscellaneous work	L.S	-		71
Sub-total				1,650
5. Penstock				
(1) Shaft excavation	cu.m	140	5,520	773
(2) Backfill concrete	cu.m	160	2,510	402
(3) Consolidation grouting	L.S	-		84
(4) Steel liner	ton	2,100	470	987
(5) Work adit	L.S	-		322
(6) Miscellaneous work	L.S	-		63
Sub-total				2,630
6. Power Station				
(1) Excavation	cu.m	7	15,160	106
(2) Concrete	cu.m	140	5,780	809
(3) Reinforcing bar	ton	1,100	300	330
(4) Superstructure	cu.m	180	11,210	2,018
(5) Generating equipment	L.S	-		7,350
(6) T/L&S/S	L.S	-		942
(7) Miscellaneous work	L.S	-		62
Sub-total				11,617
7. Access Road				
(1) New construction road	Km	200,000	3.4	680
(2) Improvement of existing road	Km	90,000	0.2	18
(3) Bridge	m	5,000	70	350
(4) Miscellaneous work	L.S	-		52
Sub-total				1,100
Total of Item I				67,652
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	0	0
(2) Bridge	m	5,000	0	0
Sub-total				0
2. Compensation				
(1) Land	sq.km	115,000	0.13	15
(2) House	nos.	7,350	7	51
Sub-total				66
Total of Item II				66
III. Administration Cost				3,383
IV. Engineering Service Cost				4,060
V. Physical Contingency				10,148
VI. Grand Total				85,309

Table III.4.2 CONSTRUCTION COST OF DALBERGIA HYDROPOWER SCHEME FOR DAM AXIS-B

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	322,400	2,257
(2) Concrete(Mass)	cu.m	80	55,100	4,408
(3) Concrete(Structure)	cu.m	140	6,730	942
(4) Reinforcing bar	ton	1,100	85	94
(5) Bridge for maintenance	L.S	-		36
(6) Curtain grouting	L.S	-		435
(7) Consolidation grouting	L.S	-		129
(8) Spillway gate	ton	4,800	630	3,024
(9) River diversion works	L.S	-		4,349
(10) Miscellaneous work	L.S	-		415
Sub-total				16,088
2. Intake				
(1) Excavation	cu.m	7	2,760	19
(2) Concrete	cu.m	140	1,000	140
(3) Reinforcing bar	ton	1,100	40	44
(4) Intake gate	ton	4,800	30	144
(5) Trashrack	ton	2,600	17	44
(6) Miscellaneous work	L.S	-		10
Sub-total				402
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	84	150,500	12,642
(2) Lining concrete	cu.m	148	33,920	5,020
(3) Reinforcing bar	ton	1,100	1,090	1,199
(4) Steel support	ton	1,220	103	126
(5) Consolidation grouting	L.S	-		106
(6) Work adit	L.S	-		1,755
(7) Miscellaneous work	L.S	-		955
Sub-total				21,802
4. Surge Tank				
(1) Shaft excavation	cu.m	100	9,000	900
(2) Lining concrete	cu.m	160	2,100	336
(3) Reinforcing bar	ton	1,100	110	121
(4) Consolidation grouting	L.S	-		21
(5) Surge tank gate	ton	4,800	34	163
(6) Miscellaneous work	L.S	-		69
Sub-total				1,610
5. Penstock				
(1) Shaft excavation	cu.m	140	5,090	713
(2) Backfill concrete	cu.m	160	2,310	370
(3) Consolidation grouting	L.S	-		78
(4) Steel liner	ton	2,100	410	861
(5) Work adit	L.S	-		322
(6) Miscellaneous work	L.S	-		58
Sub-total				2,401
6. Power Station				
(1) Excavation	cu.m	7	14,710	103
(2) Concrete	cu.m	140	5,600	784
(3) Reinforcing bar	ton	1,100	290	319
(4) Superstructure	cu.m	180	11,210	2,018
(5) Generating equipment	L.S	-		7,100
(6) T/L&S/S	L.S	-		942
(7) Miscellaneous work	L.S	-		60
Sub-total				11,326
7. Access Road				
(1) New construction road	Km	200,000	2.4	480
(2) Improvement of existing road	Km	90,000	0.2	18
(3) Bridge	m	5,000	30	150
(4) Miscellaneous work	L.S	-		32
Sub-total				680
Total of Item I				54,310
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	0	0
(2) Bridge	m	5,000	0	0
Sub-total				0
2. Compensation				
(1) Land	sq.km	115,000	0.1	12
(2) House	nos.	7,350	7	51
Sub-total				63
Total of Item II				63
III. Administration Cost				2,715
IV. Engineering Service Cost				3,760
V. Physical Contingency				8,146
VI. Grand Total				68,995

Table III.4.3 CONSTRUCTION COST OF DALBERGIA HYDROPOWER SCHEME FOR DAM AXIS-C

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	140,900	986
(2) Concrete(Mass)	cu.m	80	70,300	5,624
(3) Concrete(Structure)	cu.m	140	2,990	419
(4) Reinforcing bar	ton	1,100	90	99
(5) Bridge for maintenance	L.S	-		36
(6) Curtain grouting	L.S	-		615
(7) Consolidation grouting	L.S	-		107
(8) Spillway gate	ton	4,800	630	3,024
(9) River diversion works	L.S	-		5,571
(10) Miscellaneous work	L.S	-		394
Sub-total				16,875
2. Intake				
(1) Excavation	cu.m	7	2,760	19
(2) Concrete	cu.m	140	1,000	140
(3) Reinforcing bar	ton	1,100	40	44
(4) Intake gate	ton	4,800	30	144
(5) Trashrack	ton	2,600	17	44
(6) Miscellaneous work	L.S	-		10
Sub-total				402
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	84	149,830	12,586
(2) Lining concrete	cu.m	148	33,300	4,928
(3) Reinforcing bar	ton	1,100	825	908
(4) Steel support	ton	1,220	28	34
(5) Consolidation grouting	L.S	-		34
(6) Work adit	L.S	-		1,753
(7) Miscellaneous work	L.S	-		924
Sub-total				21,167
4. Surge Tank				
(1) Shaft excavation	cu.m	100	8,820	882
(2) Lining concrete	cu.m	160	2,000	320
(3) Reinforcing bar	ton	1,100	100	110
(4) Consolidation grouting	L.S			21
(5) Surge tank gate	ton	4,800	33	158
(6) Miscellaneous work	L.S	-		67
Sub-total				1,558
5. Penstock				
(1) Shaft excavation	cu.m	135	4,560	616
(2) Backfill concrete	cu.m	160	2,040	326
(3) Consolidation grouting	L.S	-		68
(4) Steel liner	ton	2,100	330	693
(5) Work adit	L.S	-		322
(6) Miscellaneous work	L.S	-		51
Sub-total				2,076
6. Power Station				
(1) Excavation	cu.m	7	13,180	92
(2) Concrete	cu.m	140	5,020	703
(3) Reinforcing bar	ton	1,100	260	286
(4) *Superstructure*	*cu.m*	*180*	*10,610*	1,910
(6) Generating equipment	L.S	-		6,100
(7) T/L&S/S	L.S	-		942
(8) Miscellaneous work	L.S	-		54
Sub-total				10,087
7. Access Road				
(1) New construction road	Km	200,000	3.45	690
(2) Improvement of existing road	Km	90,000	0	0
(3) Bridge	m	5,000	80	400
(4) Miscellaneous work	L.S	-		55
Sub-total				1,145
Total of Item I				53,309
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	0	0
(2) Bridge	m	5,000	0	0
Sub-total				0
2. Compensation				
(1) Land	sq.km	115,000	0.11	13
(2) House	nos.	7,350	7	51
Sub-total				64
Total of Item II				64
III. Administration Cost				2,665
IV. Engineering Service Cost				3,760
V. Physical Contingency				7,996
VI. Grand Total				67,794

Table III.5.1 CONSTRUCTION COST OF BENEDITO NOVO HYDROPOWER SCHEME FOR DAM AXIS-A

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	81,700	572
(2) Concrete(Mass)	cu.m	80	15,600	1,248
(3) Concrete(Structure)	cu.m	140	1,820	255
(4) Reinforcing bar	ton	1,100	55	61
(5) Bridge for maintenance	L.S	-		19
(6) Curtain grouting	L.S	-		89
(7) Consolidation grouting	L.S	-		27
(8) Spillway gate	ton	4,800	250	1,200
(9) River diversion works	L.S	-		2,154
(10) Miscellaneous work	L.S	-		113
Sub-total				5,737
2. Intake				
(1) Excavation	cu.m	7	1,620	11
(2) Concrete	cu.m	140	580	81
(3) Reinforcing bar	ton	1,100	24	26
(4) Intake gate	ton	4,800	14	67
(5) Trashrack	ton	2,600	8	21
(6) Miscellaneous work	L.S	-		6
Sub-total				213
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	92	32,370	2,978
(2) Lining concrete	cu.m	158	8,740	1,381
(3) Reinforcing bar	ton	1,100	130	143
(4) Steel support	ton	1,220	66	81
(5) Consolidation grouting	L.S	-		90
(6) Work adit	L.S	-		162
(7) Miscellaneous work	L.S	-		234
Sub-total				5,068
4. Surge Tank				
(1) Shaft excavation	cu.m	100	2,700	270
(2) Lining concrete	cu.m	160	790	126
(3) Reinforcing bar	ton	1,100	40	44
(4) Consolidation grouting	L.S	-		17
(5) Surge tank gate	ton	4,800		0
(6) Miscellaneous work	L.S	-		23
Sub-total				480
5. Penstock				
(1) Shaft excavation	cu.m	150	4,080	612
(2) Backfill concrete	cu.m	160	2,110	338
(3) Consolidation grouting	L.S	-		89
(4) Steel liner	ton	2,100	310	651
(5) Work adit	L.S	-		190
(6) Miscellaneous work	L.S	-		52
Sub-total				1,932
6. Power Station				
(1) Excavation	cu.m	7	10,520	74
(2) Concrete	cu.m	140	4,010	561
(3) Reinforcing bar	ton	1,100	210	231
(4) Superstructure	cu.m	180	9,420	1,696
(6) Generating equipment	L.S	-		5,300
(7) T/L&S/S	L.S	-		1,140
(8) Miscellaneous work	L.S	-		43
Sub-total				9,045
7. Access Road				
(1) New construction road	Km	200,000	1.4	280
(2) Improvement of existing road	Km	90,000	0	0
(3) Bridge	m	5,000	0	0
(4) Miscellaneous work	L.S	-		14
Sub-total				294
Total of Item I				22,769
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	1	200
(2) Bridge	m	5,000	50	250
Sub-total				450
2. Compensation				
(1) Land	sq.km	115,000	0.144	17
(2) House	nos.	7,350	88	647
Sub-total				663
Total of Item II				1,113
III. Administration Cost				1,138
IV. Engineering Service Cost				3,760
V. Physical Contingency				3,415
VI. Grand Total				32,196

Table III.5.2 CONSTRUCTION COST OF BENEDITO NOVO HYDROPOWER SCHEME FOR DAM AXIS-B

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	79,900	559
(2) Concrete(Mass)	cu.m	80	38,300	3,064
(3) Concrete(Structure)	cu.m	140	1,580	221
(4) Reinforcing bar	ton	1,100	48	53
(5) Bridge for maintenance	L.S	-		13
(6) Curtain grouting	L.S	-		120
(7) Consolidation grouting	L.S	-		36
(8) Spillway gate	ton	4,800	240	1,152
(9) River diversion works	L.S	-		2,044
(10) Miscellaneous work	L.S	-		203
Sub-total				7,466
2. Intake				
(1) Excavation	cu.m	7	2,150	15
(2) Concrete	cu.m	140	770	108
(3) Reinforcing bar	ton	1,100	31	34
(4) Intake gate	ton	4,800	15	72
(5) Trashrack	ton	2,600	9	23
(6) Miscellaneous work	L.S	-		8
Sub-total				260
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	92	29,650	2,728
(2) Lining concrete	cu.m	158	8,040	1,270
(3) Reinforcing bar	ton	1,100	150	165
(4) Steel support	ton	1,220	66	81
(5) Consolidation grouting	L.S	-		90
(6) Work adit	L.S	-		164
(7) Miscellaneous work	L.S	-		217
Sub-total				4,714
4. Surge Tank				
(1) Shaft excavation	cu.m	100	2,900	290
(2) Lining concrete	cu.m	160	840	134
(3) Reinforcing bar	ton	1,100	42	46
(4) Consolidation grouting	L.S	-		17
(5) Surge tank gate	ton	4,800		0
(6) Miscellaneous work	L.S	-		24
Sub-total				512
5. Penstock				
(1) Shaft excavation	cu.m	150	3,970	596
(2) Backfill concrete	cu.m	160	2,060	330
(3) Consolidation grouting	L.S			87
(4) Steel liner	ton	2,100	300	630
(5) Work adit	L.S	-		190
(6) Miscellaneous work	L.S	-		51
Sub-total				1,883
6. Power Station				
(1) Excavation	cu.m	7	10,380	73
(2) Concrete	cu.m	140	3,960	554
(3) Reinforcing bar	ton	1,100	205	226
(4) Superstructure	cu.m	180	9,420	1,696
(6) Generating equipment	L.S	-		5,250
(7) T/L&S/S	L.S	-		1,140
(8) Miscellaneous work	L.S	-		43
Sub-total				8,981
7. Access Road				
(1) New construction road	Km	200,000	1.4	280
(2) Improvement of existing road	Km	90,000	0	0
(3) Bridge	m	5,000	0	0
(4) Miscellaneous work	L.S	-		14
Sub-total				294
Total of Item I				24,110
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	1	200
(2) Bridge	m	5,000	50	250
Sub-total				450
2. Compensation				
(1) Land	sq.km	115,000	0.03	3
(2) House	nos.	7,350	27	198
Sub-total				202
Total of Item II				652
III. Administration Cost				1,205
IV. Engineering Service Cost				3,760
V. Physical Contingency				3,616
VI. Grand Total				33,343

Table III.5.3 CONSTRUCTION COST OF BENEDITO NOVO HYDROPOWER SCHEME FOR DAM AXIS-C

Work Item	Unit	Unit Price (US$)	Quantity	Amount (US$x1000)
I. Direct Cost				
1. Dam				
(1) Excavation	cu.m	7	75,600	529
(2) Concrete(Mass)	cu.m	80	30,500	2,440
(3) Concrete(Structure)	cu.m	140	3,270	458
(4) Reinforcing bar	ton	1,100	60	66
(5) Bridge for maintenance	L.S	-		14
(6) Curtain grouting	L.S	-		87
(7) Consolidation grouting	L.S	-		27
(8) Spillway gate	ton	4,800	490	2,352
(9) River Diversion works	L.S	-		1,275
(10) Miscellaneous work	L.S	-		181
Sub-total				7,429
2. Intake				
(1) Excavation	cu.m	7	2,920	20
(2) Concrete	cu.m	140	1,050	147
(3) Reinforcing bar	ton	1,100	42	46
(4) Intake gate	ton	4,800	16	77
(5) Trashrack	ton	2,600	9	23
(6) Miscellaneous work	L.S	-		11
Sub-total				325
3. Headrace tunnel				
(1) Tunnel excavation	cu.m	92	23,120	2,127
(2) Lining concrete	cu.m	158	6,230	984
(3) Reinforcing bar	ton	1,100	180	198
(4) Steel support	ton	1,220	46	56
(5) Consolidation grouting	L.S	-		67
(6) Work adit	L.S	-		105
(7) Miscellaneous work	L.S	-		172
Sub-total				3,709
4. Surge Tank				
(1) Shaft excavation	cu.m	100	3,170	317
(2) Lining concrete	cu.m	160	940	150
(3) Reinforcing bar	ton	1,100	47	52
(4) Consolidation grouting	L.S	-		17
(5) Surge tank gate	ton	4,800		0
(6) Miscellaneous work	L.S	-		27
Sub-total				563
5. Penstock				
(1) Shaft excavation	cu.m	150	3,720	558
(2) Backfill concrete	cu.m	160	1,930	309
(3) Consolidation grouting	L.S	-		81
(4) Steel liner	ton	2,100	270	567
(5) Work adit	L.S	-		190
(6) Miscellaneous work	L.S	-		47
Sub-total				1,752
6. Power Station				
(1) Excavation	cu.m	7	9,880	69
(2) Concrete	cu.m	140	3,760	526
(3) Reinforcing bar	ton	1,100	195	215
(4) Superstructure	cu.m	180	9,300	1,674
(6) Generating equipment	L.S	-		5,000
(7) T/L&S/S	L.S	-		1,140
(8) Miscellaneous work	L.S	-		41
Sub-total				8,665
7. Access Road				
(1) New construction road	Km	200,000	1.4	280
(2) Improvement of existing road	Km	90,000	0	0
(3) Bridge	m	5,000	0	0
(4) Miscellaneous work	L.S	-		14
Sub-total				294
Total of Item I				22,736
II. Compensation Cost				
1. Relocation Road				
(1) Road	km	200,000	0.2	40
(2) Bridge	m	5,000	40	200
Sub-total				240
2. Compensation				
(1) Land	sq.km	115,000	0.014	2
(2) House	nos.	7,350	9	66
Sub-total				68
Total of Item II				308
III. Administration Cost				1,137
IV. Engineering Service Cost				3,480
V. Physical Contingency				3,410
VI. Grand Total				31,071

Table III.6.1 MAJOR CONSTRUCTION PLANT AND EQUIPMENT FOR SALTO PILÃO (1) HYDROPOWER SCHEME

Equipment	Capacity	No. of Units
Aggregate plant	80 t/h	1
- do -	60 t/h	1
Concrete mixing plant	90 m3/h	1
- do -	50 m3/h	1
Bulldozer w/ripper	32 t	4
- do -	21 t	4
Tractor shovel	2.2 m3	4
- do -	1.6 m3	3
- do -	0.7 m3	6
Drill jumbo	6-boom	4
Raise climber	SEH-5E	1
Crawler drill	10 m3	10
Backhoe	0.4 m3	2
Dump truck	11 t	38
-do -	4 t	30
Motor grader	3.8 m	1
Macadom roller	10 t	1
Concrete pump	60 m3/h	2
Concrete pump car	70 m3/h	3
- do -	40 m3/h	3
Agitator truck	4.4 m3	37
Boring machine	5.5 kW	10
Grout mixer	200 l x 2	15
Grout pump	7.5 kW	15
Truck crane	35 t	2
Tower crane	12 t	1
Winch	20 HP	2

Table III.6.2 Construction Cost of Salto Pilão (1) Hydropower Scheme (1/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
I. Direct Cost				
1. River Diversion Works				
(1) Diversion tunnel				
(a) Portal excavation, common	cu.m	5	55,300	277
(b) Portal excavation, rock	cu.m	20	37,000	740
(c) Tunnel excavation	cu.m	90	68,300	6,147
(d) Backfill	cu.m	4	2,800	11
(e) Portal concrete	cu.m	170	2,920	496
(f) Lining concrete	cu.m	170	23,700	4,029
(g) Reinforcing bar	ton	1,300	414	538
(h) Steel support	ton	2,500	224	560
(i) Diversion closure gate	ton	6,000	9	54
(j) Plug concrete	cu.m	100	1,140	114
(2) Primary cofferdam	cu.m	6	21,300	128
(3) Secondary cofferdam				
(a) Excavation, common	cu.m	5	50,500	253
(b) Excavation, rock	cu.m	20	6,540	131
(c) Concrete	cu.m	100	32,300	3,230
(4) Downstream cofferdam	cu.m	6	36,100	217
(5) Miscellaneous works	L.S			844
Sub-total of Item 1				17,768
2. Dam and Spillway				
(1) Excavation, common	cu.m	5	86,100	431
(2) Excavation, rock	cu.m	20	24,300	486
(3) Backfill	cu.m	4	7,600	30
(4) Mass concrete	cu.m	100	66,500	6,650
(5) Structural concrete	cu.m	170	4,050	689
(6) Reinforcing bar	ton	1,300	125	163
(7) Spillway bridge	L.S			35
(8) Bridge for maintenance	L.S			7
(9) Curtain grouting	L.S			337
(10) Consolidation grouting	L.S			45
(11) Spillway gate	ton	6,000	1,095	6,570
(12) Sand scouring gate	ton	6,000	140	840
(13) Miscellaneous works	L.S			444
Sub-total of Item 2				16,725
3. Intake and Sand Trap Basin				
(1) Excavation, common	cu.m	5	126,300	632
(2) Excavation, rock	cu.m	20	190,600	3,812
(3) Backfill	cu.m	4	18,200	73
(4) Mass concrete	cu.m	100	11,400	1,140
(5) Structural concrete	cu.m	170	29,400	4,998
(6) Reinforcing bar	ton	1,300	380	494
(7) Intake gate	ton	6,000	196	1,176
(8) Trashrack	ton	3,500	133	466
(9) Sand flush gate	ton	6,000	4	24
(10) Rim grouting	L.S			205
(11) Miscellaneous works	L.S			568
Sub-total of Item 3				13,586

(to be continued)

Table III.6.2 Construction Cost of Salto Pilão (1) Hydropower Scheme (2/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
4. Headrace Tunnel				
(1) Tunnel excavation	cu.m	90	216,870	19,518
(2) Lining concrete	cu.m	170	57,260	9,734
(3) Reinforcing bar	ton	1,300	2,020	2,626
(4) Steel support	ton	2,500	315	788
(5) Consolidation grouting	L.S			154
(6) Work adit	L.S			1,586
(7) Miscellaneous works	L.S			1,641
Sub-total of Item 4				36,047
5. Surge Tank				
(1) Open excavation, common	cu.m	5	99,600	498
(2) Open excavation, rock	cu.m	20	64,600	1,292
(3) Shaft excavation	cu.m	130	19,000	2,470
(4) Lining concrete	cu.m	190	3,300	627
(5) Structural concrete	cu.m	170	850	145
(6) Reinforcing bar	ton	1,300	195	254
(7) Consolidation grouting	L.S			22
(8) Surge tank gate	ton	6,000	65	390
(9) Miscellaneous works	L.S			265
Sub-total of Item 5				5,962
6. Penstock				
(1) Open excavation, rock	cu.m	20	10,400	208
(2) Shaft excavation	cu.m	130	8,650	1,125
(3) Backfill	cu.m	4	8,600	34
(4) Open concrete	cu.m	170	1,450	247
(5) Backfill concrete	cu.m	200	2,640	528
(6) Reinforcing bar	ton	1,300	15	20
(7) Consolidation grouting	L.S			106
(8) Steel liner	ton	3,800	1,710	6,498
(9) Work adit	L.S			446
(10) Miscellaneous works	L.S			438
Sub-total of Item 6				9,649
7. Power Station & Tailrace				
(1) Excavation, common	cu.m	5	216,700	1,084
(2) Excavation, rock	cu.m	20	65,700	1,314
(3) Backfill	cu.m	4	6,000	24
(4) Concrete	cu.m	190	24,300	4,617
(5) Backfill concrete	cu.m	100	700	70
(6) Reinforcing bar	ton	1,300	830	1,079
(7) Superstructure	cu.m	190	15,600	2,964
(8) Draft tube gate	ton	6,000	20	120
(9) Generating equipment	L.S			26,990
(10) T/L & S/S	L.S			3,417
(11) Tailrace bridge	L.S			13
(12) Miscellaneous works	L.S			410
Sub-total of Item 7				42,102

(to be continued)

Table III.6.2 Construction Cost of Salto Pilão (1) Hydropower Scheme (3/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
8. Access Road				
(1) New construction road	Km	360,000	3.5	1,260
(2) Improvement of existing road	Km	180,000	1.5	270
(3) Bridge	m	5,700	10	57
(4) Miscellaneous works	L.S			79
Sub-total of Item 8				1,666
Total of Item I				143,506
II. Compensation Cost				
1. Land				
(1) Agricultural land	sq.Km	220,000	0.15	33
(2) Forest	sq.Km	80,000	0.10	8
(3) Grass land	sq.Km	195,328	0.067	13
(4) Reforestation	sq.Km	150,000	0.017	3
Sub-total of Item 1				57
2. House				
(1) Wooden house	Nos.	9	12,672	114
Total of Item II				171
III. Administration Cost				7,175
IV. Engineering Service Cost				4,100
V. Physical Contingency (15 %)				23,243
Grand Total				178,195

Table III.6.3 DISBURSEMENT SCHEDULE

(Unit: thousand US$)

Year/Scheme	Salto Pilão (1)	Dalbergia	Benedito Novo
1st	55,398	30,202	17,392
2nd	19,900	23,036	16,076
3rd	54,231	35,807	23,061
4th	48,666	13,152	-
Total	178,195	102,197	56,529

Table III.6.4 MAJOR CONSTRUCTION PLANT AND EQUIPMENT FOR DALBERGIA HYDROPOWER SCHEME

Equipment	Capacity	No. of Units
Aggregate plant	80 t/h	1
- do -	60 t/h	1
Concrete mixing plant	90 m3/h	1
- do -	50 m3/h	1
Bulldozer w/ripper	32 t	4
- do -	21 t	4
Tractor shovel	2.2 m3	4
- do -	1.6 m3	2
Drill jumbo	4-boom	6
Raise climber	SEH-5E	1
Crawler drill	10 m3	10
Rocker shovel	0.4 m3	7
Battery locomotive	6 t	19
Muck car	4.5 m3	21
Train loader	200 t/h	6
Backhoe	0.2 m3	2
Dump truck	11 t	33
Motor grader	3.8 m	1
Macadom roller	10 t	1
Concrete placer	3 m3	4
Concrete pump car	70 m3/h	2
- do -	45 m3/h	3
- do -	40 m3/h	2
Agitator truck	4.4 m3	33
Agitator car	3 m3	15
Boring machine	5.5 kW	10
Grout mixer	200 l x 2	15
Grout pump	7.5 kW	15
Winch	20 HP	2
Tower crane	12 t	1
Truck crane	35 t	2

Table III.6.5 Construction Cost of Dalbergia Hydropower Scheme (1/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
I. Direct Cost				
1. River Diversion Works				
(1) Diversion tunnel				
(a) Portal excavation, common	cu.m	5	41,500	208
(b) Portal excavation, rock	cu.m	20	41,300	826
(c) Tunnel excavation	cu.m	90	9,330	840
(d) Backfill	cu.m	4	2,100	8
(e) Portal concrete	cu.m	170	3,000	510
(f) Lining concrete	cu.m	170	3,390	576
(g) Reinforcing bar	ton	1,300	110	143
(h) Steel support	ton	2,500	45	113
(i) Diversion closure gate	ton	6,000	7.5	45
(j) Plug concrete	cu.m	100	550	55
(2) Primary cofferdam	cu.m	6	2,000	12
(3) Secondary cofferdam				
(a) Excavation, common	cu.m	5	20,900	105
(b) Excavation, rock	cu.m	20	2,540	51
(c) Concrete	cu.m	100	18,700	1,870
(4) Downstream cofferdam	cu.m	6	11,700	70
(5) Miscellaneous works	L.S			269
Sub-total of Item 1				5,700
2. Dam and Spillway				
(1) Excavation, common	cu.m	5	250,700	1,254
(2) Excavation, rock	cu.m	20	118,000	2,360
(3) Backfill	cu.m	4	16,200	65
(4) Mass concrete	cu.m	100	109,700	10,970
(5) Structural concrete	cu.m	170	3,750	638
(6) Reinforcing bar	ton	1,300	390	507
(7) Riverbed protection	cu.m	70	1,650	116
(8) Spillway bridge	L.S			35
(9) Bridge for maintenance	L.S			7
(10) Curtain grouting	L.S			473
(11) Consolidation grouting	L.S			141
(12) Spillway gate	ton	6,000	680	4,080
(13) Sand scouring gate	ton	6,000	120	720
(14) Miscellaneous works	L.S			828
Sub-total of Item 2				22,193
3. Intake and Sand Trap Basin				
(1) Excavation, common	cu.m	5	51,300	257
(2) Excavation, rock	cu.m	20	39,100	782
(3) Backfill	cu.m	4	4,440	18
(4) Structural concrete	cu.m	170	9,210	1,566
(5) Reinforcing bar	ton	1,300	120	156
(6) Intake gate	ton	6,000	60	360
(7) Trashrack	ton	3,500	50	175
(8) Sand flush gate	ton	6,000	4	24
(9) Miscellaneous works	L.S			139
Sub-total of Item 3				3,476

(to be continued)

Table III.6.5 Construction Cost of Dalbergia Hydropower Scheme (2/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
4. Headrace Tunnel				
(1) Tunnel excavation	cu.m	110	149,340	16,427
(2) Lining concrete	cu.m	180	35,820	6,448
(3) Reinforcing bar	ton	1,300	1,060	1,378
(4) Steel support	ton	2,500	110	275
(5) Consolidation grouting	L.S			106
(6) Work adit	L.S			1,982
(7) Miscellaneous works	L.S			1,232
Sub-total of Item 4				27,848
5. Surge Tank				
(1) Open excavation, common	cu.m	5	39,700	199
(2) Open excavation, rock	cu.m	20	2,900	58
(3) Shaft excavation	cu.m	130	11,600	1,508
(4) Lining concrete	cu.m	190	2,700	513
(5) Structural concrete	cu.m	170	670	114
(6) Reinforcing bar	ton	1,300	160	208
(7) Consolidation grouting	L.S			24
(8) Surge tank gate	ton	6,000	45	270
(9) Miscellaneous works	L.S			131
Sub-total of Item 5				3,025
6. Penstock				
(1) Open excavation, common	cu.m	5	27,300	137
(2) Open excavation, rock	cu.m	20	9,500	190
(3) Shaft excavation	cu.m	130	5,100	663
(4) Backfill	cu.m	4	34,900	140
(5) Open concrete	cu.m	170	1,540	262
(6) Backfill concrete	cu.m	200	1,810	362
(7) Reinforcing bar	ton	1,300	16	21
(8) Consolidation grouting	L.S			95
(9) Steel liner	ton	3,800	645	2,451
(10) Work adit	L.S			280
(11) Miscellaneous works	L.S			216
Sub-total of Item 6				4,816
7. Power Station & Tailrace				
(1) Excavation, common	cu.m	5	24,500	123
(2) Excavation, rock	cu.m	20	5,500	110
(3) Backfill	cu.m	4	5,000	20
(4) Embankment	cu.m	6	22,000	132
(5) Concrete	cu.m	190	14,600	2,774
(6) Backfill concrete	cu.m	100	620	62
(7) Reinforcing bar	ton	1,300	350	455
(8) Superstructure	cu.m	190	5,600	1,064
(9) Draft tube gate	ton	6,000	6	36
(10) Generating equipment	L.S			7,363
(11) T/L & S/S	L.S			654
(12) Miscellaneous works	L.S			184
Sub-total of Item 7				12,976

(to be continued)

Table III.6.5 Construction Cost of Dalbergia Hydropower Scheme (3/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
8. Access Road				
(1) New construction road	Km	360,000	1.85	666
(2) Improvement of existing road	Km	180,000	0.15	27
(3) Bridge	m	5,700	20	114
(4) Miscellaneous works	L.S			40
Sub-total of Item 8				847
Total of Item I				80,880
II. Compensation Cost				
1. Land				
(1) Forest	sq.Km	120,000	0.065	8
(2) Grass land	sq.Km	601,625	0.183	110
Sub-total of Item 1				118
2. House				
(1) Wooden house	Nos.	12,946	5	65
Total of Item II				183
III. Administration Cost				4,044
IV. Engineering Service Cost				3,760
V. Physical Contingency (15 %)				13,330
Grand Total				102,197

Table III.6.6 MAJOR CONSTRUCTION PLANT AND EQUIPMENT FOR BENEDITO NOVO HYDROPOWER SCHEME

Equipment	Capacity	No. of Units
Aggregate plant	60 t/h	2
Concrete mixing plant	70 m3/h	1
- do -	50 m3/h	1
Bulldozer w/ripper	32 t	4
- do -	21 t	4
Tractor shovel	2.2 m3	4
- do -	1.6 m3	2
Drill jumbo	2-boom	2
Raise climber	SEH-5E	1
Crawler drill	10 m3	8
Rocker shovel	0.4 m3	3
Battery locomotive	6 t	6
Muck car	3.0 m3	9
Train loader	200 t/h	2
Backhoe	0.2 m3	2
Dump truck	11 t	16
Motor grader	3.8 m	1
Macadom roller	10 t	1
Concrete placer	3 m3	3
Concrete pump car	45 m3/h	4
- do -	40 m3/h	3
Agitator truck	4.4 m3	20
Agitator car	3 m3	6
Boring machine	5.5 kW	10
Grout mixer	200 l x 2	10
Grout pump	7.5 kW	10
Winch	20 HP	2
Tower crane	12 t	1
Truck crane	35 t	2

Table III.6.7 Construction Cost of Benedito Novo Hydropower Scheme (1/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
I. Direct Cost				
1. River Diversion Works				
(1) Diversion tunnel				
(a) Portal excavation, common	cu.m	5	5,500	28
(b) Portal excavation, rock	cu.m	20	6,000	120
(c) Tunnel excavation	cu.m	100	4,200	420
(d) Backfill	cu.m	4	1,700	7
(e) Portal concrete	cu.m	170	1,500	255
(f) Lining concrete	cu.m	180	1,600	288
(g) Reinforcing bar	ton	1,300	55	72
(h) Steel support	ton	2,500	30	75
(i) Diversion closure gate	ton	6,000	5.5	33
(j) Plug concrete	cu.m	100	320	32
(2) Primary cofferdam	cu.m	6	3,000	18
(3) Secondary cofferdam				
(a) Excavation, common	cu.m	5	6,400	32
(b) Excavation, rock	cu.m	20	900	18
(c) Concrete	cu.m	100	3,930	393
(4) Miscellaneous works	L.S			88
Sub-total of Item 1				1,878
2. Dam and Spillway				
(1) Excavation, common	cu.m	5	43,000	215
(2) Excavation, rock	cu.m	20	25,500	510
(3) Mass concrete	cu.m	100	50,300	5,030
(4) Structural concrete	cu.m	170	2,800	476
(5) Reinforcing bar	ton	1,300	85	111
(6) Riverbed protection	cu.m	70	2,050	144
(7) Spillway bridge	L.S			14
(8) Bridge for maintenance	L.S			3
(9) Curtain grouting	L.S			87
(10) Consolidation grouting	L.S			27
(11) Spillway gate	ton	6,000	650	3,900
(12) Sand scouring gate	ton	6,000	70	420
(13) Miscellaneous works	L.S			331
Sub-total of Item 2				11,267
3. Intake and Sand Trap Basin				
(1) Excavation, common	cu.m	5	83,200	416
(2) Excavation, rock	cu.m	20	31,100	622
(3) Backfill	cu.m	4	20,640	83
(4) Mass concrete	cu.m	100	2,540	254
(5) Structural concrete	cu.m	170	18,800	3,196
(6) Reinforcing bar	ton	1,300	235	306
(7) Intake gate	ton	6,000	120	720
(8) Trashrack	ton	3,500	72	252
(9) Sand flush gate	ton	6,000	4	24
(10) Miscellaneous works	L.S			244
Sub-total of Item 3				6,116

(to be continued)

Table III.6.7 Construction Cost of Benedito Novo Hydropower Scheme (2/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
4. Headrace Tunnel				
(1) Tunnel excavation	cu.m	140	21,520	3,013
(2) Lining concrete	cu.m	192	6,100	1,171
(3) Reinforcing bar	ton	1,300	155	202
(4) Steel support	ton	2,500	51	128
(5) Consolidation grouting	L.S			66
(6) Work adit	L.S			118
(7) Miscellaneous works	L.S			229
Sub-total of Item 4				4,926
5. Surge Tank				
(1) Open excavation, common	cu.m	5	20,000	100
(2) Open excavation, rock	cu.m	20	6,300	126
(3) Shaft excavation	cu.m	130	3,300	429
(4) Lining concrete	cu.m	190	990	188
(5) Structural concrete	cu.m	170	80	14
(6) Reinforcing bar	ton	1,300	55	72
(7) Consolidation grouting	L.S			18
(8) Miscellaneous works	L.S			47
Sub-total of Item 5				994
6. Penstock				
(1) Open excavation, common	cu.m	5	7,150	36
(2) Open excavation, rock	cu.m	20	5,860	117
(3) Shaft excavation	cu.m	130	3,000	390
(4) Backfill	cu.m	4	10,600	42
(5) Open concrete	cu.m	170	1,050	179
(6) Backfill concrete	cu.m	200	1,290	258
(7) Reinforcing bar	ton	1,300	10	13
(8) Consolidation grouting	L.S			84
(9) Steel liner	ton	3,800	350	1,330
(10) Work adit	L.S			201
(11) Miscellaneous works	L.S			122
Sub-total of Item 6				2,772
7. Power Station & Tailrace				
(1) Excavation, common	cu.m	5	9,600	48
(2) Excavation, rock	cu.m	20	4,000	80
(3) Backfill	cu.m	4	700	3
(4) Embankment	cu.m	6	5,100	31
(5) Concrete	cu.m	190	9,000	1,710
(6) Backfill concrete	cu.m	100	1,300	130
(7) Reinforcing bar	ton	1,300	220	286
(8) Superstructure	cu.m	190	4,600	874
(9) Draft tube gate	ton	6,000	4	24
(10) Generating equipment	L.S			5,650
(11) T/L & S/S	L.S			2,657
(12) Miscellaneous works	L.S			114
Sub-total of Item 7				11,607

(to be continued)

Table III.6.7 Construction Cost of Benedito Novo Hydropower Scheme (3/3)

Work Item	Unit	Unit Price (US$)	Quantity	Amount (thousand US$)
8. Relocation Road				
(1) Excavation, common	cu.m	5	71,600	358
(2) Excavation, rock	cu.m	20	46,200	924
(3) Slope Protection	sq.m	3.3	11,400	38
(4) Bridge	m	5,700	20	114
(5) Miscellaneous works	L.S			72
Sub-total of Item 8				1,505
9. Access Road				
(1) New construction road	Km	360,000	1.4	504
(2) Improvement of existing road	Km	180,000	0	0
(3) Bridge	m	5,700	0	0
(4) Miscellaneous works	L.S			25
Sub-total of Item 9				529
Total of Item I				41,593
II. Compensation Cost				
1. Land				
(1) Agricultural land	sq.Km	675,000	0.005	3
(2) Forest	sq.Km	400,000	0.011	4
(3) Urban area	sq.Km	7,700,000	0.012	92
Sub-total of Item 1				100
2. House				
(1) Brick house	Nos.	37,586	7	263
(2) Wooden house	Nos.	13,300	6	80
Sub-total of Item 2				343
3. Existing Power Station	KW	500	3,120	1,560
Total of Item II				2,003
III. Administration Cost				2,080
IV. Engineering Service Cost				3,480
V. Physical Contingency (15 %)				7,373
Grand Total				56,529

FIGURES

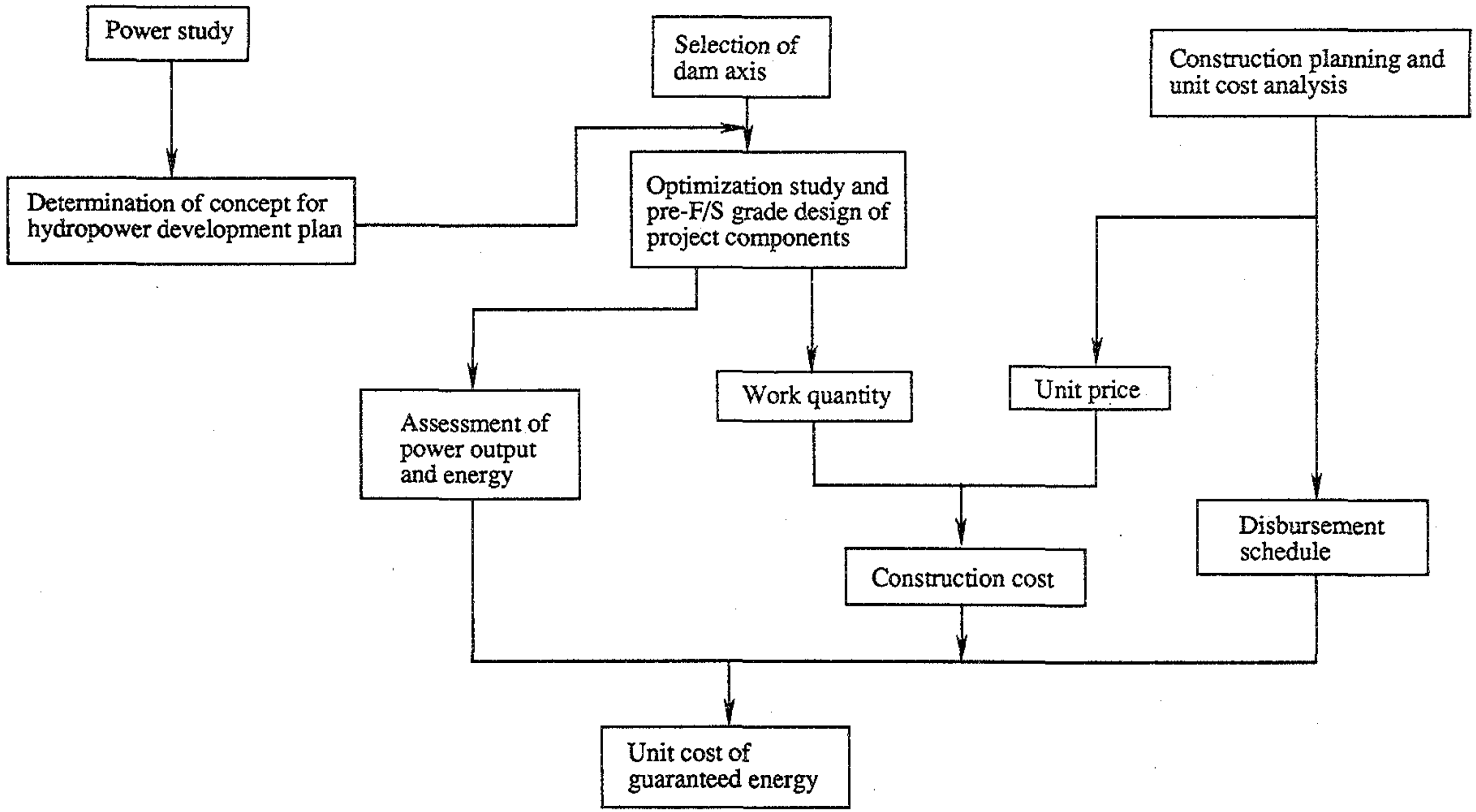
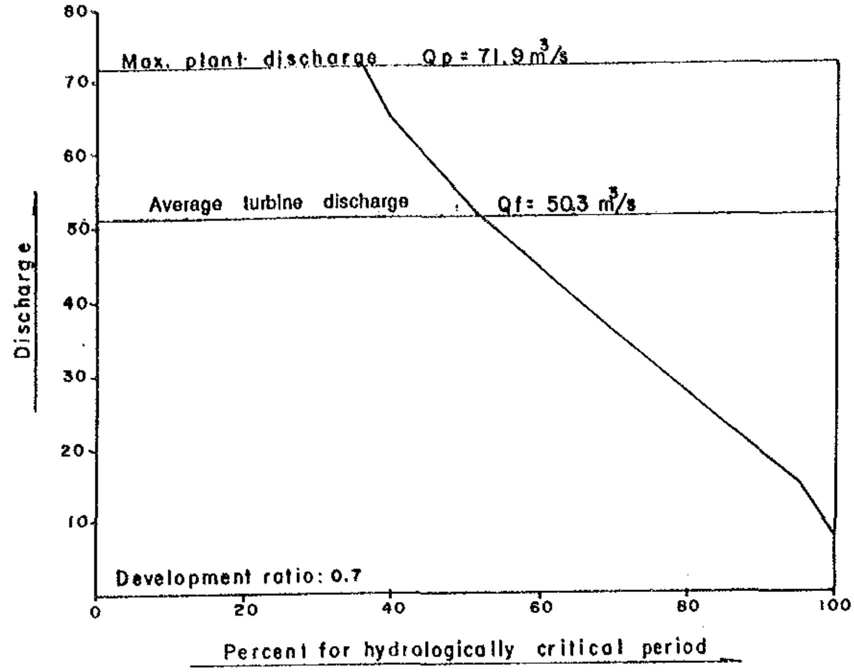
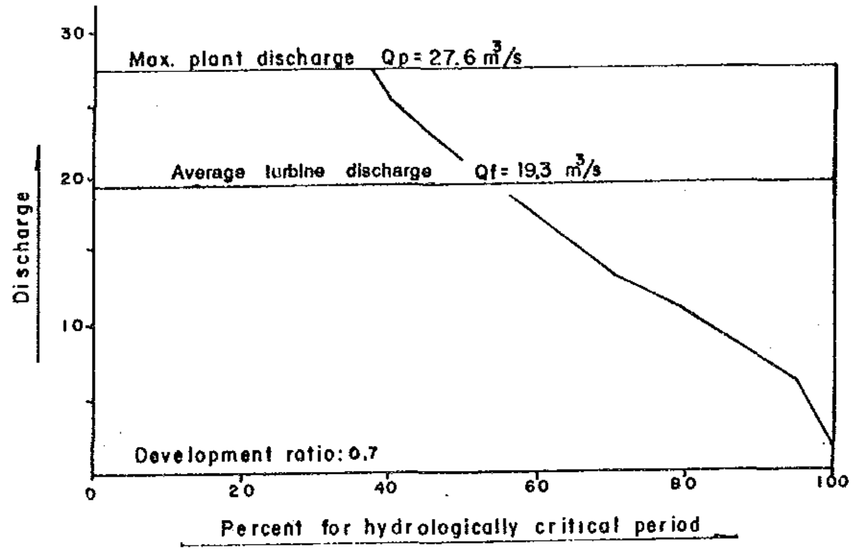


Fig. III.2.1 STUDY PROCEDURE



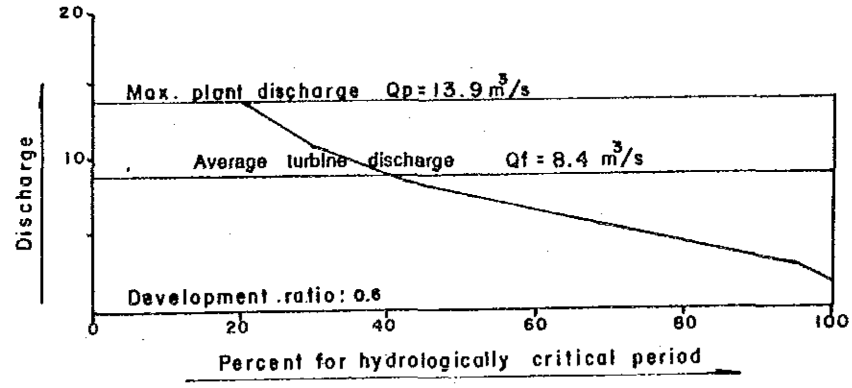
Salto Pilão (I) Scheme

Excess Percent (%)	Flow Discharge (m³/s)
1	562.0
5	318.0
10	204.0
15	155.0
20	121.0
30	83.9
40	64.6
50	53.1
60	44.4
70	35.7
80	27.4
90	18.9
95	15.0
100	7.3



Dalbergia Scheme

Excess Percent (%)	Flow Discharge (m³/s)
1	280.3
5	145.0
10	82.8
15	60.6
20	48.2
30	33.2
40	25.4
50	20.8
60	17.0
70	13.1
80	10.6
90	7.5
95	5.7
100	1.2



Benedito Novo Scheme

Excess Percent (%)	Flow Discharge (m³/s)
1	75.5
5	31.7
10	20.6
15	16.5
20	14.0
30	10.8
40	9.1
50	7.6
60	6.6
70	5.6
80	4.6
90	3.4
95	2.8
100	1.9

Fig. III.2.2 RELATION BETWEEN MAXIMUM PLANT DISCHARGE AND AVERAGE TURBINE DISCHARGE

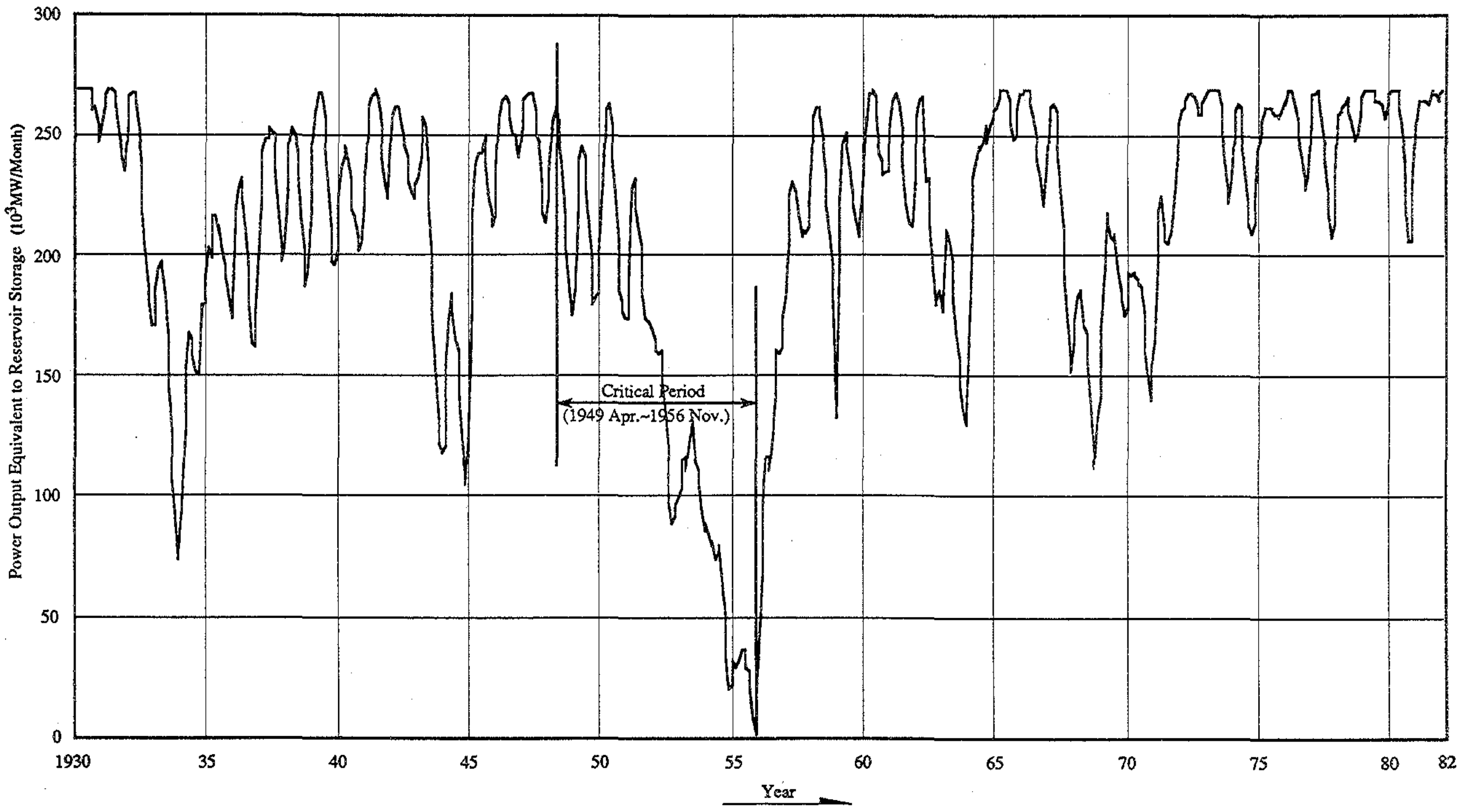
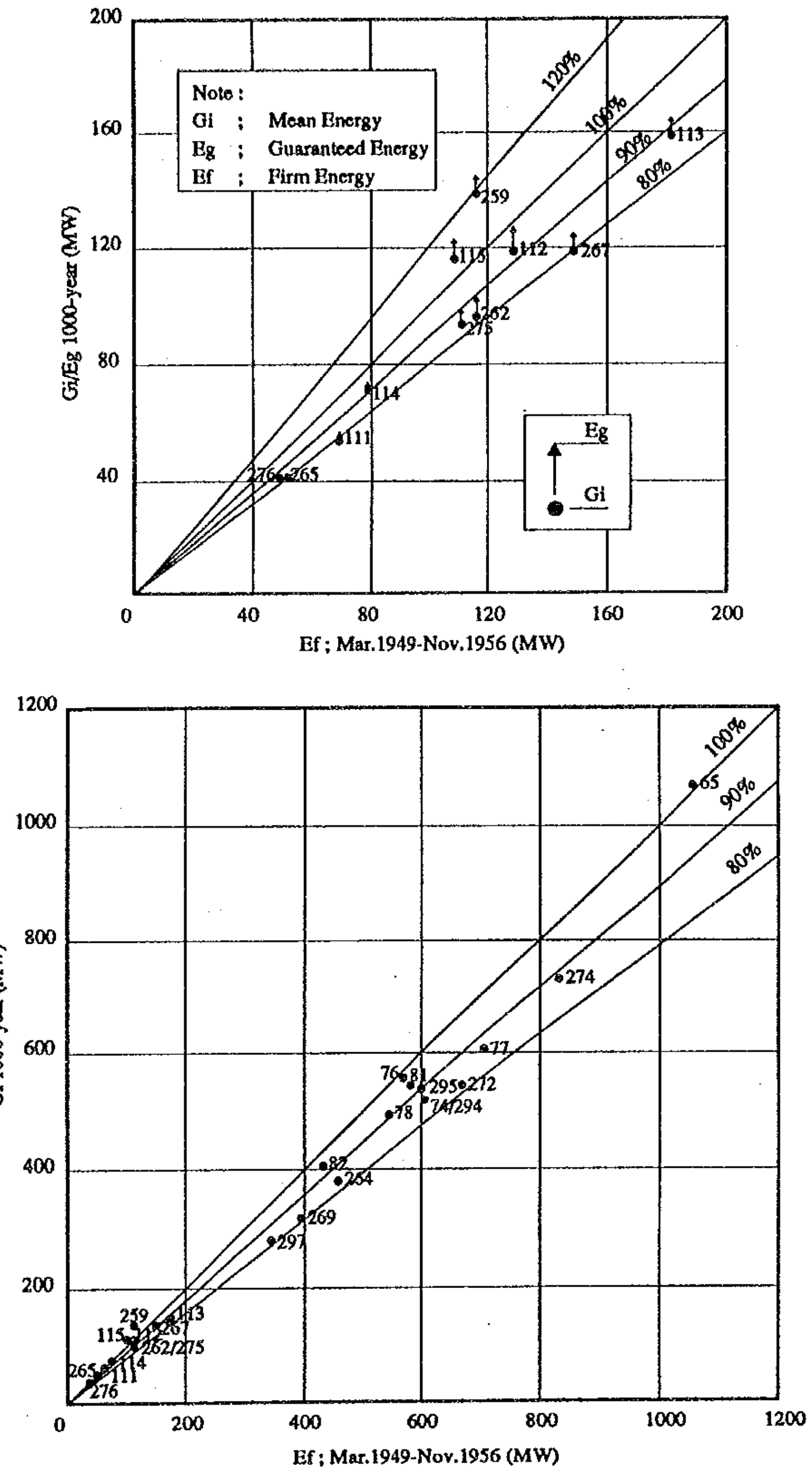


Fig. III.2.3 CRITICAL PERIOD IN INTERCONNECTION OF SOUTH AND SOUTHEAST SYSTEMS



Note ; Figures show the number code of station.

Fig. III.2.4 FIRM ENERGY BASED ON CRITICAL MAR.1949-NOV.1956 RECORD V.S. MEAN ENERGY PROBABLY OCCURING LESS THAN FIVE % BASED ON 1000-YEAR SYNTHETIC FLOW

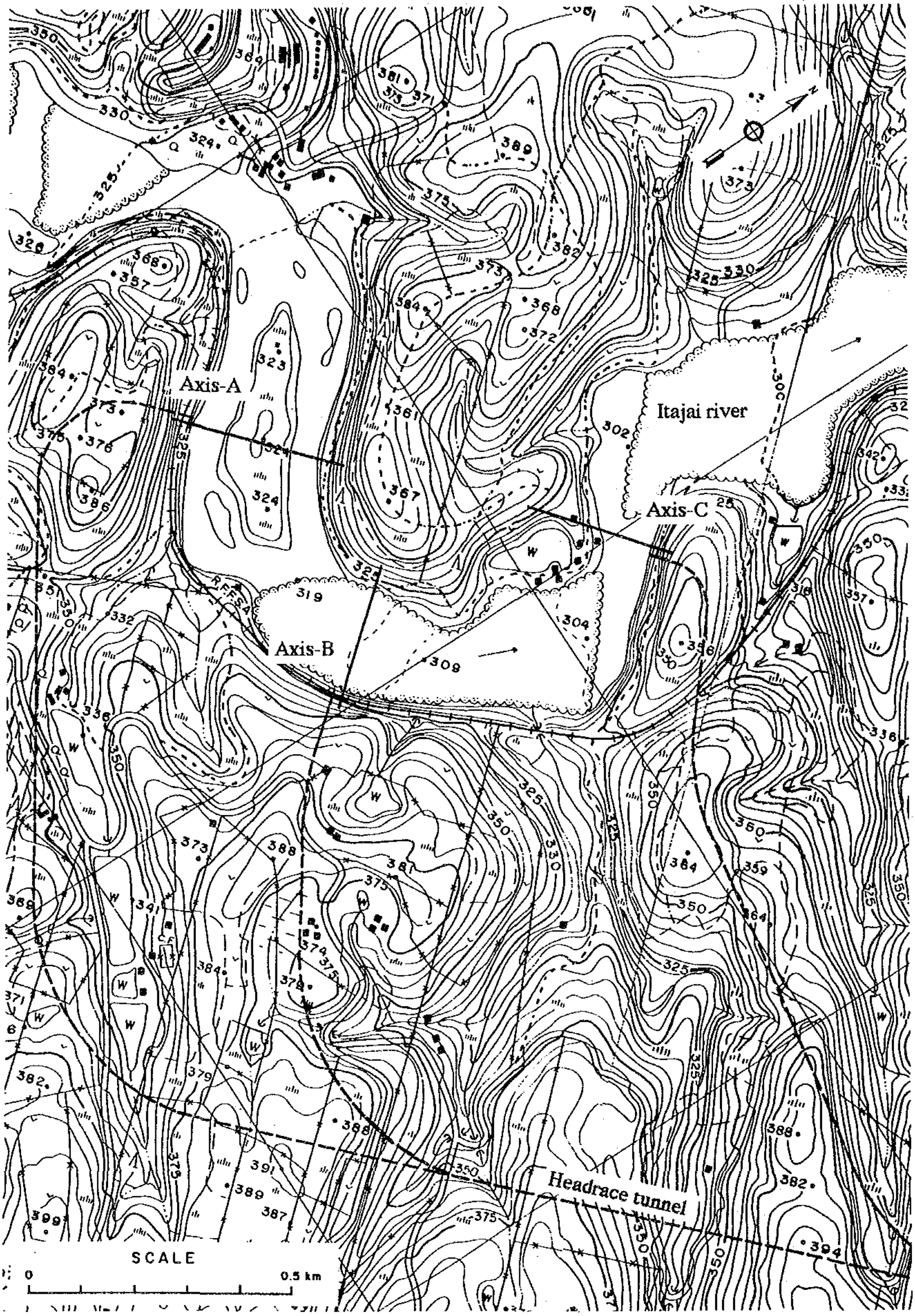


Fig. III.3.1 LOCATION OF ALTERNATIVE DAM AXES FOR SALTO PILÃO (1) HYDROPOWER SCHEME

Fig. III.3.2 GENERAL LAYOUT OF SALTO PILÃO (1) HYDROPOWER SCHEME FOR COMPARATIVE STUDY

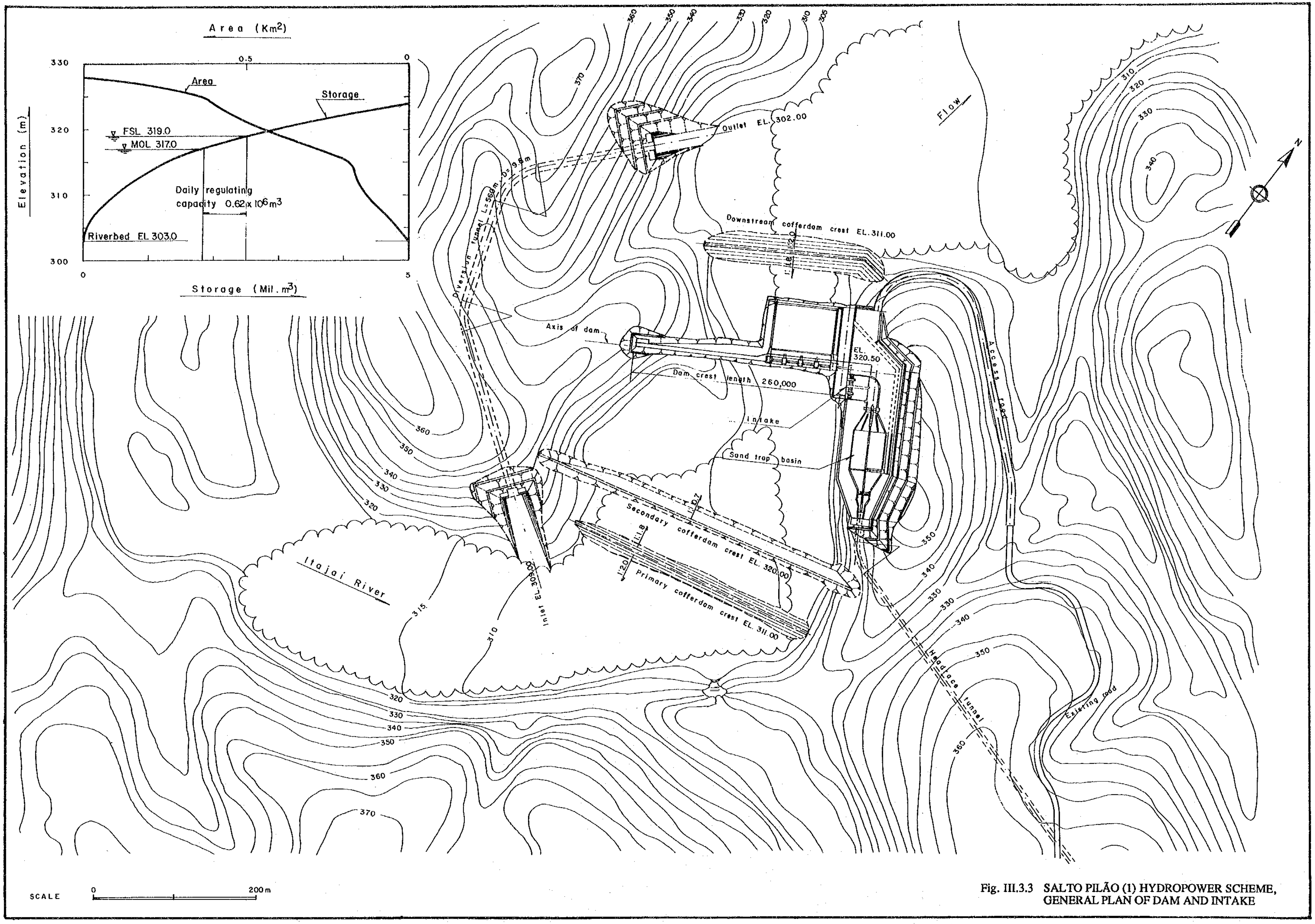


Fig. III.3.3 SALTO PILÃO (1) HYDROPOWER SCHEME, GENERAL PLAN OF DAM AND INTAKE

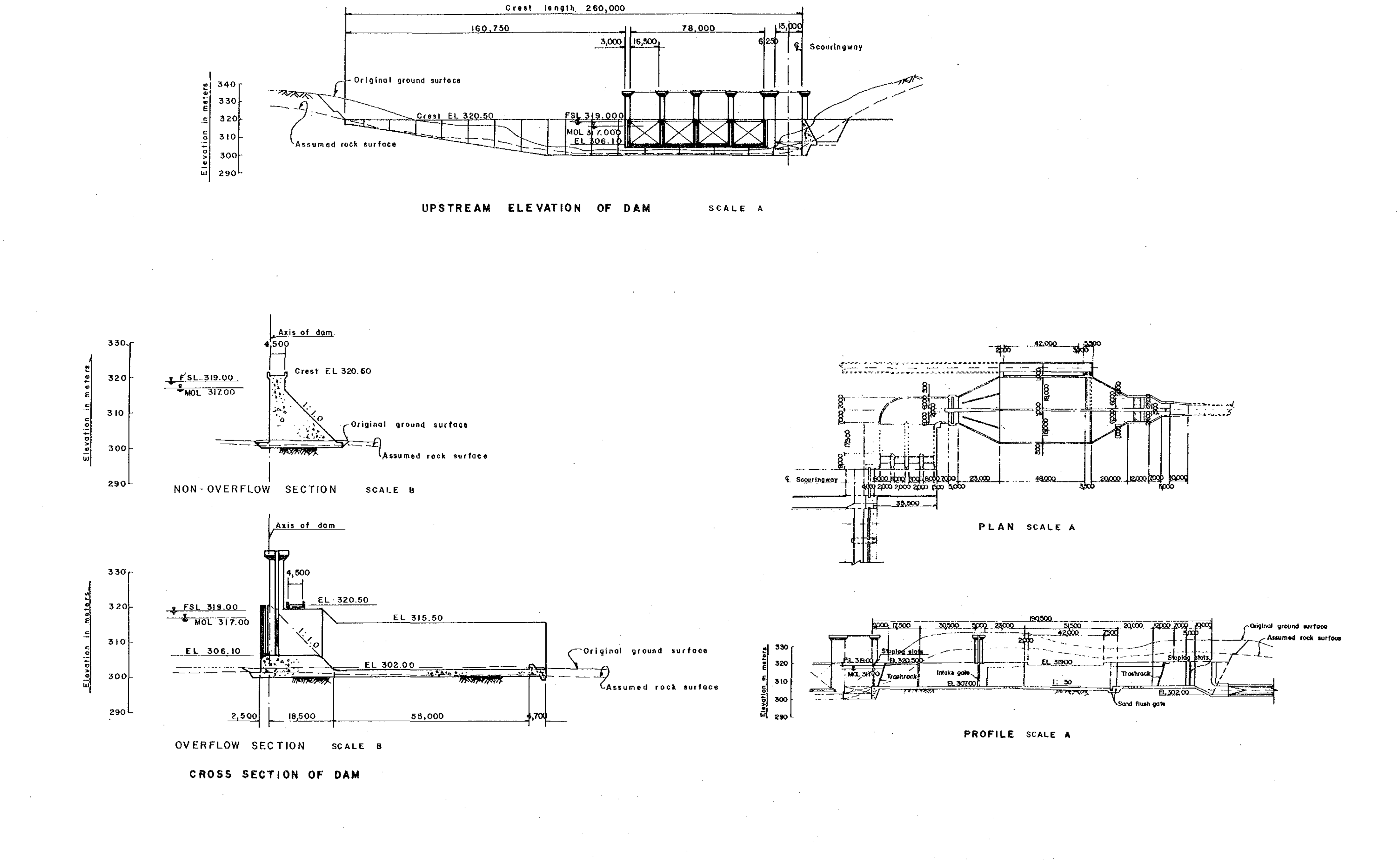
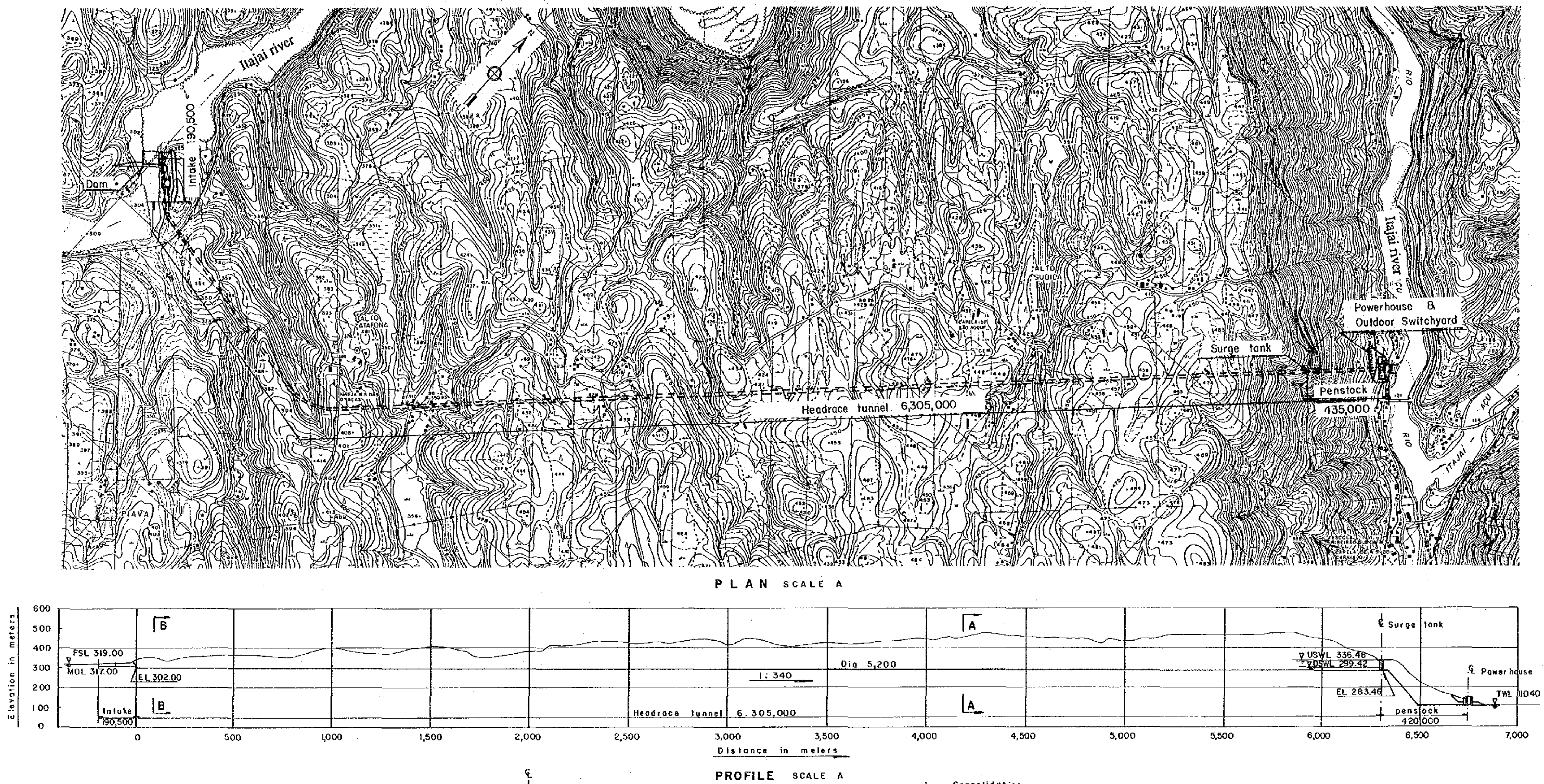


Fig. III 3.4 SALTO PILÃO (1) HYDROPOWER SCHEME, DAM AND INTAKE



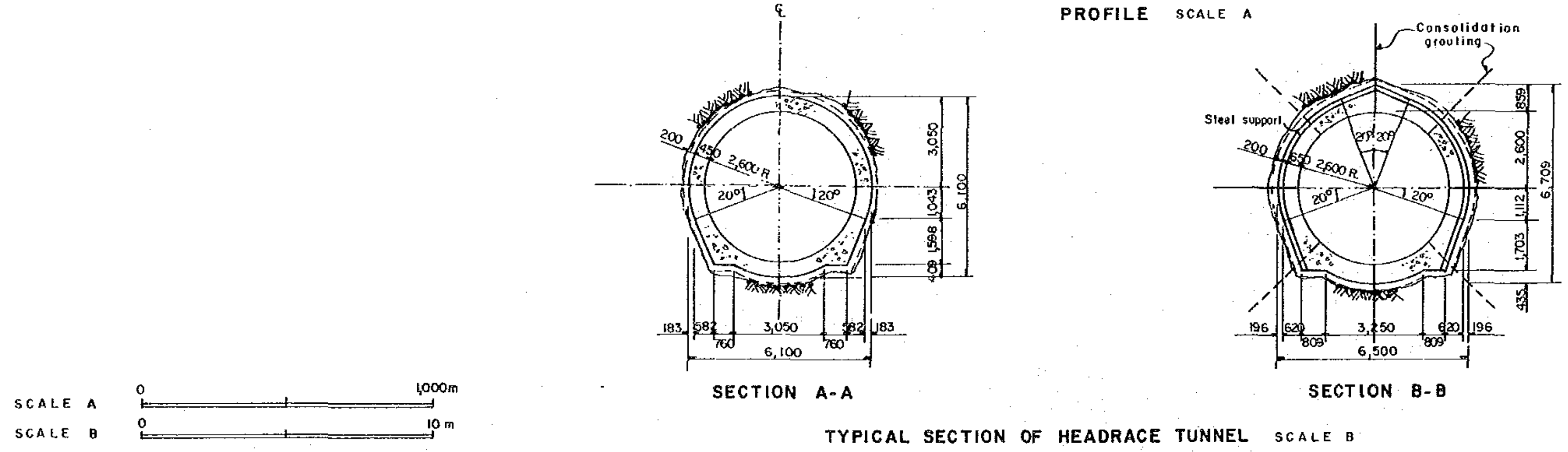
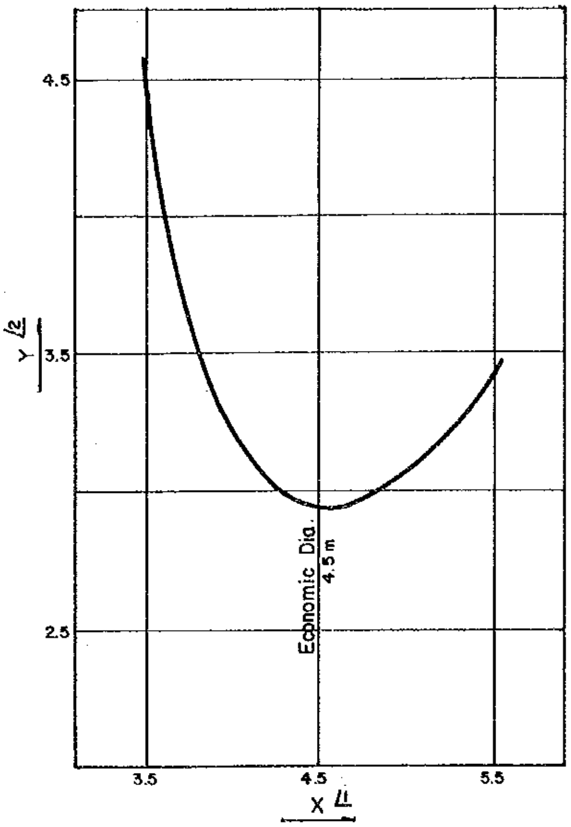
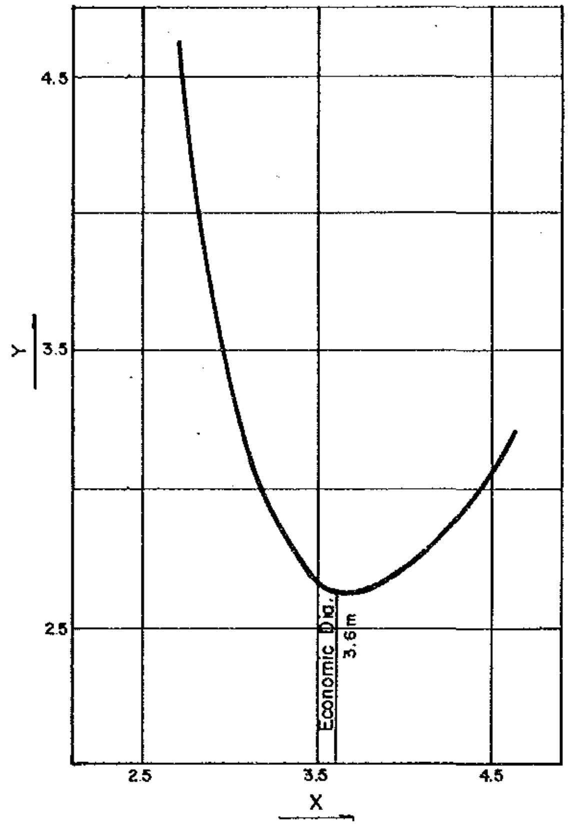


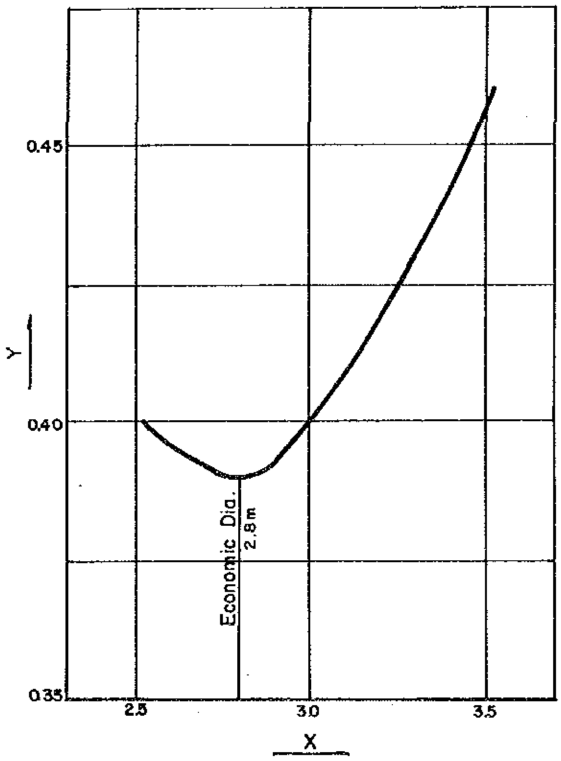
Fig. III.3.5 SALTO PILÃO (1) HYDROPOWER SCHEME, GENERAL PLAN AND PROFILE OF WATERWAY



Salto Pilão (I) Scheme



Dalbergia Scheme



Benedito Novo Scheme

Notes : /1 X means diameter of headrace tunel in meter.

/2 Y means sum of annual cost based on construction cost and annual tariff equivalent to loss head in mil. US$.

Fig. III.3.6 ECONOMIC DIAMETER OF HEADRACE TUNNEL

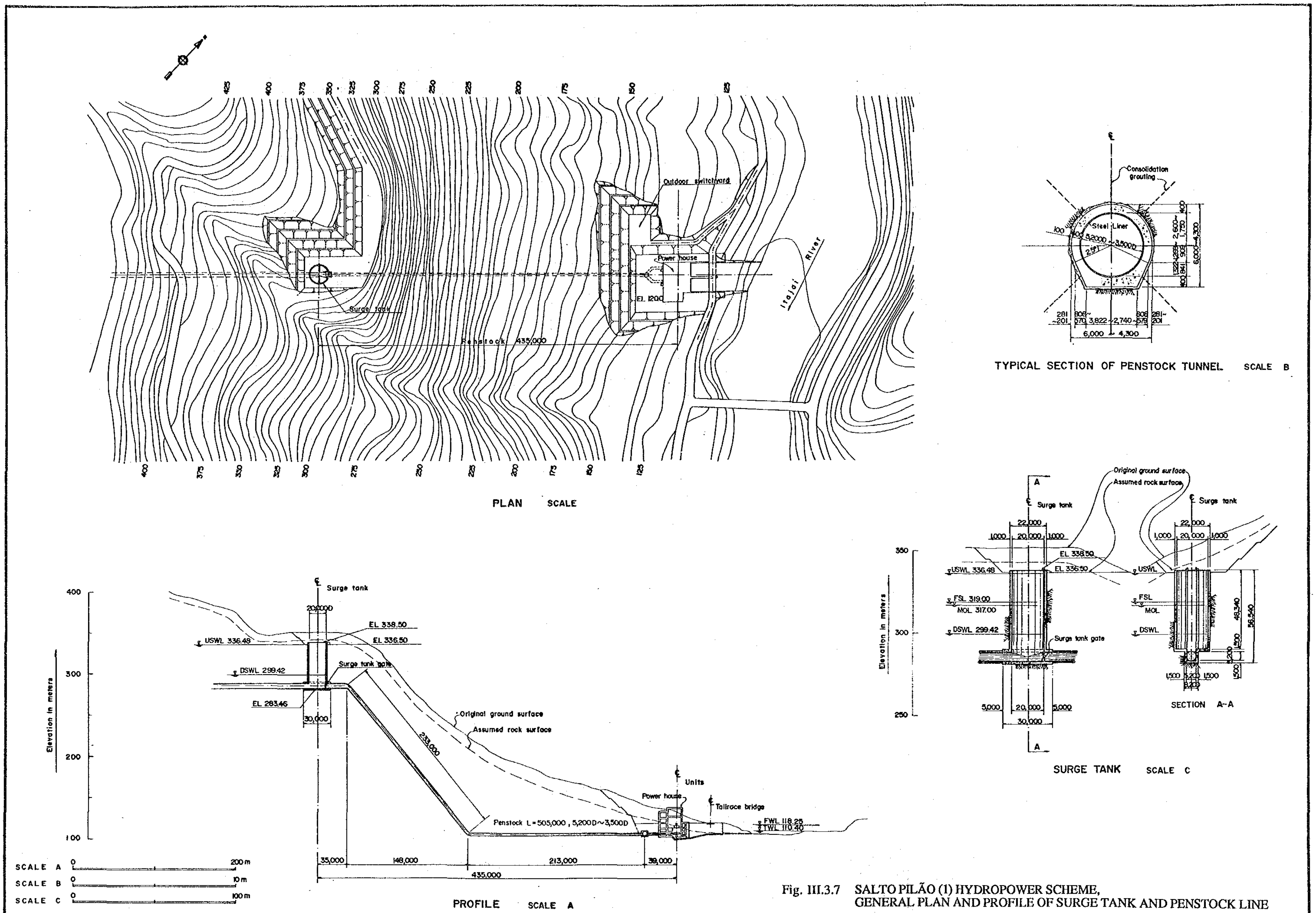
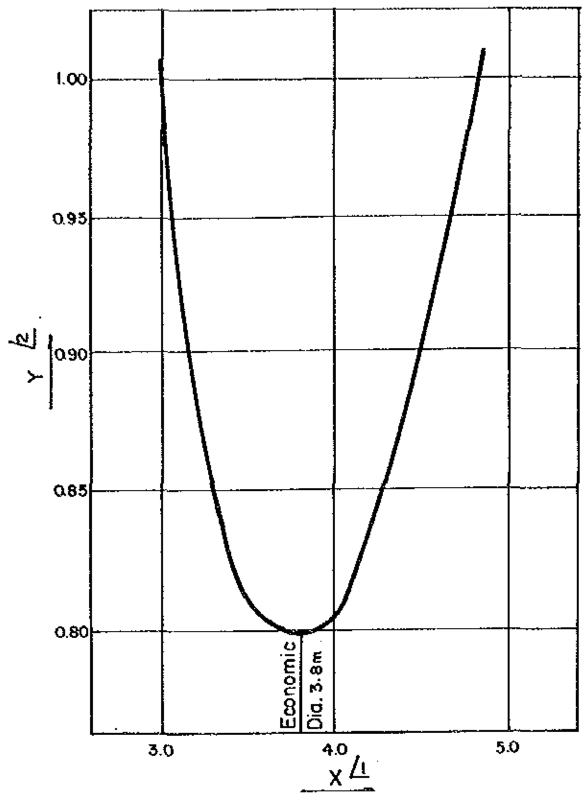
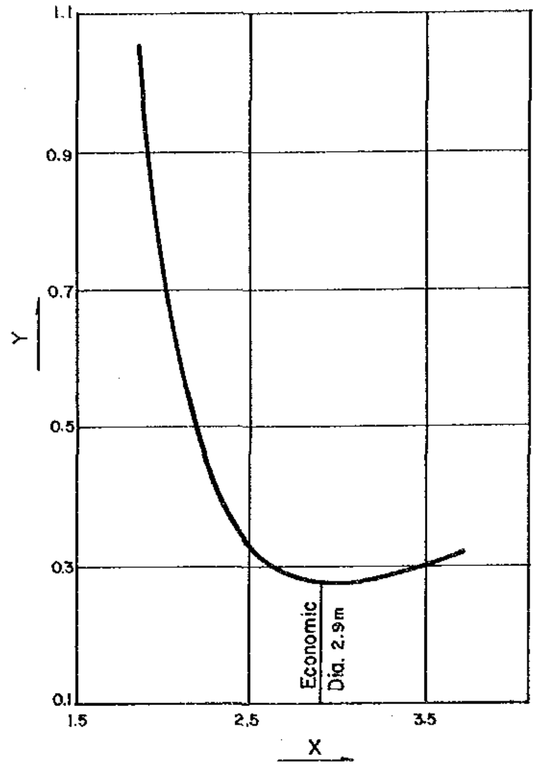


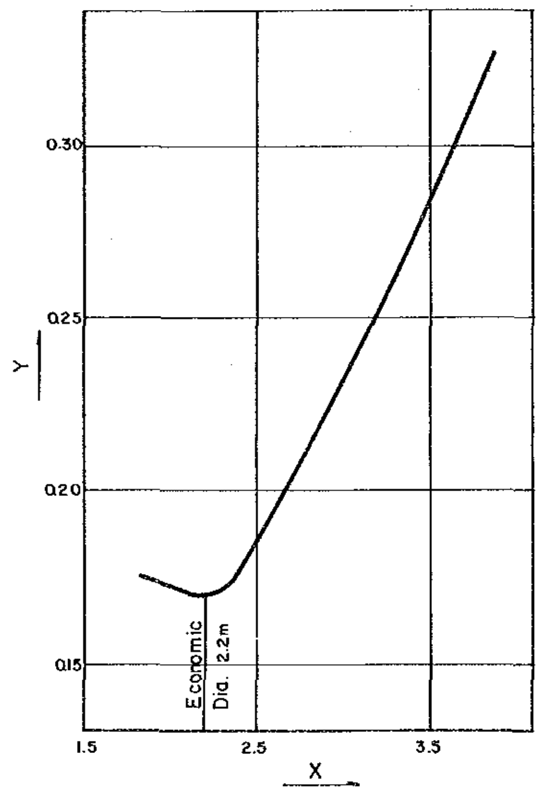
Fig. III.3.7 SALTO PILÃO (1) HYDROPOWER SCHEME, GENERAL PLAN AND PROFILE OF SURGE TANK AND PENSTOCK LINE



Salto Pilão (I) Scheme



Dalbergia Scheme



Benedito Novo Scheme

Notes : /1 X means average diameter of penstock in meter.

/2 Y means sum of annual cost based on construction cost and annual tariff equivalent to loss head in mil US$.

Fig. III. 3. 8 ECONOMIC DIAMETER OF PENSTOCK

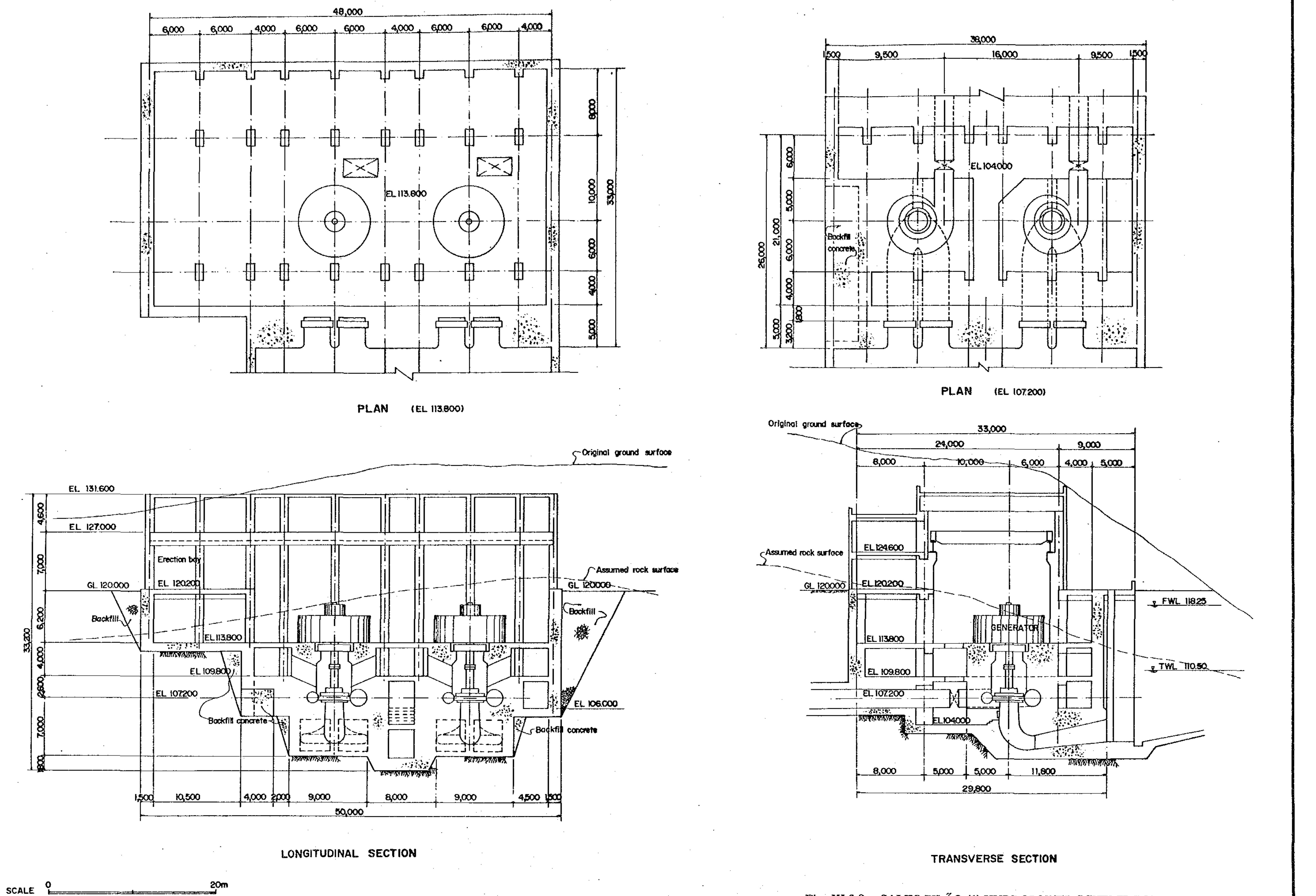


Fig. III.3.9 SALTO PILÃO (1) HYDROPOWER SCHEME, POWERHOUSE

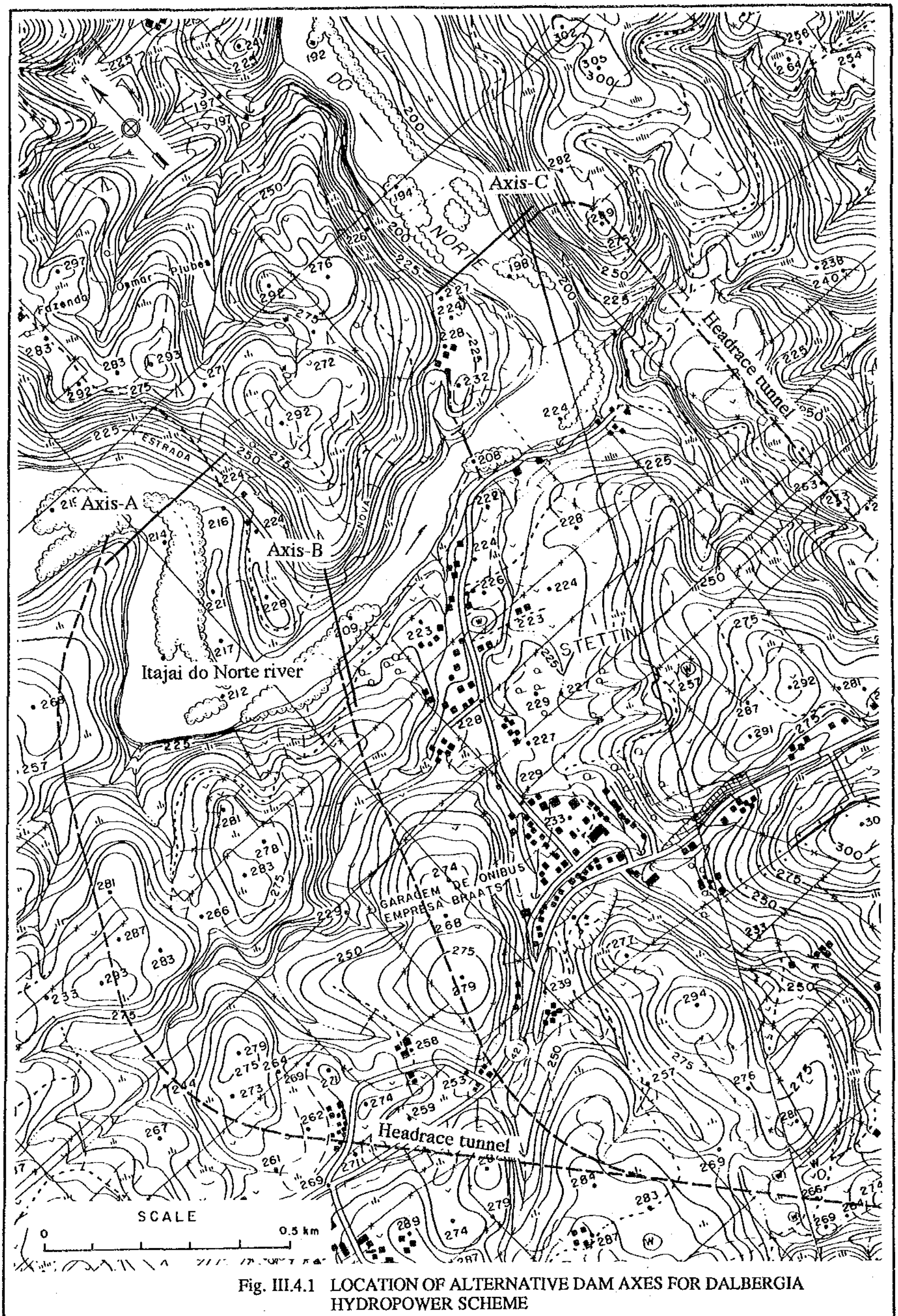


Fig. III.4.1 LOCATION OF ALTERNATIVE DAM AXES FOR DALBERGIA HYDROPOWER SCHEME