7.5 Dependable Discharge

The results of operation of Julumito Reservoir based on the reservoir operation rules described in the preceding section are as shown in Fig. 7-6. In view of these facts (during the 15-year period from 1962 through 1976) there is discharges less than the minimum operation discharge of 25 m^3/sec in 47 days during the 15-year period from 1962 through 1976 (3.1 days average per year), that is for 11 days in March-April of 1964, a dry year, and 36 days from April to June of 1973, but on all other days there are available discharges of 25 m^3/sec or more.

Therefore, the dependable discharge of Julumito Power Station is to be 25 m^3/sec.

7.6 Optimum Scale and Maximum Available Discharge of Power Station

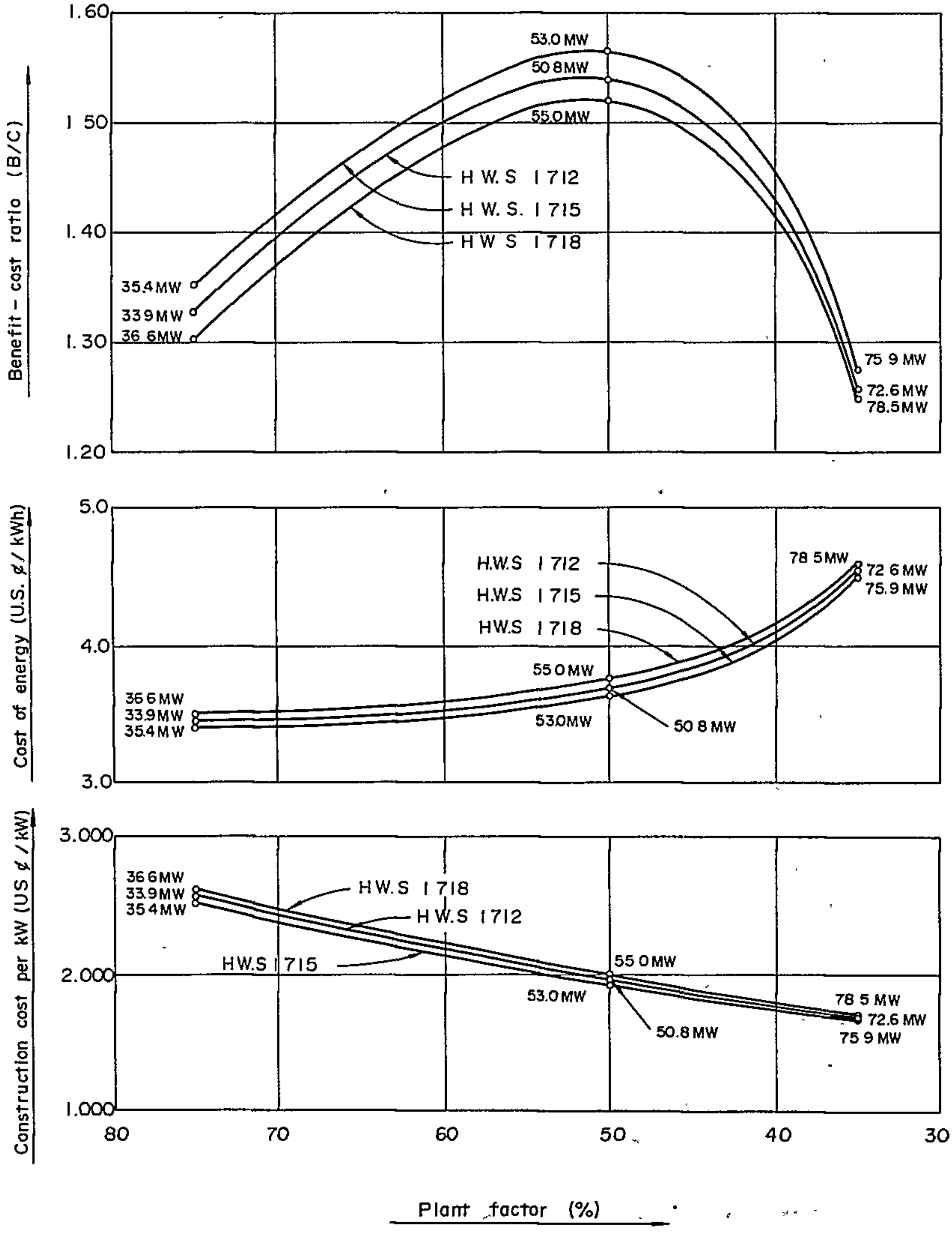
In case of a reservoir-type power station the scale of the power station must be determined taking into consideration two aspects. In essence, one aspect is that of supply capability as a power station determined from height of the dam, inflow to the reservoir, regulating capacity of the reservoir, etc., while the other aspect is that of desired supply capability of a new power station called for from the standpoint of load curve (shape of load) of power demand. And it is most desirable to coincide above two supply capabilities. In the future load curve, it would be desirable that the existing run-of-river and daily regulating pond-type power stations share base load and Julumito Hydro-electric Power Station, that is an only reservoir-type power station in the CEDELCA and CEDENAR power systems, shares the remainder (balance load).

7.6.1 Supply Capability of Julumito Hydro-electric Power Station and its Characteristics

Julumito Hydro-electric Power Station is a reservoir-type power station having an effective storage capacity of 50 million m^3 capable of regulating seasonally and annually the inflow to its reservoir. The available head is 126 m, the length of the pressure tunnel from the reservoir is relatively long at 1,770 m and the diversion waterway is 7,040 m long. Therefore, it has a economical limit as a peak-load power station. In other words, it possesses a site characteristic that its economic nature will be sharply impaired if the plant factor were to be lowered too much.

It is necessary to determine the plant factor of Julumito Power Station thorough examination of the above-mentioned site characteristics. The benefit-cost ratios in comparisons with an alternative thermal power station (see Chapter 11, Economic Analysis) of various plant factors are shown in Table 7-3 and Fig. 7-7.

Fig. 7-7 Result of Study on Optimum Max. Discharge and Installed Capacity



7.6.2 Load Curve and New Supply Capability

As seen in Fig. 7-8, the load curve of a workday in December 1978 shows that the power was received from the ISA Power Systems during the daytime and lighting time. As described in Chapter 4, "Load Forecast," it is expected that the load curves of the CEDELCA and CEDENAR power systems will indicate more or less the same shapes in the future also. Therefore, it is desirable that a new power station possesses a supply capability (dependable output, dependable energy) matching the part of the load which is being received from ISA.

The annual load factor of Julumito Hydro-electric Power Station is 65%, which is higher than the estimated annual load factor of 54 - 52% of the CEDELCA and CEDENAR power systems. On the other hand, the dependable discharge of Julumito Hydro-electric Power Station is 25 m^3/sec.day, and it has the capacity (load factor 50%) to supply the power, which is being received presently from ISA, for the balance load even during the low-water season.

Fig. 7-8 (1) Load Curve in Dec. 1978

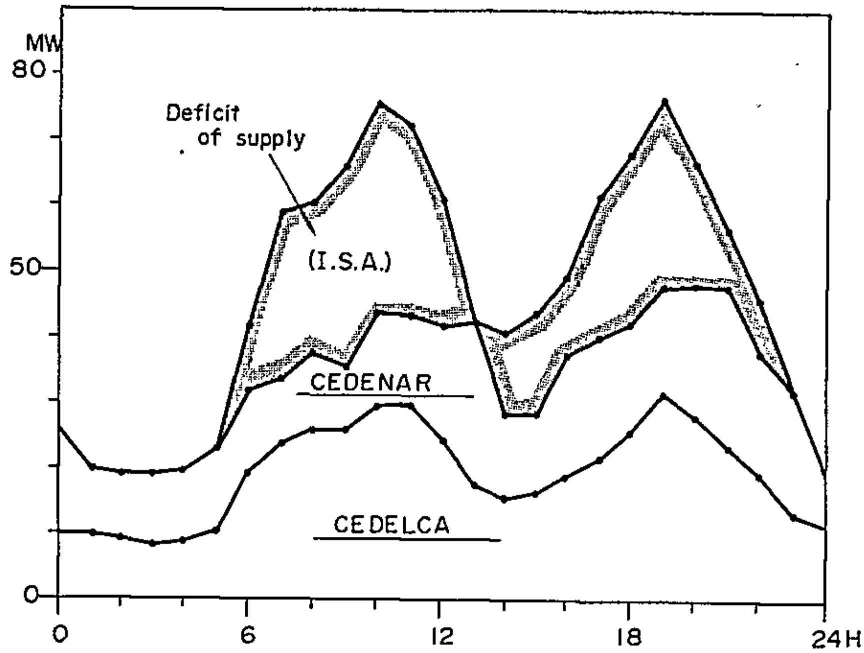


Fig. 7-8 (2) Load Duration Curve in Dec. 1978

Max. Demand ; 76.6 MW
Daily Energy ; 1,121 MWh
Load factor ; 61 %

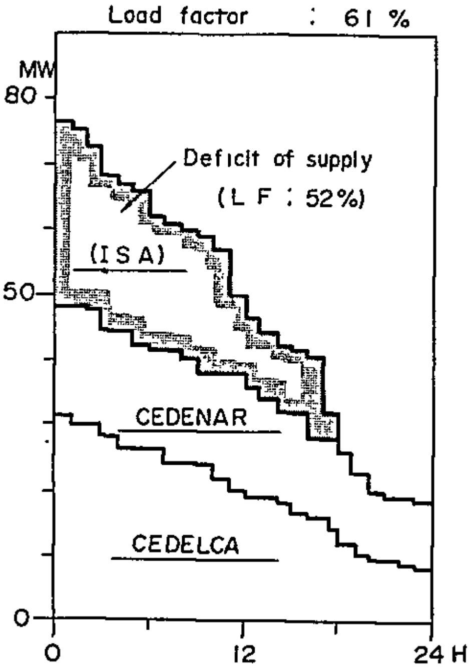


Table 7-3 Study on Optimum

Case	H. W. S. (m)	E. Storage Capacity (10^6m^3)	Drawdown (m)	Firm Discharge (m^3/sec.)	Plant Factor (%)	M Disc (m^3/
1					75	3
2	1,712	40.0	13.4	24.4	50	4
3					35	6
4					75	3
5	1,715	50.0	15.0	25.0	50	5
6					35	7
7					75	3
8	1,718	50.0	11.1	25.0	50	5
9					35	7

(the number in parenthesis is energy at new substation)

Table 7-3 Study on Optimum Maximum Discharge and Installed Capacity

Case	H.W.S. (m)	E. Storage Capacity ($10^6 m^3$)	Drawdown (m)	Firm Discharge (m^3/sec.)	Plant Factor (%)	Max. Discharge (m^3/sec.)	Installed Capacity (MW)	Annual Energy Production (10^6 kWh)	Construction Cost Total (10^3U.S.$)	Construction Cost (U.S.$/kW)	Annual Cost (10^3U.S.$)	Cost of Energy (U.S.¢/kWh)	B/C
1	1,712	40.0	13.4	24.4	75	32.5	33.9	270.5 (264.3)	87,190	2,572	9,181	3.474	1.324
2					50	48.8	50.8	294.5 (287.7)	100,650	1,981	10,666	3.707	1.540
3					35	69.7	72.6	294.5 (287.7)	123,870	1,706	13,120	4.560	1.252
4	1,715	50.0	15.0	25.0	75	33.3	35.4	282.4 (275.9)	89,380	2,525	9,408	3.410	1.349
5					50	50.0	53.0	307.0 (300.0)	103,200	1,947	10,935	3.645	1,567
6					35	71.4	75.9	307.0 (300.0)	127,400	1,679	13,492	4.497	1.270
7	1,718	50.0	11.1	25.0	75	33.3	36.6	294.1 (287.3)	96,040	2,624	10,111	3.519	1,301
8					50	50.0	55.0	316.2 (308.9)	109,970	1,999	11,657	3.774	1.522
9					35	71.4	78.5	316.2 (308.9)	134,200	1,710	14,216	4.602	1.248

(the number in parenthesis is energy at new substation)

7.7 Firm Peak Discharge

The firm peak discharge for calculating the firm peak output of Julumito Power Station is the available discharge 48.1 m^3/sec that the turbines of power station can draw in at the low water level of the reservoir.

7.8 Installed Capacity and Firm Peak Output

The installed capacity, the firm peak output and the calculations to arrive at them are shown in Table 7-4. The intake water level is to be the standard water level of the reservoir for installed capacity and low water level for firm peak in the outlet at maximum available discharge.

Table 7-4 Calculation of Output

Unit	Installed Capacity	Firm Peak Output (Dependable capability)
Available Discharge ($m^3/sec.$)	50.0	48.1
Intake Water Level (m)	1,710.0	1,700.0
Tailrace Water Level (m)	1,577.0	1,577.0
Effective Head (m)	126.0	116.5
Output (kW)	53,000	47,200

7.9 Number of Main Equipment Units

The number of units of main equipment must be determined to be the most advantageous in consideration of transportation limits, equipment manufacturing limitations, economy and harmony with the power system.

In the case of the Julumito Hydro-electric Power Project, there won't be problems in transportations of Colombia judging from the results of reconnaissance of roads and bridges. Consequently, the number of units of equipment may be determined from the viewpoints of economy and harmony with the electric power system.

Generally there is a trend that the larger the unit capacity becomes, the more economical the total of civil work costs and equipment costs becomes. Therefore, it is desirable that unit capacity is as large as possible taking into consideration reliability of the equipment, that is, performance records and manufacturing technology. However, in the small power system, to have a too large unit capacity will result in hindrance of power supply during faulting and scheduled shutdowns.

About 20% of the power demand of the service area is generally said to be the desirable limit for maximum unit capacity from the standpoint of power system operation. The largest unit capacity in the CEDELCA and CEDENAR power systems is the 12.0 MW of Florida II Power Station that was completed in 1975. And that corresponded to 22% of maximum power demand 53.9 MW in 1975. As the maximum power demand of the CEDELCA and CEDENAR power systems in 1985 when Julumito Hydro-electric Power Station is to be completed will be 131 MW, the unit capacity of 26.5 MW of Julumito will be 20% of the maximum power demand.

Meanwhile, the CEDELCA and CEDENAR power systems will be interconnected by a 230-kV transmission line so that even during faulting of Julumito Hydro-electric Power Station it will be possible to instantaneously receive substitute power from the Central Power System. The Julumito Hydro-electric Power Station will in most cases be operated at a high water level throughout the year, and the plant factor will be as high as 66%. Therefore in case of a single unit proposal, there would be a possibility of overflow from the reservoir when the unit were to be stopped. However, in case of a two-units proposal, in view of the regulating capacity of Julumito Reservoir, it would be possible to prevent overflow from the reservoir by operating one unit.

Referring to the high water level, high plant factor operation of Julumito Hydro-electric Power Station, see Fig. 7-7.

As described above, the number of units of main equipment is to be two units from the standpoints of power system operation and the regulating capacity of the reservoir of Julumito Hydro-electric Power Station.

7.10 Possible Power Generation

Possible power generation was calculated based on the operation rules of Julumito Reservoir set forth in 7.4. The available discharges and reservoir water levels are used to calculate the monthly possible power generation for the 15-year period from 1962 through 1976. The results are as shown in Table 7-5 and Fig. 7-9, and the average power generation for the above 15-year period would be the following:

Firm energy	259,400 MWh
Secondary energy	47,600 MWh
Total	307,000 MWh

As firm energy, the energy in the driest year, 1973, during the 15-years period from 1962 through 1976 is taken.

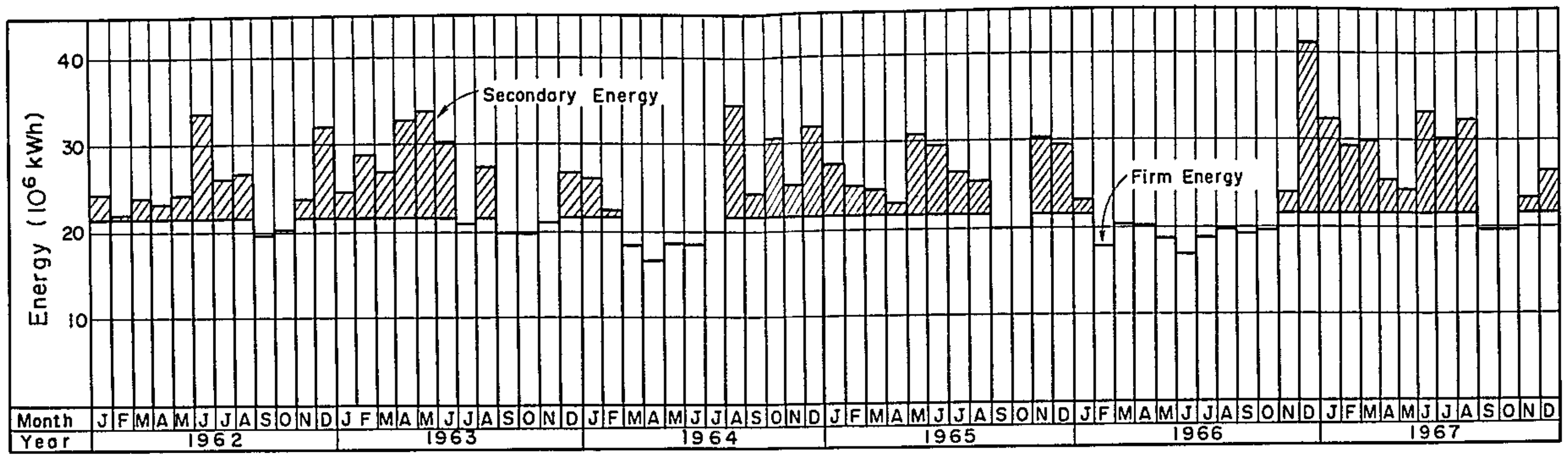
The secondary energy is the rest of the 15-years average energy in excess of the above firm energy.

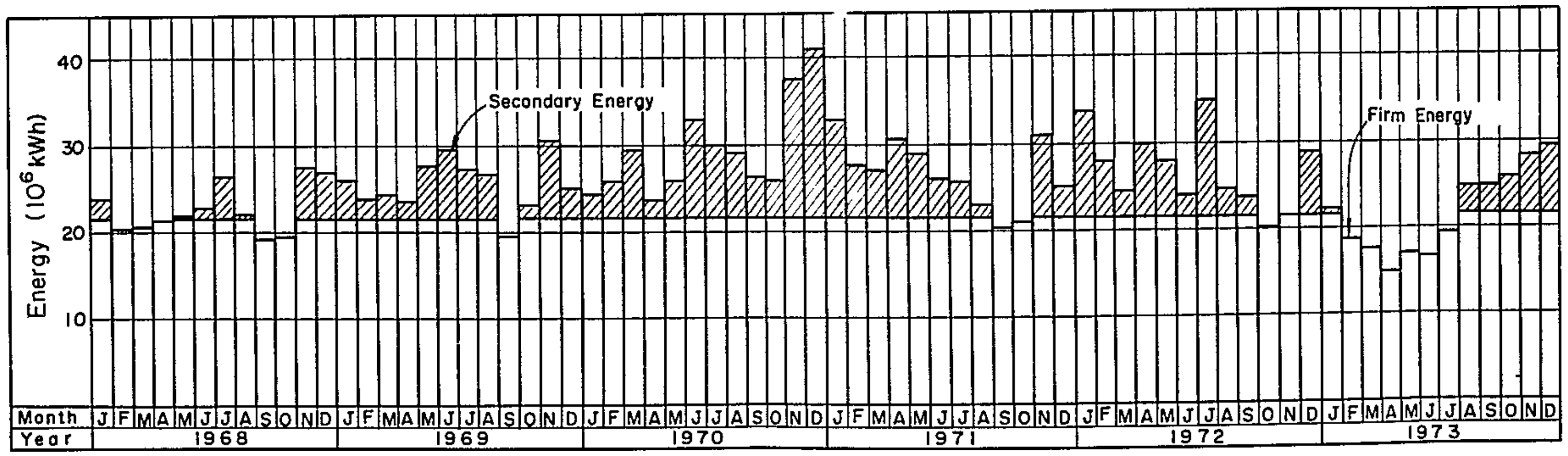
Table 7-5 Energy Production

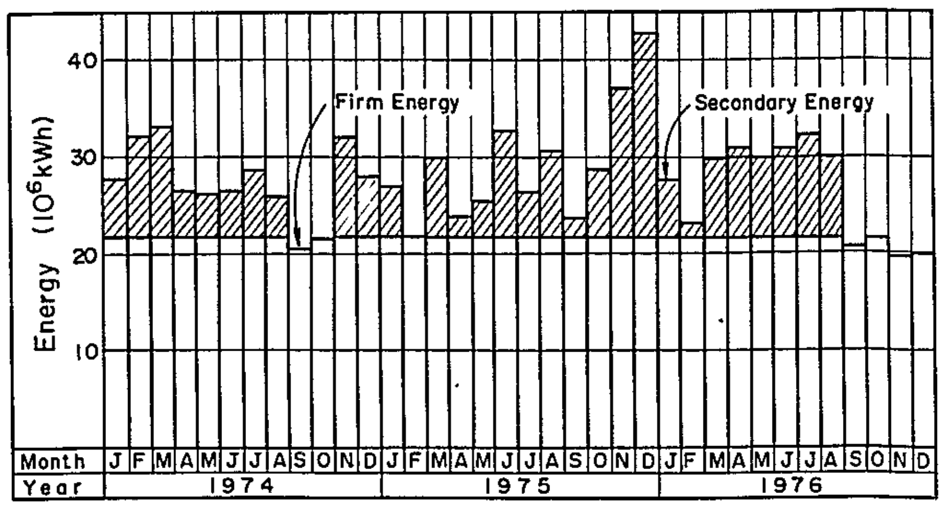
(Unit: 10^3 kWh)

Month / Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1962	24,518	21,911	24,125	23,408	24,262	33,610	26,219	26,650	19,776	20,264	23,904	31,968	300,616
1963	24,622	28,926	26,775	32,861	33,711	30,146	20,844	27,306	19,738	19,597	20,989	26,574	312,088
1964	25,988	20,800	18,380	16,484	18,487	18,429	19,878	34,750	24,333	30,638	25,284	31,785	285,236
1965	27,851	25,060	24,513	22,944	30,810	29,388	26,486	25,356	19,815	19,860	30,415	29,590	312,087
1966	23,214	17,769	20,388	20,143	18,665	16,831	18,900	19,656	19,261	19,643	24,005	41,130	259.605
1967	32,381	28,295	29,752	25,354	24,262	33,180	30,118	32,297	19,761	19,667	23,263	26,454	324,783
1968	23,790	20,481	20,588	21,312	21,940	22,849	26,408	21,999	19,168	19,491	27,549	27,021	272,595
1969	25,975	23,772	24,171	23,565	27,835	29,355	27,148	26,610	19,500	23,139	30,571	24,900	306,541
1970	24,407	25,977	29,411	23,664	25,807	32,916	29,703	28,985	26,255	25,900	37,463	30,980	341,469
1971	32,761	27,507	26,964	30,630	28,822	26,002	25,649	23,007	20,359	20,924	31,029	25,088	318,744
1972	33,789	27,950	24,468	30,064	27,854	23,888	34,890	24,725	23,696	20,111	21,443	28,764	321,641
1973	22,396	18,841	17,544	14,841	17,003	16,616	19,339	24,764	24,792	25,737	28,238	29,331	259,442
1974	27,860	31,912	32,990	26,432	26,258	26,423	28,631	25,839	20,291	21,465	32,018	28,038	328,158
1975	26,931	21,741	29,767	23,669	25,343	32,396	26,456	30,491	23,580	28,449	36,917	40,265	346,004
1976	27,547	23,136	29,664	30,963	29,130	30,816	32,275	30,172	20,580	21,494	19,728	19,878	315,383
Average	26,935	24,272	25,300	24,422	25,346	26,856	26,196	26,840	21,394	22,425	27,521	29,451	

Fig. 7-9 Energy Production







(10^6 kWh)

Item / Year	Firm Energy	Secondary Energy	Total
1962	259 442	41.174	300.616
1963	〃	52.646	312.088
1964	〃	25.794	285.236
1965	〃	52.645	312.087
1966	〃	163	259.605
1967	〃	65.381	324.783
1968	〃	13.153	272.595
1969	〃	47.099	306 541
1970	〃	82.027	341.469
1971	〃	59.301	318.743
1972	〃	62.199	321.641
1973	〃	0	259.442
1974	〃	68.716	328.158
1975	〃	86.562	346004
1976	〃	55.941	315.383
Average	〃	47.517	306.959

CHAPTER 8

PRELIMINARY DESIGN

CHAPTER 8 PRELIMINARY DESIGN

CONTENTS

FIGURE LIST

CHAPTER 8 PRELIMINARY DESIGN

8.1 Design

8.1.1 Civil Structure

(1) Main Dam

a) Outline

It is judged that a fill type is the most suitable dam type in consideration of the topography and the geological conditions of the dam site described in Chapter 6, "Geology and Construction Materials", the characteristics of the dam construction materials available in the surroundings of the dam site, and the behavior of the dam after completion.

It is considered that construction of a concrete dam is technically and economically infeasible in view of the geology of the ground to be the foundation of the dam being mainly andesite lava but partially including volcanic ash deposits as described in Chapter 6, and that concrete aggregates of good quality cannot be obtained in sufficient quantity.

Impervious soil materials available in the vicinity of the dam site are weathered residual soils of volcanic ash deposits and andesite lava. Both have fairly high impermeabilities as core materials, but generally have very fine-gradations and high natural water contents.

In consideration of the above mentioned properties (particularly compressibility) of the impervious materials, the properties of the rock materials to be used, and further, the topography and bedrock exposure conditions of the dam site, the dam is designed as a rockfill embankment with an inclined core and an arch-shape axis. The upstream slope is 1:2.3 and the downstream slope is 1:1.8.

At the next stage of detail design, the final design must be made based on the results of detailed investigations of the dam construction materials and the foundation ground.

b) Determination of Freeboard

The freeboard of a rockfill dam is determined by the formula below in consideration of wind, waves caused by earthquake, water level rises due to unforeseen accidents during spillway gate operation and the dam type.

$$H_f \geqq h_w + \frac{h_e}{2} + h_a + h_i$$

where

H_f : freeboard (m)

h_w : wave height due to wind (m)

h_e : wave height due to earthquake (m)

h_a : water level rise due to unforeseen accident during spillway gate operation

h_i : safety height according to dam type

(i) Wave Height Due to Wind, h_w

This is determined by the S.M.B. Method under the conditions below.

F : fetch, 4.0 km

V : wind velocity, 30 m/sec

Upstream slope gradient, 1: 2.3

From Fig. 8-1

h_w = 1.10 m

(ii) Wave Height Due to Earthquake, h_e

This is determined by the equation below.

$$h_e = \frac{K\tau}{2\pi}\sqrt{g \cdot H_o}$$

where

K : seismic coefficient, 0.05

τ : earthquake wave period, 1 sec

g : gravity acceleration, 9.8 m/sec^2

H_o : reservoir water depth, 78 m

Based on the above,

h_e = 0.22 m

(iii) Water Level Rise Due to Unforeseen Accident During Spillway Gate Operation, h_a

This amount will be proportional to flood discharge and duration of accident, and inversely proportional to surface area and number of gates. It is therefore assumed that a small value will be adopted for h_a at this dam, but to be conservative, the standard value of 0.5 m shall be taken.

h_a = 0.5 m

(iv) Safety Height According to Dam Type, h_i

With a rockfill dam, overtopping of the dam crest will be fatal so that a larger h_i than for a concrete dam is taken. The standard height of 1.0 m will be taken here.

h_i = 1.0 m

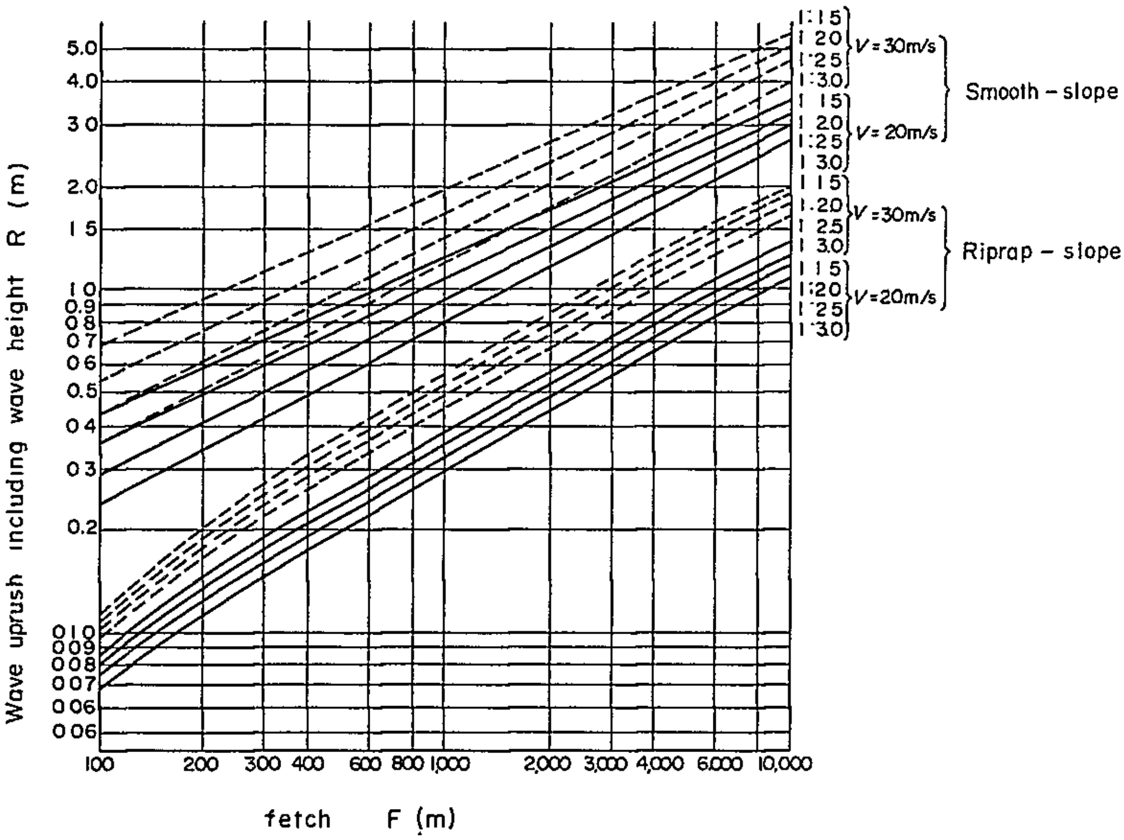
(v) Freeboard, H_f

As a result of the above

$$H_f \geqq 1.10 + 0.22/2 + 0.50 + 1.00 = 2.71 \text{ m}$$

To be conservative, H_f = 3.00 m is to be taken. Consequently, the elevation of the dam crest will be 1,718.00 m.

Fig. 8-1 Wave Uprush (Including Wave Height) Obtained by Combining The S.M.B. Method with Saville Method



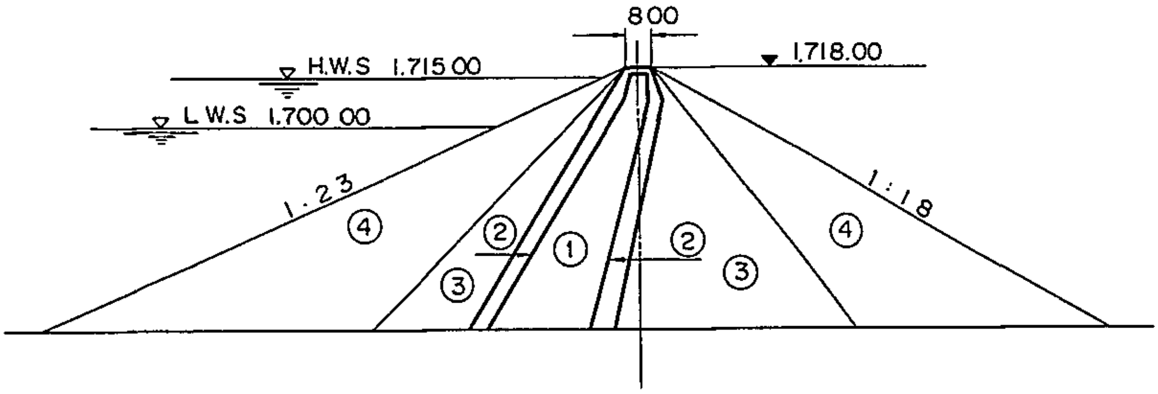
c) Stability Calculations for Dam

(i) Outline

Stability calculations are made for the stages of full reservoir, rapid draw-down, and immediately after completion. Calculations are made by the slice method on assumed circular slip surfaces.

The cross section for calculations is as shown in Fig. 8-2.

Fig. 8-2 Typical Cross Section of Main Dam



The physical properties of the embankment materials used for calculations were assumed as shown below considering test results and the physical properties of similar materials for dams constructed in the past.

Zone	Material	Spec. Grav.	Unit Weight (t/m^3) Dry	Wet	Saturated	Angle of Internal Friction	Cohesion (t/m^2)
1	Core	2.52	1.26	1.71	1.76	0.577	0
2	Filter	2.35	1.81	1.90	2.04	0.700	0
3	Smaller rock	2.35	1.68	1.76	1.97	0.781	0
4	Larger rock	2.35	1.62	1.67	1.93	0.839	0

(ii) Stability Analysis

The stability calculations are made by a calculation method developed by the Electric Power Development Co., Ltd., based on the Swedish Method usinr an IMB Model 370-148.

The safety factor is given by the following equations:

Normal condition:

$$SF(N) = \frac{\Sigma(N + H_n - U)\tan\phi + \Sigma C\cdot L}{\Sigma(T - H_t)}$$

Earthquake condition:

$$SF(E) = \frac{\Sigma(N + H_n - U)\tan\phi - \Sigma Ne\tan\phi + \Sigma C\cdot L}{\Sigma(T - H_t) + \Sigma T_e}$$

where

N : normal force

T : tangential force

$\tan \phi$: coefficient of internal friction

H_n : normal component of hydrostatic pressure

H_t : tangential component of hydrostatic pressure

U : pore pressure

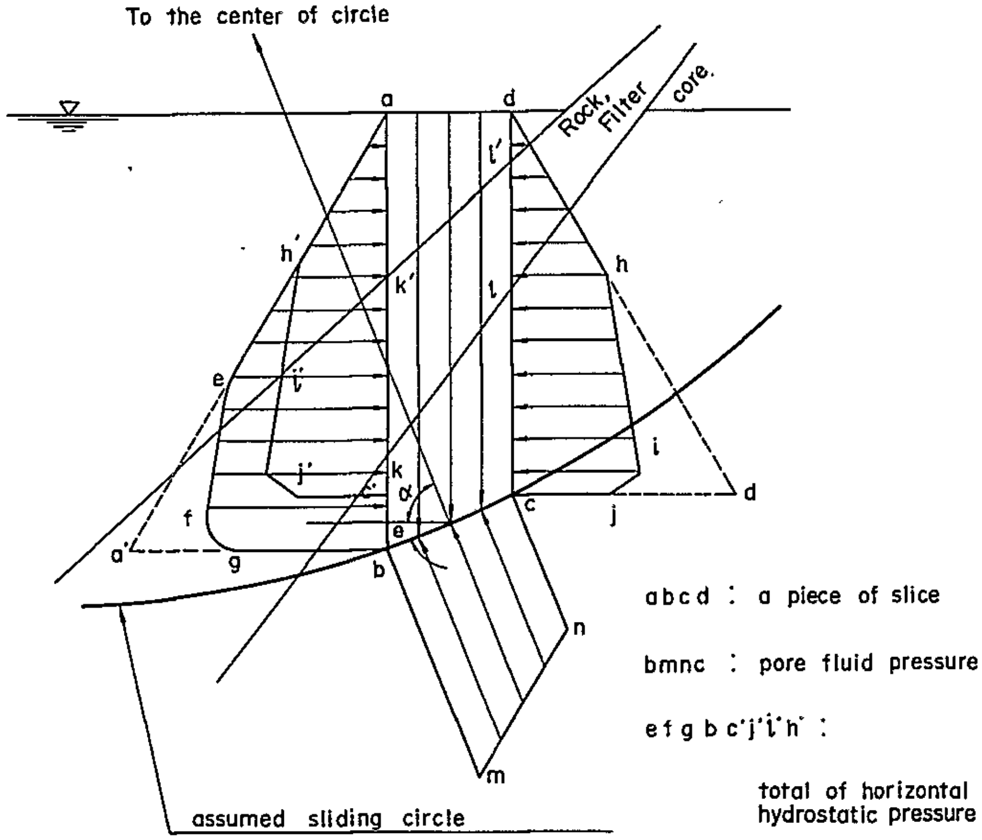
C : cohesion

L : sliding arc length

N_e : normal component of earthquake force

T_e : tangential component of earthquake force

Fig. 8-3 Water Pressure in the Core



$\Sigma N \cdot \tan\phi = \Sigma \{\gamma(k'bc l') + \gamma w(abcd)\} \cos\theta \cdot \tan\phi$

$\Sigma Hn - \tan\phi = \Sigma (efgbc'j'i'h') \cos\alpha \cdot \tan\phi$

$\Sigma U \quad \tan\phi = \Sigma (bmnc) \tan\phi$

$\Sigma T = \Sigma \{\gamma(k'bc l') + \gamma w(abcd)\} \sin\theta$

$\Sigma Ht = \Sigma (efgbc'j'i'h') \sin\alpha$

$\Sigma Ne \quad \tan\phi = \Sigma \beta i (k'bcl')\lambda \sin\theta \quad \tan\phi$

$\Sigma Te = \Sigma \beta i (k'bcl')\lambda \cos\theta$

unit weight

γ : normal condition

βi : earthquake condition

coefficient of earthquake : k

(iii) Calculation Conditions

Stability calculations were made on the upstream and downstream slopes for the cases below. The safety factors for normal and earthquake conditions were calculated for each case.

Case (1)	Full reservoir	Water level 1,715.00
Case (2)	Rapid drawdown	Water level 1,700.00
Case (3)	Immediately after completion	

The pore pressures assumed were those shown in Fig. 8-4 - Fig. 8-6. As horizontal seismic coefficient, K, 0.05 was used.

Fig. 8-4 Pore Pressure at Full Reservoir Stage

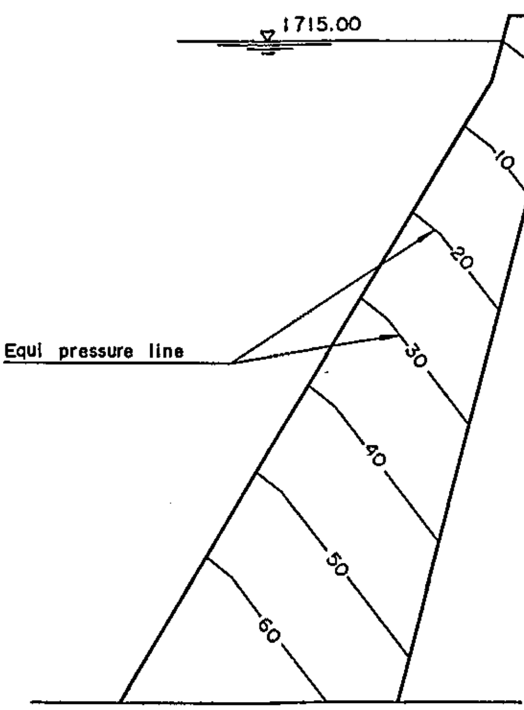


Fig. 8-5 Pore Pressure at Draw-Down Stage

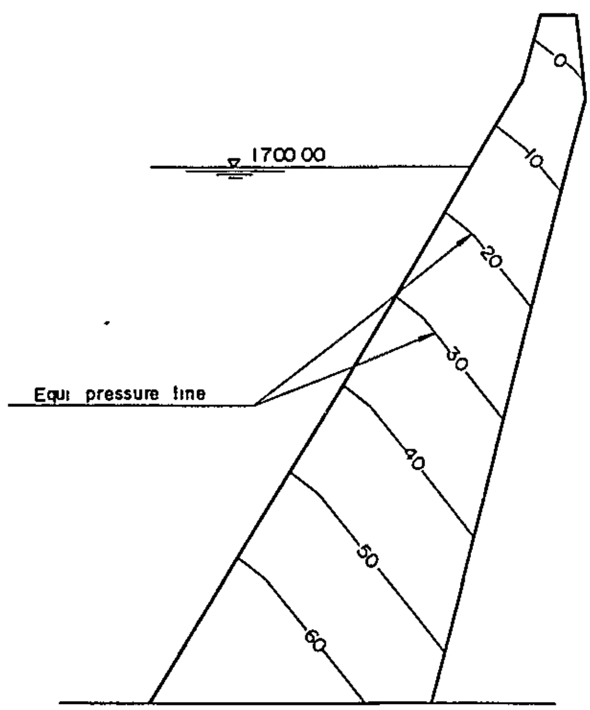
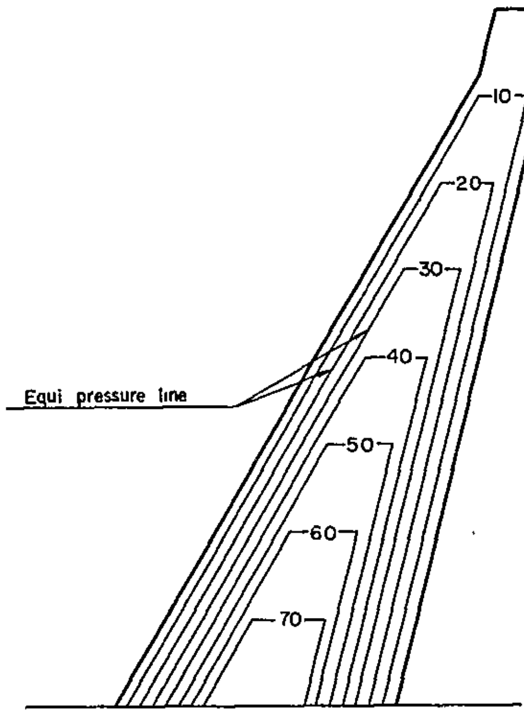


Fig. 8-6 Pore Pressure at Construction Stage



(iv) Results of Calculations

Slope	Case	Safety Factor (F)	
		Normal condition	Earthquake condition
Upstream	(1)	1.82	1.45
	(2)	1.73	1.43
	(3)	1.63	1.41
Downstream	(1)	1.52	1.35
	(2)	1.52	1.35
	(3)	1.27	1.10

Fig. 8-7 Stability Analysis Case (1)

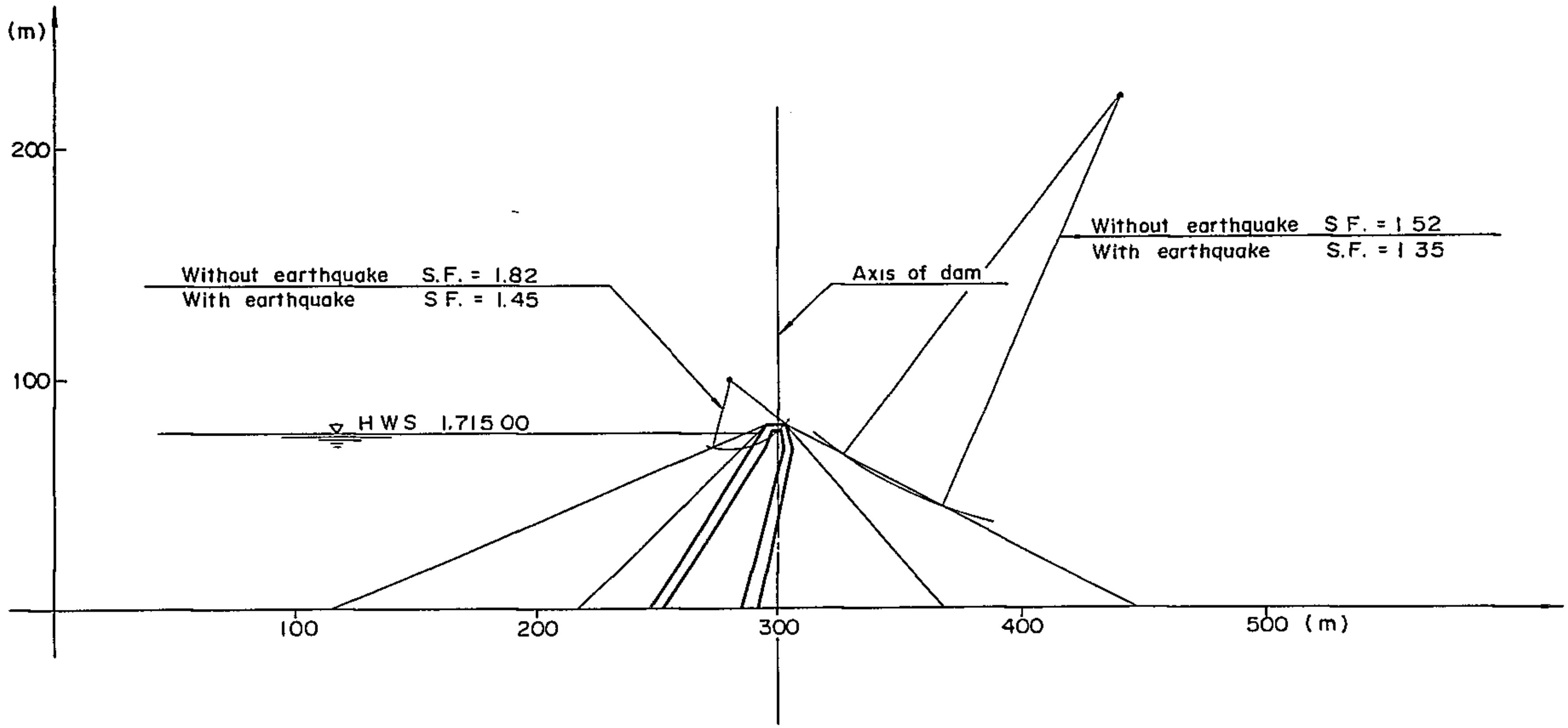


Fig. 8-8 Stability Analysis Case (2)

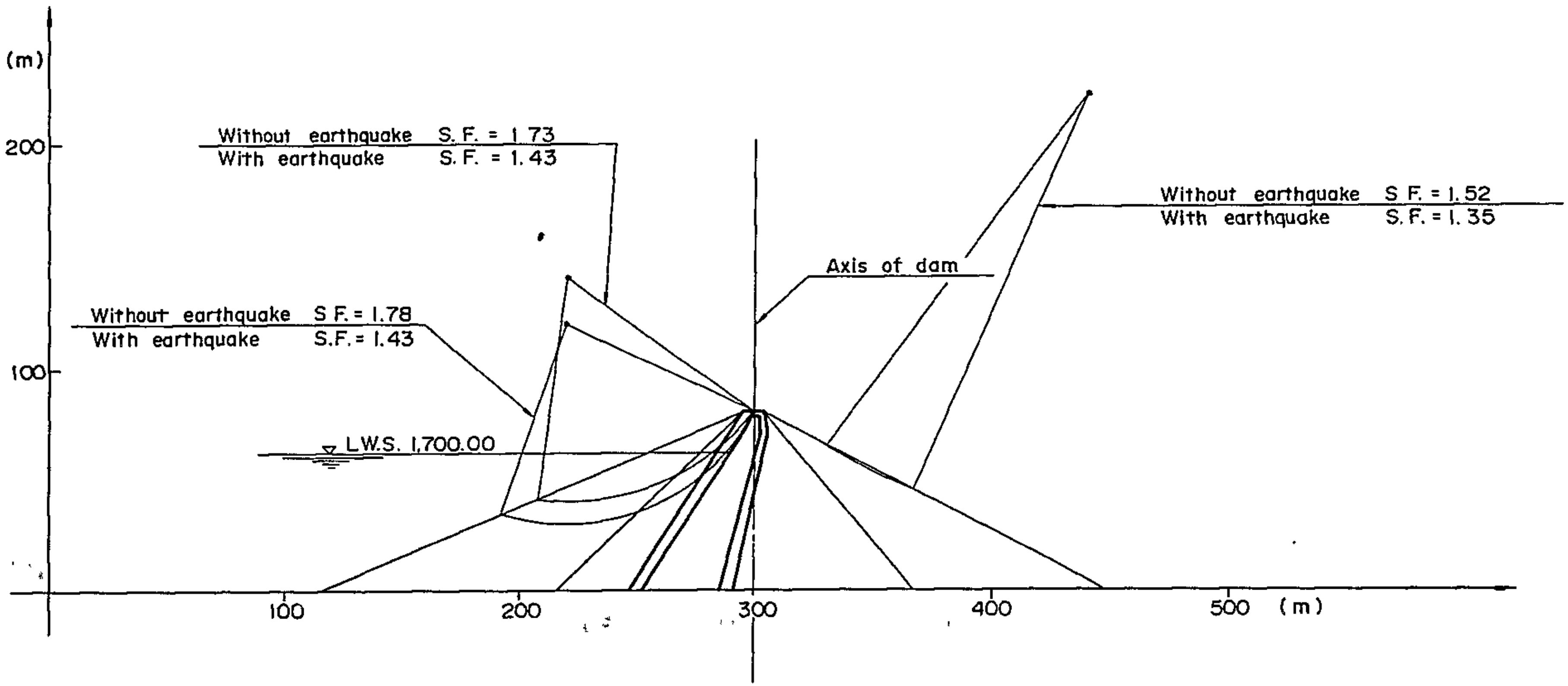
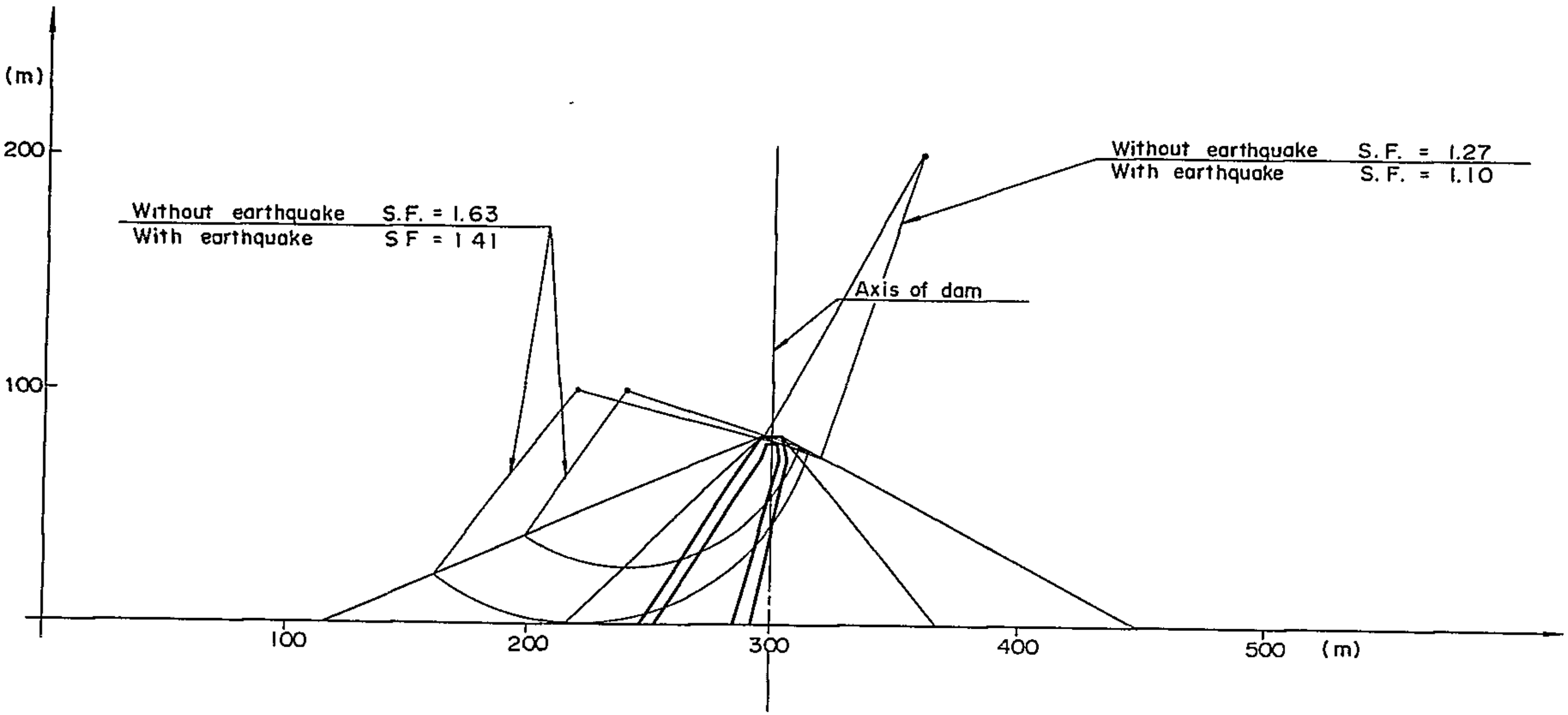


Fig. 8-9 Stability Analysis Case (3)



d) Seepage through Impervious Core

(i) Quantity of Seepage through Impervious Core

Quantity of seepage was calculated under the following, conditions.

Reservoir water level	1,715.00 m
Dam height	80 m
Coefficient of permeability of impervious core	1×10^{-7} m/sec

The flow net is as shown in Fig. 8-10.

The quantity of seepage is determined by the equation below.

$$Q = K \cdot H \cdot \left(\frac{n_c}{n_d}\right) \cdot L$$

where

Q : quantity of seepage (m^3/sec)

K : coefficient of permeability, 1×10^{-7} m/sec

H : head differential between upstream and downstream sides, 77 m

n_c : number of stream tubes 17

n_d : number of partitions divided by equipotential lines, 6

L : dam axis length, 340 m

$Q = 7.4\,\ell/\text{sec}$

(ii) Calculation of Seepage Velocity

The seepage velocity is determined on the streamline with maximum hydraulic gradient.

$$V = K \cdot \frac{H}{L}$$

where

V : seepage velocity (m/sec)

K : coefficient of permeability, 1×10^{-7} m/sec

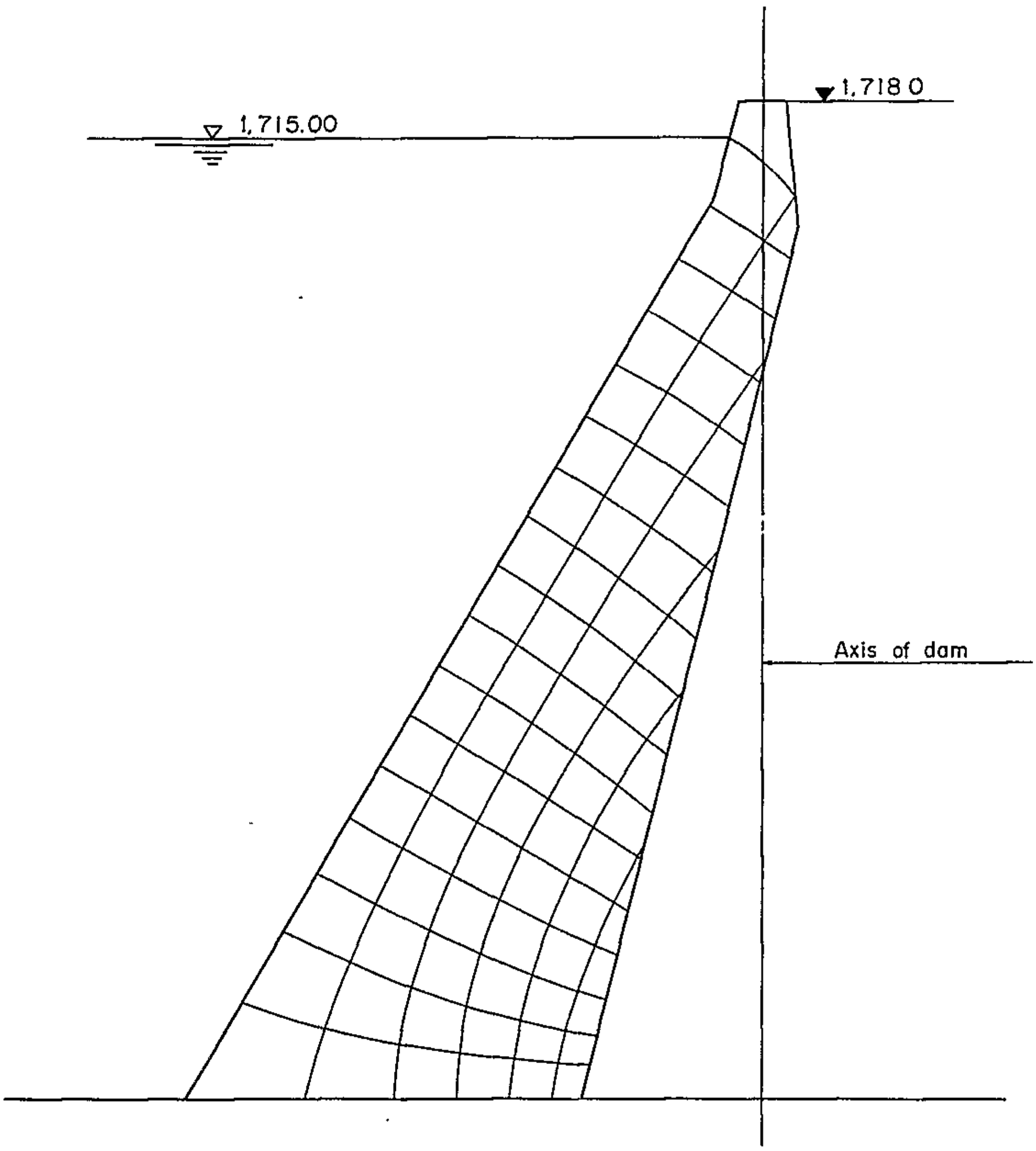
H : head differential between upstream and downstream sides, 77 m

L : streamline length, 32 m

$V = 2.41 \times 10^{-5}$ cm/sec

The seepage velocity is sufficiently low, and there will be safety against piping.

Fig. 8-10 Flow - Net (Full Reservoir)



e) Temporary Diversion Tunnel and Outlet Tunnel

Diversion of the river flow during construction is to be done by a single temporary diversion tunnel of inside diameter of 2.5 m provided at the left bank. The capacity of the temporary diversion tunnel is to be 25 m^3/sec in consideration of safety during the period of dam construction.

A tunnel of inside diameter of 2.5 m is to be driven from the upstream side parallel to the temporary diversion tunnel, and after completion of the construction work this tunnel is to be connected with the temporary diversion tunnel at the lower reaches of a stream of the dam axis for use as an outlet tunnel.

(2) Spillway

As described in Chapter 5, "Hydrology," the design flood discharge (Rio Sate) of the main dam site is 95 m^3/sec. The spillway is to be a chute type of a structure capable of releasing 95 m^3/sec at water level of 1,715.00 m. The spillway is to be provided at the right bank of the main dam where economical construction can be done topographically, and the flow regulation is to be done by one flap gate of 4.00 m x 5.00 m.

(3) Intake

The intake, in order to prevent inflow of sediment, is to be provided at a gully at the left bank of the Rio Sate approximately 200 m upstream from the dam site, and is to be an inclined type having a slope of 1: 0.7. The bed elevation of the intake, taking into consideration the geological conditions of this site, the center elevation of the pressure tunnel, and the construction method, is to be at El. 1,685 m, 15 m below the low water level of the reservoir. In front of the intake the trash rack and guide rail will be provided for a trash remover. A roller gate is to be provided along the intake slope for uses such as maintenance and inspection of the headrace tunnel.

(4) Headrace Tunnel

The headrace tunnel is to have a circular cross section and the inside diameter was determined so that the sum of the annual cost related to the tunnel construction cost and the annual electricity charge revenue loss related to the hydraulic gradient would be a minimum. The results of the study are as shown in Fig. 8-11, and the inside diameter of the tunnel is to be 4.20 m.

The foundation between the intake and the powerhouse where the headrace tunnel is to be provided, as described in Chapter 6, "Geology and Construction Materials," consists of a volcanic ash deposit of approximately 30 - 40 m depth from the surface, underlying which there is thick andesite lava. The center line of the headrace tunnel is selected to be in the andesite lava, with moreover, ample cover.

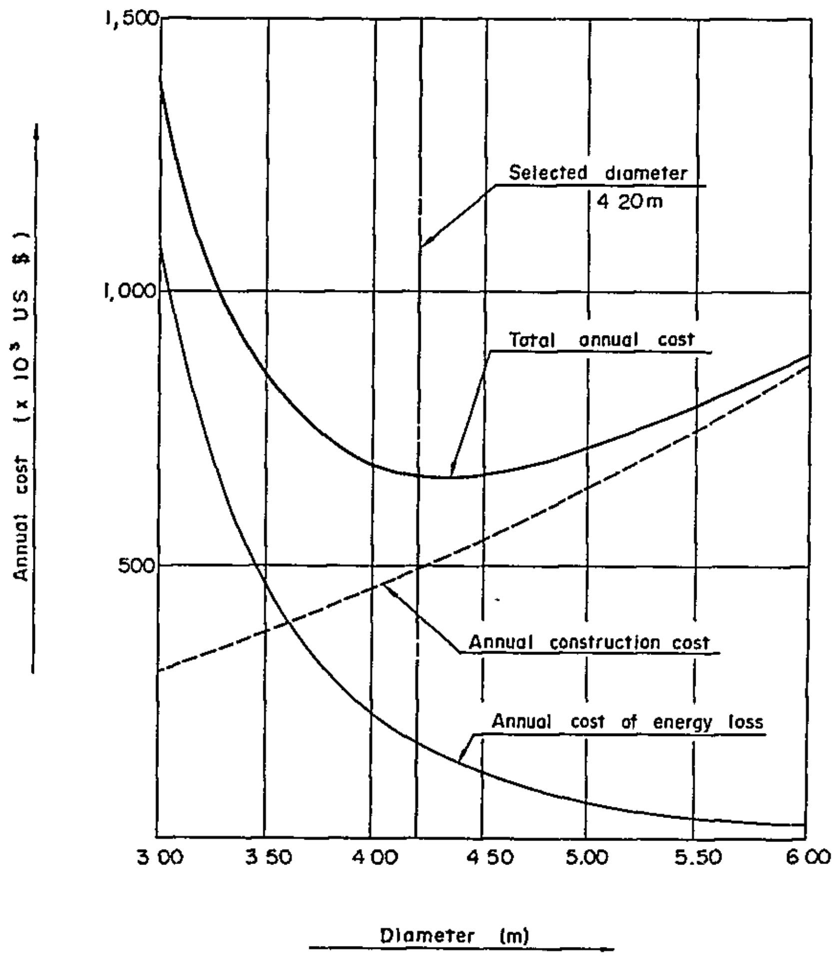
The tunnel is to be with reinforced concrete lining over its entire length, and as necessary, at the intake, the surge tank vicinity and places of adverse geological conditions, by steel pipe. Also as described in Chapter 6, it is thought there will be no fear of faults and water springing, and there should be no special problems in construction. It is planned for work adits for construction to be provided at the intake and powerhouse sides.

(5) Surge Tank

a) Outline

The surge tank is to be a orifice type to avoid a complex structure in consideration of the topography of the site, the geological conditions because of which the upper half must be located in a volcanic ash deposit, and the accompanying difficulties in construction. The cross-sectional shape is to be circular with an inside diameter of 8.00 m.

Fig. 8-11 Economical Diameter Diagram (for Q = 50 m^3/sec.) (Headrace Tunnel)

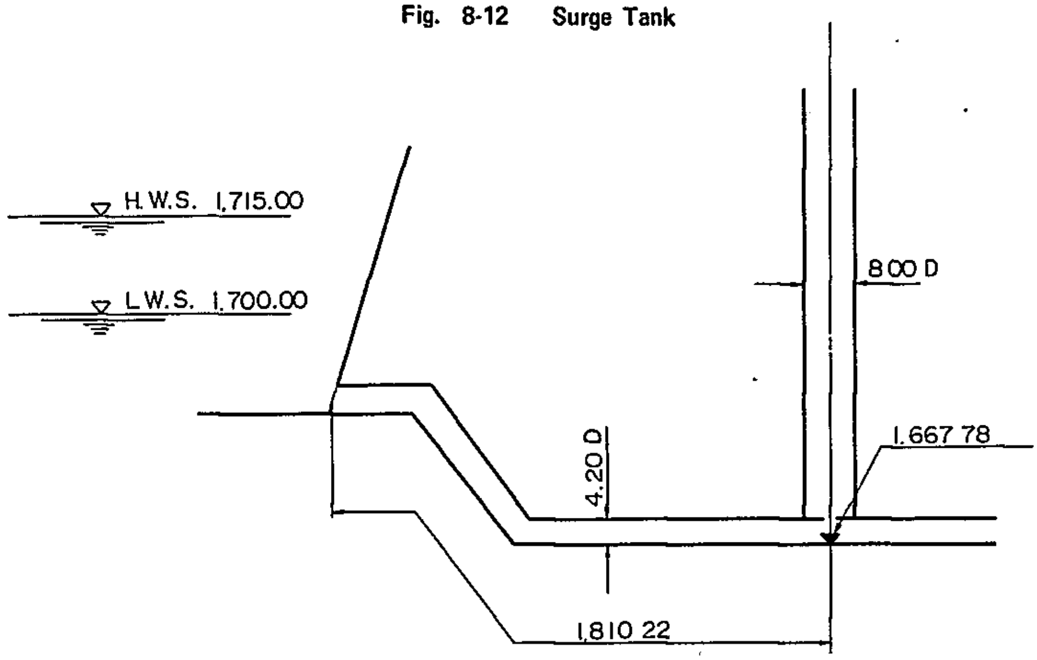


Note: Construction cost of headrace tunnel is direct cost excluding indirect cost (interest contingency and escalation ; 60 % direct cost)

The surge tank was designed in case of full load rejection at reservoir high water level, and in case of half load demand at reservoir low water level that adverse effects will not be inflicted on the headrace tunnel and the turbines.

b) Surging Calculations

Fig. 8-12 Surge Tank



(i) Basic Equations

$$\frac{dV}{dt} = \frac{z - \varepsilon \cdot |V| \cdot V - K}{L/g}$$

$$\frac{dz}{dt} = \frac{Q - f.V}{F}$$

$$K = \frac{1}{2g \cdot (C_d \cdot F_p)^2} |f \cdot V - Q| \cdot (f \cdot V - Q)$$

where

- z : surge tank water level
- V : flow velocity inside tunnel (m/sec)
- L : tunnel length between intake and surge tank (m)
- f : cross-sectional area of tunnel (m^2)
- F : cross-sectional area of surge tank (m^2)
- ε : tunnel head loss coefficient
- Q : available discharge (m^3/sec)
- C_d : runn-off coefficient of orifice
- F_p : cross-sectional area of port (m^2)

(ii) Basic Numerical Values

$F = 50.265 \text{ m}^2$

$f = 13.854 \text{ m}^2$

$L = 1,810.22 \text{ m}^2$

$F_p = 3.801 \text{ m}^2$

$C_d = 0.8$ (inflow), 0.7 (outflow)

Case of Full Load Rejection

Closure time $T_1 = 0 \rightarrow 5$ sec, $Q = 50 \text{ m}^3/\text{sec} \rightarrow 0$

Reservoir water level $H_o = 1,715.00$ m

Tunnel head loss coefficient $\varepsilon_{up} = 0.2120$

Case of Half Load Demand

Opening time $T_1 = 0 \rightarrow 2$ sec, $Q = 25 \rightarrow 50$ m /sec

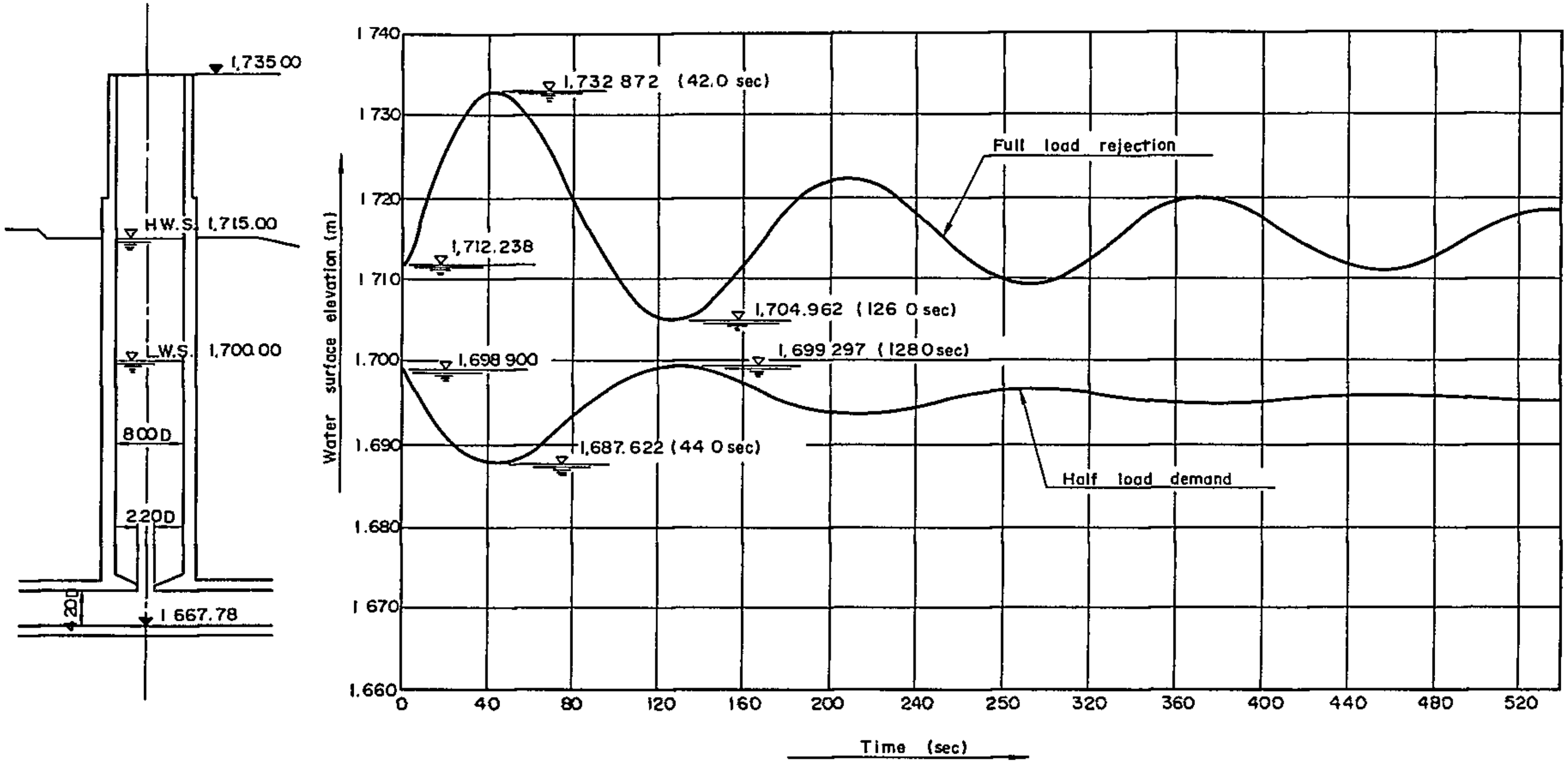
Reservoir water level $H_o = 1,700.00$ m

Tunnel head loss coefficient $\varepsilon_{down} = 0.3378$

(iii) Results of Calculations

The calculations were made by the Runge-Kutta Method at 0.5 sec intervals using an IBM Model 370-148. The results of calculations are shown in Fig. 8-13.

Fig. 8-13 Surging Curve



In the case of
full load rejection

$Q = 50\ m^3/s \longrightarrow 0\ m^3/s$

$n = 0.0110$

$\varepsilon_{up} = 0.2120$

W.S.E. = 1,715.00

In the case of
half load demand

$Q = 25\ m^3/s \longrightarrow 50\ m^3/s$

$n = 0.0140$

$\varepsilon_{down} = 0.3378$

W.S.E. = 1,700.00

(6) Penstock

a) Outline

The design of the penstock was studied for the two alternatives of a surface type and a vertical-shaft type upon consideration of the topography, especially, the slope of the ground surface, and geology. As a result, it was decided to adopt the surface type since compared with the vertical-shaft type it would be slightly lower in construction cost, construction would be easier, and maintenance would not be difficult. However, depending on the results of further investigations of topography and geology, there is a possibility of changing to the vertical-shaft alternative in the definite study if there were to be found problems about the surface type.

The penstock is a single line of welded steel pipe at the upper part and bifurcates immediately before entering the powerhouse. The shell materials is to be mainly SM 53 or SM 58 (JIS), or equivalent material. Similarly to the case of the headrace tunnel, the inside diameter of the penstock was selected calculating the annual costs and annual benefit loss due to head loss for various inside diameters so that the sum of the two would be a minimum. The results are as shown in Fig. 8-14, and an average inside diameter of 3.70 m was selected. Based on this, the inside diameter is to be varied between 4.2 m and 3.2 m.

The design of the penstock is to be considered with hydrostatic pressure, head due to surging, and water-hammer pressure as the interior pressure.

The rock excavated for the penstock is planned to be used as rock embankment material for the main dam as previously described.

b) Water-Hammer Calculations

(i) Method of Calculation

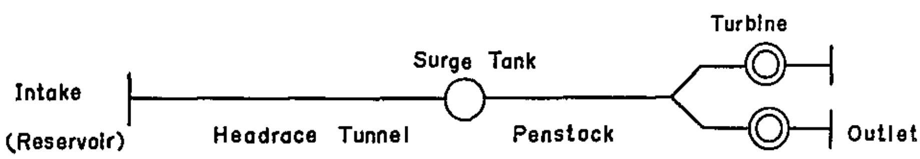
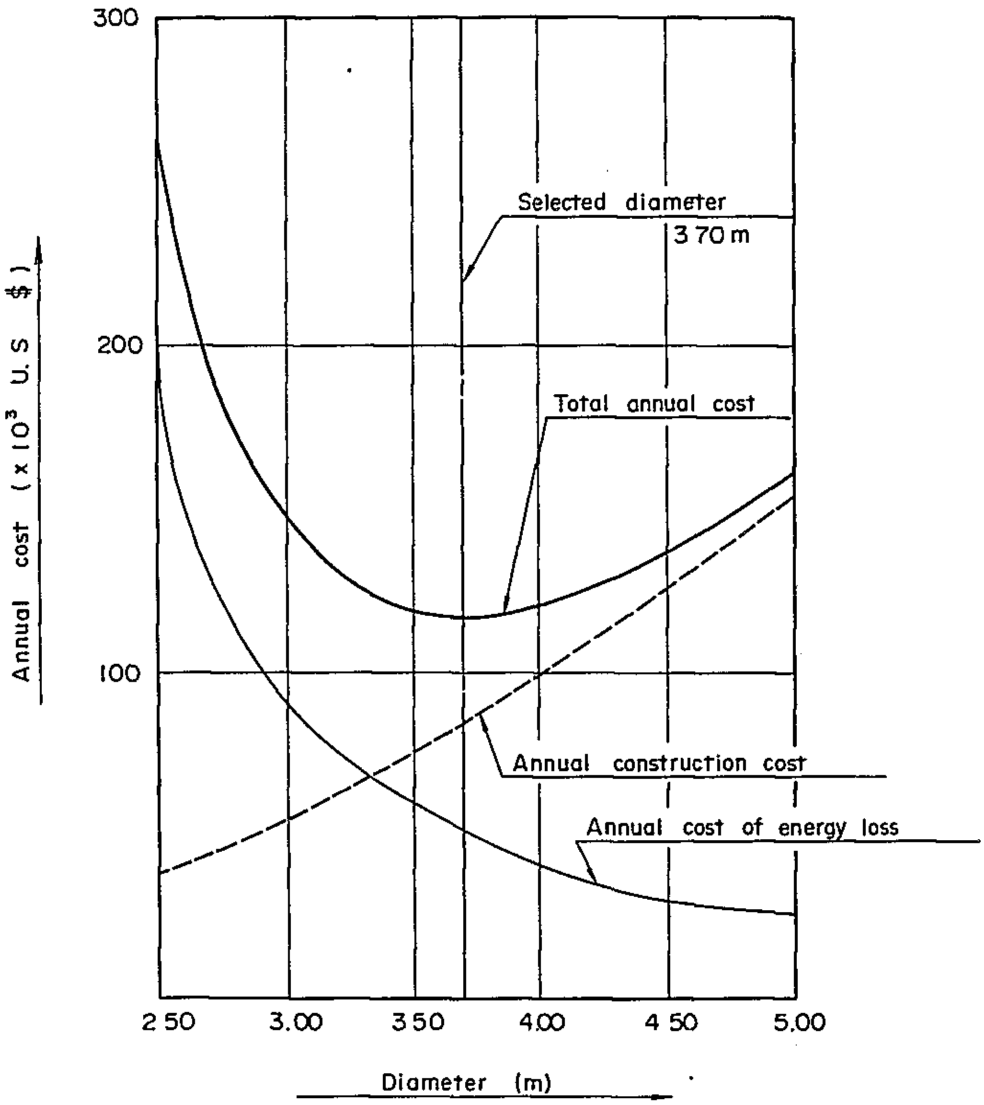


Fig. 8-14 Economical Diameter Diagram (for $Q = 50\ m^3/sec$) (Penstock)

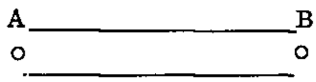


Note : Construction cost of penstock is direct cost excludeing indirect cost (interest, contingency and escalation : 60% of direct cost)

For the penstock system shown in the diagram, the pressure and discharge variation produced when openings of turbine guide vanes are varied are obtained solving the fundamental quation shown in the following sub-paragraph by the successive approximation method for every 0.01 sec.

The openings of guide vanes are considered as varying linearly, with head losses produced concentrated at the assumed penstock end, and are calculated based on the actual penstock length. Calculations are also to include the influence of surging.

(ii) Fundamental Equation



The fundamental equation for pressure waves in a single basic penstock as shown in the above diagram is as given below.

$$H_a,(t) \pm S\cdot Q_a,(t) = H_b,(t - \frac{L}{a}) \pm S\cdot Q_b,(t - \frac{L}{a})$$

where

$H_a,(t)$: pressure at point A at time (t)

$Q_a,(t)$: discharge at point A at time (t)

$H_b,(t - \frac{L}{a})$: pressure at point B at time $(t - \frac{L}{a})$

$Q_b,(t - \frac{L}{a})$: discharge at point B at time $(t - \frac{L}{a})$

S : constant $= \frac{a}{gA}$

a : propagation velocity of pressure wave in inside pipe

g : acceleration of gravity

A : cross-sectional area of pipe

L : length of pipe

(iii) Boundary Conditions

① Boundary Condition at Closing Equipment

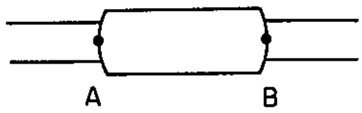
The following boundary condition will prevail in case of linear closing at the closing equipment, namely, guide vanes:

$$Q_a,(t) = (1 - \frac{t}{T})\cdot\sqrt{H_a,(t) - H_b,(t)}$$

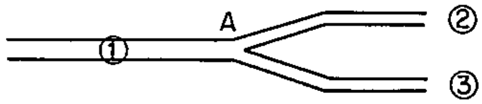
where

t : any time within closing time of closing equipment $(0 \leq t \leq T)$

T : closing time of closing equipment



② Boundary Condition at Bifurcation



In bifurcation, the pressures of the 3 lines of pipe should be equal at point A and discharge should be continuous.

In effect, the boundary conditions are to be such that

$$Q_{1,t} = Q_{2,t} + Q_{3,t}$$

will hold true.

③ Boundary Condition at Intake (Reservoir)

The following boundary condition will hold true at the intake:

$$H_{A,t} = H_{A,0}$$

(iv) Basic Numerical Values

The basic numerical values used for calculations are as follows:

Pressure Tunnel

Length	1,810.22 m
Cross-sectional area	13.854 m^2 (inside diameter 7.00 m)

Surge Tank

Cross-sectional area	50.265 m^2 (inside diameter 8.00 m)
Vertical shaft base elevation	1,669.95 m

Penstock

	Length	Cross-sectional Area
Surge Tank-Bifurcation		
Section 1	92.86 m	13.854 m^2
Section 2	69.35 m	10.752 m^2
Section 3	81.98 m	8.042 m^2
Bifurcation - Turbine	14.37 m	3.142 m^2

Tailrace	
Length	9.73 m
Cross-sectional area	6.870 m^2
Turbine	
Max. available discharge	25.0 x 2 = 50.0 m^3/sec
Number of units	2
Center elevation	1,572.00 m
Closing time	5 sec
Water Level	
Reservoir	1,715.00 m
Tailrace	1,577.00 m
Pressure Wave Propagation Velocity	900 m/sec

(v) Results of Calculations

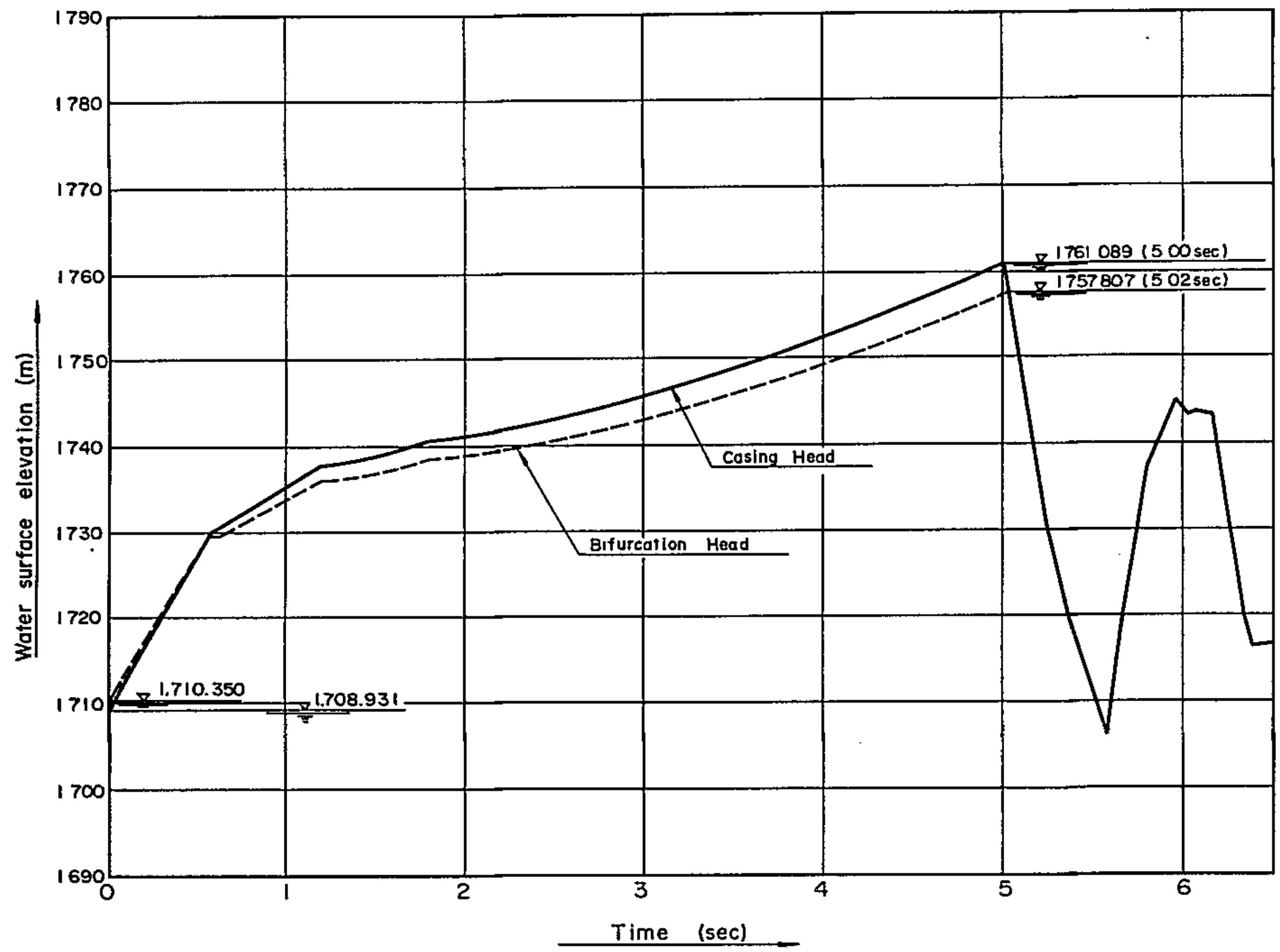
Calculations were made by electronic computer for every 0.01 sec.

The results of calculations are as shown in Fig. 8-15.

The maximum value of water-hammer pressure indicated as a ratio to hydrostatic pressure will be as follows:

$$H_{A,(5.0)} / H_{A,(0)} = \frac{189.089}{143.0} = 1.322$$

Fig. 8-15 Penstock Water-Hammer



Reservoir water surface elevation	1,715.00 m
Tailrace water surface elevation	1,577.00 m
Maximum discharge	25 m³/s x 2 = 50 m³/s
Number of generator	2 units
Closing time	5 sec
Pressure wave propagation velocity	900 m/sec

c) Strength Calculations of Steel Penstock Pipe

(i) Design Heads at Principal Points

The water pressure rises due to water-hammer and surging, based on b), "Water-Hammer Calculations," is to be approximately 40% of the hydrostatic head at the turbine center, and approximately 55% at the starting point of the surge tank. The water pressure in the intermediate parts is assumed to show linear variation.

(ii) Calculation Formula for Shell Thickness

$$\sigma = \frac{P.D}{2(t-\varepsilon)\cdot\eta}$$

where

σ : circumferential stress of steel pipe (kg/cm^2)

P : acting internal pressure (kg/cm^2)

D : inside diameter of pipe (cm)

t : shell thickness (cm)

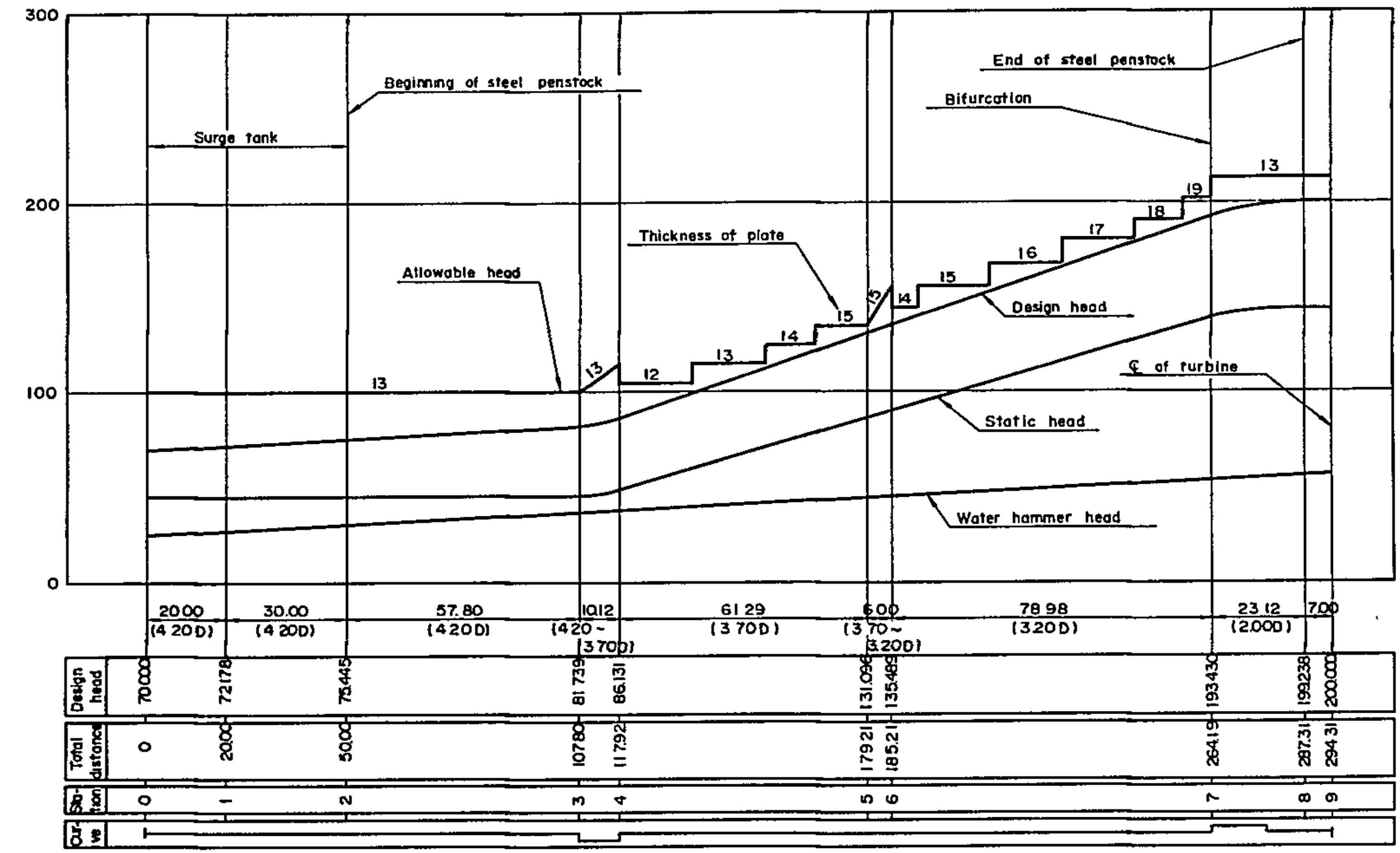
ε : corrosion allowance = 0.15 (cm)

η : longitudinal direction welded joint efficiency = 0.95

(iii) Results of Calculations

The design head and the shell thickness of the steel penstock are shown in Fig. 8-16.

Fig. 8-16 Steel Penstock Design Head Diagram



SPECIFICATION

Maximum discharge	50 m^3/sec	Material	
Maximum static head	143.00 m	Allowable tensile stress	1 950 kg/cm
Water hammer (at turbine)	189.09 m	Welding efficiency	9 5 %
Closing time	5.0 sec	Corrosion allowance	1 5 mm

(7) Powerhouse, Outdoor Switchyard

The powerhouse and switchyard, in consideration of topography and geology, are to be a surface type and an outdoor type, respectively. Draft tubes are to be of elbow type with gates provided at outlets. The outdoor switchyard is to be provided on the roof of the powerhouse. Rock excavated for construction of the powerhouse is to be diverted as much as possible for use in embankment of the main dam.

(8) Rio Cauca Diversion Dam

The type is to be a free overflow concrete gravity dam, and the design of the water passage section will be to have an overflowing section allowing complete overflow of 1,490 m /sec at a water level of 4.9 m. A sand flush gate, 6.0 x 3.0 m, is to be provided near the right-bank intake allowing water to be drawn in a sure manner.

In order to prevent permeation of water at the sand-gravel layer which is to serve as the foundation of the dam, steel sheet piles are to be continuously driven to a depth of 11 m along the dam axis. A sand flush channel of 1.0 x 1.5 m is to be provided at the intake bay that sediment flowing in can be flushed out.

(9) Rio Palace Diversion Dam

The structural type is to be the same as for Rio Cauca Diversion Dam. The water passage section is to be of a design to release flood discharge of 450 m^3/sec at a water level of 3.5 m.

A sand flush gate of 4.0 m x 3.0 m is to be provided near the left-bank intake. A sand flush channel of 1.0 m x 1.0 m is to be provided at the intake bay.

(10) Rio Blanco Diversion Dam

The type is to be a free overflow concrete gravity dam similarly to the two diversion dams described above. The water passage section is to have a design enabling a flood discharge of 175 m^3/sec to be released at a water level of 2.0 m.

A sand flush gate of 3.0 m x 3.0 m is to be provided near the left-bank intake. A sand flush channel of 1.0 x 1.0 m is to be provided at the intake bay.

(11) Cauca Waterway

Cauca Waterway is to consist of an open concrete canal of 2,400 m in length and a tunnel of 220 m. The cross section of the open concrete canal is to be selected to safely and economically conduct a maximum 40.0 m^3.sec of water at a gradient of 1: 600. A side overflow section is to be provided near the intake of the open concrete canal section, the design being for inflow in excess of the maximum capacity of the waterway to be allowed to overflow. The tunnel section is to be of a design for a standard horseshoe-shape non-pressure tunnel with concrete lining.

(12) Palace Waterway, Blanco Waterway

Palace Waterway is to be a semi-circular top, square bottom non-pressure tunnel of length of 770 m and inside diameter of 2.8 m with concrete lining, designed to safely and economically conduct a maximum 12 m^3/sec of water at a gradient of 1: 800.

Blanco Waterway is to be a semi-circular top, square bottom non-pressure tunnel of length of 3,650 m and inside diameter of 3.0 m with concrete lining, designed to safely and economically conduct a maximum 13.8 m^3/sec of water at a gradient of 1: 800.

Both the Palace and Blanco waterways are to be provided with side overflow sections immediately downstream of their intakes, the designs being for inflows in excess of the maximum capacities of the waterways to be allowed to overflow.

8.1.2 Turbine and Generator

The normal effective head of Julumito Hydro-electric Power Station is 126.0 m, the available drawdown 15.0 m and the maximum available discharge per turbine 25.0 m^3/sec. A vertical-shaft Francis turbine is most suitable for these conditions. The turbine output is to be 27,500 kW per unit with a speed of 400 rpm. A butterfly valve is to be installed as the inlet valve. The generator capacity is to be 29,500 kVA per unit at rated power factor of 0.9 (lagging), the voltage 11 kV, and the type a rotating field closed type. The unit system is to be adopted for auxiliary station equipment.

The outdoor switchyard is to be provided on the roof of the powerhouse, and the main transformers of 29,500-kVA, 3-phase, oil-immersed, self-cooled type are to be installed at the upstream side adjacent to the powerhouse. The secondary-side voltage of the transformers is to be 115 kV, and the 115-kV outgoing facility to New Popayan Substation is to consist of a single circuit. A circuit breaker is to be provided at the outlet of Julumito Power Station and a transfer trip relaying system is to be adopted for the section between it and the circuit breaker to be installed at the receiving side at New popayan Substation. In effect, relay equipment to be installed between New Popayan Substation and Julumito Power Station will not operate during external faulting, but will operate only during internal faulting (during faulting on the Julumito Power Station side), and when this relay equipment operates, simultaneously with tripping the circuit breaker at its own end, it will transmit a tripping signal to Julumito Power Station to trip the circuit breaker at the 115-kV line side of Julumito Power Station.

Regarding outdoor switchyard equipment other than the two main transformers, they will be arranged on the roof of the powerhouse to reduce the amount of excavation of the slope in the vicinity of the powerhouse.

A one-man control system is to be adopted for the above turbines, generators and outdoor switchyard equipment, and the design will be such as to enable operation of all equipment to be done from the distribution panel room. The single line diagram of the power station is shown in Fig. 8-17.

8.1.3 Transmission Line and Telecommunication Facilities

In order to transmit the power generated at Julumito Hydro-electric Power Station to New Popayan Substation, a 115-kV transmission line, single circuit, 10 km is to be constructed between Julumito Power Station and New Popayan Substation. With the exception of the outgoing point of the transmission line at the power station, the route is through a gentle hill area and there will be no problem in particular, but in view of the location of the powerhouse, a dead-end tower is to be constructed on a hill at the opposite bank to cross the Rio Cauca once and then recross to the right-bank side of the Rio Cauca to take a straight-line route to New Popayan Substation. The route map is given in Fig. 8-18. The standard steel tower designs to be used for this transmission line are shown in Fig. 8-19.

Regarding telecommunication facilities, the following are to be provided for protection and communication between Julumito Power Station and New Popayan Substation, the receiving substation.

- Load dispatching communication facilities
- Transfer trip relaying facilities

In order to make up the above-mentioned telecommunication facilities, power line carrier equipment is to be provided between Julumito Power Station and New Popayan Substation, and this telecommunication system is shown in Fig. 8-20.

As described above, the scope of work in this Project extends as far as the incoming steel structure at the outdoor switchyard of New Popayan Substation. However, with regard to telecommunication facilities, opposite-end facilities to those at Julumito Hydro-electric Power Station will be installed inside New Popayan Substation.

Fig. 8-17 Power House Single-Line Diagram

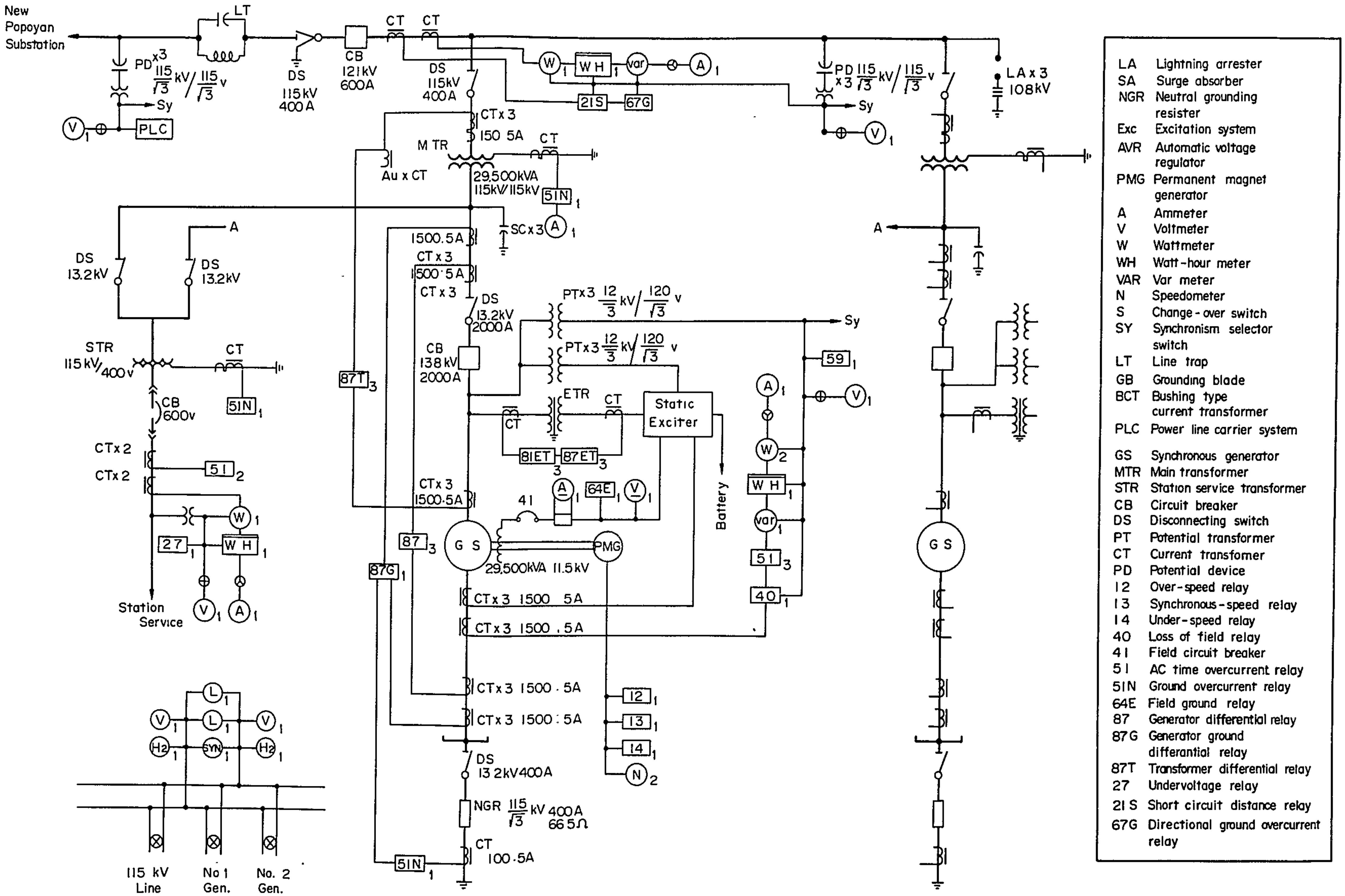


Fig. 8-18 Transmission Line Route

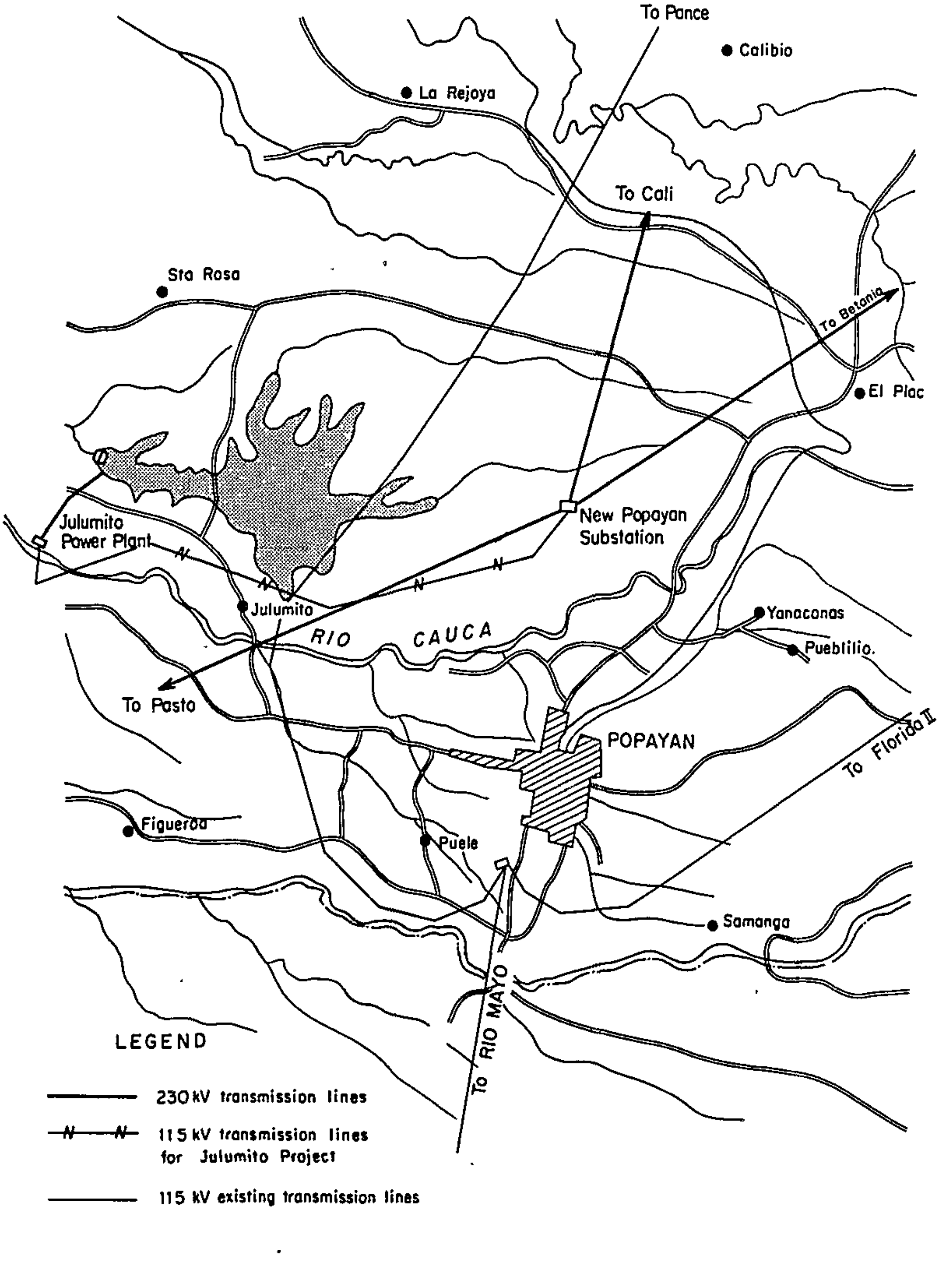


Fig. 8-19 Transmission Line Tower Configration

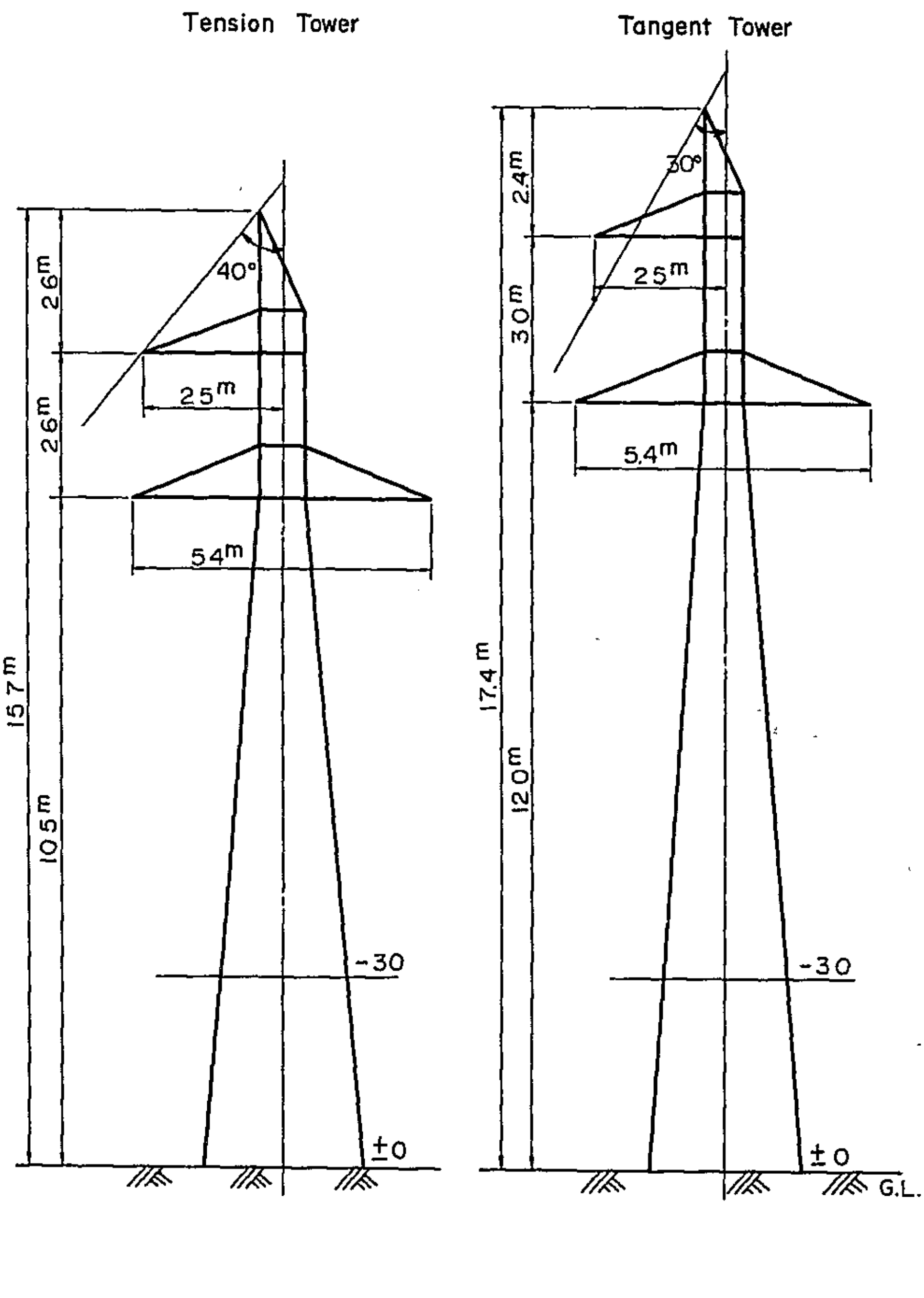
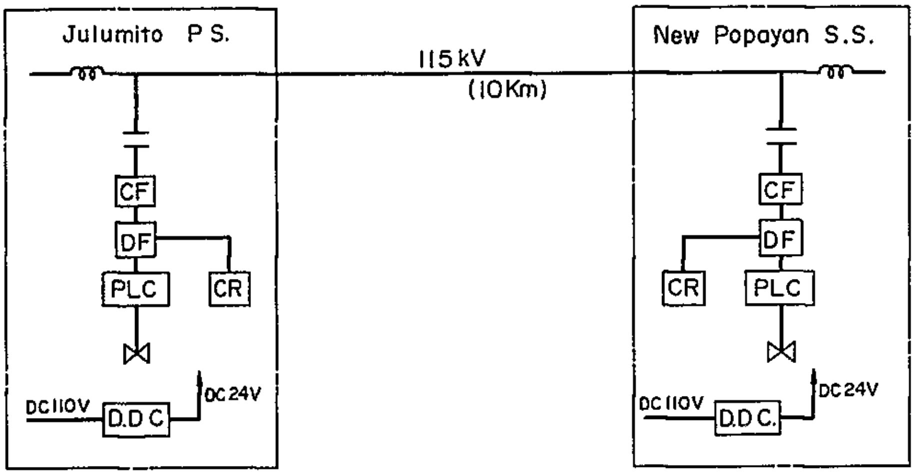
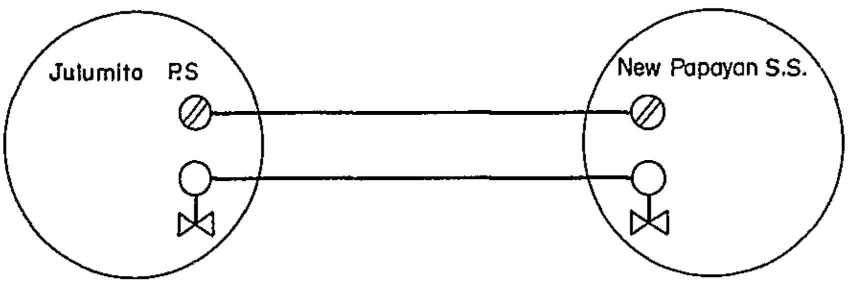


Fig. 8-20 Telecommunication System Diagram



Legend

- Line trap
- Coupling capacitor
- CF Coupling filter
- PLC Power line carrier telephone terminal equipment
- DF
- CR Carrier transfer trip relaying equipment
- Telephone handset
- DDC DC-DC Converter



Legend

- Load dispatching telephone channel
- Carrier transfer trip relaying channel

8.1.4 Major Specifications

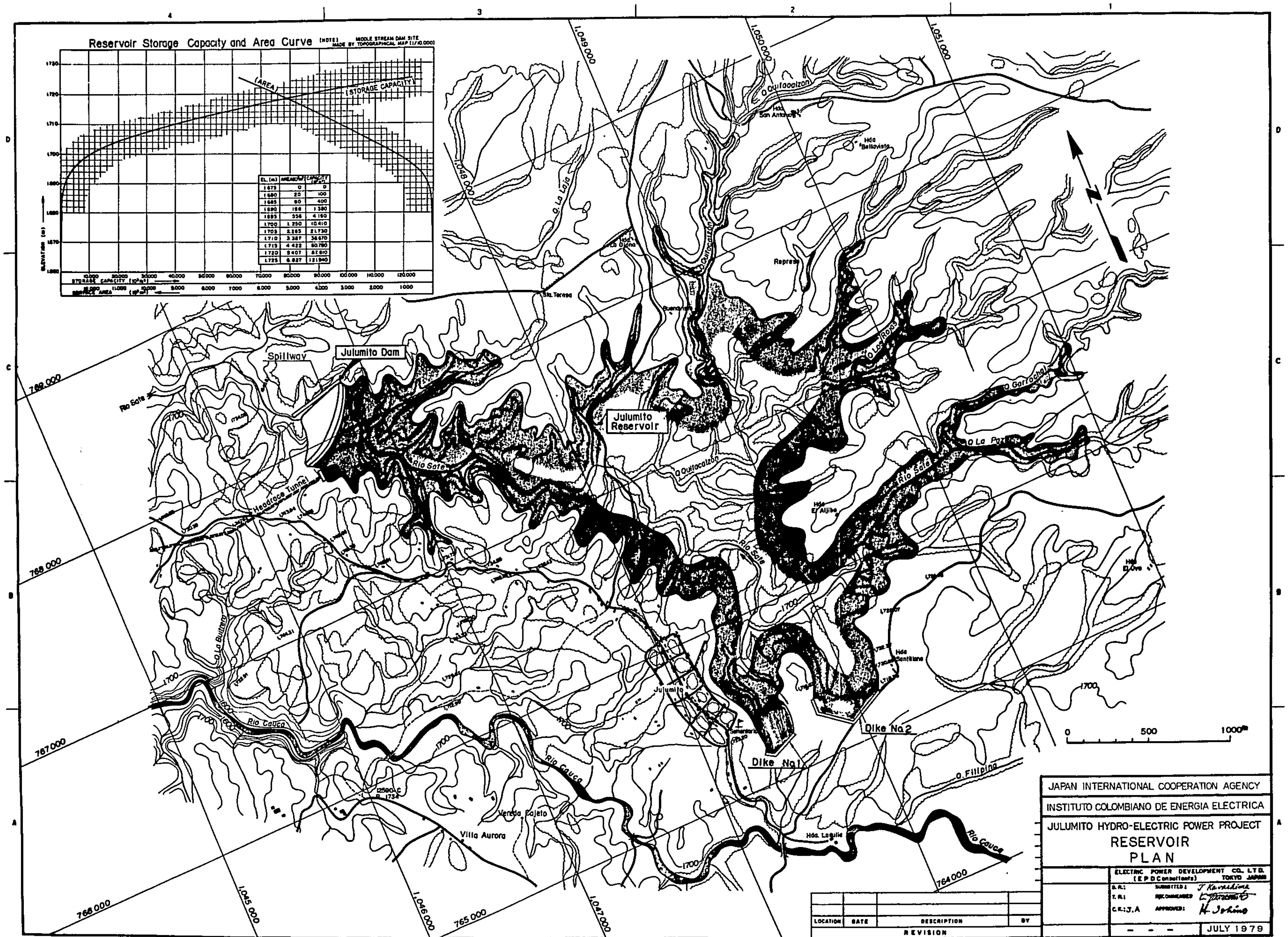
The major specifications of the various structures in the Julumito Hydro-electric Power Project are as indicated below.

Item	Description	
CIVIL STRUCTURE		
Dam	Inclined core type, Rockfill dam	
	Elevation of crest	1,718.0 m
	Height	82.0 m
	Length of crest	340.0 m
	Width of crest	8.0 m
	Slope of upstream face	1 : 2.3
	Slope of downstream face	1 : 1.8
	Volume of dam	1,254,000 m^3
Diversion	Tunnel type	
	Length	380.0 m
Outlet	Tunnel type	
	Length	365.0 m
	Type of valve	Hollow Jet Valve
Spillway	Chute spillway with control gate	
	Design flood discharge	95.0 m^3/sec
	Type of gate	Flap gate
	Number of gate	1
	Dimension of gate	4.00 x 5.00
Dike No. 1	Earthfill	
	Height	4.0 m
	Length of crest	224.64 m
	Slope of upstream face	1 : 2.0
	Slope of downstream face	1 : 2.0
	Volume of dike No. 1	12,500 m^3

Item	Description	
Dike No. 2	Earthfill	
	Height	7.0 m
	Length of crest	603.80 m
	Slope of upstream face	1 : 2.0
	Slope of downstream face	1 : 2.0
	Volume of dike No. 2	35,000 m^3
Intake	Inclined type, reinforced concrete structure	
	Maximum discharge	50.0 m^3/sec
	Type of gate	Roller gate
	Dimension of gate	4.5 m x 7.0 m
	Screen	8.0 m x 40.3 m
Headrace	Pressure tunnel	
	Length	1,775.0 m
	Shape	Circular
	Inside diameter	4.2 m
Surge tank	Orifice type	
	Inside diameter of tank	8.0 m
	Inside diameter of orifice	2.2 m
	Height	63.0 m
Penstock	Welded steel, ring girder type	
	Materials	SM 58 or SM 53 (JIS)
	Length	287.31 m
	Number of line	1
	(Bifuracate into 2 lines at No. 3 anchor blocks)	
	Inside diameter	4.2 3.2 m
	(2.0 1.6 m after bifuracation)	
Powerhouse	Reinforced concrete structure	
	Length	31.4 m
	Width	20.4 m
	Height	30.0 m

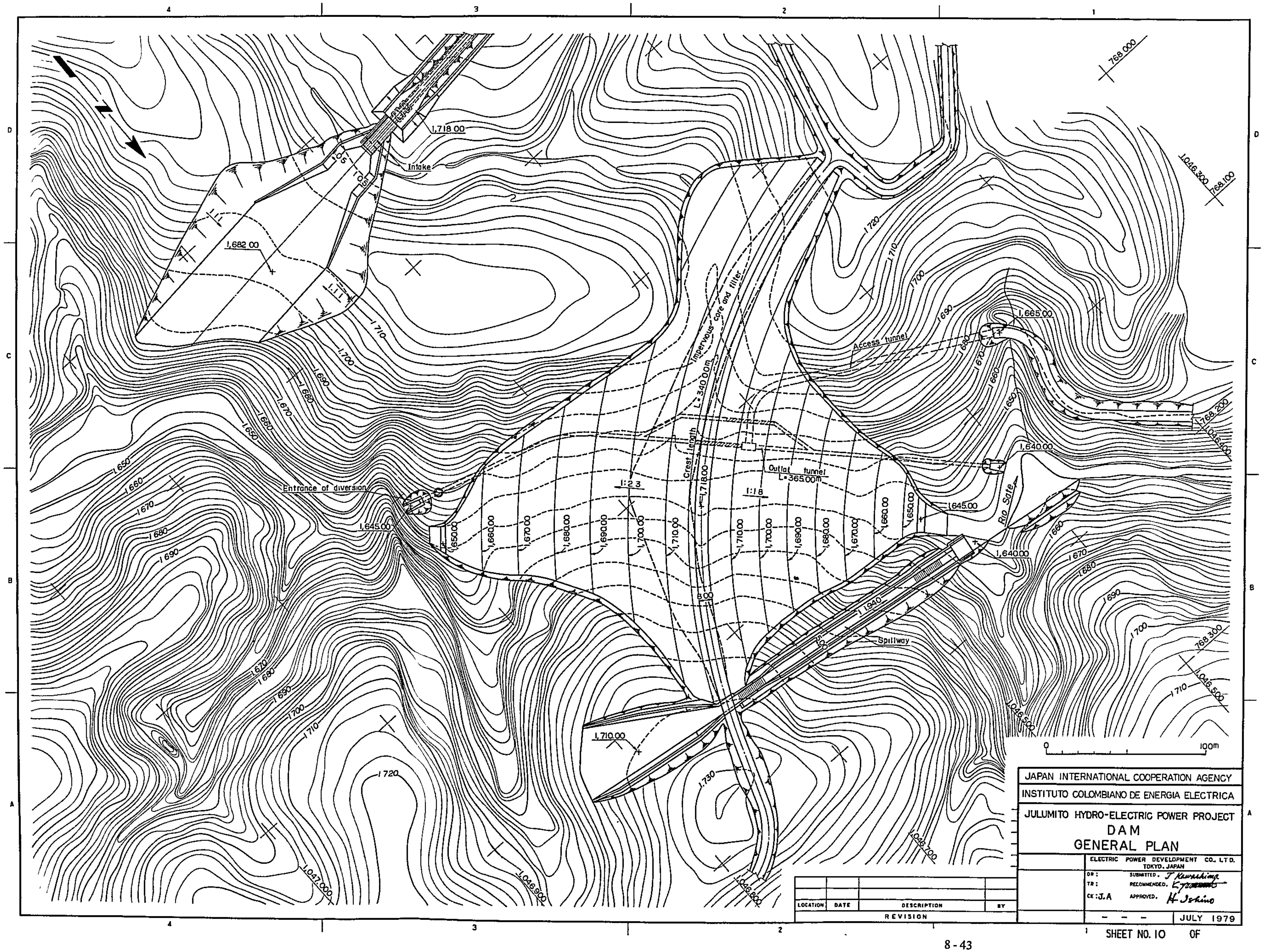
Item	Description	
Rio Cauca diversion dam	Free overflow type concrete gravity dam	
	Height	12.5 m
	Length	75.5 m
	Sand flash gate	Slide gate 6.0 x 3.0 m
	Volume of concrete	9,300 m^3
Rio Palace diversion dam	Free overflow type concrete gravity dam	
	Height	8.7 m
	Length	34.4 m
	Sand flash gate	Slide gate 4.0 x 3.0 m
	Volume of concrete	1,400 m^3
Rio Blanco Diversion dam	Free overflow type concrete gravity dam	
	Height	7.5 m
	Length	42.4 m
	Sand flash gate	Slide gate 3.0 x 3.0 m
	Volume of concrete	900 m^3
Cauca diversion waterway	Open channel and tunnel	
	Capacity	40.0 m^3/sec
	Length of open channel	2,400.0 m
	Length of tunnel	220.0 m
Palace diversion waterway	Tunnel	
	Capacity	12.0 m^3/sec
	Length	770 m
Blanco diversion waterway	Tunnel	
	Capacity	13.8 m^3/sec
	Length	3,650 m
ELECTRIC EQUIPMENT		
Turbine	Vertical shaft Fransis type	
	Output	27,500 kW
	Maximum discharge	25.0 m^3/sec
	Revolution	400 rpm
	Number of units	2

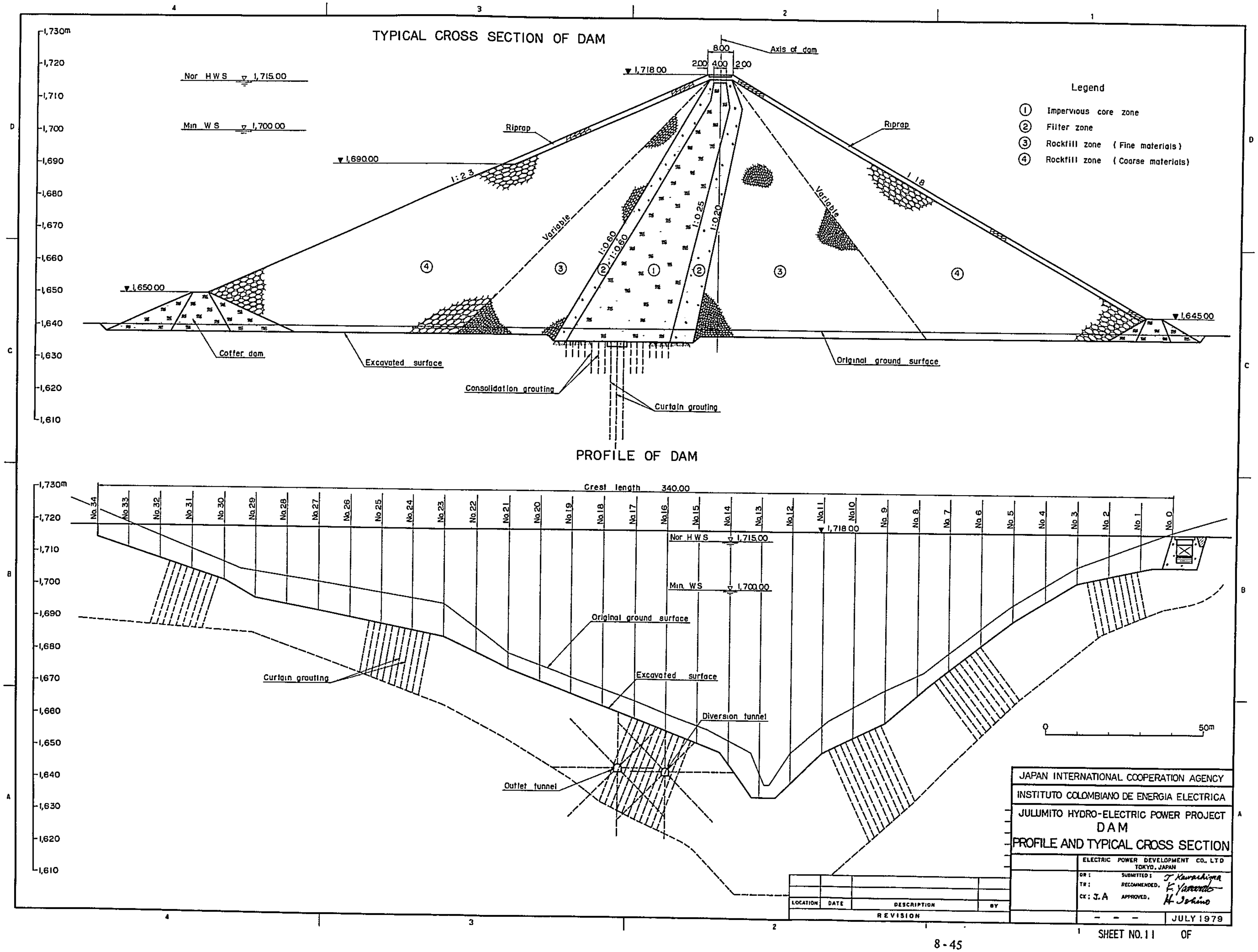
Item	Description	
Generator	Three-phase, synchronous generator, vertical shaft, rotating field closed type	
	Capacity	29,500 kVA
	Voltage	11.0 kV
	Frequency	60 Hz
	Number of units	2
Transformer	Three-phase outdoor, oil-immersed self-cooled type	
	Capacity	29,500 kVA
	Voltage	10.5/115 kV
	Number of units	2
Outdoor Switchyard	Transmission voltage	115 kV
Transmission Line	Distance	10 Km
	Voltage	115 kV
	Number of circuits	1 cct
	Conductor	160 mm^2 ACSR
	Insulator	250 mm suspension insulator ball and socket type
	Overhead ground wire	45 mm^2 GSC
	Support	Steel tower
TELECOMMUNICATION SYSTEM	For communication system	1 ch
	For Carrier relaying system	1 ch

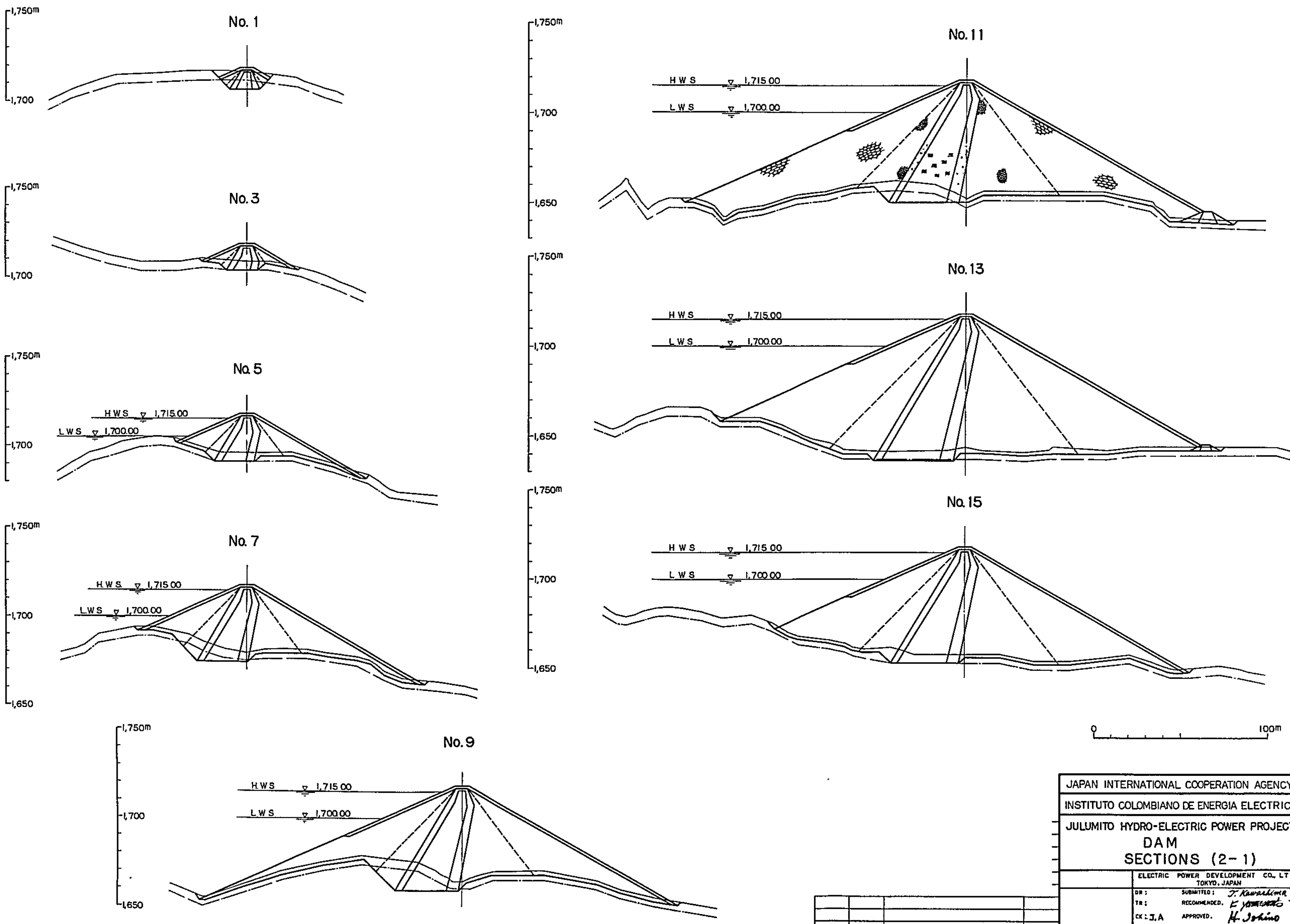


Reservoir Storage Capacity and Area Curve
(NOTE) MIDDLE STREAM DAM SITE MADE BY TOPOGRAPHICAL MAP (1/10,000)
(AREA)
(STORAGE CAPACITY)
EL. (m) | AREA | CAPACITY
1675 | 0 | 0
1680 | 20 | 100
1685 | 80 | 400
1690 | 188 | 1 380
1695 | 556 | 4 190
1700 | 1 250 | 10 410
1705 | 2 265 | 21 730
1710 | 3 387 | 38 670
1715 | 4 422 | 60 780
1720 | 5 407 | 87 610
1725 | 6 827 | 121 940
ELEVATION (m)
STORAGE CAPACITY
SURFACE AREA
Spillway
Julumito Dam
Julumito Reservoir
Headrace Tunnel
Rio Sate
Rio Cauca
Q. Quitacalzon
Q. La Laja
Q. Los Rojas
Q. Garrocha
Q. La Paz
Q. La Builtrera
Q. Filipina
Hda San Antonio
Hda Bellavista
Hda La Diana
Represa
Sta. Teresa
Buenavista
Hda El Aljibe
Hda El Ovo
Hda Santillana
Hda. Laguna
Julumito
Cementerio
Dike No.1
Dike No.2
Vereda Cajete
Villa Aurora
1,045 000
1,046 000
1,047 000
1,048 000
1,049 000
1,050 000
1,051 000
769 000
768 000
767 000
766 000
765 000
764 000
1700
0
500
1000m
N
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
RESERVOIR
PLAN
ELECTRIC POWER DEVELOPMENT CO., LTD.
(E P D Consultants) TOKYO JAPAN
SUBMITTED: J. Kawashima
RECOMMENDED:
C.K.: J.A
APPROVED: H. Johino
JULY 1979
LOCATION | DATE | DESCRIPTION | BY
REVISION

SHEET NO. 9 OF


1,718.00
Intake
1,682.00
1,665.00
Access tunnel
Impervious core and filter
L=340.00m
Crest length
1,718.00
Outlet tunnel
L=365.00m
1,640.00
Entrance of diversion
1:2.3
1:1.8
1,645.00
Rio Sate
1,640.00
Spillway
1,710.00
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
DAM
GENERAL PLAN
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN
LOCATION
DATE
DESCRIPTION
BY
REVISION
JULY 1979
SHEET NO. 10 OF


TYPICAL CROSS SECTION OF DAM
Axis of dam
8.00
2.00 4.00 2.00
▼1,718.00
Nor HWS ▽ 1,715.00
Min WS ▽ 1,700.00
▼1,690.00
▼1,650.00
▼1,645.00
Riprap
1:2.3
1:1.8
Variable
1:0.60
1:0.25
1:0.20
Cofter dam
Excavated surface
Original ground surface
Consolidation grouting
Curtain grouting
Legend
① Impervious core zone
② Filter zone
③ Rockfill zone (Fine materials)
④ Rockfill zone (Coarse materials)
PROFILE OF DAM
Crest length 340.00
Diversion tunnel
Outlet tunnel
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
DAM
PROFILE AND TYPICAL CROSS SECTION
ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN
JULY 1979
SHEET NO.11 OF
LOCATION DATE DESCRIPTION BY
REVISION



JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

DAM
SECTIONS (2-1)

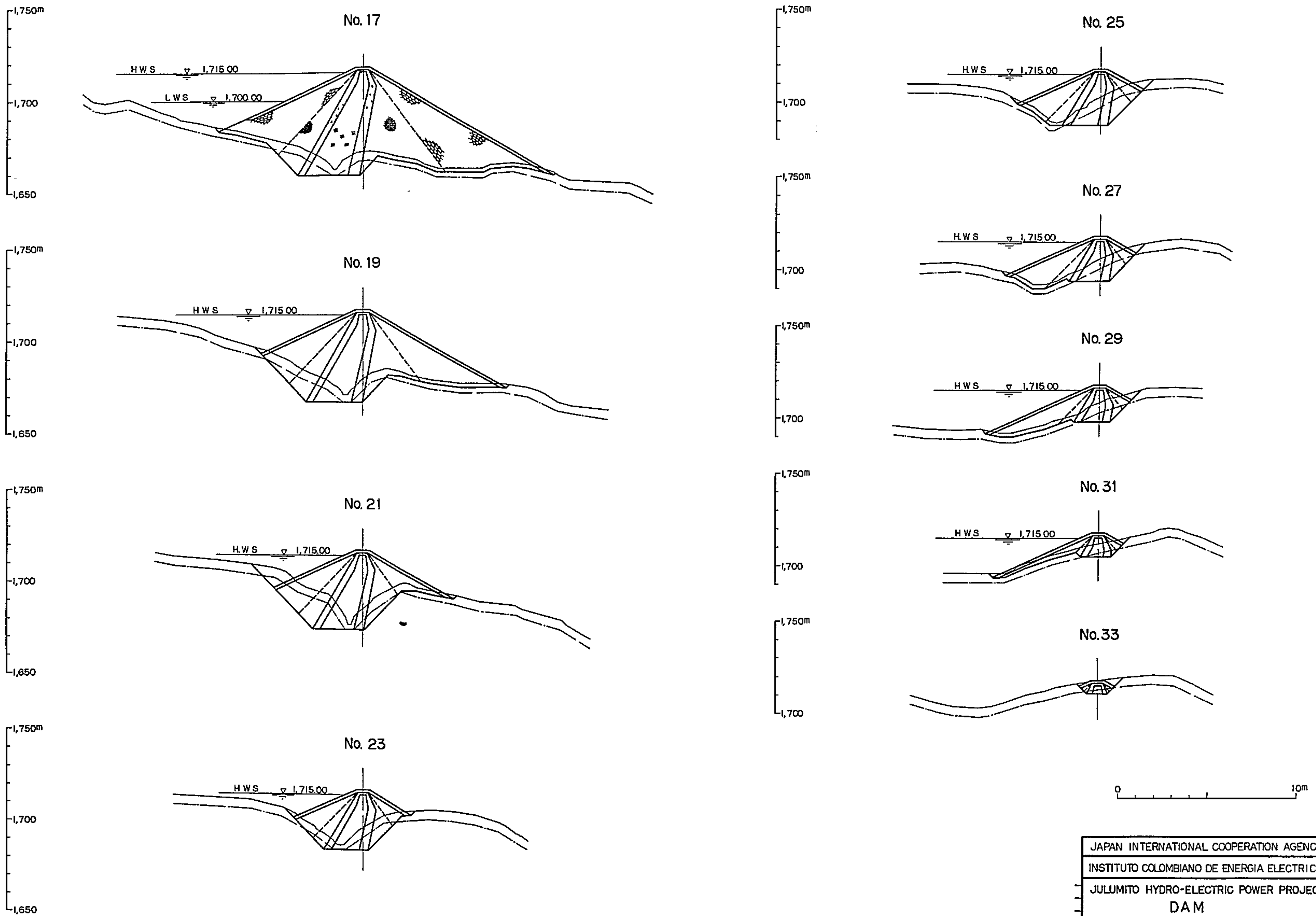
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN

LOCATION	DATE	DESCRIPTION	BY
REVISION			

DR : SUBMITTED : T. Kawashima
TR : RECOMMENDED. F. Yamamoto
CK : J.A APPROVED. H. Ishino

— — — JULY 1979

SHEET NO. 12 OF



LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

DAM
SECTIONS (2-2)

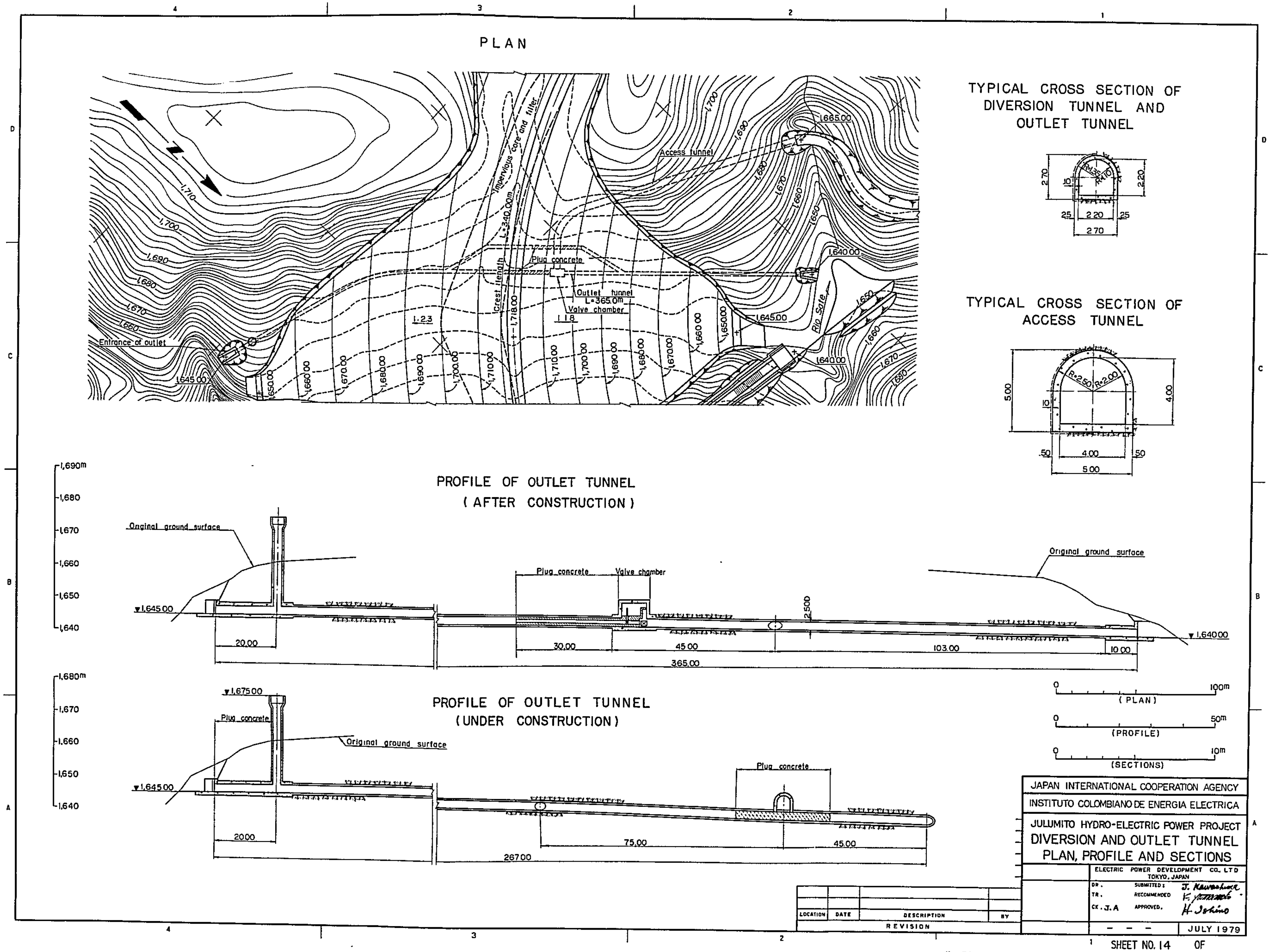
ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN

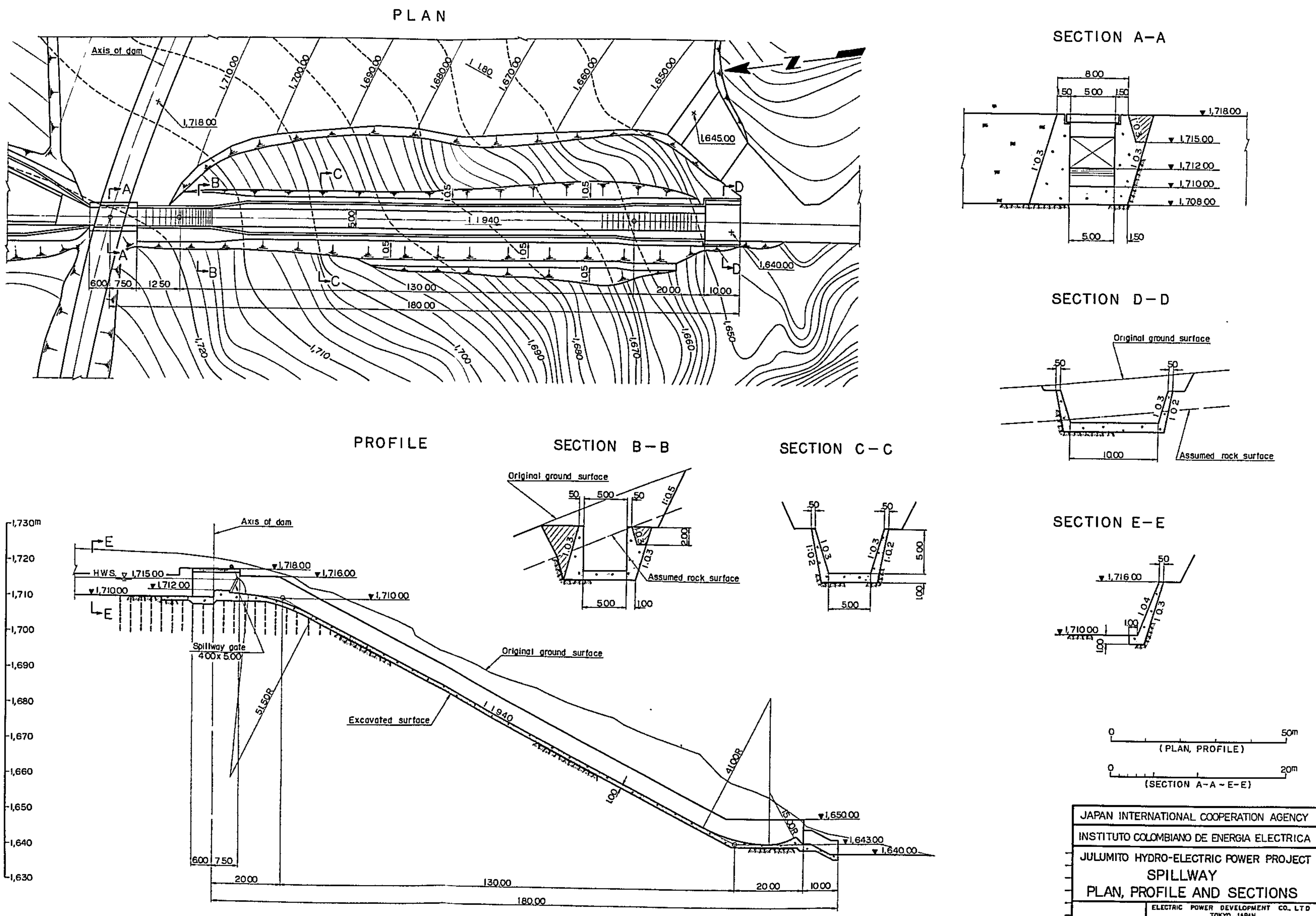
DR :
TR :
CK : J.A

SUBMITTED : T. Kawashima
RECOMMENDED. K. Yamamoto
APPROVED. H. Ishino

— — — JULY 1979

SHEET NO. 13 OF


PLAN
Access tunnel
Impervious core and filter
L=340.00m
Crest length
Plug concrete
Outlet tunnel
L=365.0m
Valve chamber
Entrance of outlet
Rio Sote
1:2.3
1:1.8
1,665.00
1,640.00
1,645.00
TYPICAL CROSS SECTION OF DIVERSION TUNNEL AND OUTLET TUNNEL
TYPICAL CROSS SECTION OF ACCESS TUNNEL
PROFILE OF OUTLET TUNNEL
(AFTER CONSTRUCTION)
Original ground surface
Plug concrete
Valve chamber
▼1,645.00
▼1,640.00
20.00
30.00
45.00
103.00
10.00
365.00
PROFILE OF OUTLET TUNNEL
(UNDER CONSTRUCTION)
▼1,675.00
Plug concrete
Original ground surface
▼1,645.00
20.00
75.00
45.00
267.00
(PLAN)
(PROFILE)
(SECTIONS)
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
DIVERSION AND OUTLET TUNNEL
PLAN, PROFILE AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN
SUBMITTED:
RECOMMENDED
APPROVED.
LOCATION
DATE
DESCRIPTION
BY
REVISION
JULY 1979
SHEET NO. 14 OF



LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

SPILLWAY
PLAN, PROFILE AND SECTIONS

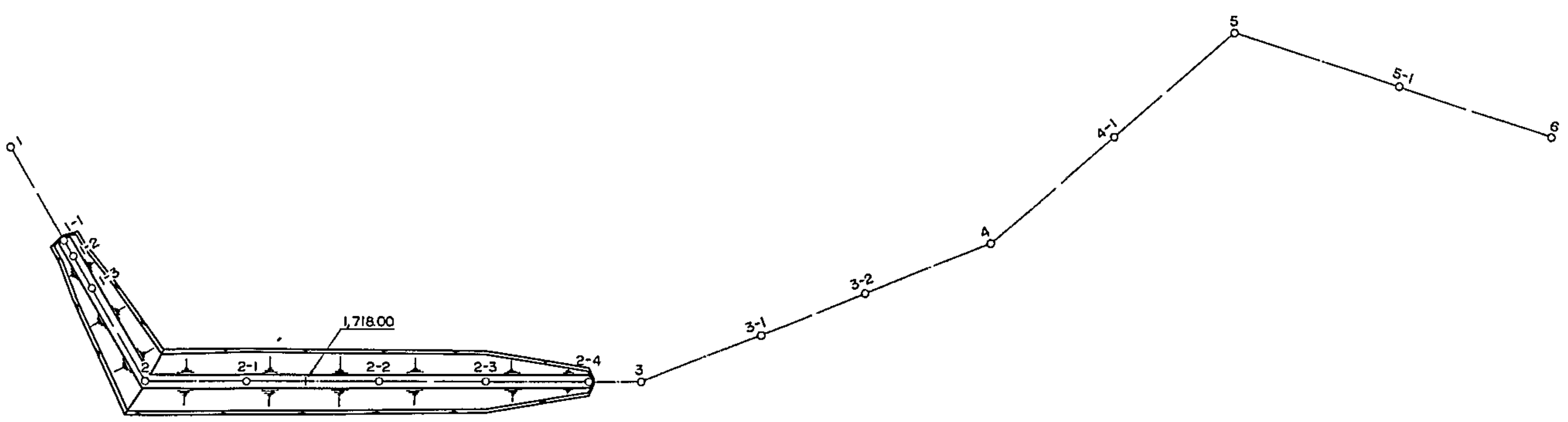
ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN

DR. SUBMITTED. T. Kawashima
TR: RECOMMENDED. K. Yamamoto
CK J.A APPROVED. H. Ishino

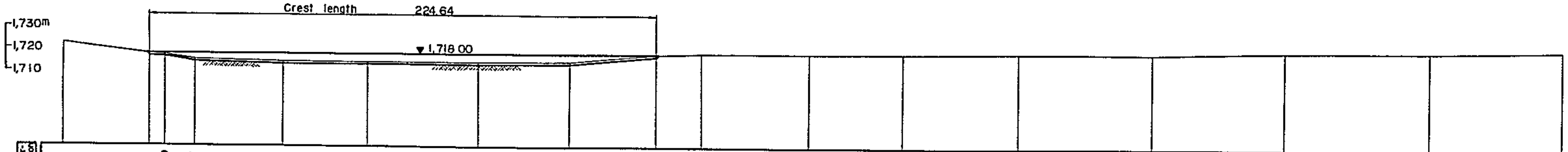
— — — JULY 1979

SHEET NO. 15 OF

PLAN



PROFILE

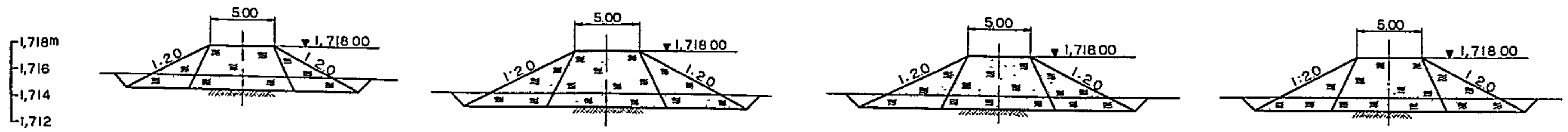


No	Distance	Ground surface elevation
No. 1	0	1,722.41
1-1	39.40	1,717.81
1-2	6.65	1,717.79
1-3	13.60	1,715.67
No 2	39.25	1,714.80
2-1	37.63	1,714.93
2-2	49.32	1,714.93
2-3	39.40	1,715.09
2-4	38.79	1,718.28
No. 3	20.23	1,718.88
3-1	46.24	1,718.88
3-2	41.57	1,718.99
No. 4	51.20	1,719.57
4-1	59.90	1,719.69
No 5	58.30	1,720.62
5-1	63.70	1,720.59
No 6	58.30	1,721.53

0 100m (Plan and profile)

0 20m (Sections)

SECTION 1-3 SECTION 2 SECTION 2-1 SECTION 2-3



LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

DIKE NO. 1

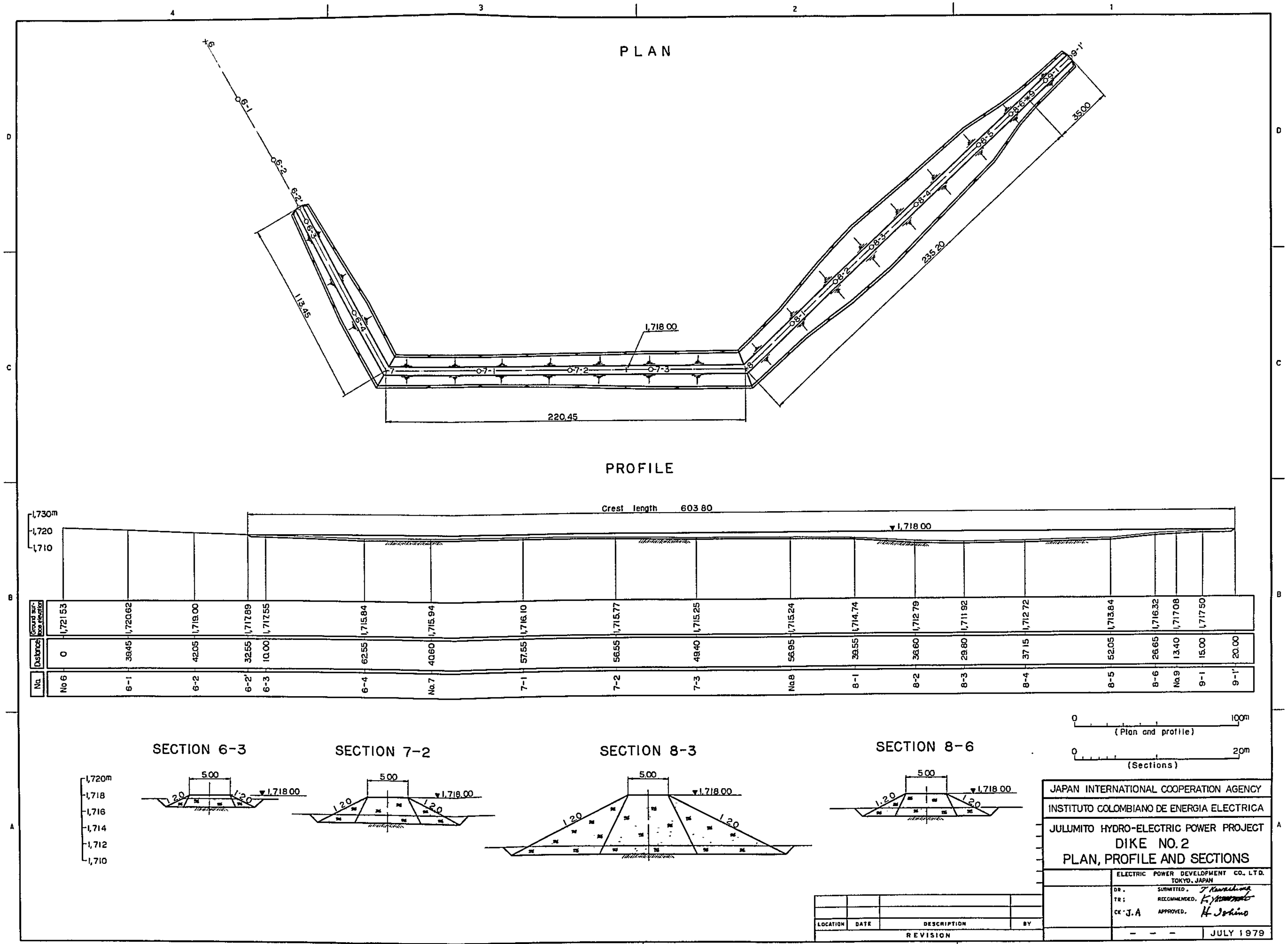
PLAN, PROFILE AND SECTIONS

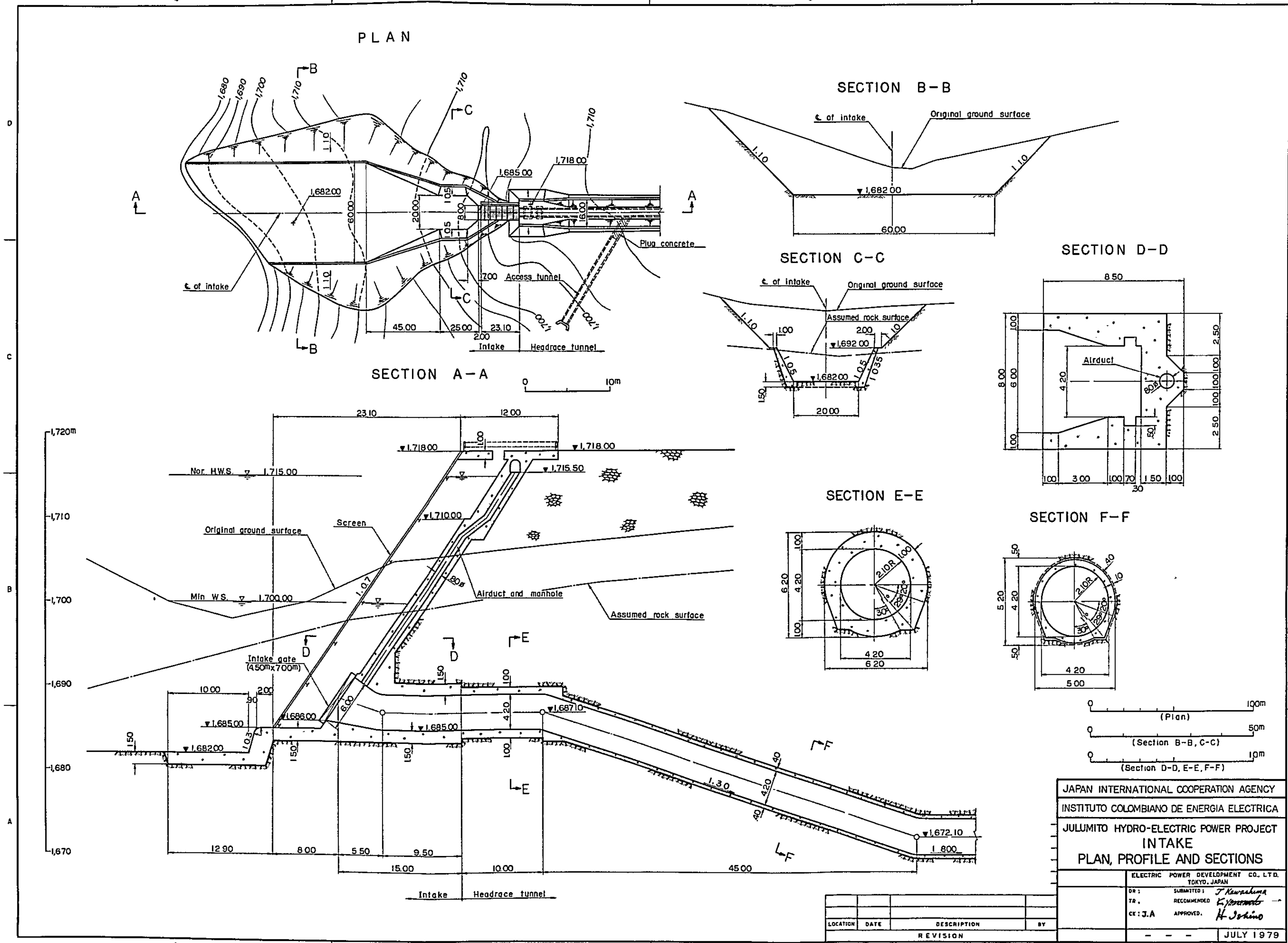
ELECTRIC POWER DEVELOPMENT CO., LTD TOKYO, JAPAN

DR. SUBMITTED: T. Kawashima
TR: RECOMMENDED:
CK: J.A APPROVED: H. Ishino

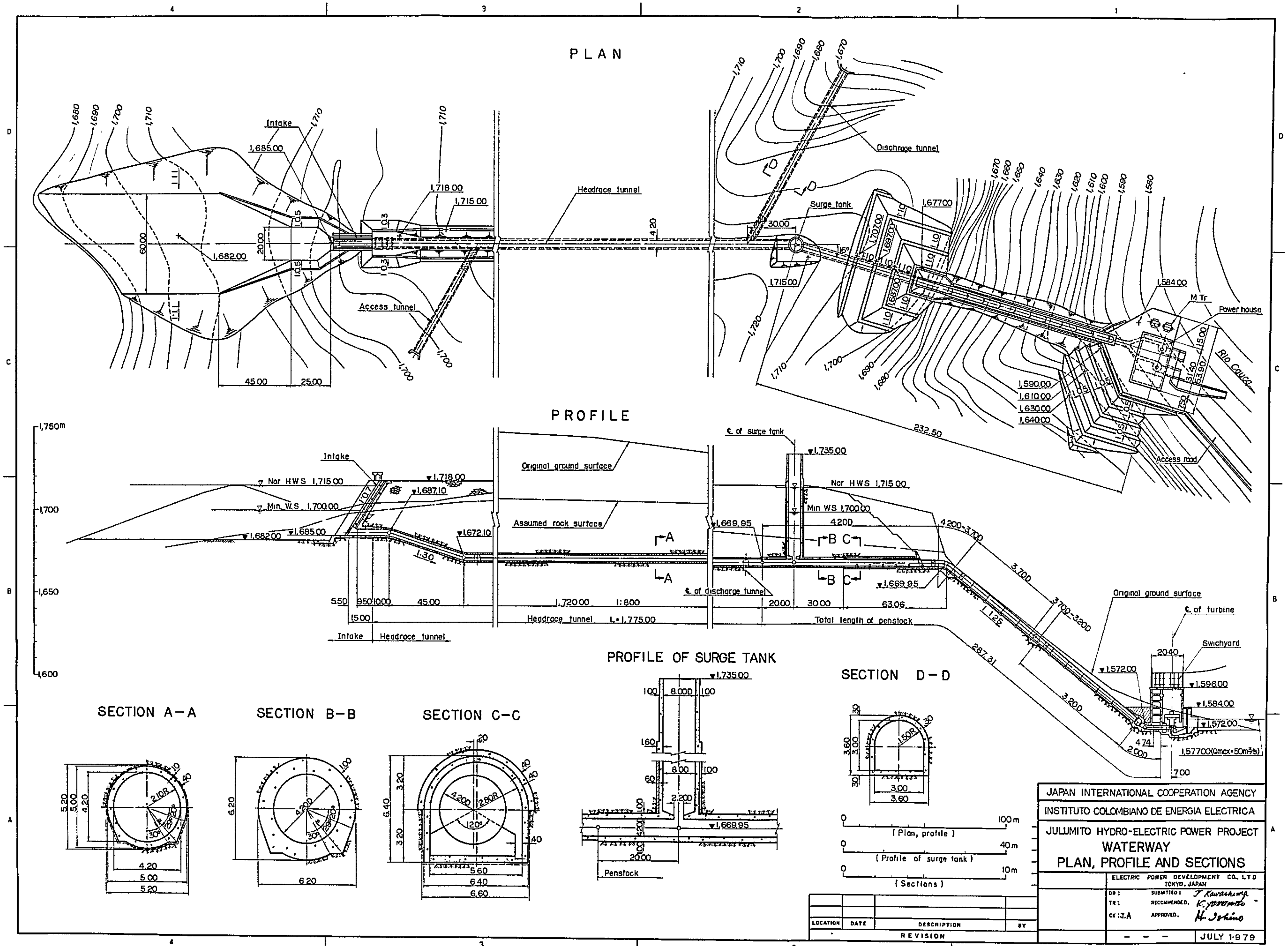
JULY 1979

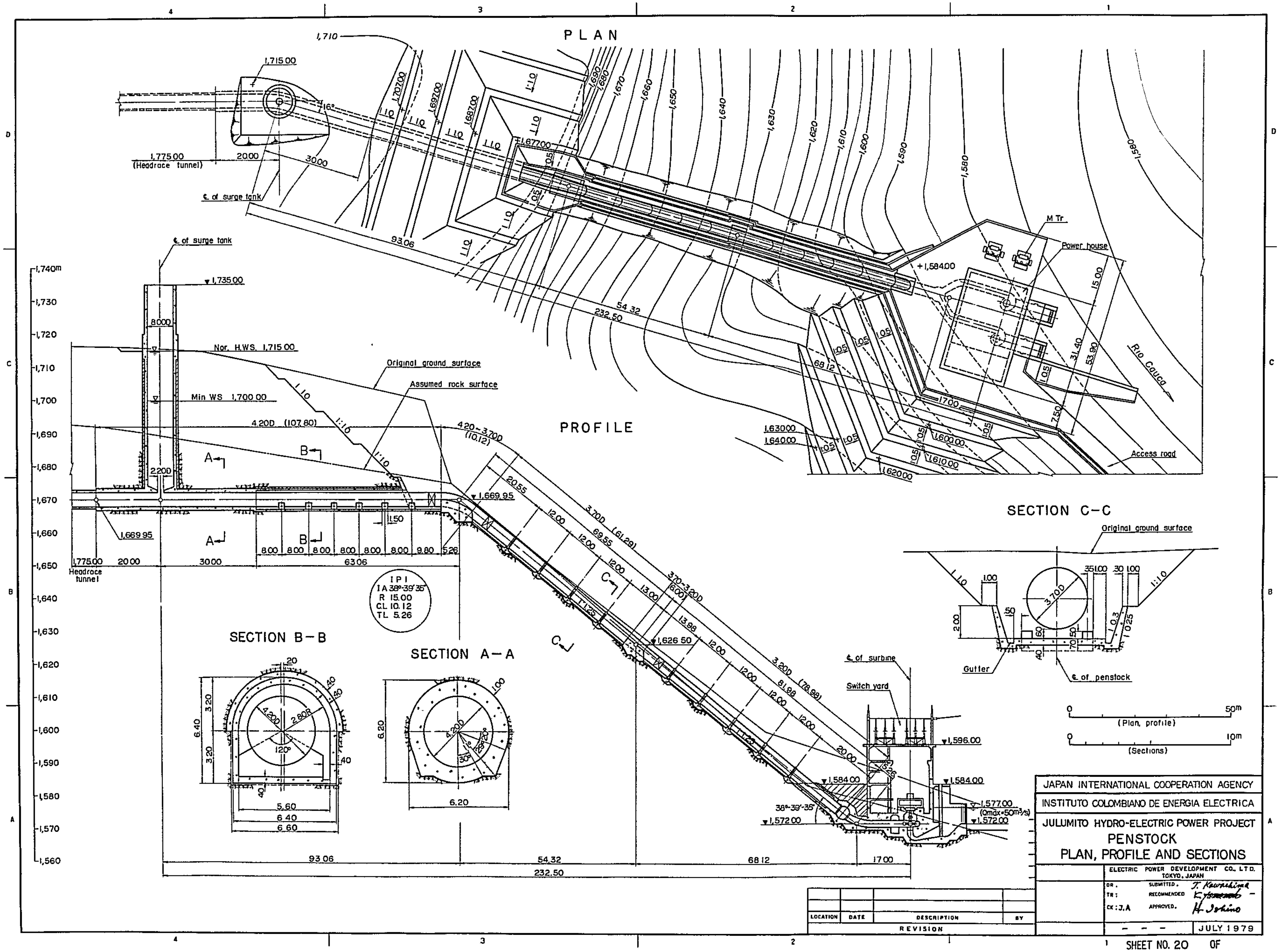
SHEET NO. 16 OF

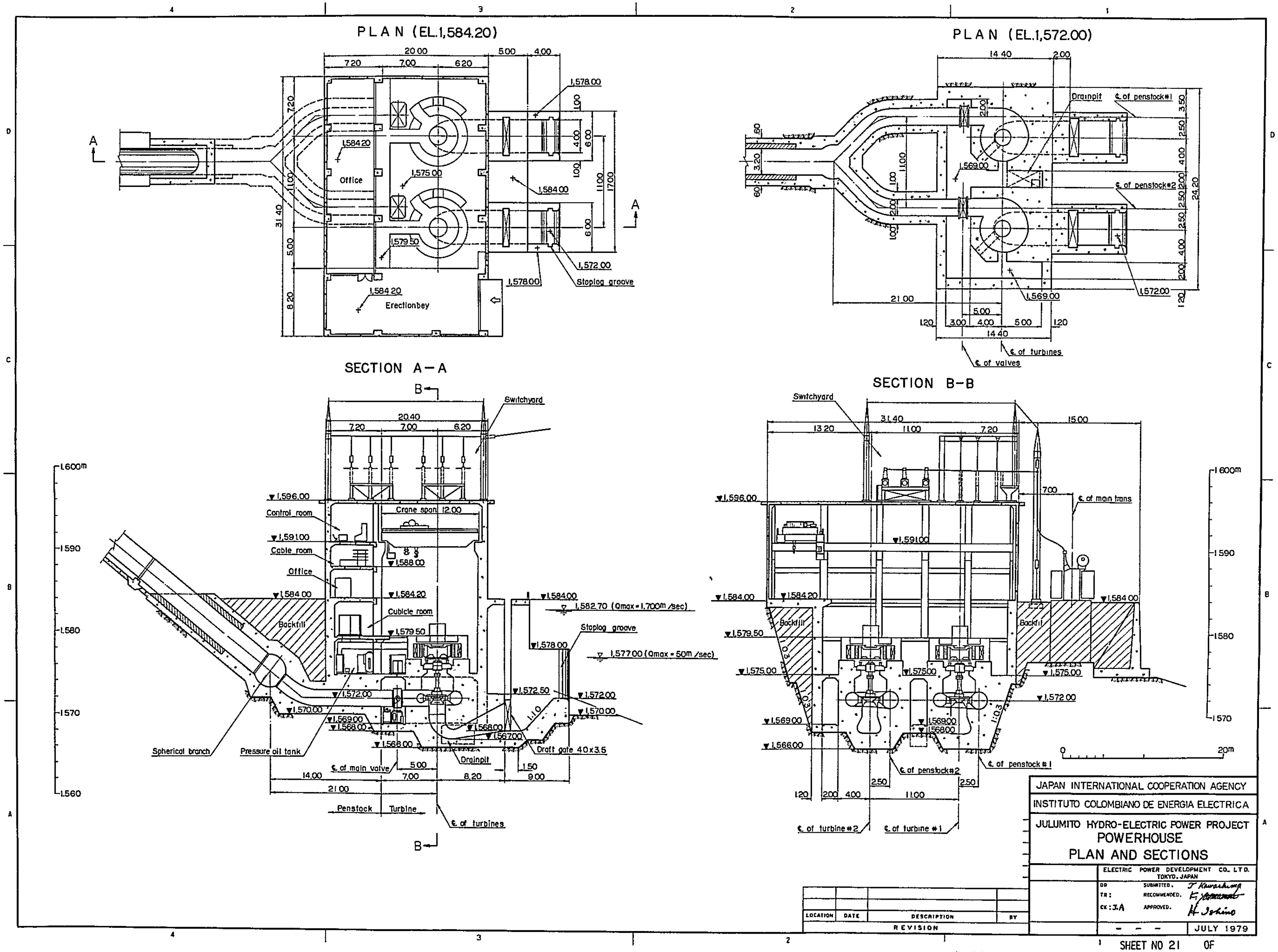

PLAN
1,718.00
113.45
220.45
235.20
35.00
PROFILE
Crest length 603.80
▼1,718.00
1,730m
1,720
1,710
Ground surface elevation
Distance
No.
No.6 6-1 6-2 6-2' 6-3 6-4 No.7 7-1 7-2 7-3 No.8 8-1 8-2 8-3 8-4 8-5 8-6 No.9 9-1 9-1'
1,721.53 1,720.62 1,719.00 1,717.89 1,717.55 1,715.84 1,715.94 1,716.10 1,715.77 1,715.25 1,715.24 1,714.74 1,712.79 1,711.92 1,712.72 1,713.84 1,716.32 1,717.08 1,717.50
0 39.45 42.05 32.55 10.00 62.55 40.60 57.55 56.55 49.40 56.95 39.55 36.60 29.80 37.15 52.05 26.65 13.40 15.00 20.00
0 100m (Plan and profile)
0 20m (Sections)
SECTION 6-3
SECTION 7-2
SECTION 8-3
SECTION 8-6
5.00
1:2.0
▼1,718.00
1,720m
1,718
1,716
1,714
1,712
1,710
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
DIKE NO. 2
PLAN, PROFILE AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD. TOKYO, JAPAN
DR.
TR.
CK. J.A
SUBMITTED. T. Kawashima
RECOMMENDED.
APPROVED. H. Ishino
LOCATION DATE DESCRIPTION BY
REVISION
JULY 1979
SHEET NO. 17 OF

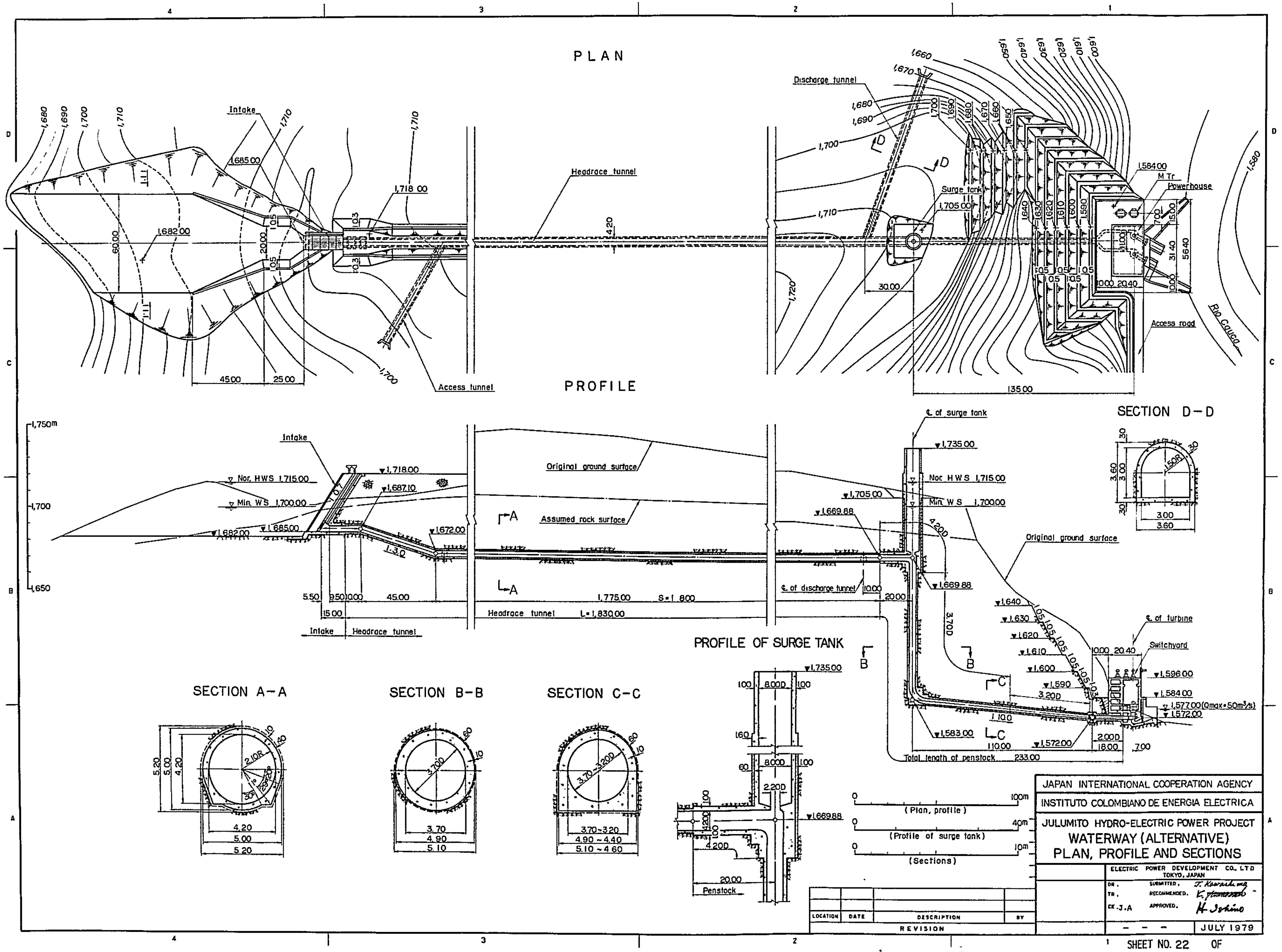


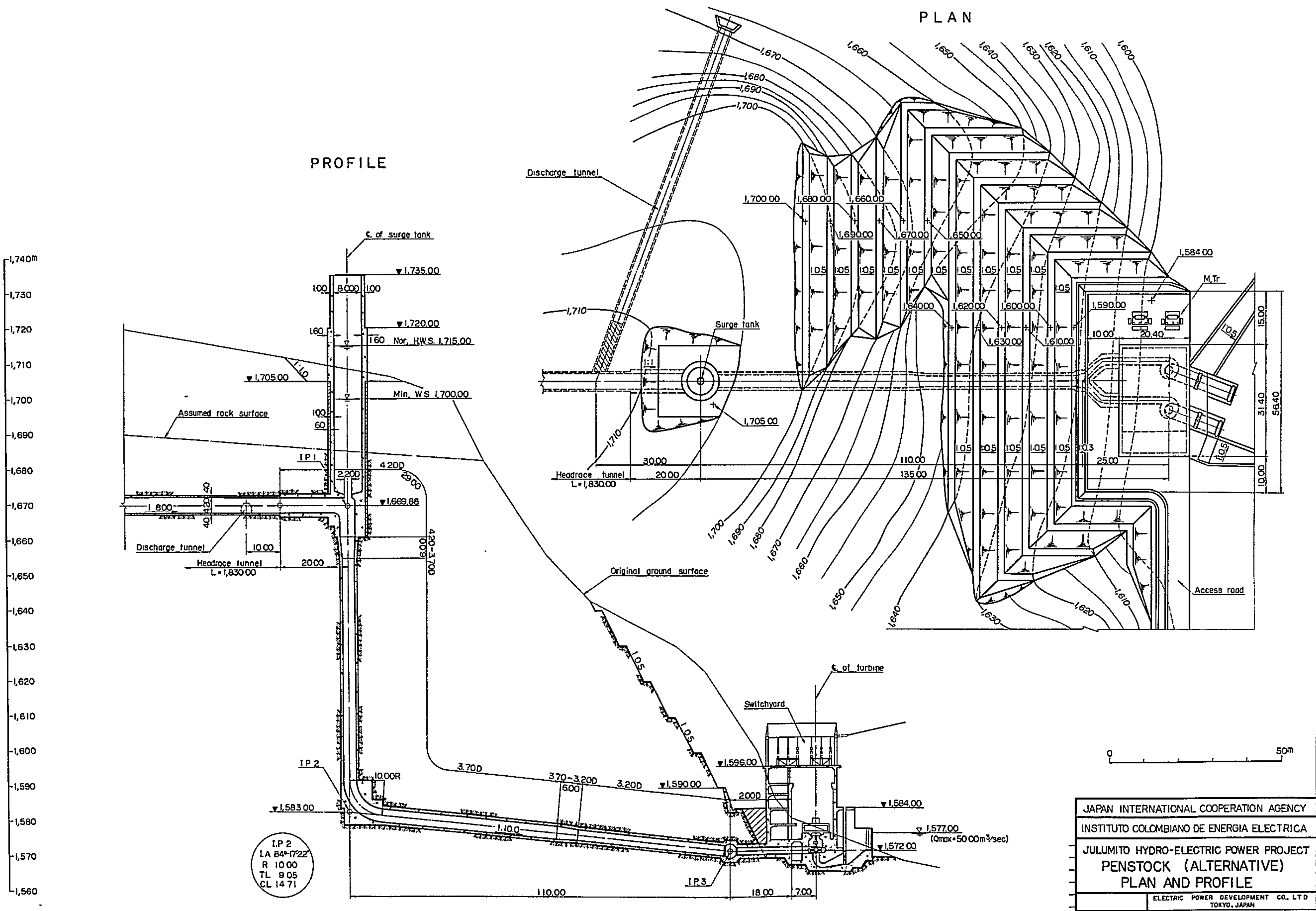
SHEET NO. 18 OF


PLAN
Intake
Headrace tunnel
Access tunnel
Discharge tunnel
Surge tank
Power house
M Tr
Rio Cauca
Access road
PROFILE
Nor HWS 1,715.00
Min. W.S 1,700.00
Original ground surface
Assumed rock surface
℄ of surge tank
℄ of discharge tunnel
℄ of turbine
Swichyard
Headrace tunnel L=1,775.00
Total length of penstock
Intake
Headrace tunnel
PROFILE OF SURGE TANK
Penstock
SECTION A−A
SECTION B−B
SECTION C−C
SECTION D−D
(Plan, profile)
(Profile of surge tank)
(Sections)
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
WATERWAY
PLAN, PROFILE AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN
LOCATION
DATE
DESCRIPTION
BY
REVISION
JULY 1979
SHEET NO. 19
OF


PLAN
PROFILE
SECTION C-C
SECTION B-B
SECTION A-A
Original ground surface
Assumed rock surface
Nor. H.W.S. 1,715.00
Min WS 1,700.00
℄ of surge tank
(Headrace tunnel)
Headrace tunnel
Power house
M Tr
Rio Cauca
Access road
Switch yard
℄ of turbine
℄ of penstock
Gutter
IP1
IA 38°-39'35"
R 15.00
CL 10.12
TL 5.26
(Qmax=50m³/s)
(Plan, profile)
(Sections)
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
PENSTOCK
PLAN, PROFILE AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN
SUBMITTED.
RECOMMENDED
APPROVED.
CK: J.A
JULY 1979
LOCATION
DATE
DESCRIPTION
BY
REVISION
SHEET NO. 20 OF


PLAN (EL.1,584.20)
Office
Erectionbey
Stoplog groove
PLAN (EL.1,572.00)
Drainpit
℄ of penstock#1
℄ of penstock#2
℄ of turbines
℄ of valves
SECTION A–A
Switchyard
Crane span 12.00
Control room
Cable room
Office
Cubicle room
Backfill
1,582.70 (Qmax = 1,700m³/sec)
1,577.00 (Qmax = 50m³/sec)
Spherical branch
Pressure oil tank
℄ of main valve
Drainpit
Draft gate 4.0 x 3.5
Penstock
Turbine
℄ of turbines
SECTION B–B
Switchyard
℄ of main trans
Backfill
℄ of penstock#2
℄ of penstock#1
℄ of turbine#2
℄ of turbine#1
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
POWERHOUSE
PLAN AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN
SUBMITTED.
RECOMMENDED.
APPROVED.
LOCATION
DATE
DESCRIPTION
BY
REVISION
JULY 1979
SHEET NO 21 OF


PLAN
Intake
Headrace tunnel
Access tunnel
Discharge tunnel
Surge tank
Powerhouse
M Tr
Access road
Rio Cauca
PROFILE
Original ground surface
Assumed rock surface
Nor. HWS 1,715.00
Min. WS 1,700.00
Headrace tunnel L = 1,830.00
℄ of surge tank
℄ of discharge tunnel
℄ of turbine
Switchyard
Total length of penstock 233.00
1,577.00 (Qmax = 50m³/s)
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
PROFILE OF SURGE TANK
Penstock
(Plan, profile)
(Profile of surge tank)
(Sections)
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
WATERWAY (ALTERNATIVE)
PLAN, PROFILE AND SECTIONS
ELECTRIC POWER DEVELOPMENT CO., LTD TOKYO, JAPAN
SUBMITTED, RECOMMENDED, APPROVED
LOCATION DATE DESCRIPTION BY
REVISION
JULY 1979
SHEET NO. 22 OF



LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

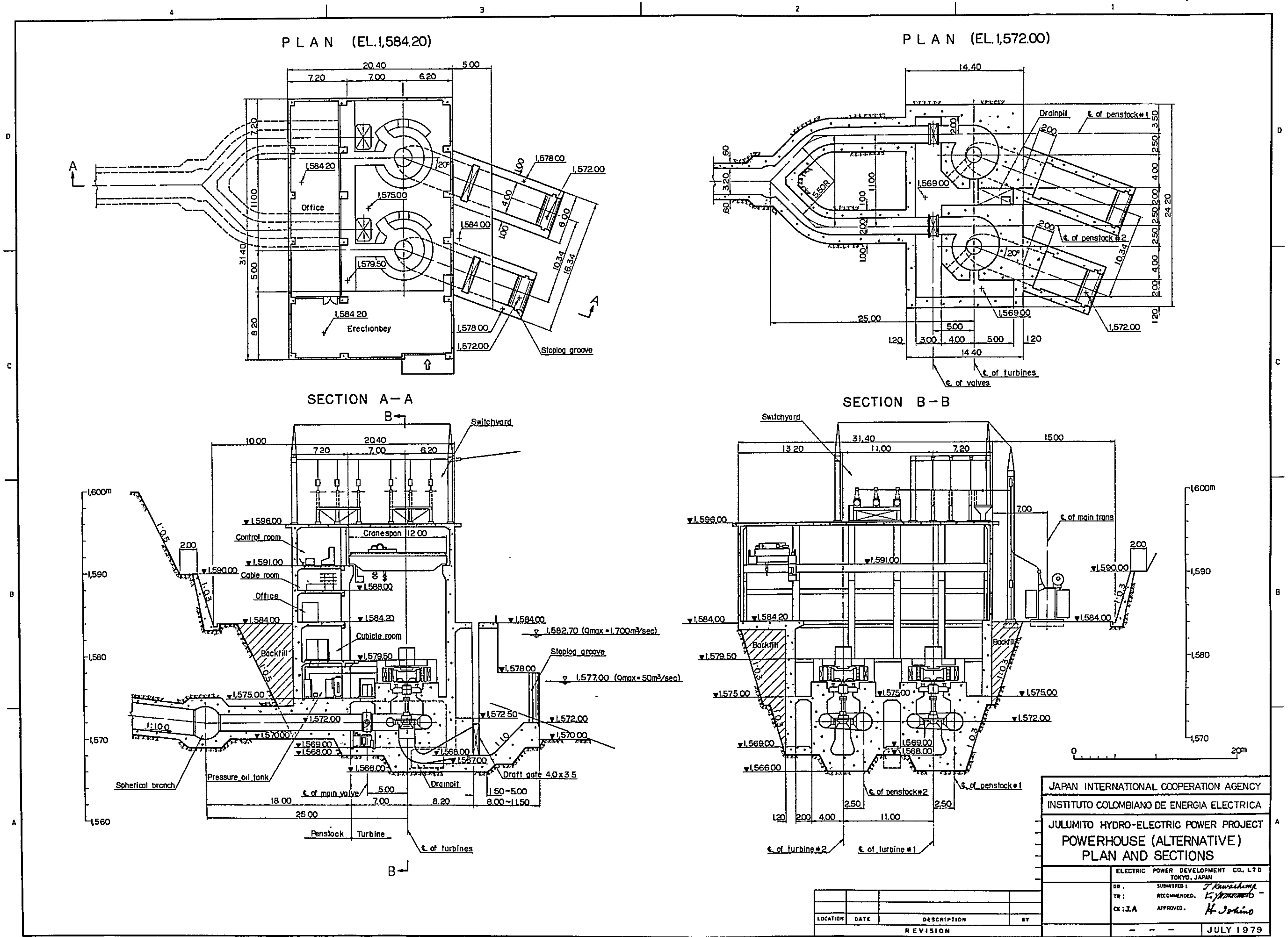
PENSTOCK (ALTERNATIVE)
PLAN AND PROFILE

ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN

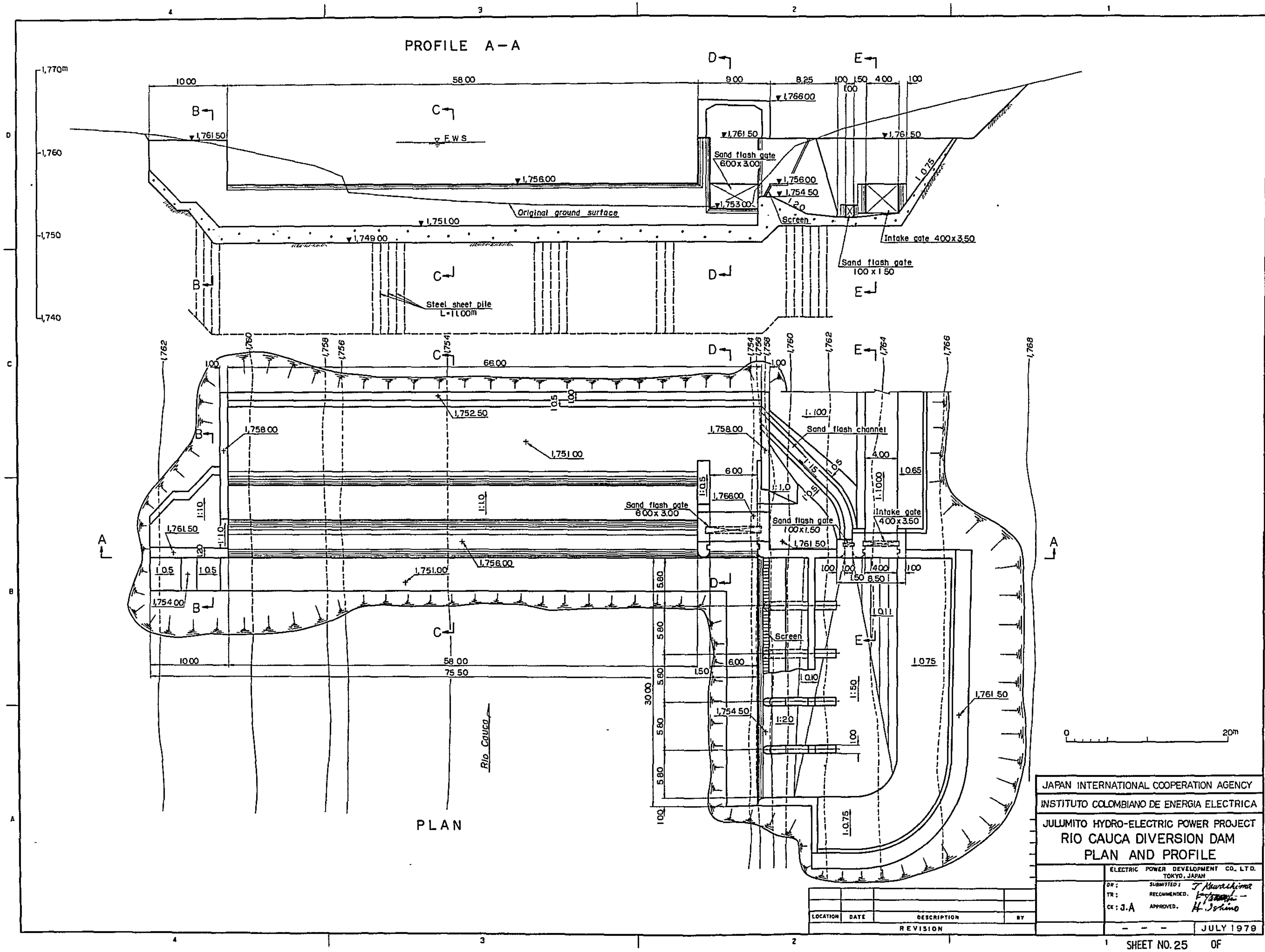
DR. SUBMITTED: T Kawashima
TR. RECOMMENDED: K. [illegible]
CK. J.A APPROVED: H. Ishino

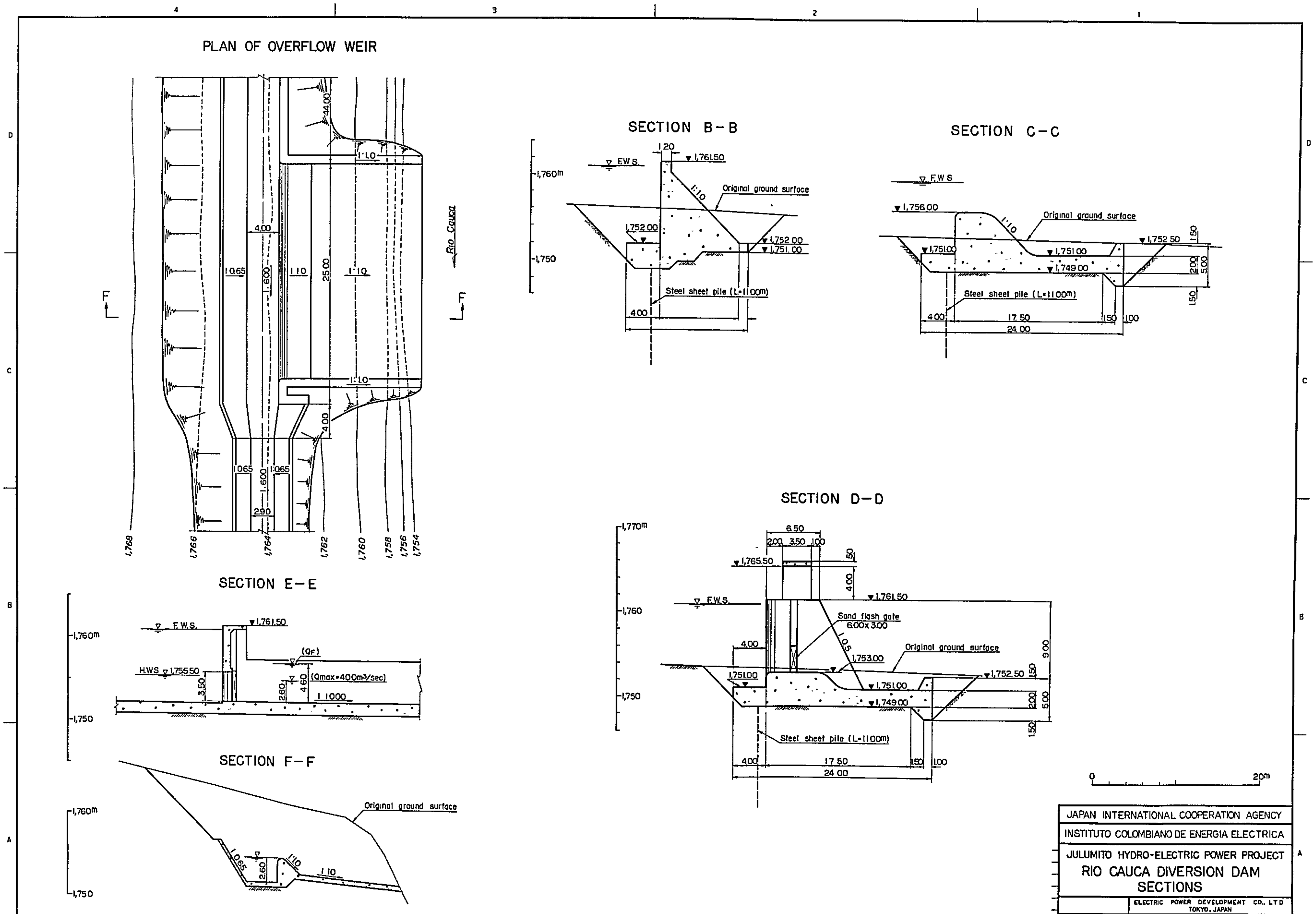
— — — JULY 1979

SHEET NO. 23 OF



SHEET NO. 24 OF


PROFILE A－A
Original ground surface
Sand flash gate 6.00 x 3.00
Screen
Intake gate 4.00 x 3.50
Sand flash gate 1.00 x 1.50
Steel sheet pile L=11.00m
F.W.S
Sand flash channel
Rio Cauca
PLAN
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
RIO CAUCA DIVERSION DAM
PLAN AND PROFILE
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN
JULY 1979
SHEET NO. 25 OF



JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

RIO CAUCA DIVERSION DAM
SECTIONS

ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN

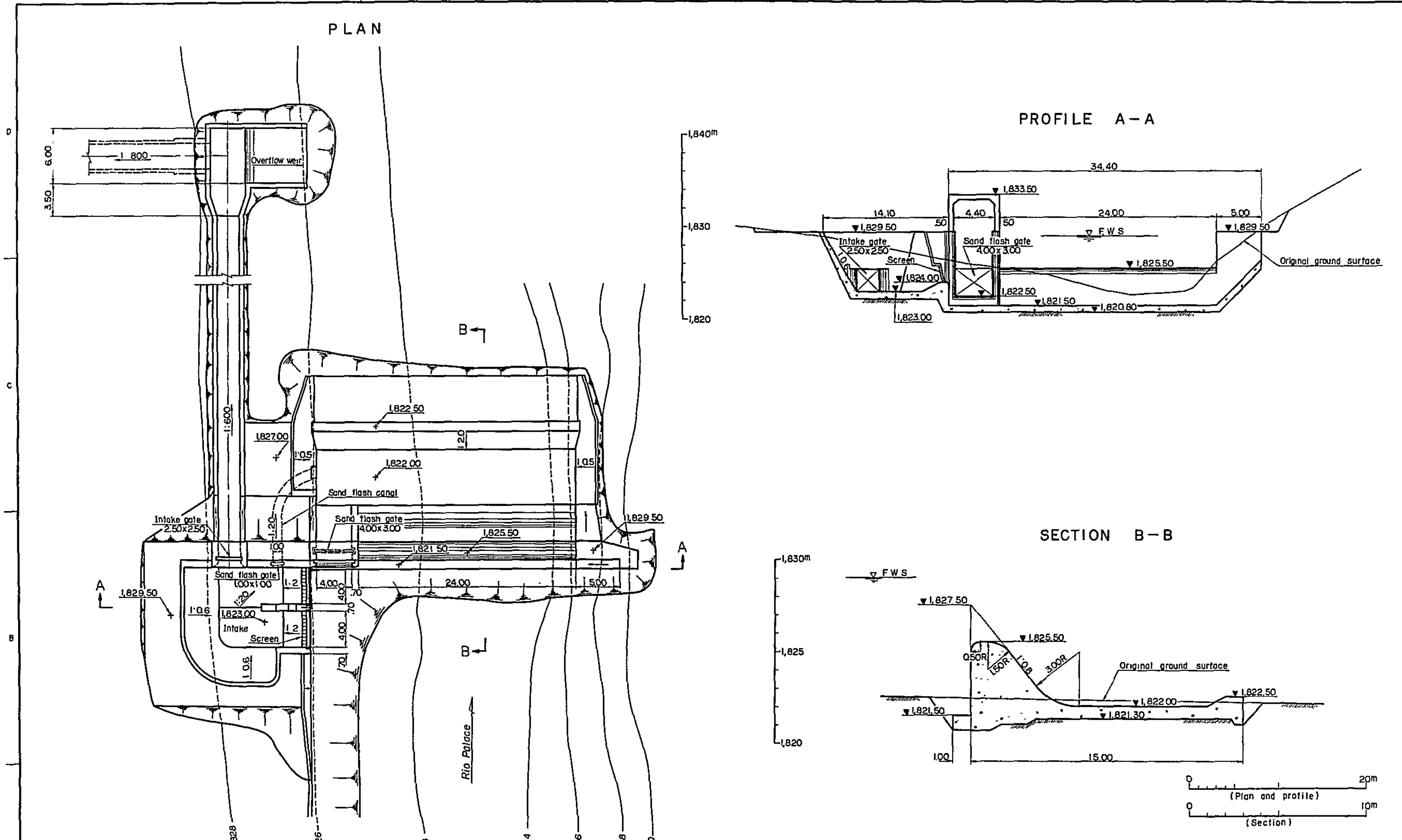
DR.		SUBMITTED.	T Kawashima
TR:		RECOMMENDED.	K. Yamamoto
CK.	J.A	APPROVED.	H. Ishino

LOCATION	DATE	DESCRIPTION	BY

REVISION

JULY 1979

SHEET NO. 26 OF



LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

RIO PALACE DIVERSION DAM

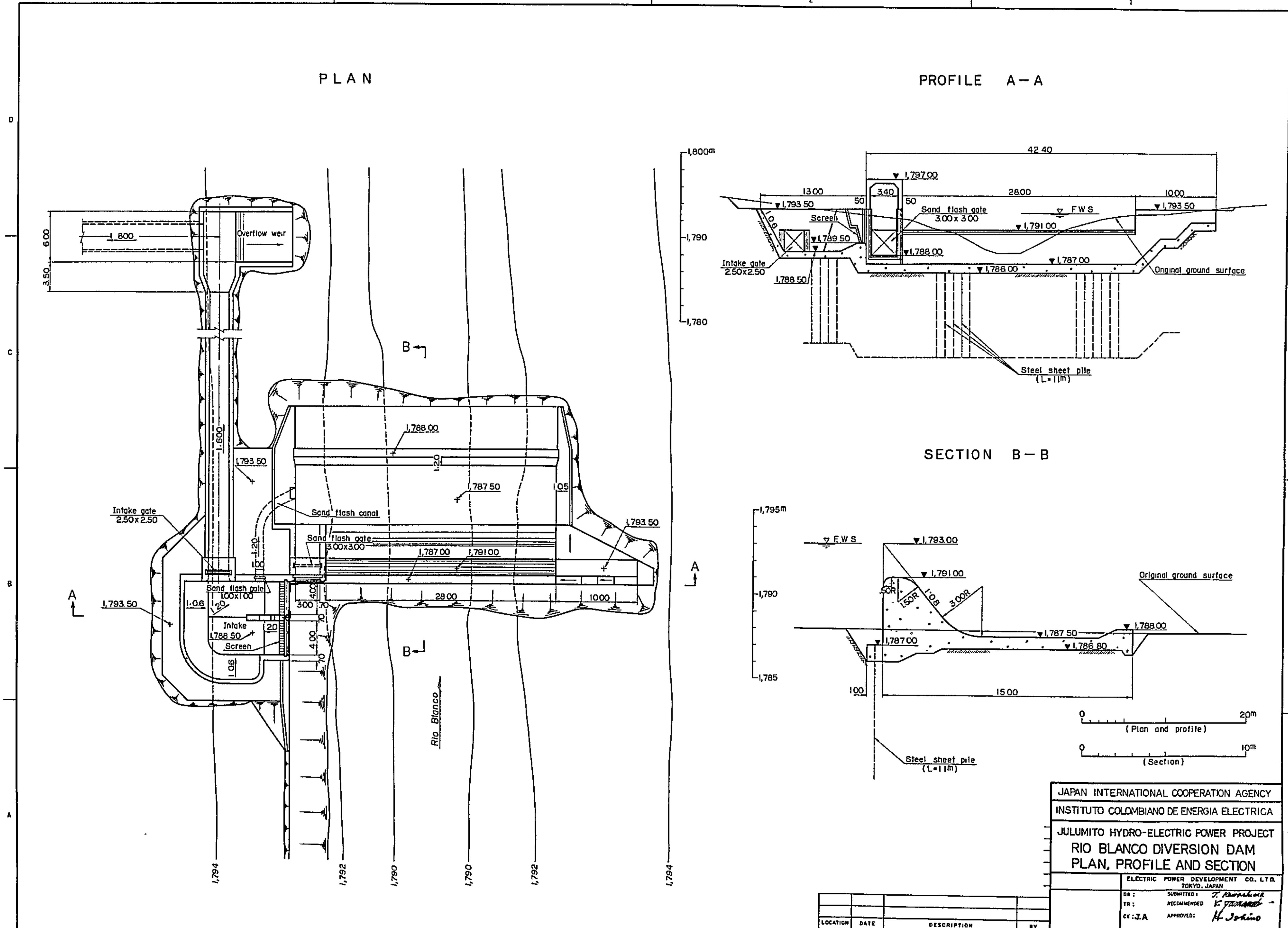
PLAN, PROFILE AND SECTION

ELECTRIC POWER DEVELOPMENT CO., LTD
TOKYO, JAPAN

DR: SUBMITTED: T. Kawashima
TR: RECOMMENDED:
CK. J.A APPROVED: H. Ishino

– – – JULY 1979

SHEET NO. 27 OF



JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

RIO BLANCO DIVERSION DAM
PLAN, PROFILE AND SECTION

ELECTRIC POWER DEVELOPMENT CO., LTD. TOKYO, JAPAN

DR: SUBMITTED: T. Kawashima
TR: RECOMMENDED: F. Yamamoto
CK: J.A APPROVED: H. Jokino

LOCATION	DATE	DESCRIPTION	BY

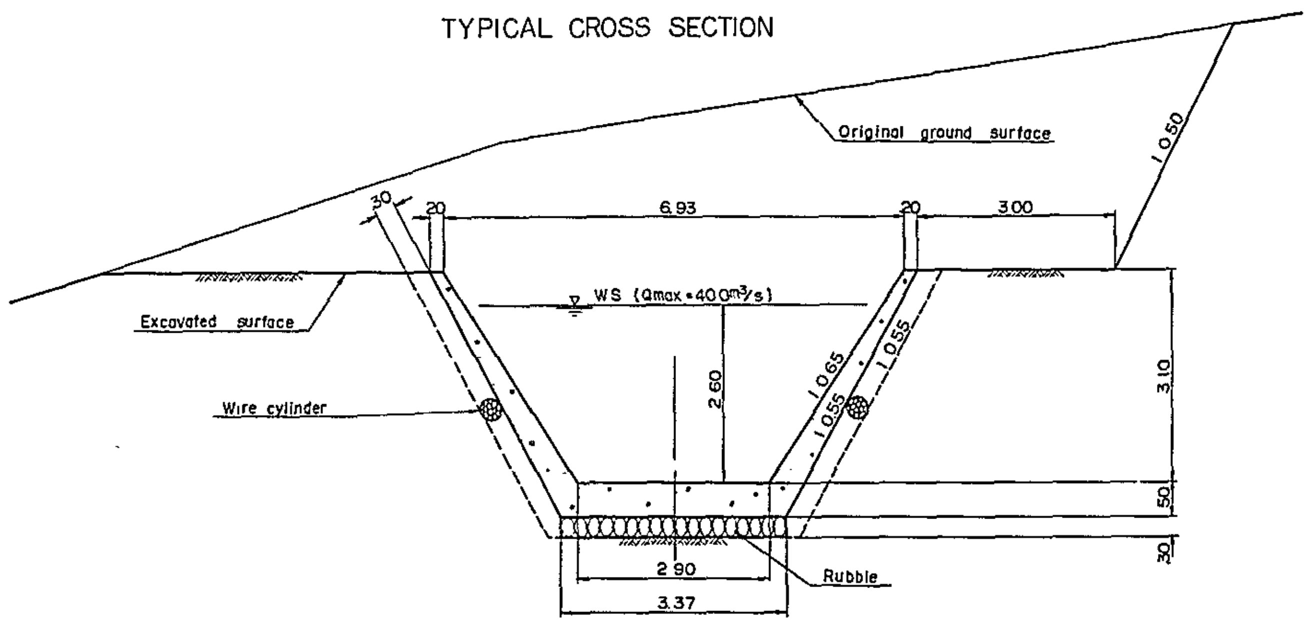
REVISION

JULY 1979

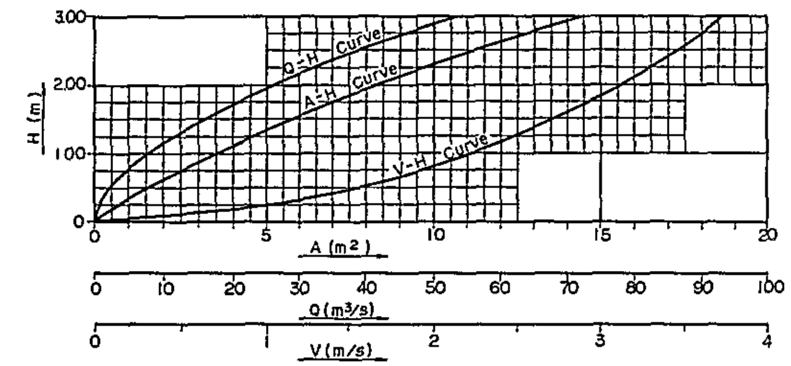
SHEET NO. 28 OF

CAUCA DIVERSION OPEN CHANNEL

TYPICAL CROSS SECTION



HYDRAULIC CHARACTERISTIC CURVE

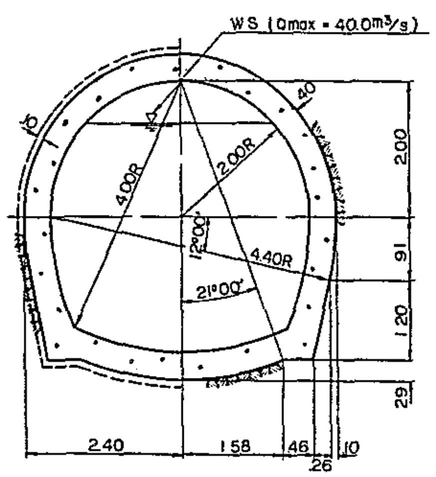


Legend

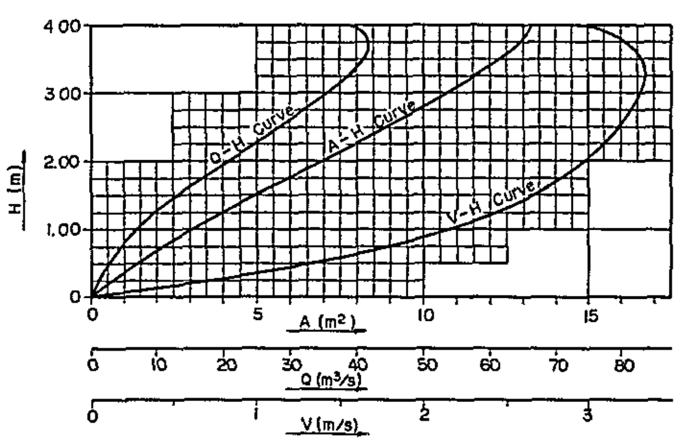
H · Water depth		(m)
A	Area	(m²)
Q	Discharge	(m³/s)
V	Velocity	(m/s)

CAUCA DIVERSION TUNNEL

TYPICAL CROSS SECTION

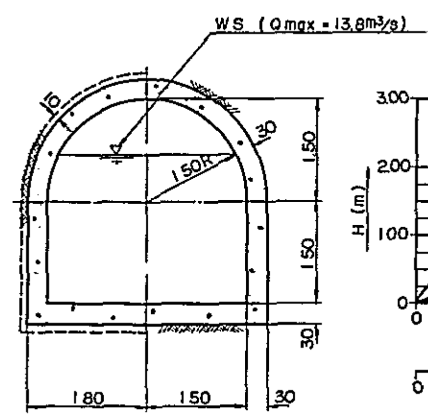


HYDRAULIC CHARACTERISTIC CURVE

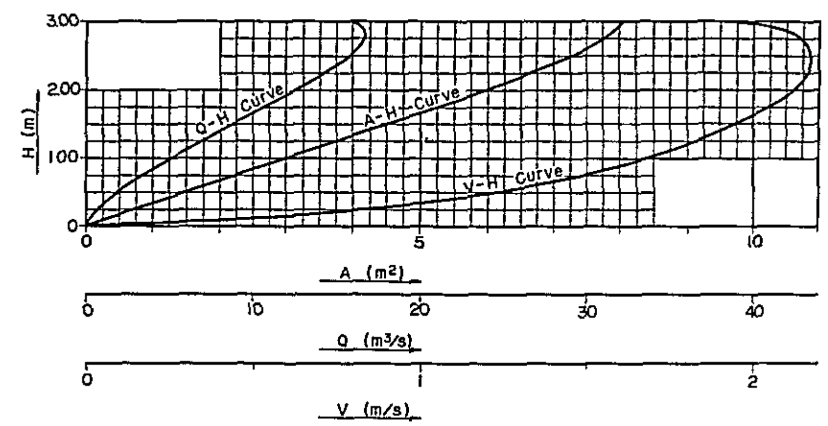


BLANCO DIVERSION TUNNEL

TYPICAL CROSS SECTION

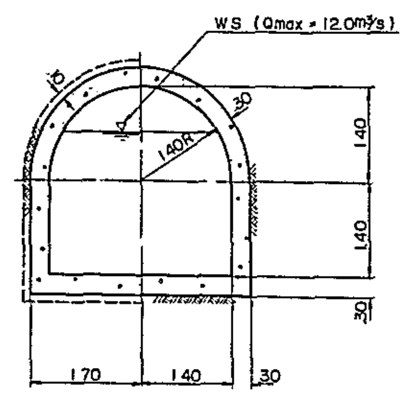


HYDRAULIC CHARACTERISTIC CURVE

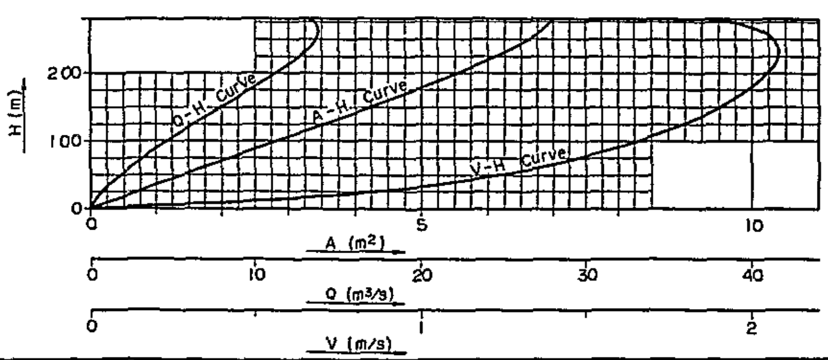


PALACE DIVERSION TUNNEL

TYPICAL CROSS SECTION



HYDRAULIC CHARACTERISTIC CURVE



0 5m

LOCATION	DATE	DESCRIPTION	BY
REVISION			

JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

CAUCA, PALACE AND BLANCO DIVERSION WATERWAY

ELECTRIC POWER DEVELOPMENT CO., LTD. TOKYO, JAPAN

DR. SUBMITTED: T. Kawashima
TR. RECOMMENDED: F. Yamamoto
CK: J.A APPROVED: H. Johino

— — — JULY 1979

SHEET NO. 29 OF

8.2 Power System Analysis

8.2.1 Objective

ICEL is proceeding with a 230-kV interconnecting transmission line plan in connection with the Betania Hydro-electric Power Project (ultimate output 500 MW). It is planned for this transmission line to start from Yumbo Substation, a substation of ISA, run parallel to an existing 115-kV transmission line to connect with New Popayan Substation, and further be extended to Betania Hydro-electric Power Station. This 230-kV transmission line is being planned with the target for 1985 when Betania Hydro-electric Power Station is to be completed. Meanwhile, in step with the increase in power demand of the CEDENAR Power System, it is planned for a 230-kV transmission line to be extended from New Popayan Substation to Catambuco Substation.

As indicated above, in 1985, when Julumito Hydro-electric Power Station is to be completed, a 230-kV transmission line will have been completed running parallel to the existing 115-kV transmission line, and it is expected that the power flow of the existing 115-kV transmission line will be greatly changed.

Accordingly, the objective of the power system analysis is to examine the voltage distributions of the related power stations and substations in 1985 when Julumito Hydro-electric Power Station is to go into operation, the power flows of the various transmission lines, the reactive power capacities required, breaking capacities of circuit breakers required during 3-phase short-circuit, and further, the transient stabilities of generators of principal power stations during 3-phase short-circuit to solve the various problems of system operation of the CEDELCA and CEDENAR power systems.

8.2.2 Preconditions

(1) Power System

The CEDELCA and CEDENAR Power Systems as of 1985, other than that New Popayan Substation will have been constructed at Popayan and an interconnection made with the Central Power System by a 230-kV transmission line, will fundamentally be not very much changed from their present power system structures.

In power system analysis, the influence of the CVC Power System which affects the CEDELCA and CEDENAR power systems was taken into consideration, with power systems north of Esmeralda Power Station totalized together and eliminated as direct objects of study. In effect, the power systems shown in Fig. 8-21 were made the objects of study.

Further, although it is not clearly known regarding the timing of construction of the

230-kV, single-circuit, 170-km transmission line between New Popayan Substation and Catambuco Substation, a study is to be made whether this would be necessary in 1985.

The 230-kV transmission line to be constructed between Pance Substation and New Popayan Substation and the 230-kV transmission line between New Popayan Substation and Catambuco Substation are both to be loop-operated through existing 115-kV transmission lines and tie transformers, and therefore, the section between the New Popayan Substation secondary-side 115-kV bus and the existing Popayan Substation is to be connected by a 115-kV transmission line, single-circuit.

(2) Increase in Power Demand and Transformer Capacity

The transformer capacities of substations will increase in accordance with increase in power demand. For substations in CEDELCA and CEDENAR power systems scheduled to have transformers additionally installed or replaced, expansion plans were taken into consideration, while other substations were considered as having transformer capacities corresponding to the power demands forecast as of 1985. Regarding the substations for power supply to Cali in the CVC Power System, it was not possible to determine transformer capacities forecasting power demand by substation due to lack of data, and therefore, transformer capacities were determined assuming power demands of the existing San Antonio, Juanchito and Pance Substations all to be 150 MW.

(3) Line Constant, Generator Transient Reactance and Generator Unit Inertia Constant

Regarding line constants of the CEDELCA and CEDENAR power systems, since conductor sizes are known and clearances between conductors are also known, the values given in the 1972 Feasibility Study Report on this Project were used without alteration for line resistance, line reactance and ground capacitance.

Regarding line constants, generator transient reactances xd' and generator unit inertia constants M of the CVC Power System and newly-constructed 230-kV transmission lines, standard values were adopted according to generator capacities.

These values are shown in Fig. 8-21.

8.2.3 Results of Study

(1) Power Flow

In study of power flows, as indicated in Fig. 8-22 through Fig. 8-24, voltages at substation buses where reactive power phase modifying equipment are shown were fixed, and the load power factors of the various substations were taken to be 90% and the transformer taps at power stations and substations as $\pm 10\%$ changeable.

a) Case of Betania Hydro-electric Power Station and Catambuco Substation Interconnection

As shown in Fig. 8-22, the power flow distributions of the 220-kV transmission line and the 115-kV transmission line are within the transmission capacities of the transmission lines and pose no special problem. The bus voltages of the various substations are also in the range of 100% ± 5%. The substation bus voltages of the CEDENAR Power System at the end of the wide-area power system are on the low side and in order to maintain substation secondary-side bus voltages at 95%, leading reactive power facilities (power condensers) totalling 23 MVar are necessary.

Further, when power generation of 200 MW is being done at Betania Hydro-electric Power Station, lagging reactive power facilities (reactor) of 50 MVar are necessary to maintain the 230-kV bus voltage of New Popayan Substation at 100%.

In order to maintain the 115-kV-side bus voltage of Pance Substation at 97.0%, it will be necessary to install leading reactive power facilities (power condenser) of 158 MVar, although they can also be installed divided among San Antonio and Juanchito Substations.

b) Case of Only New Popayan Substation and Pance Substation Interconnected by 230-kV Transmission Line

The power flow diagram with the electric power system composition of Fig. 8-22 unaltered and the transmission lines to Betania and to Catambuco Substation shut down is shown in Fig. 8-23.

The bus voltages of the CEDENAR Power System compared with the case of interconnection at Catambuco Substation by the 230-kV transmission line show the 115-kV voltages at Pasto Substation and Catambuco Substation to be lowered 4.4% and 5.3%, respectively. In order to maintain the secondary-side bus voltages of the 115-kV substations of the CEDENAR Power System at 95%, it will be necessary to additionally install 37 MVar of leading reactive power facilities (power condensers) compared with the case of interconnection by the 230-kV transmission line.

Fig. 8-24 shows the results of adjustments of reactive power of generators and transformer taps of the various substations in the power system in order to reduce the leading reactive power facilities (power condensers) within the CEDELCA and CEDENAR power systems.

Reactive Power Required at Substations

Unit: MVAR

	Yumbo	Pance	New Popayan	Pasto	Tumaco	Ipiales	Total
Before adjustment	51	165	22	36	6	18	298
After adjustment	33	136	0	27	5	13	214
Difference	18	29	22	9	1	5	84

Seen from the results of the above power flows studies, it is desirable from the standpoint of voltage maintenance at the various substations of the CEDENAR Power System for the 230-kV transmission line, single-circuit, 170 km, between New Popayan Substation and Catambuco Substation to be constructed at a relatively early time (within 3 or 4 years from 1985).

(2) Short-Circuit Capacity

The CEDELCA and CEDENAR power systems are situated at the southern end of the national interconnected power system of Colombia and are located at a considerable distance from the Central Power System. Consequently, the short-circuit capacity in 1985 in terms of that at the 230-kV bus side of New Popayan Substation is 1,760 MVA, and there is no special problem in determining the rated breaking capacities of circuit breakers. In effect, it will suffice to adopt circuit breakers of the ratings below for the 230-kV and 115-kV sides.

	Rated Current	Rated Current Broken	Rated Breaking Capacity
230-kV circuit breaker	1,200 A	25 kA	10,400 MVA
115-kV circuit breaker	1,200 A	12.5 kA	2,600 MVA

(3) Transient Stability

Calculations of transient stabilities of generators were made for the power stations below.

Name of Power Plant	Symbol number	Remarks
Betania	1	
Julumito	2	
Florida No. 2	3	
Rio Mayo	4	
Hydro P	5	Small hydro in CEDELCA
Hydro C	6	Small hydro in CEDENAR
Salvajina	7	

The conditions for calculations were the three cases of 3-phase short-circuit, 3-phase reclosure failure at the Betania Power Station side of New Popayan Substation (see Fig. 8-26 to 28), short-circuit, final breaking of the transmission line, double-circuit, to Cali at the 115-kV bus of the existing Popayan Substation, and 3-phase short-circuit, final breaking of the 115-kV outgoing transmission line at Julumito Hydro-electric Power Station, and transient stability calculations were made for the above-mentioned 7 power stations.

The generators of Julumito Hydro-electric Power Station will be stable except in the case of faulting of the outgoing 115-kV transmission line. The six other power stations are all stable against faulting in the above-mentioned three cases, and if the 230-kV transmission lines and 115-kV transmission lines were to be loop-operated, there would be no step-out accidents of generators in the CEDELCA and CEDENAR power systems due to transmission line faults and the supply reliabilities of the power systems would be greatly enhanced.

Fig. 8-21 Impedance Map of Power System Relateed with Julumito Power Project in Beginning of 1985

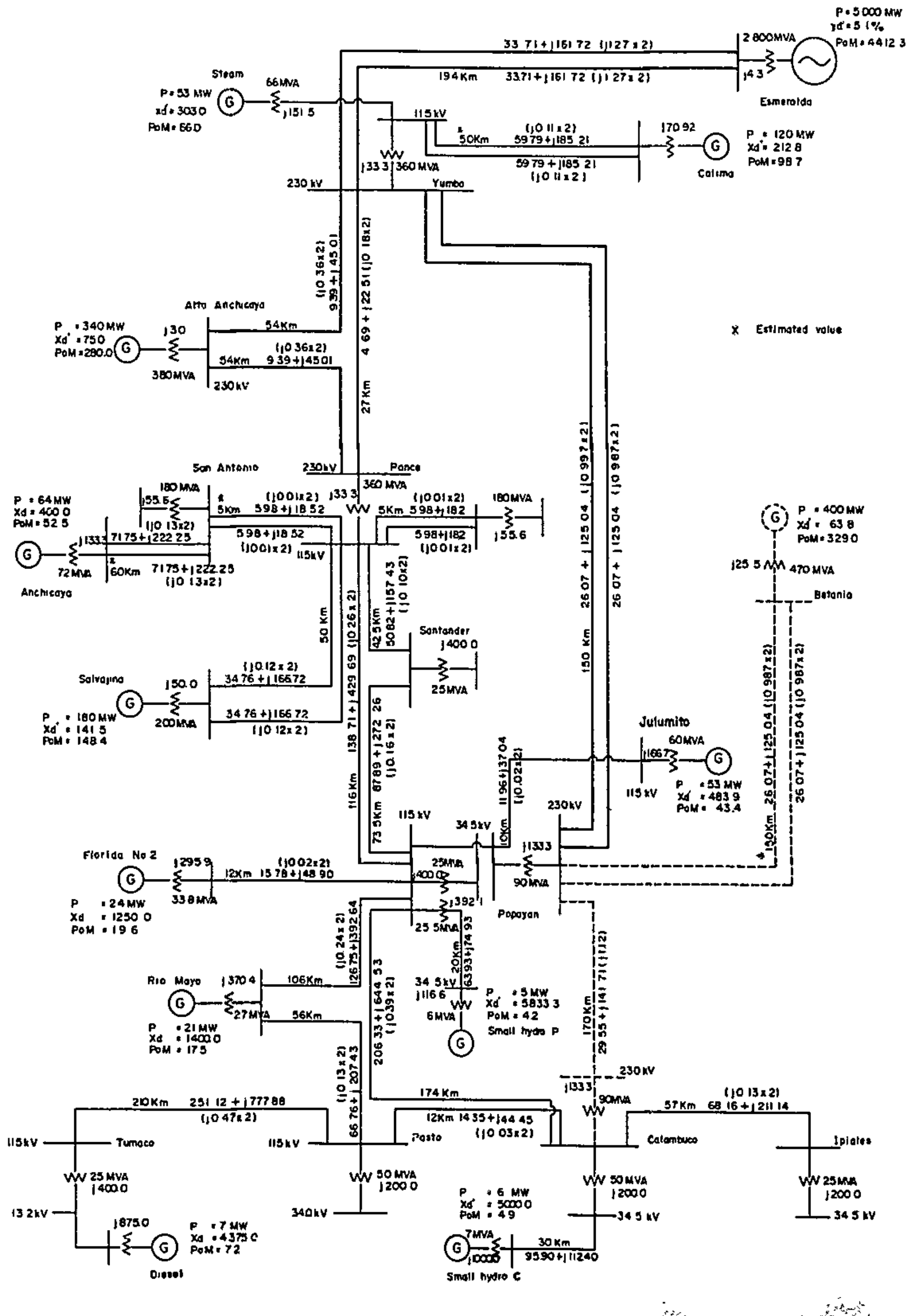


Fig. 8-22 Power Flow and Voltage Regulation in 1985 (Betania and Catambuco Interconnection)

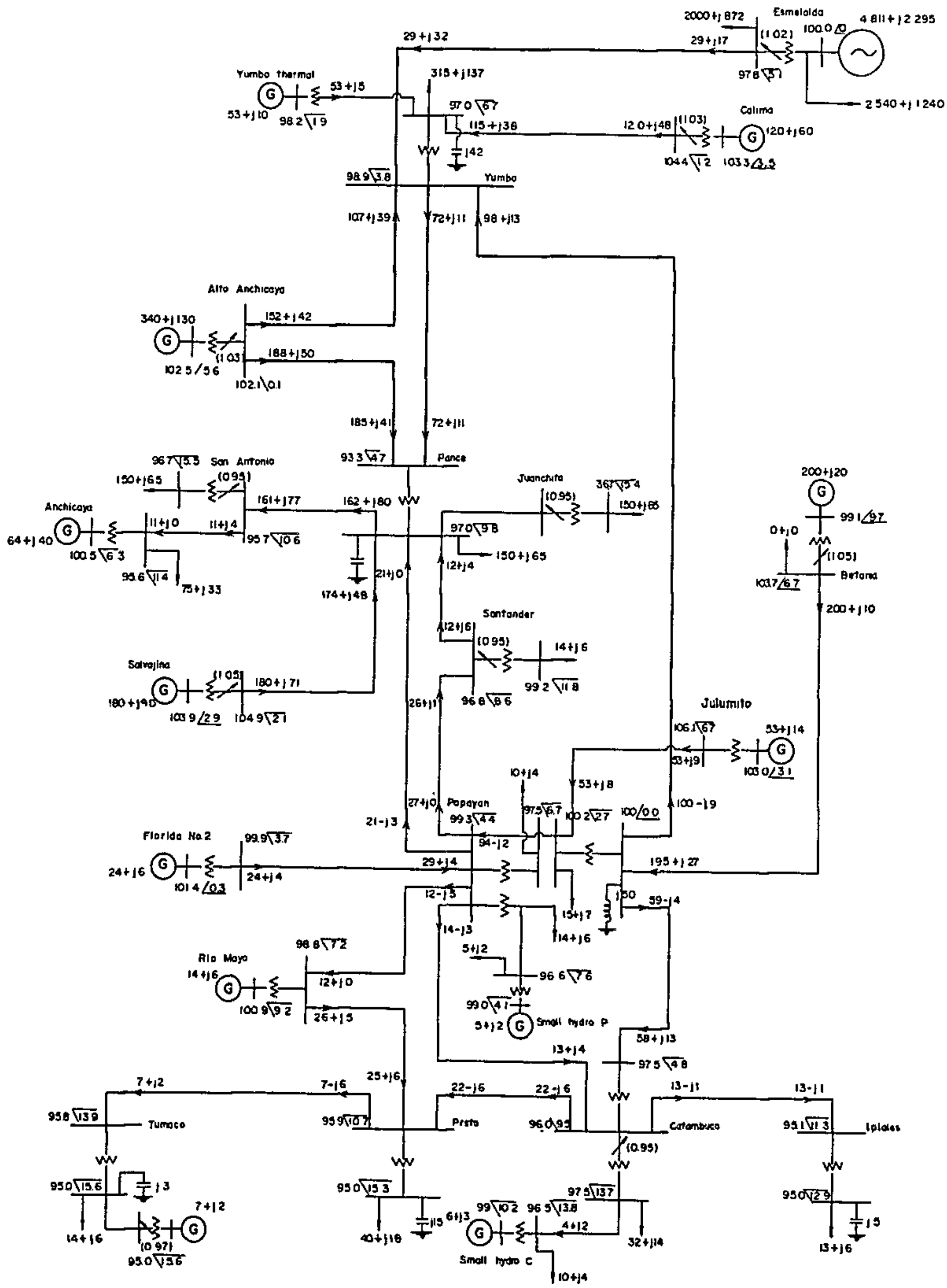


Fig. 8-23 Power Flow and Voltage Regulation in 1985 (Without Voltage Adjustment)

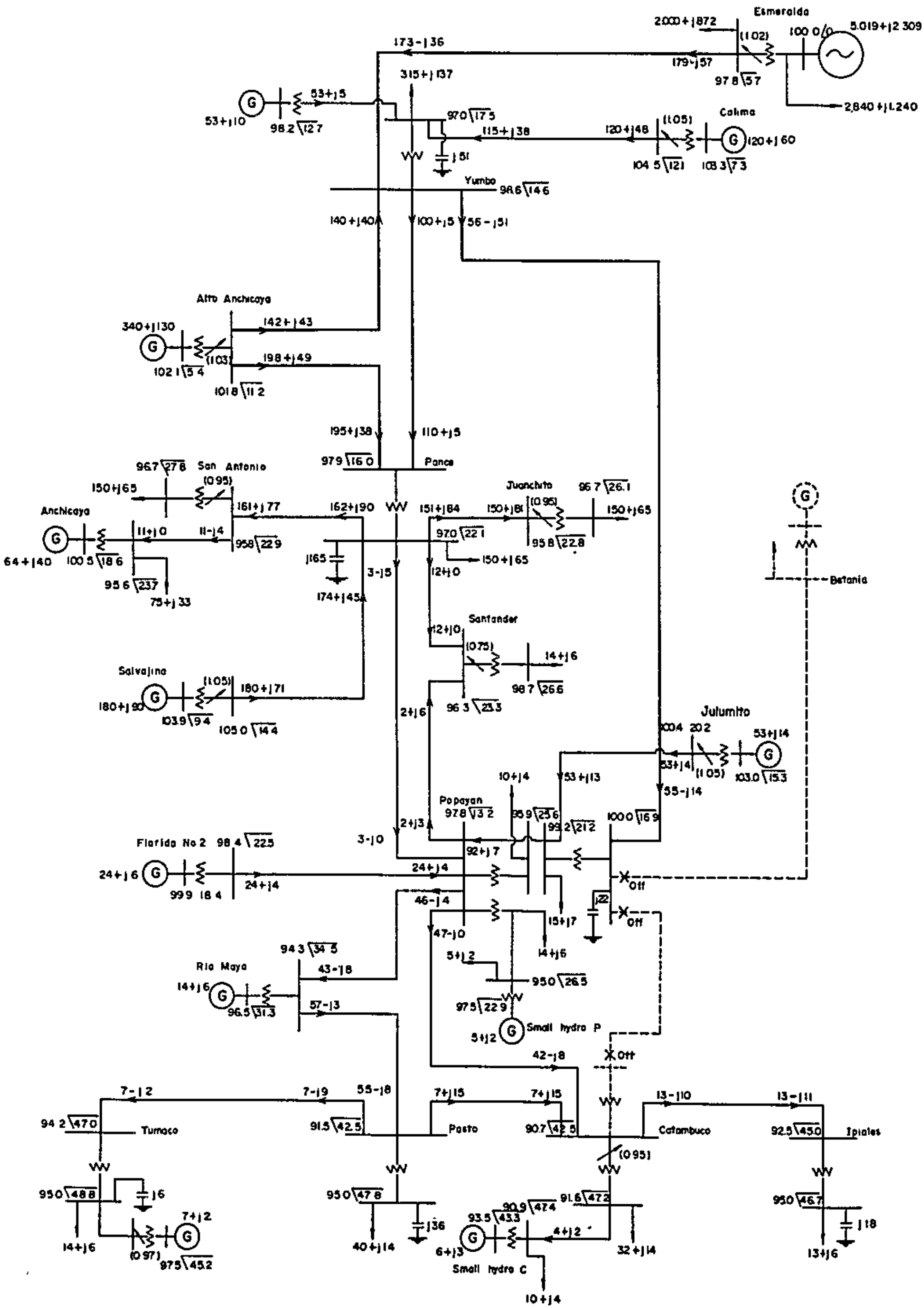


Fig. 8-24 Power Flow and Voltage Regulation in 1985 (With Voltage Adjustment)

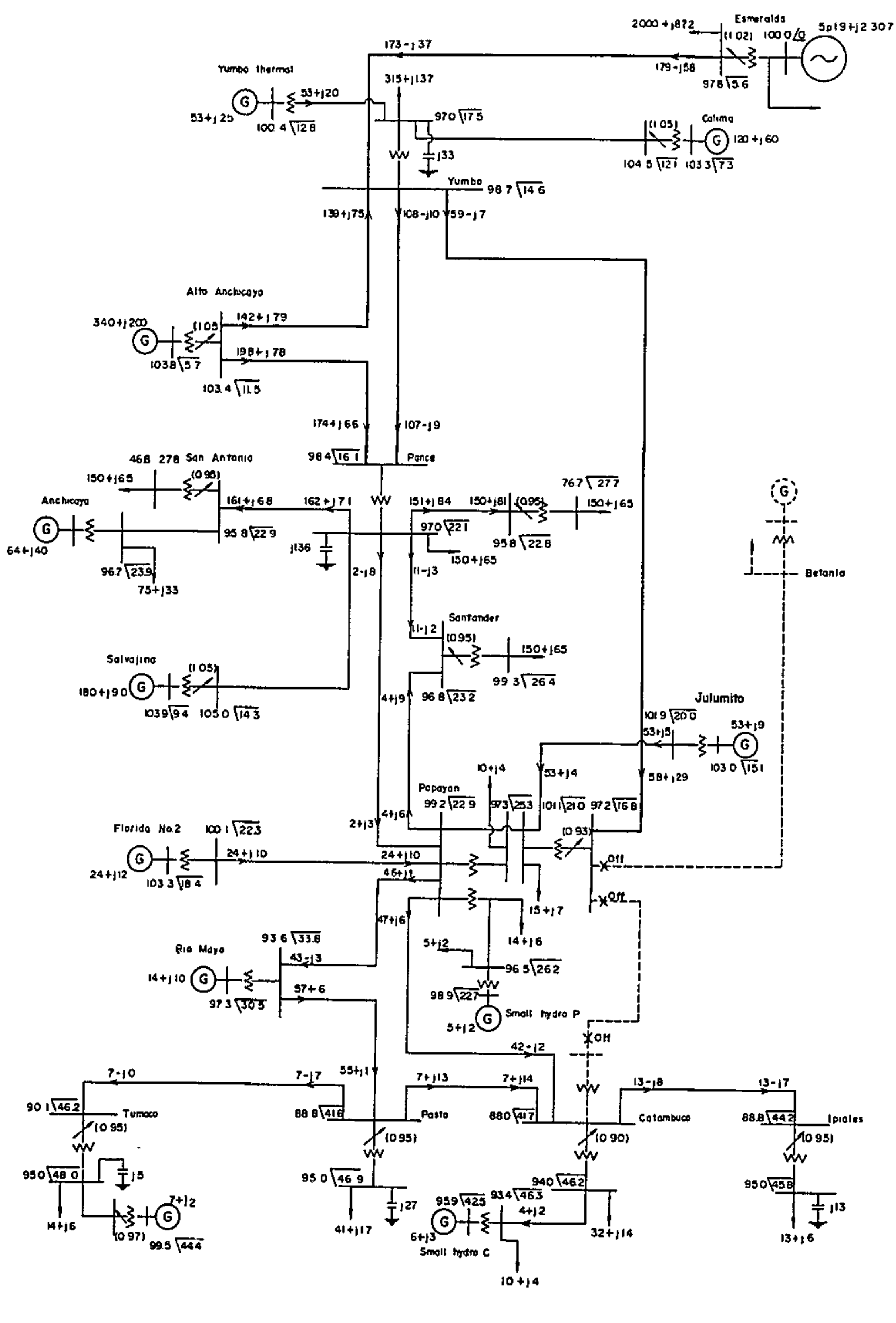


Fig. 8-25 Short Circuit Capacity in 1985 Unit: MVA

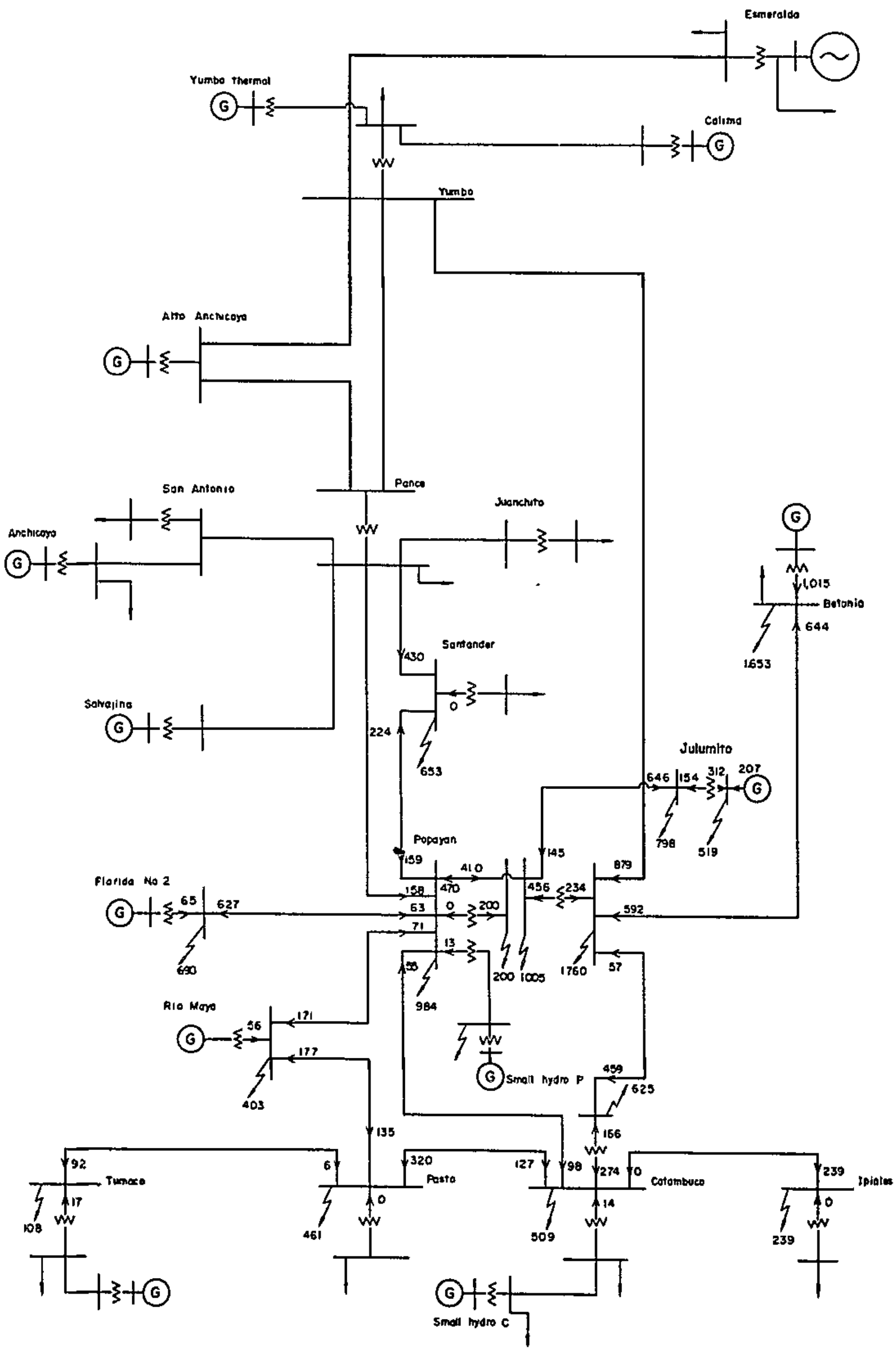


Fig. 8-26 Transient Stability in 1985

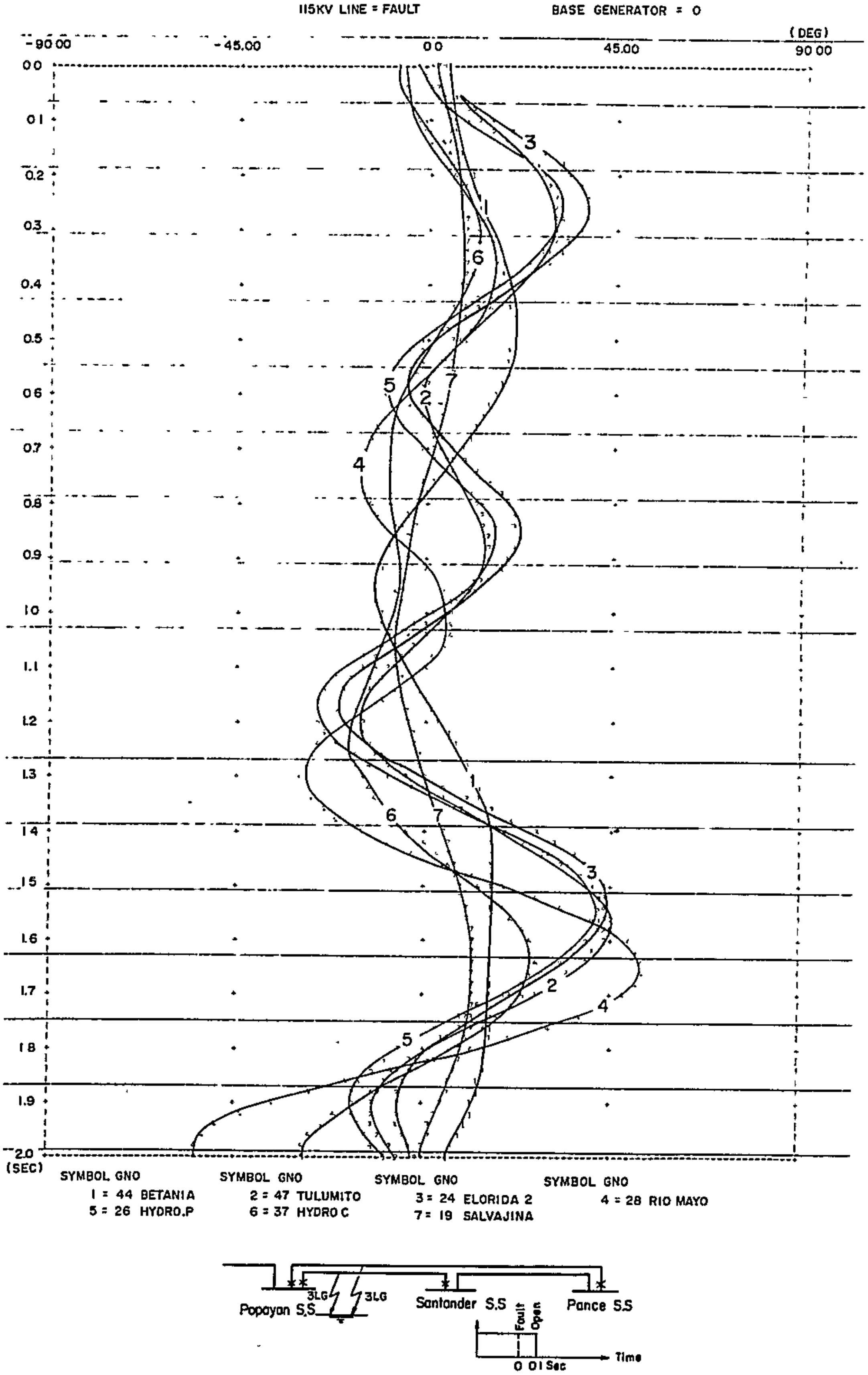


Fig. 8-27 Transient Stability in 1985

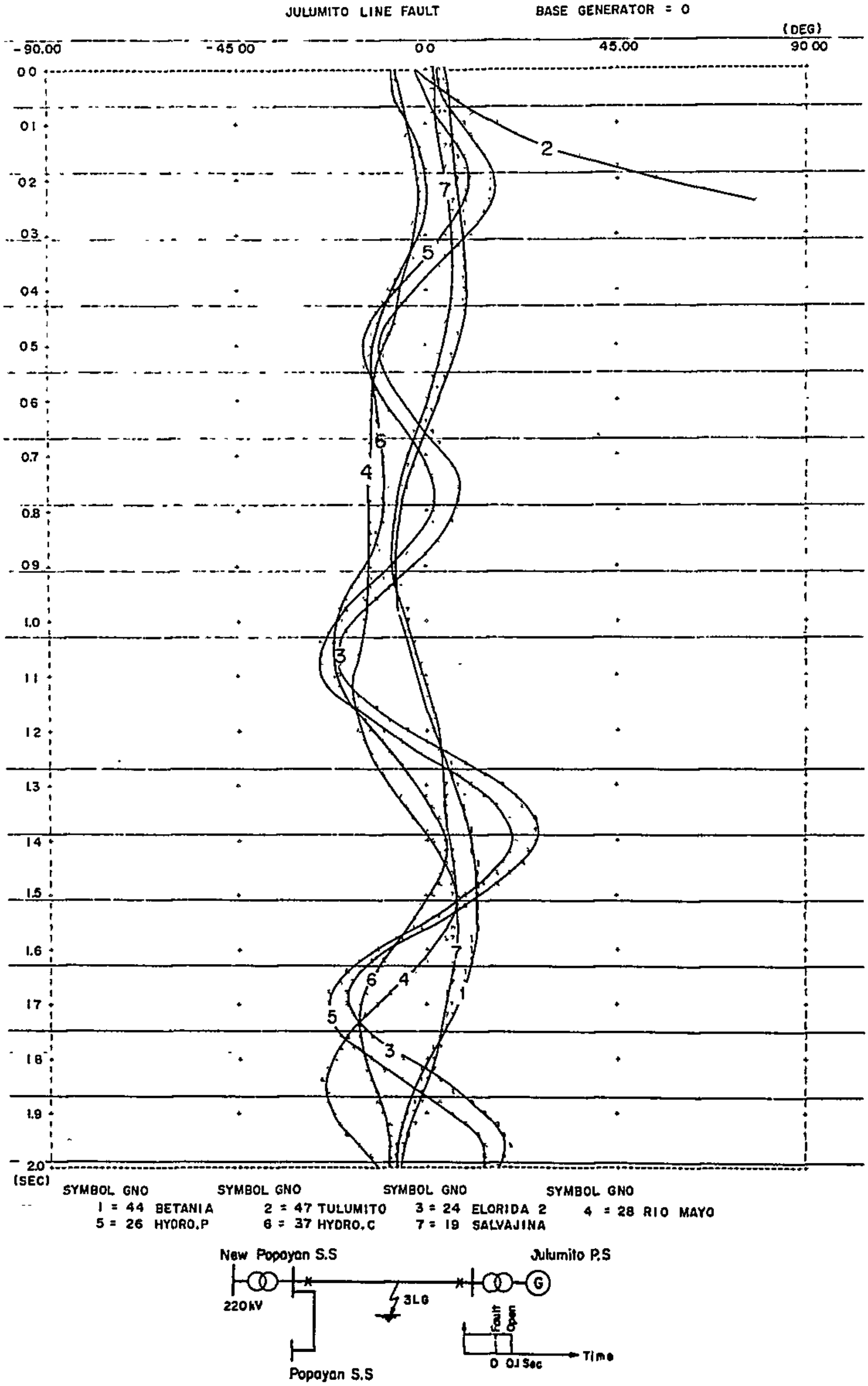
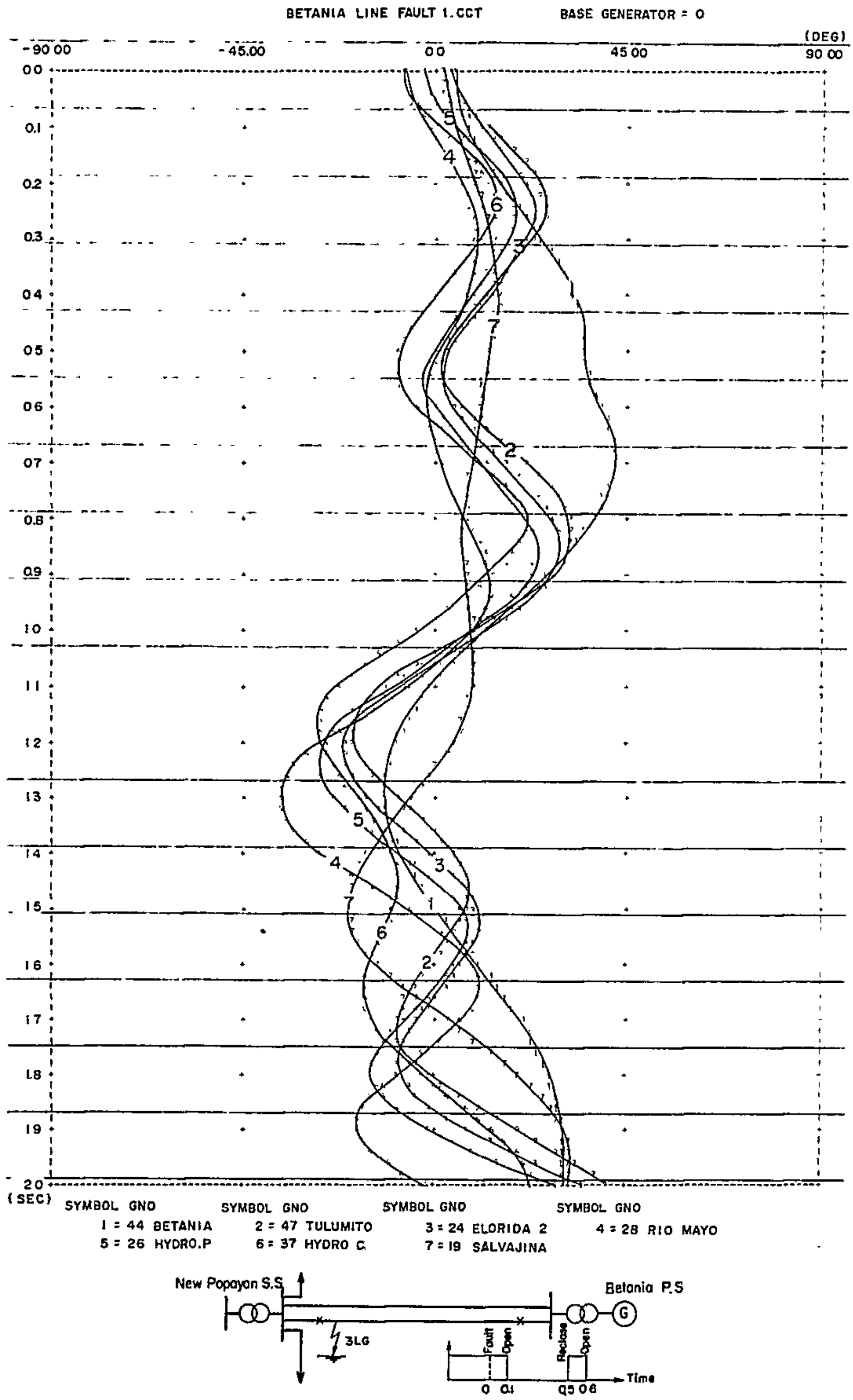


Fig. 8-28 Transient Stability in 1985



CHAPTER 9

CONSTRUCTION SCHEDULE AND CONSTRUCTION SCHEME

CHAPTER 9 CONSTRUCTION SCHEDULE AND CONSTRUCTION SCHEME

CONTENTS

FIGURE LIST

TABLE LIST

CHAPTER 9 CONSTRUCTION SCHEDULE AND CONSTRUCTION SCHEME

9.1 Construction Schedule

9.1.1 General

The Julumito Hydro-electric Power Project, as previously stated, is now strongly desired to be started at an early date.

In order to do so, preparatory works such as topographical surveying, geological investigations, and material tests must be completed, and detail design and preparation of specifications be started after above-mentioned works are completed.

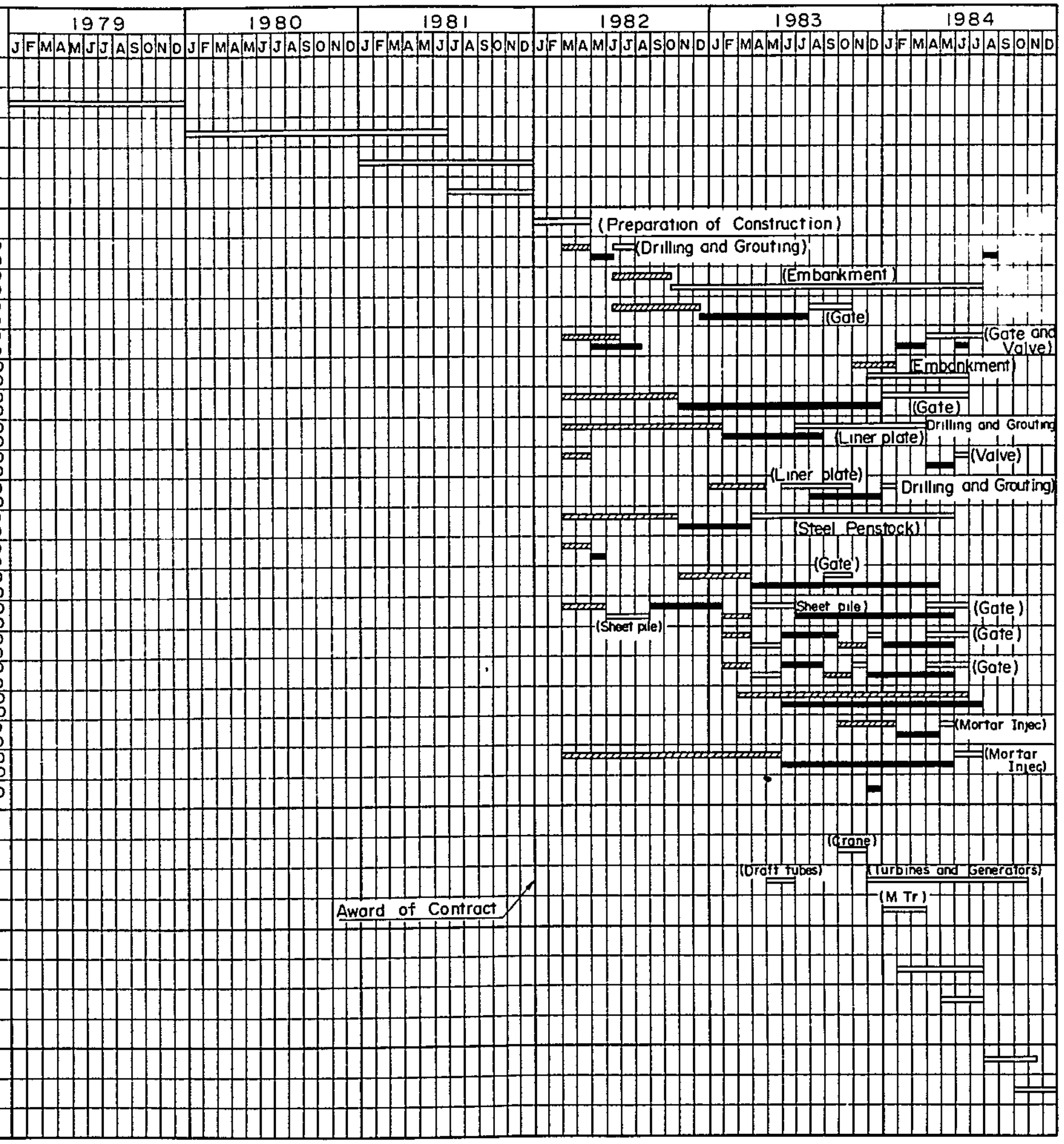
Taking into account the present situation, it is estimated that it will be June 1981 when these works will be completed. Assuming at least a period of 6 months for tendering and contracting, it was estimated that the start of main works would be January 1982 at the earliest.

Preparatory works necessary for starting construction work, namely, construction of access roads to the dam site, the powerhouse site, materials borrow areas, etc., or repairs of existing roads, and provision of facilities such as motive power facilities for construction must all be completed by the end of 1981.

Predicated on the above conditions, the construction schedule of this Project was planned as shown in Table 9-1.

Table 9-1 Construction Schedule

Item			Quantity
I Preparation works			
	Field investigation works (Geological Invetigation, Topographical)		
	Detail design and preparation of tender ducument		
	Preparatory works (Access road others)		
	Tender and contract		
II Civil works and hydraulic equipment			(Unit = m³)
	Dam	Diversion tunnel (L=365m)	Ex = 4 420 Con = 1 700
		Dam	Ex = 225 400 Emb = 1254 000
		Spillway	Ex = 73 100 Con = 5 090
		Outlet tunnel (L=267m)	Ex = 8 190 Con = 3 560
	Dike No 1, No 2		Ex = 19 800 Emb = 47 500
	Intake		Ex = 256 700 Con = 10 400
	Headrace tunnel	Headrace tunnel (L=1775m)	Ex = 41 050 Con = 15 210
		Drainnage tunnel (L=103m)	Ex = 1 390 Con = 660
	Surge tank		Ex = 6 580 Con = 3 830
	Penstock		Ex = 94 730 Con = 2 830
	Powerhouse and Tailrace	Diversion tunnel (L=200m)	Ex = 8 700 Con = 3 480
		Powerhouse and Tailrace	Ex = 76 900 Con = 7 460
	Diversion dam and Intake	Cauca	Ex = 24.600 Con = 9 830
		Palace	Ex = 2 700 Con = 1 460
		Blanco	Ex = 2 900 Con = 1 090
	Diversion waterway	Cauca (L=2620m)	Ex = 133 690 Con = 11 860
		Palace tunnel (L= 770m)	Ex = 8 840 Con = 3 180
		Blanco tunnel (L=3650m)	Ex = 46 840 Con = 16 040
	Foundation of switchyard		Con = 150
III. Electrical equipment			
	Crane		
	Turbines and generators		
	Main transformers and other equipment		
IV. Transmission line and others			
	Transmission line		
	Telecommunication system		
Water storage			
Test			
Start of operation			



LEGEND
- Excavation
- Concrete
- Other work

9.1.2 Outline of Work Execution by Year

(1) First Year of Construction (1982)

Regarding the main dam work, the temporary diversion tunnel is to be completed, and after diverting the flow of the Rio Sate, excavation of the dam foundation is to be started.

The open-cut excavation for the spillway work is to be executed at the same time as the dam foundation excavation, and from the end of 1982 drilling and grouting of the dam foundation, dam embankment and placement of spillway concrete is to be started.

Open-cut excavation for the intake work is to be completed similarly to the spillway and concrete placement is to be started.

For the headrace tunnel work, excavation is to be started from the work adit on the intake side and from the lower portal work adit cum sand flush channel.

Open-cut excavation for the penstock is to be started and concrete placement is to be executed from the end of 1982.

Regarding the powerhouse work, foundation excavation is to be carried out in succession after completion of the open-cut excavation for the penstock, and for this purpose it will be necessary for the temporary diversion waterway (tunnel) to be completed and the flow of the Rio Cauca switched in advance.

For waterway work, besides carrying out care of river, foundation excavation, sheet pile driving and concrete placement for Rio Cauca Diversion Dam, excavation of the Rio Blanco Waterway tunnel is to be started.

Ordering of hydraulic equipment such as the penstock and main electrical equipment will need to be done during this year.

(2) Second Year of Construction (1983)

Regarding the main dam work, dam embankment is to be continued in succession to the previous year, preceding which drilling and grouting is to be done.

In the spillway work, after completion of concrete placement, gate installation is to be done.

For the intake work, concrete placement is to be done continued from the previous year.

In the headrace tunnel work, after completion of excavation and concrete lining, mortar injection and grouting are to be carried out.

For the surge tank work, excavation of the base tunnel and vertical shaft excavation are to be completed, and installation of steel lining and placement of backfill concrete are to be carried out.

In penstock work, installation of the penstock is to be started after completing concrete placement.

With respect to the powerhouse work, besides placing concrete for the foundation, side walls and slab after completion of foundation excavation, installation of equipment such as draft tubes, crane and turbine-generators is to be carried out and accompanying concrete placement and building work are to be done.

Regarding waterway work, the right-bank side of Rio Cauca Diversion Dam is to be constructed following the procedure of the previous year. The Rio Palace and Rio Blanco diversion dams and the Cauca and Palace waterways are to be started in this year. As for Blanco Waterway, after continuing with tunnel driving from the previous year, lining concrete is to be placed.

The works for Dike No. 1 and Dike No. 2 are to be started at the end of the year.

(3) Third Year of Construction (1984)

Besides civil work such as embankment of the main dam and dikes, concreting work of the various structures, drilling and grouting at the main dam and tunnel, all works such as installation of hydraulic equipment consisting of dam outlet facilities, intake gate, diversion dam gates, penstock, etc., and electrical equipment consisting of turbines, generators, transformers, etc., and further, all other work such as the transmission line and telecommunication facilities necessary for start of operation of the power station are to be completed.

Water impoundment is to be started from August, water passage tests through the headrace tunnel to the power station are to be carried out in October - November, and commercial operation of the power station is to be started at the end of December.

9.2 Construction Scheme

9.2.1 Regional Conditions and Transportation Route

The site of this Project is located approximately 10 km northwest of the city of Popayan and is favored with conditions of location from the standpoint of construction work.

The distance from Cali, which is the main source of construction materials, is approximately 150 km, and there is a concrete-paved road in good condition from Cali.

To the west of Cali, the port of Buenaventura on the Pacific Ocean Coast will be the landing port for imported materials, machinery and equipment. The port has facilities for unloading heavy articles and the road to Cali is well-maintained.

There is a road existing over the approximately 10 km between the center of Popayan and the dam site, and it will be adequate with repairs made at a section in the village of Julumito.

9.2.2 Temporary Facilities for Construction

Regarding temporary facilities for construction, their varieties and scales are determined by conditions of location and the scales, construction schedules, topographical and geological conditions, etc. of the structures, and outlines of the major facilities are given in Fig. 9-1.

(1) Access Roads

There will be some amount of new construction of access roads required for hauling rock over the approximately 6 km between the main dam site and the planned quarry site, access roads to the intake and powerhouse, and access roads to the Rio Cauca and Rio Palace diversion dam vicinities and portals of waterway tunnels, but these can be constructed very easily and there are no problems.

(2) Electric Power for Construction

The electric power required for construction of Julumito Power Station is estimated to be a maximum of approximately 3,000 kW. This electric power for construction is to be supplied through a transmission line for construction (34.5 kV) of approximately 10 km from Popayan Substation of CEDELCA, to a substation for construction (34.5 kV/6.6 kV) to be provided in the vicinity of the dam site. It is planned for power to be supplied to the various construction sites by 6.6-kV distribution lines from this substation.

(3) Concrete Facilities

Concrete aggregates are planned to be manufactured at an aggregate plant to be provided downstream of the powerhouse site. The raw materials for aggregates are mainly planned to be the excavation much from the penstock route and the powerhouse foundation.

A concrete plant is to be provided near the aggregate plant with concrete supplied to the various work sites by 3.0 to 4.5 m^3 transit mixer trucks.

With regard to the capacities required of the aggregate plant and the concrete plant, it will be economical for them to be 50 t/hr and 25 m^3/hr, respectively, taking into account factors such as concrete placement schedules.

(4) Air Supply Facilities

Supply of compressed air required for construction work is to be done using both stationary and portable types. Stationary types of the necessary capacities are planned to be provided at the Rio Sate work sites such as the main dam and intake headrace tunnel upper portal are located, and the Rio Cauca side having the headrace tunnel lower portal, surge tank, penstock, powerhouse, quarry site and concrete facilities. Portable type are to be used for other works.

(5) Water Supply Facilities

Water stored upstream of the cofferdam at the main dam site on the Rio Sate is to be used as the source of construction and drinking water.

This water is to be pumped up to a storage tank, and besides supplying to the dam, intake, headrace tunnel, etc., drinking water is also to be supplied.

Construction water for the powerhouse, aggregate plant, concrete plant and other points in the vicinity of the powerhouse is to be intaken and supplied from the cofferdam upstream of the powerhouse site.

Regarding the construction water for the waterway works, water is to be drawn and supplied from upstream of the individual diversion dam sites.

(6) Temporary Buildings

The engineers' office, and the contractors' offices, materials warehouses, repair shops, etc. will be required to be provided in the preparatory works, and since there are favorable topographical conditions, construction can be readily accomplished.

As for quarters for engineers and laborers, it will be possible to commute from Popayan so-that they will not be particularly needed, and hospitals in Popayan can be used as medical facilities, so special facilities are not required to be built.

(7) Other Facilities

Regarding the temporary site workshop for the penstock and other hardwares, preparation of lots will be easy and there will be no problems in particular.

9.2.3 Procurement of Construction Materials

The principal materials to be used for construction are estimated to be 29,800 tons of cement, 1,900 tons of reinforcing bar, and 3,600 kl of oils such as light oil, gasoline and heavy oil.

Of these materials, the greater part is produced in Colombia so that domestic products are to be used, but hydraulic equipment such as gates and penstock, electrical equipment such as gates and penstock, electrical equipment such as turbines and generators, and steel from, steel support, rods, bits, etc. are to be imported.

9.2.4 Construction of Principal Structures

(1) Construction of Main Dam

Firstly, a temporary diversion tunnel is to be provided at the left bank of the Rio Sate for diversion of the river flow. Excavation of the dam foundation is to be carried out after removing topsoil, cutting down in succession from parts of higher elevation, preparing the dam foundation by carrying out river-bed excavation.

At the bedrock to serve as the foundation for the impervious soil core, curtain grouting is to be performed to prevent water permeation. Consolidation grouting is to be performed at parts requiring improvements in foundation bearing capacity and ground improvements.

The embankment volume of the dam is to be a total of 1,254,000 m^3 consisting of 177,000 m^3 of impervious core zone at the center, 113,000 m^3 of filter zones on both sides, and further, on both outer sides, rock zones to be embanked of 894,000 m^3, and riprap amounting to 70,000 m^3 to be placed below El. 1,690 m of the upstream of the dam and at the downstream.

As previously stated, it is planned for rock materials and filter materials to be collected from a quarry site near the powerhouse site on the Rio Cauca and hauled to the dam site.

Impervious core materials are to be collected from a borrow area on the left bank approximately 500 m downstream from the dam site on the Rio Sate. Of the materials collected, those requiring adjustments in gradation and water content are planned to be used upon adjustment at a temporary stockpile to be provided between the dam and the borrow areas.

The embankment schedule of the dam, considering the quantity of embankment, the types of heavy construction machinery to be used, hauling distance, weather, etc., is to be 22 months.

The types of heavy machinery used for embankment will be shovels of dipper capacity 1.5 to 3.0 m^3 class, dump trucks of 18 ton class, bulldozers of 15 to 20 ton class and vibrating rollers of 15-ton class.

The road for hauling material from the wuarry site to the dam is to have ample width and gentle grade enabling trucks to pass each other at high speeds.

Lift heights of embankment and the method of compacting will need to be decided upon carrying out test banking to obtain good results.

After plugging of the temporary diversion tunnel, water is to be impounded in a period of 3 months collecting the Rio Sate's own flow and water drawn from diversion waterways, and water passage tests are to be performed.

(2) Construction of Headrace Tunnel

Construction of the headrace tunnel approximately 1,800 m in length is to be done from both the intake and surge tank ends. In effect, work adits are to be provided at the sides of the two portals and roughly one half, or 900 m, is to be worked from each portal by the rail method.

It is planned for tunnel driving to be completely mechanized, and it is surmised that full-face cutting can be adopted for roughly the entire length.

Concrete is to be lined after completion of tunnel excavation. Concrete is to be placed using steel travelling forms with the full section placed simultaneously.

After lining of concrete, mortar injection and high-pressure grouting are to be successively done.

(3) Construction of Surge Tank

For the surge tank, vertical shaft excavation is to be started after completion of open-cut excavation. Since more than half of the vertical shaft would involve earth excavation, both mechanical excavation and manual excavation are to be employed, and sub-lining concrete is to be placed immediately after completing excavation. The excavation cycle is to be about 1 m/time.

The base tunnel is to be completed before the vertical shaft excavation reaches the bottom.

(4) Construction of Penstock

Open-cut excavation is first to be started from the top, and rock excavation muck is to be utilized as concrete aggregate and bedding gravel.

After completion of excavation, concrete for culverts, anchor blocks (primary) and saddles is to be placed.

Installation of the penstock is to be started from the bottom after finishing concrete placement. The concrete lining of the upper horizontal tunnel is to be completed before installation of the penstock.

(5) Construction of Powerhouse

Excavation of the powerhouse foundation is to be done immediately after completion of open-cut excavation for the penstock and concrete placement to be started.

Prior to this, diversion of the river flow to a temporary diversion waterway planned at the left bank of the Rio Cauca is to be done.

Necessary concrete placement of the powerhouse is to be completed by the time the crane and main equipment are to be installed.

(6) Construction of Rio Cauca Diversion Dam and Other Diversion Dams

Care of river is to be accomplished by the half-river cofferdamming method. The river flow is first to be directed to the left-bank side, water cutoff wall concrete placed along the middle of the river, and after embanking cofferdams at the upstream and downstream on the right-bank side, foundation excavation and sheet pile driving is to be done, followed by placing of dam concrete. It will be necessary to leave a cut-out block to serve as a temporary diversion channel after the river flow has been switched to the right-bank side.

The river flow is next to be switched to the right-bank side and foundation, excavation, sheet pile driving and concrete placement on the left-bank side are to be carried out.

The Rio Palace and Rio Blanco diversion dams are also to be constructed in the manner described above.

(7) Construction of Waterways

The excavation of open canal is to be done mainly by using a backhoe and other machines. After completion of excavation, works necessary before concrete placement such as foundation cobblestone laying, gabion setting and reinforcement placing are to be performed followed by concrete placement of the canal invert. The side concrete is to be placed using steel slipforms.

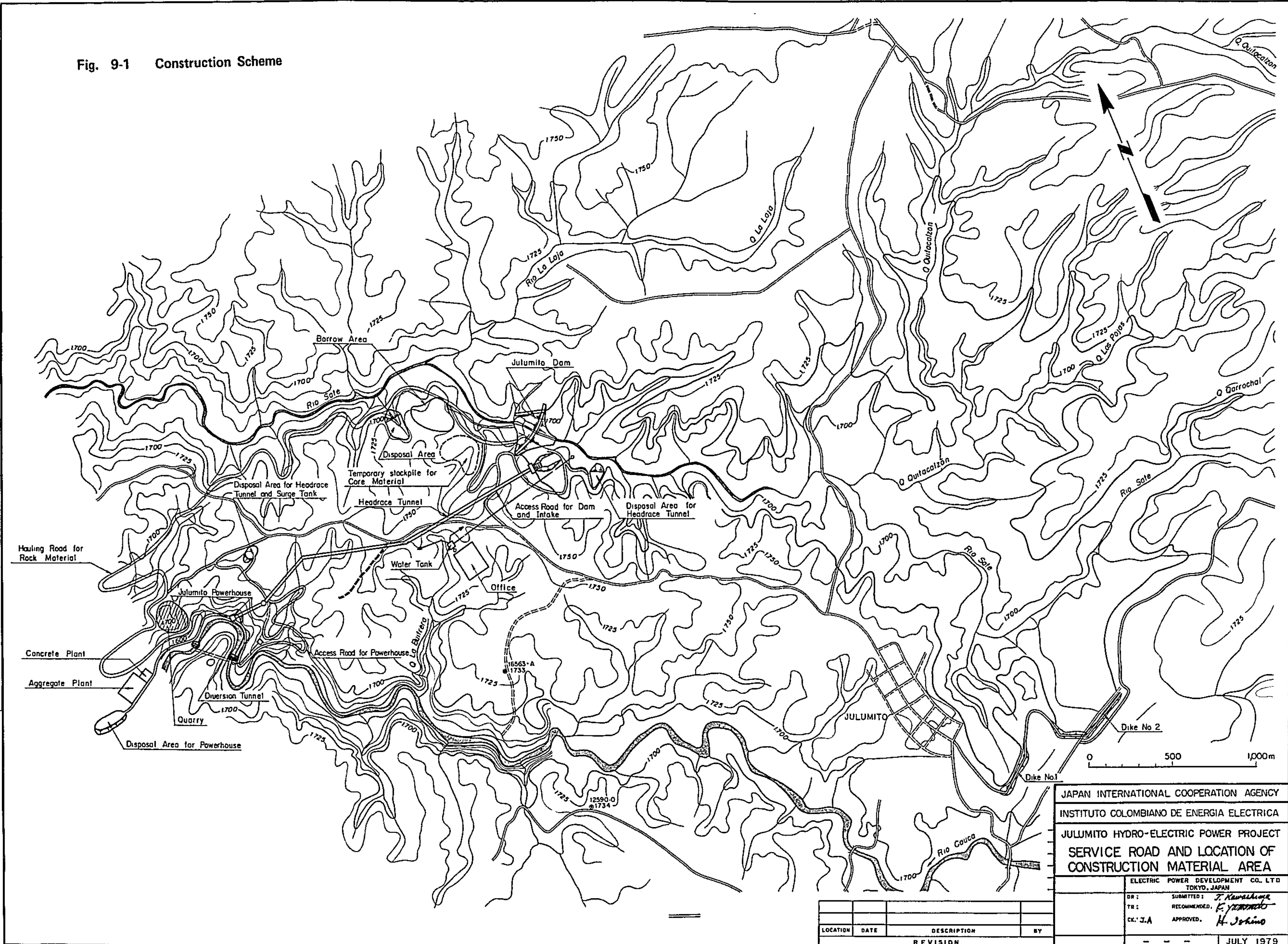
Excavation of tunnel is to be done by the rail method from both portals. Concrete is to be placed using steel travelling forms with the full section placed simultaneously.

Injection of mortar is to be done after lining of concrete.

(8) Other Works

Nothing special is required to be discussed concerning other works.

Fig. 9-1 Construction Scheme



JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

SERVICE ROAD AND LOCATION OF CONSTRUCTION MATERIAL AREA

ELECTRIC POWER DEVELOPMENT CO., LTD TOKYO, JAPAN

DR :	SUBMITTED :	T. Kawashima	
TR :	RECOMMENDED.	K. Yamamoto	
CK.: J.A	APPROVED.	H. Ishino	
	– – –	JULY 1979	

LOCATION	DATE	DESCRIPTION	BY
REVISION			

SHEET NO. OF