CHAPTER 7

WATER SUPPLY SCHEME

CHAPTER 7 WATER SUPPLY SCHEME

CONTENTS

TABLE LIST

FIGURE LIST

CHAPTER 7 WATER SUPPLY SCHEME

7.1 BASIC CONSIDERATIONS

7.1.1 BASIC PRINCIPLE

(1) The Andean Altiplano which includes the project area has a wide distribution of the Maure Formation and Capillune Formation consisting of conglomerate and siltstone from lacustrine deposits formed in the Late Tertiary according to geologic age. These are aquifer formations promising as water sources which are actually being utilized in some areas.

For groundwater to be taken up as the main source of water supply, it is necessary for the safety yield to be determined based on the following:

a) Wide-area investigations of structural geology

b) Investigations of regional piezometric level

c) Investigations of transmissivity of groundwater

Generally speaking, such investigations will require much time and expense, and therefore, the main water source considered in the present study will be surface water which also includes, in part, sub-surface water at shallow depths.

(2) The points which should be considered in the layout of the principal facilities for water supply are the following:

a) As many as possible of the available catchment areas are to be connected by intake facilities consisting of intake dams, and if necessary, combinations of pump-up facilities and waterways, to cover a large catchment area.

b) The intake facilities, moreover, are to be arranged in concentrated manner and not be scattered over a wide area.

c) The pump-up facilities are to be of scales as small as possible so that power costs and operation and maintenance costs for pump-up will be a minimum.

d) Or, for this purpose, a regulating pond where intake quantities will be averaged, and waste discharge losses will be made a minimum is to be economically constructed.

These factors have mutually conflicting relationships, and it is thought the optimum plan can be selected through adjustments of the individual factors.

(3) Based on such a basic principle, comparison studies were made of Alternative Plan A (A-I, A-II) for construction of a dam at Pasto Grande, Alternative Plan B (B-I, B-II, B-III) for conduction of water to the Rio Callazas mainly utilizing the Loriscota Basin, and Alternative Plan C for conduction of water to the Rio Salado mainly utilizing the Loriscota Basin, and as a result Alternative B-III was selected as being the most advantageous fundamental plan.

These circumstances were explained in the form of an interim report (July 1983) and an agreement in principle was obtained (see Appendix-II).

7.1.2 BASIC CONDITIONS

(1) The intake sources for the water supply scheme according to B-III are the two of the Loriscota Basin which is a closed basin with no outgoing stream, and the adjacent Rio Tocco Basin (Pampa Ventillo). The catchment area is 319 km^2 combining the 234 km^2 of the Loriscota Basin and the 85 km^2 of the Tocco Basin.

For this purpose,

a) Laguna Loriscota which presently holds water unsuitable for irrigation is to be drained, after which the lake bottom is to be utilized as a facility having a water collection function, and this is the basic condition of the project.

b) These two catchment areas are separated by low hills at Loripongo, and it is necessary for connection of the two to be made so that they will act as one catchment area.

Since topographical maps of about 1/2,000 considered to be necessary for a feasibility study were not available on the B-III project area, a 1/100,000 aerial photographic map and a 1/25,000 topographical map were used. Also, a 1/5,000 map prepared by the Ministerio de Agricultura y Alimentacion in 1981 covers a part of the B-III project area, and this too, was utilized.

(2) Direct observations of hydrological quantities in the intake catchment areas had not been made. Therefore, the technique adopted was to estimate the necessary hydrological data of the projected catchment areas based on the topographical features in the Loriscota Basin and the hydrological data of neighboring catchment areas, and to verify the proprieties by a suitable technique. The concrete calculation processes are as described in Chapter 5. According to the calculations, the annual average runoff (recharge) is 1.40 m^3/sec, and adding the annual average runoff of 0.50 m^3/sec of the Rio Tocco catchment area, the amount of 1.90 m^3/sec would be the object of intake.
For the quantity to be actually conducted to Laguna Aricota, the transport losses at the waterways to be constructed for the water supply shceme and at the downstream natural streams must be taken into consideration. In concrete terms, the intake quantity available for utilization was determined assuming that 90% of the previously mentioned recharge could be collected at the Loriscota

lake bottom and be conducted to Tocco Intake Dam, then discharge more than $3m^3/sec$ would be spilled out. The quantity of supply to Laguna Aricota was determined assuming a further 10% loss form the previously mentioned intake quantity avaialble for utilization at the waterway and natural streams from Tocco Intake Dam to Laguna Aricota.

7.2 OUTLINE OF WATER SUPPLY SCHEME TO THE LAGUNA ARICOTA

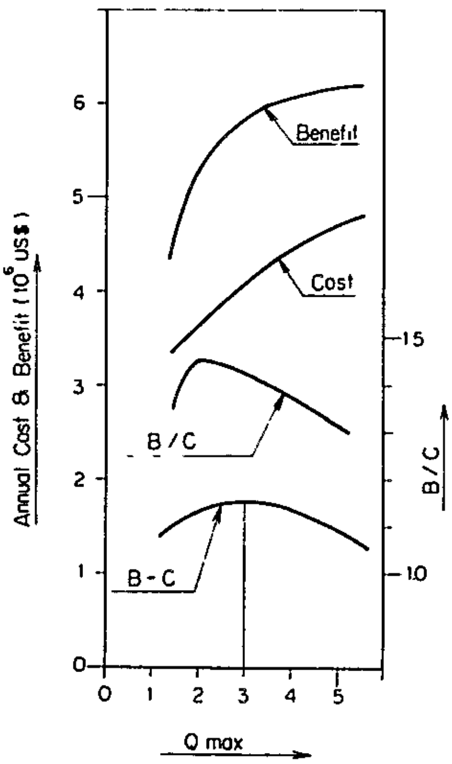
7.2.1 EXAMINATION OF OPTIMUM DEVELOPMENT SCALE

(1) The water supply scheme according to B-III consists of a) water collection at the Loriscota Basin and the Rio Tocco, b) intake by an intake dam and a pumping station provided on the Rio Tocco, and c) water conduction by the facilities such as open canals.

Here, a study will be made of the optimum development scale (Qmax) by examining the effects of variations in the scales of the abovementioned series of intake and conduction facilities on "obtainable benefit".

(2) As cost (annual expense), there are the depreciation costs, maintenance and repair costs, and operation costs of the series of facilities. The relation between maximum intake quantity of these facilities, or capacity of the waterway system and the cost is thought to depend on the topographical conditions.

Fig. 7-1 Optimum Qmax



As benefit, the electric energy produced and the irrigation benefit due to power generation discharge can be considered. Assuming that the benefit per cubic meter of water is constant, the relation between maximum intake and benefit will directly reflect the trend of the discharge duration curve, and when a certain point is exceeded the gradient will be greatly changed.

Table 7-1 Available Quantity of Water Intaken at Tocco Pumping Station

Unit: m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Average
1966	0.87	1.72	1.73	1.42	1.94	1.54	1.44	1.40	0.99	1.17	1.26	1.40	512.91	1.41
67	1.72	3.00	3.00	2.61	1.50	1.75	1.56	1.60	1.28	1.27	0.85	1.53	656.28	1.80
68	3.00	2.48	2.23	1.94	1.65	1.69	1.23	1.66	1.50	1.21	1.81	1.71	673.51	1.84
69	1.71	2.86	2.52	1.50	1.93	1.53	1.35	1.35	1.36	1.05	1.08	1.25	590.14	1.62
70	2.34	2.29	2.70	1.58	1.51	1.36	1.31	1.18	1.01	0.98	0.98	1.25	561.39	1.52
71	2.06	3.00	3.00	1.51	1.25	1.17	1.32	1.22	0.91	0.81	0.98	1.61	570.47	1.56
72	3.00	3.00	3.00	2.95	1.05	0.88	0.87	0.83	0.84	0.82	1.23	1.86	618.33	1.69
73	3.00	3.00	3.00	2.06	1.94	1.76	1.63	1.52	1.57	1.54	1.49	1.44	726.57	1.99
74	3.00	3.00	3.00	2.26	1.53	1.84	1.51	3.00	1.24	0.69	0.65	0.96	688.09	1.89
75	3.00	3.00	3.00	1.63	2.14	1.69	1.56	1.25	0.97	0.76	0.97	1.68	656.89	1.80
76	3.00	2.76	2.58	1.49	1.47	1.27	1.11	1.15	1.59	0.79	0.67	0.89	571.33	1.56
77	1.22	2.99	3.00	1.48	1.52	2.15	1.76	1.28	1.04	0.81	1.70	1.72	625.43	1.71
78	3.00	2.63	1.56	2.10	1.50	1.63	1.52	1.25	1.01	0.84	1.11	1.14	584.25	1.60
79	2.13	1.48	2.25	1.36	1.46	1.34	1.43	1.88	1.07	1.11	1.02	1.11	537.61	1.47
1980	2.72	2.82	2.17	1.29	1.18	1.06	1.05	1.04	0.98	1.23	0.98	0.68	523.25	1.43
Average	2.39	2.67	2.58	1.81	1.57	1.51	1.38	1.44	1.16	1.01	1.12	1.35	606.43	1.66

Examination of the optimum development scale consists of selecting the point at which the effect of capital investment is a maximum through implementation of the water supply scheme, and in concrete terms, it can be said to be a scale that the difference between the benefit curve and the cost curve (B - C) is a maximum. According to the results of examination, the optimum development scale based on Altenative B-III is Qmax = 3.0 m^3/sec, and this is shown in Fig. 7-1.

7.2.2 LAGUNA LORISCOTA

(1) Drainage of Laguna Loriscota

Draining the water unsuitable for irrigation contained in Laguna Loriscota is a basic item of the water supply scheme in Alternative B-III. The methods of drainage conceivable are:

a) Direct drainage by diluting the lake water and providng a drainage waterway

b) Cutting off the two rivers (Rio Lorisa, Rio Putijane) flowing into the lake and diverting to another basin, causing the lake to dry up naturally

c) Otherwise, a chemical treatment method

Of these, chemical treatment is known to be unsuitable for handling the water of Laguna Loriscota since the equipment cost will be enormous in case the quantity to be treated is large. As for the method of shutting off inflow to the lake to cause it to dry up naturally, it was included in Alternatives B-I and B-II and studied. According to simulation calculations, intake of surface water and a part of shallow sub-surface water by a waterway provided along the north shore of the lake will lower the water level of the lake approximately 2 m, but complete drying up will not be achieved.

Consequently, the water of Laguna Loriscota is to be drained by a drainage canal to be provided at an elevation equal to the lake bottom to dry up the lake bottom. What kind of effects the bottom material will have on the water collected on drying or inflow due to precipitation cannot be predicted at present. As described in Chapter 6 (Hydrogeology in Surroundings of Laguna Loriscota), and the considerations concerning water quality of rivers (Chapter 10), there is a possibility that the origin of the lake water was thermal spring water, which may be springing even now. The pit excavations and pump-up tests carried out at the north shore of the lake suggest that giological structure around the lake is not homogeneous at shallow part and distribution of sub-surface flow which share about 55% of quantity of water caltivating the lake is maldistribution. Taking points abovementioned into consideration, it is judged that water collection at the lake bottom after completion of drainage is possible using comparatively simple facility.

Consequently horizontal arrangement of waterways like fish-bone combining collecting canals and conducting canal are constructed at the lake bottom certain area at the deepest part of the lake bottom may be remained as a deposit area for the lacustrine material to be treated if necessary.

(2) Loriscota Canal

Loriscota canal from the lake bottom area to Tocco intake dam must run through the hills of Loripongo which is watershed separating Loriscota Basin and Tocco Basin.

According to the report on geological survey achieved in the vicinity (drilling hole SA-4), sand-gravel layers and gravel layer are continuing to a fairly great depth under the lake so that at suggest a possibility of seepage of underground water. Since topographic surveying has not been carried out as yet, the details are unknown, but it is thought the height from the elevation of the lake bottom to the top of the hill is about 15 to 20 m.

Taking into consideration such geological and topographical conditions, the waterway to pass through Loripongo will be an open canal with gently sloped. The earthwork involved in this method is seen to be a maximum of 1,400,000 m^3 in all, and will be a large-scale works. In the future, if a detailed geological investigation is carried out and it becomes possible to adopt a tunnel driving method or a special tunnelling method, an examination of the construction method should be made anew.

The Loriscota waterway would also be utilized as a drainage canal on diluting the water of Laguna Loriscota, and discharge will be made into the Rio Tambo via the Rio Viscachas.

7.2.3 TOCCO INTAKE FACILITIES AND WATERWAY

(1) Tocco Intake Dam

An intake dam is to be constructed at the downstream most part of the Pampa Ventillo at a narrow point at the foots of Co. Cuesllampo and Co. Pacchiauqui. The geology to be the dam foundation is a fluvio-glacial deposit with properties such as thickness of distribution, permeability, and deformability unknown, but it is estimated that the gradation is comparatively fine-grained.

Therefore, considering such conditions, the dam is to be a fill-type dam using local materials, and moreover, since material for an impervious core is not available in the vicinity, it is to be an asphalt facing type.

Since the water sources which can be utilized are limited, water cut-off at the foundation is important despite the fact that the scale of the dam is small. Although the groutability of the dam foundation consisting of fluvio-glacial deposits is unknown, a water barrier must be formed by a grouting method or some other method.

The flood flows of the Tocco and Loriscota catchment areas must be safely released by a spillway (concrete overflow weir) adjacent to the dam. In consideration of operation and administration of facilities in the plateau area, control gates will not be provided. The dam crest length including the spillway will be 135 m, with the maximum height 11.5 m.

(2) Pumping Station and Pumping Pipeline

The water taken in is to be pumped up by pump and pipeline to obtain head for conducting it to the Rio Mataza and Rio Callazas. The pumping station is to be a semi-underground reinforced concrete structure in which two 1,600-kW pumps, necessary auxiliary equipment, and control apparatus are accomodated. In general, a greater suction head is required for a high altitude compared with a low one, and 3 m is adopted here. The principal dimensions of the pumping station are width of 11.00 m, length of 19.00 m, and height of 9.50 m. Pump-up pipelines are connected at the discharge sides of the pumps. The two pipeliens are immediately joined after leaving the pumps to become a single line of diameter of 1.20 m. The pipeline is supported by anchor blocks, ring girders and saddles.

(3) Conduction Waterway

The conduction waterway is to be an open canal of length of 30 kM, have a gradient of 1/1,500, and go from the end of the pipeline to pass the north shore and west shore of Laguna Viscacha to reach the Rio Mataza. On this route, at the north of Laguna Viscacha, there will be an inverted shiphon at the canyon connecting to the Rio Japopunco, while at Loma Tasujane at the south of Laguna Viscacha, there will be a tunnel.

Various structural materials can be considered for this open canal of maximum capacity of 3.0 m^3/sec, but in view of the following:

a) The canal is long so that the materials must be readily available at the site,
b) The construction cost must be cheap, and maintenance and administration at this remote highland must be easy,

the lining is to be a combination of concrete (bottom slab) and cobblestone concrete (side walls).

A waterway of this type is already in use for condition of water from the Laguna Blanca area to the Rio Uchusuma, is one of the accomplished technique as construction method.

7.2.4 PUMP-UP EQUIPMENT

(1) Pump-up Equipment

Tocco Pumping Station is planned for a total head of 85.0 m, and maximum pump-up capacity of 3.0 m^3/sec. Two pump units are to be provided considering economy, inspection, overhauling, etc. The comparisons of installation costs by number of pumps are shown in Table 7-2.

Table 7-2 Economic Comparison of Pump Unit

Number of Pump	1	2	3
Total Pumping Head (m)	85.0	85.0	85.0
Pumping Discharge (per unit) (m^3/sec)	3.0	1.5	1.0
Capacity of Mortar (kW)	3,200	1,600	1,100
Revolution Speed (rpm)	504	750	880
Construction Cost ($x10^3$US$)			
Electrical Equipment	1,422	1,647	1,883
Civil Structure	1,677	1,540	1,596
Total	3,089	3,187	3,484

As can be seen in Table 7-2, when the numbers of pumps are compared only by construction cost, the single-unit proposal is the cheapest. However, in the case of one unit, pump-up will not be possible when there is trouble with the pump, and when inspection and overhauling are considered, a plural number of pumps will be absolutely necessary. Furthermore, even if the number of pumps is made two, the cost will be only about 3% higher compared with the proposal for one unit. With three units the cost would be approximately 9% higher compared with the two-unit proposal, and therefore, two units were adopted.

Pump-up operation is to be controlled with pumps operated intermittently based on the water level of Tocco Regulating Pond.

(2) Power Transformation Facilities

The power supply to Tocco Pumping Station is to be the 69 kV transmission line from Suches Substation owned by SPCC. The switchyard is to be an outdoor type provided next to the pumping station. The receiving transformer is to be one outdoor-type, 3-phase, oil-immersed transformer of capacity of 4,500 kVA, and voltage of 69 kV/33 kV.

Table 7-3 gives the general specifications of Tocco Pumping Station, and Fig 7-7 the single line diagram of the station.

Table 7-3 Dimension of Tocco Pumping Facilities

Output of Pumping Station	
Pump	3,200 kW
Type	Horizontal shaft, double suction, volute pump
Number of Unit	2
Total Pumping Head	85 m
Pumping Discharge	1.5 m^3/sec
Revolving Speed	705
Pump Input	1,460 kW
Motor	
Type	Wound-rotor type, 3-phase, Induction motor
Number of Unit	2
Output	1,600 kW
Voltage	3,300 V
Frequency	60 Hz
Transformer	
Type	Outdoor, 3-phase, oil-immersed transformer
Number of Unit	1
Capacity	4,500 kVA
Voltage	69 kV/3.3 kV

7.2.5 POWER TRANSMISSION AND TELECOMMUNICATION FACILITIES

(1) Transmission Line

The transmission line for power supply to the pumping station to be provided at the Tocco site for water supply to Laguna Aricota is planned for 69 kV, 1 cct, 35 km from Suches S-2 Substation of SPCC. Details are described in Chapter 9, "Transmission Line Plan and System Analysis."

(2) Telecommunication Facilities

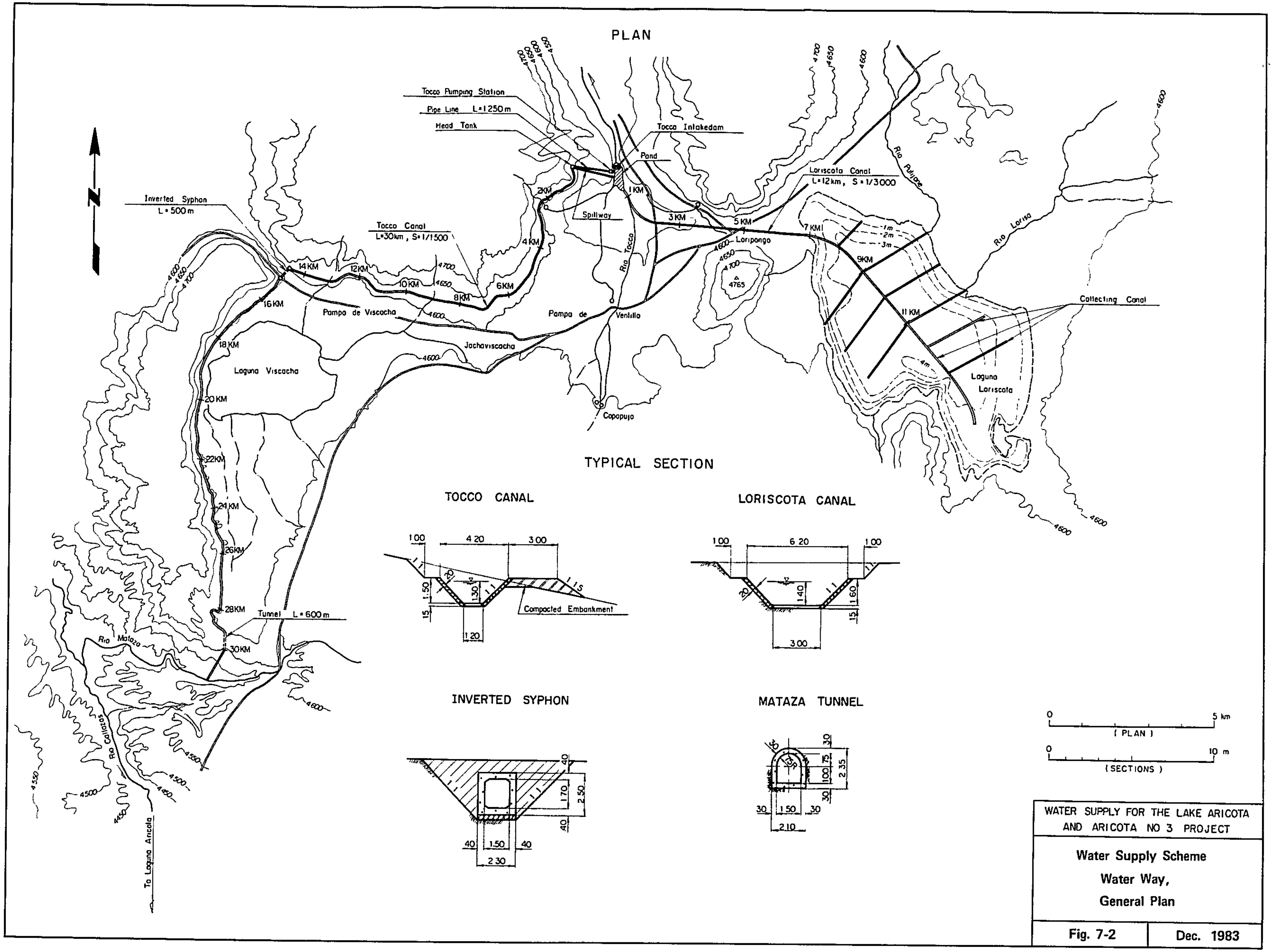
The telecommunication channels between Aricota No. 2 Power Station and the pumping station is to be by short wave. The frequency is to be the 5 MHz band, 7 MHz band or 9 MHz band used by CORDETACNA, but it will be necessary to decide this upon carrying out transmission tests at the time of carrying out detail design.

With regard to other means of communication, power line carrier channel (PLC) usually employed will require many PLC apparatus to be installed and connected since there is no direct transmission line between the pumping station and Aricota No. 2 Power Station. As for VHF, this would require a relay station at a point at elevation of approximately 3,500 m because of the topography, and there would be difficulties about securing power supply, and structuring, constructing, and maintaining the station.

The equipment required for a short wave channel would be shown in Table 7-4.

Table 7-4 Telecommunication Facilities

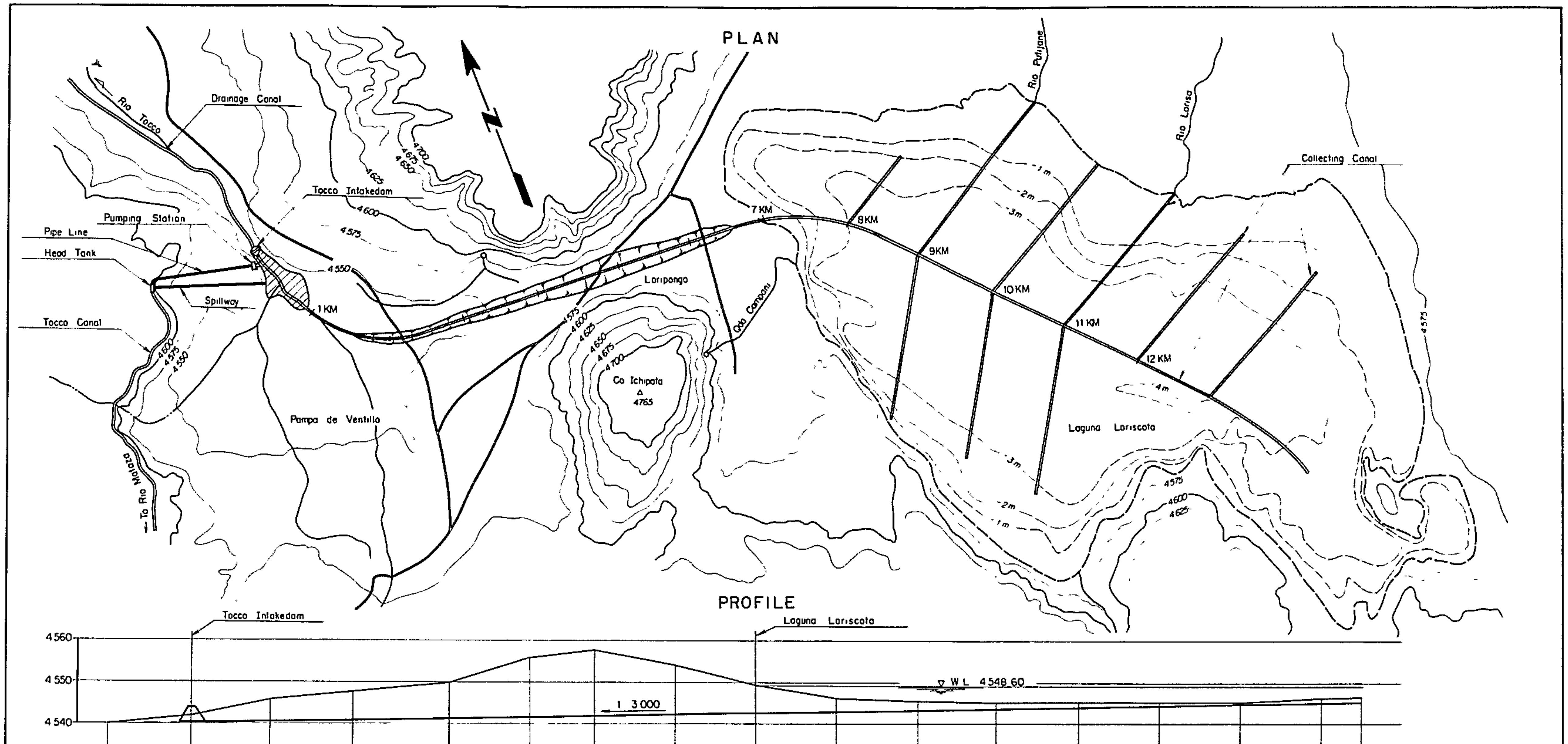
Item	Pump Station	Aricota No. P/S
HF Radio Equipment	1 set	1 set
Antenna with pole and feeder	1 set	1 set
Power supply equipment	1 set	1 set



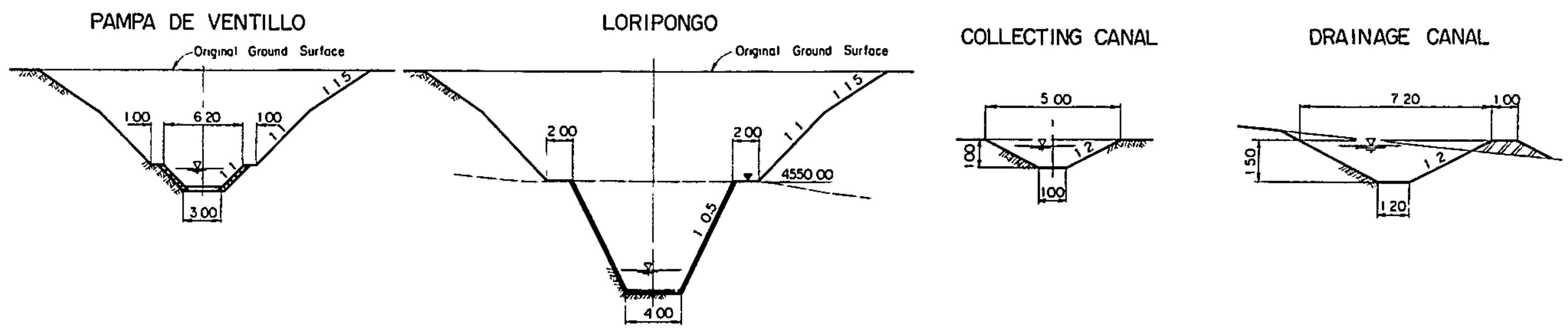
WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT

Water Supply Scheme
Water Way,
General Plan

Fig. 7-2	Dec. 1983



Distance (KM)	-1.0	0	1.0	2.0	3.2	4.2	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	14.5
Formation Height		4540.00	4540.33	4540.66	4541.07	4541.40	4541.66	4542.00	4543.30	4542.66	4543.00	4543.33	4543.66	4544.00	4544.33	4544.66	4544.83
Ground Height	4 549	4 540	4 546	4 548	4 550	4 556	4 558	4 554	4 549	4 546	4545.30	4 545	4 545	4 545	4 545	4 546	4546.10



0 — 2.5 km (PLAN)

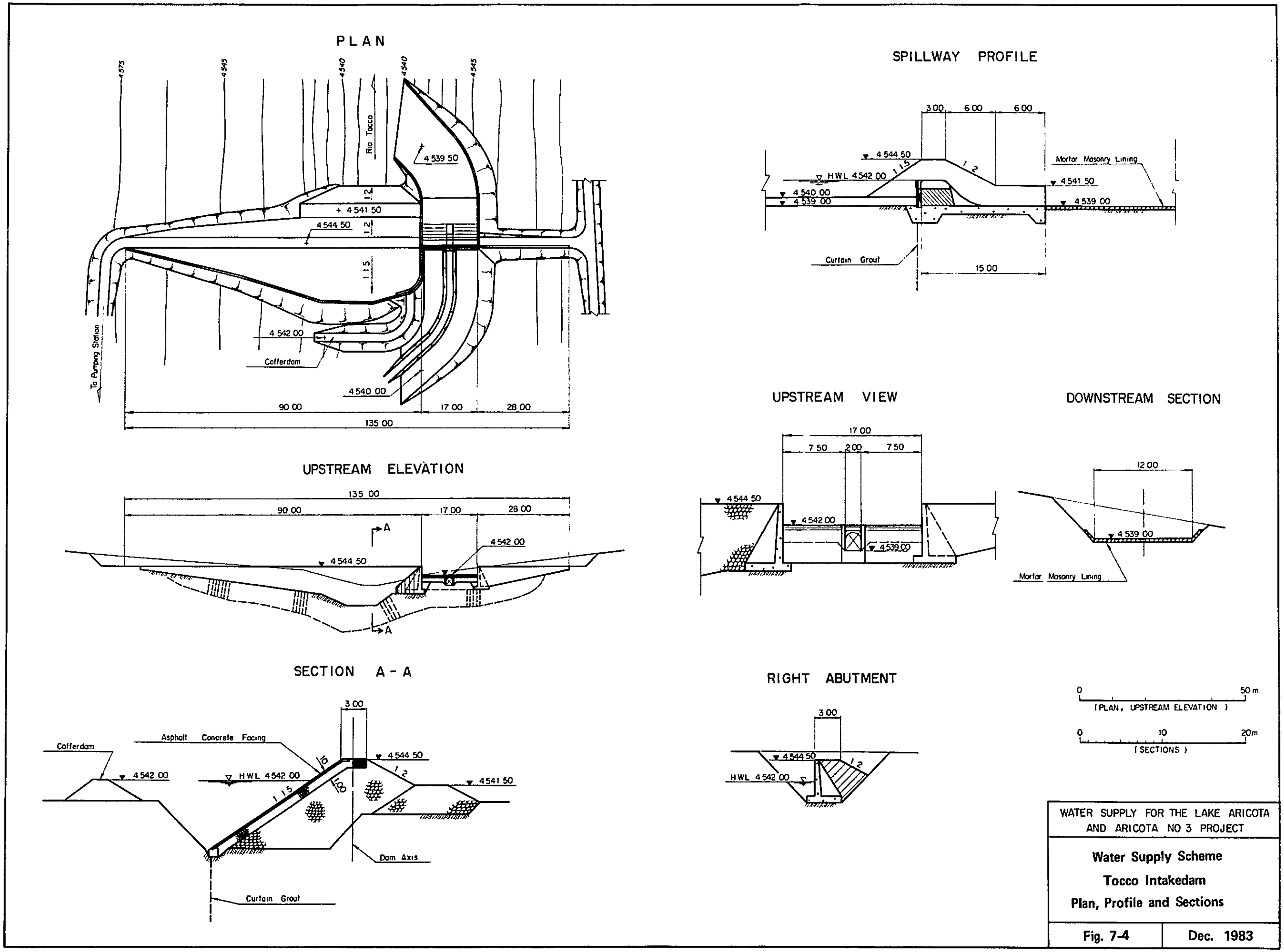
0 — 20 m (PAMPA DE VENTILLO, LORIPONGO)

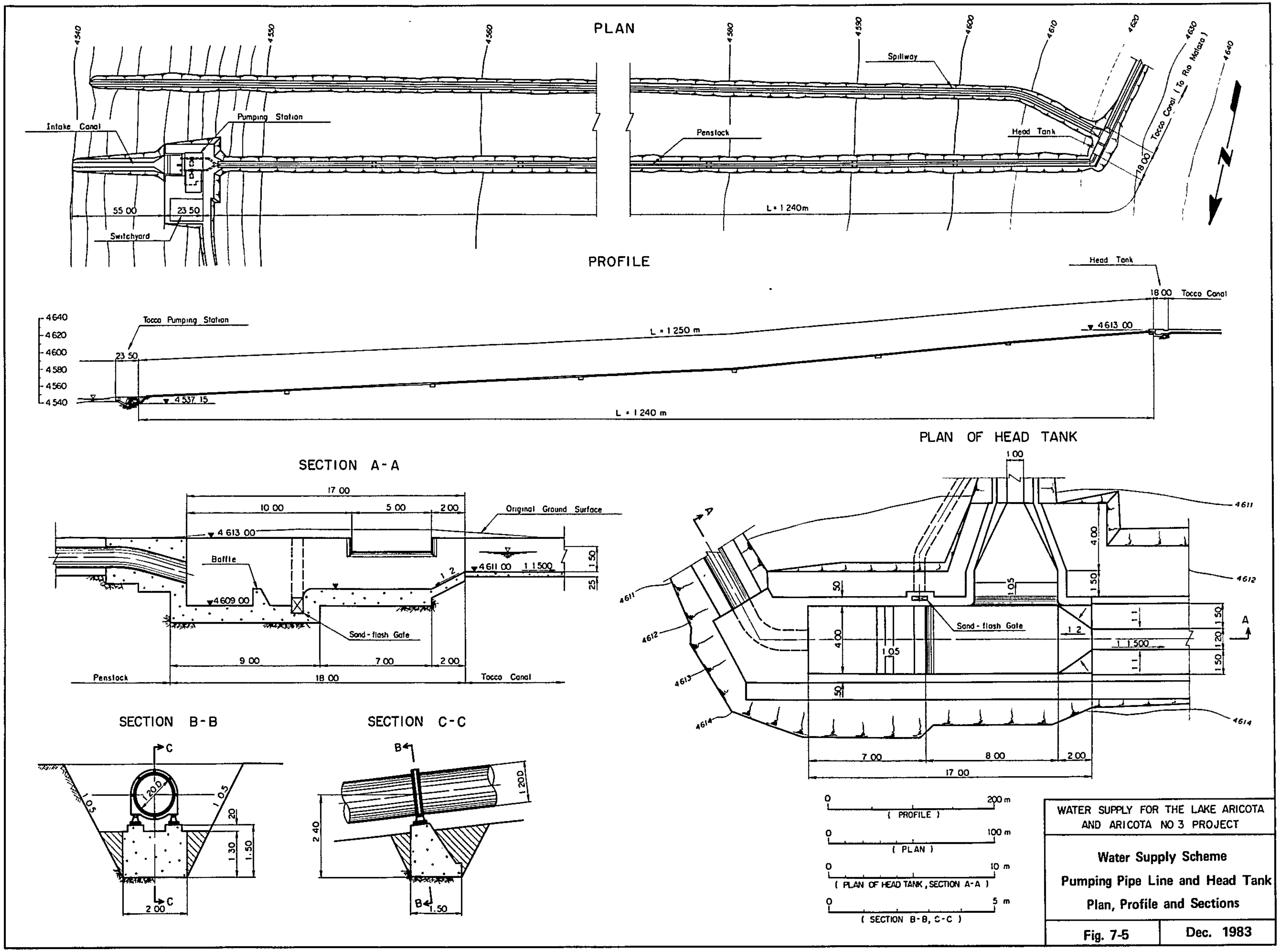
0 — 10 m (COLLECTING CANAL, DRAINAGE CANAL)

WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT

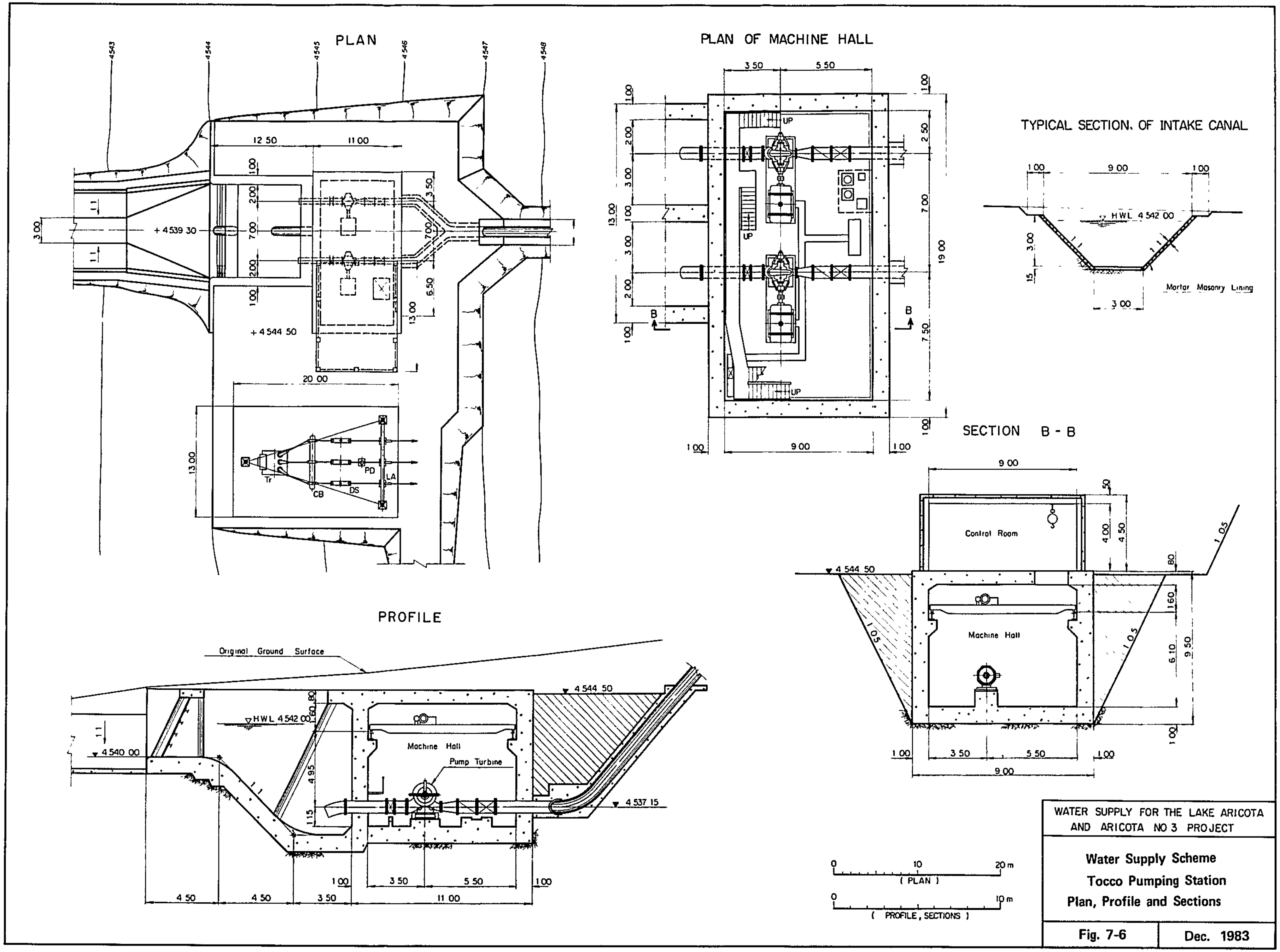
Water Supply Scheme
Loriscota Canal
Plan, Profile and Sections

Fig. 7-3 | **Dec. 1983**


PLAN
4575
4545
4540
4540
4545
Rio Tocco
4 539 50
+ 4 541 50
4 544 50
1.2
1.2
1.1.5
4 542 00
Cofferdam
4 540 00
To Pumping Station
90 00
17 00
28 00
135 00
SPILLWAY PROFILE
3 00
6 00
6 00
4 544 50
HWL 4 542 00
1.1.5
1.2
4 540 00
4 539 00
Mortar Masonry Lining
4 541 50
4 539 00
Curtain Grout
15 00
UPSTREAM ELEVATION
135 00
90 00
17 00
28 00
A
A
4 542 00
4 544 50
UPSTREAM VIEW
17 00
7 50
2 00
7 50
4 544 50
4 542 00
4 539 00
DOWNSTREAM SECTION
12 00
4 539 00
Mortar Masonry Lining
SECTION A - A
3 00
Cofferdam
Asphalt Concrete Facing
4 542 00
HWL 4 542 00
4 544 50
4 541 50
1.1.5
1.2
1.00
Dam Axis
Curtain Grout
RIGHT ABUTMENT
3 00
4 544 50
HWL 4 542 00
1.2
0
50 m
(PLAN, UPSTREAM ELEVATION)
0
10
20m
(SECTIONS)
WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT
Water Supply Scheme
Tocco Intakedam
Plan, Profile and Sections
Fig. 7-4
Dec. 1983



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT	
Water Supply Scheme Pumping Pipe Line and Head Tank Plan, Profile and Sections	
Fig. 7-5	Dec. 1983

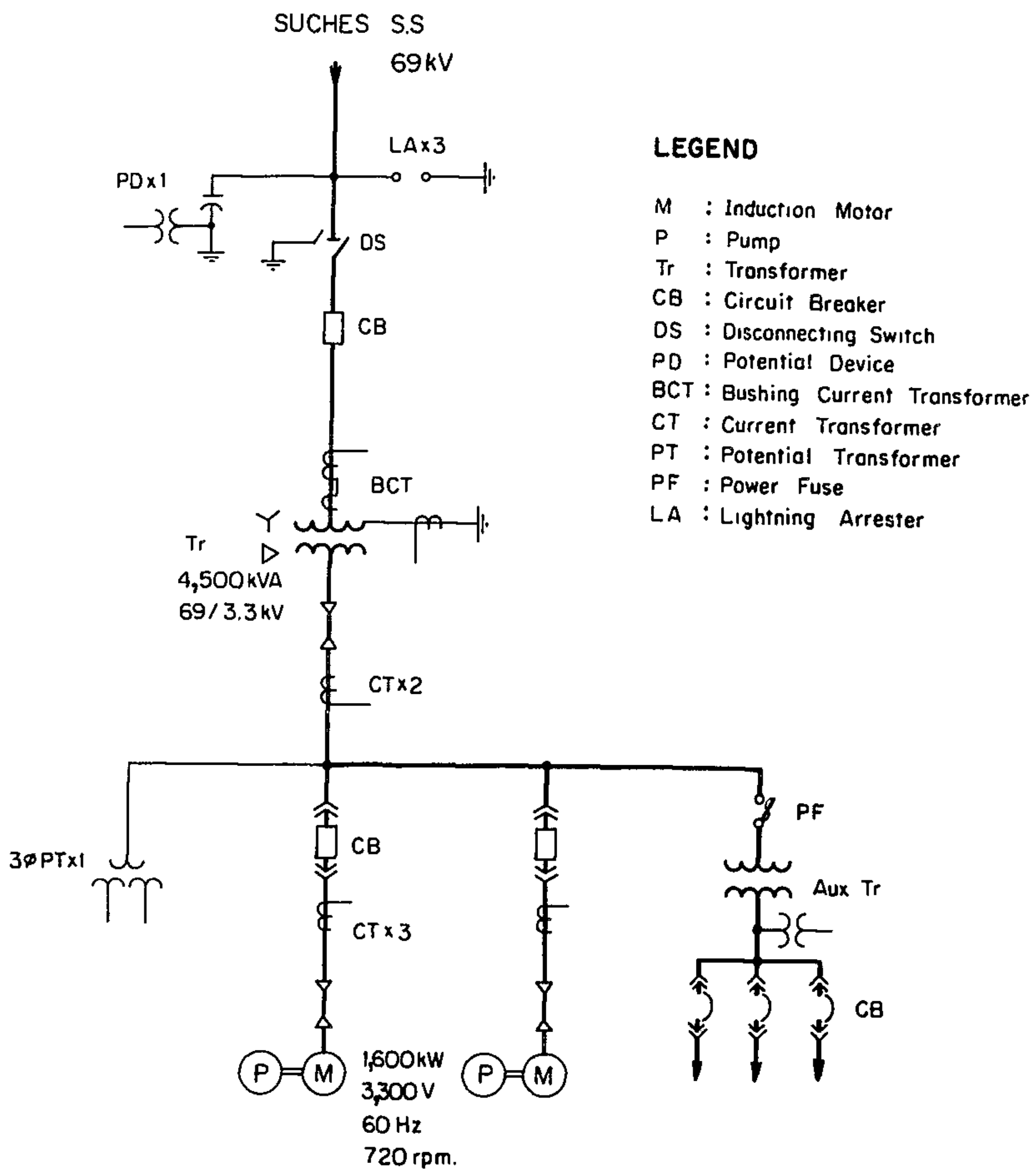


WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT

Water Supply Scheme
Tocco Pumping Station
Plan, Profile and Sections

Fig. 7-6	**Dec. 1983**

Fig. 7-7 Tocco Pumping Station Single Line Diagram



CHAPTER 8

ARICOTA NO.3 HYDROELECTRIC POWER SCHEME

CHAPTER 8 ARICOTA NO.3 HYDROELECTRIC POWER SCHEME

CONTENTS

TABLE LIST

FIGURE LIST

CHAPTER 8 ARICOTA NO.3 HYDROELECTRIC POWER SCHEME

8.1 BASIC CONSIDERATIONS

The Aricota No. 3 Hydroelectric Power Scheme was studied based on the fundamental conditions and considerations given below.

(1) As described in Chapter 4, "Power Demand Forecast," in the Aricota System in the southwestern part of Peru which is the object area of this Project, development of new electric source is expected to cope with the rapidly increasing power demand.

(2) The Aricota System has the existing Aricota No. 1 and No. 2 hydroelectric power stations with a maximum of 4.6 m^3/sec pumped up from laguna Aricota for use in power generation, and a total of 35,700 kW of power is being generated. Aricota No. 3 Power Station planned here is a scheme effectively utilizing the remaining head of the Rio Curibaya downstream of the tailrace of the existing Aricota No. 2 Power Station and the water pumped up from Laguna Aricota.

(3) The Aricota No. 3 Hydroelectric Power Scheme has been subjected to a basic study regarding five alternatives in "Report on Modified Second Stage Development of Plan Tacna November, 1971," and as shown in Appendix, the conclusion was drawn that Alternative III-C is the most economical plan. The outline of Alternative III-C is that an intake dam is to be provided immediately downstream of the tailrace of Aricota No. 2 Power Station, a maximum of 4.6 m^3/sec is conducted to a power station provided at the Chulibaya site on the right bank of the Rio Curibaya by a headrace tunnel of length of 6.7 km, and 14,000 kW of power is generated. The Survey Mission, taking into consideration the results of field investigations, reexamined

these conditions, and as a result judged that Alternative III-C, compared with other alternatives is appropriate on condition that it has the highest head per unit length of waterway and is economical. Consequently, in this chapter, a still further detailed examination will be made with the before-mentioned Alternative III-C as the fundamental plan.

(4) The existing Aricota No. 1 and No. 2 power stations are conduit-type power stations using non-pressure tunnels, and in principle are for meeting base load. However, if the power demands of the Aricota System in the near future are predicted, the arrival times of water flowing down the headrace tunnels are considered, pump-up from Laguna Aricota is started beforehand, and planned operation of the two power stations is carried out, these power stations can serve to meet peak loads.

(5) Determination of the reservoir operation plan for Laguna Aricota aims mainly at effective utilization for the water delivered to Laguna Aricota by the water supply scheme. In other words, this reservoir operation not only should make the available energy production of Aricota No. 3 Power Station as large as possible, but also, the operation method more comprehensive and more economical must be reviewed including the characters and capacities of the existing Aricota No. 1 and No. 2 power stations, and the effects on the irrigated land in the downstream area of the Rio Curibaya.

(6) In determination of the reservoir operation plan for Laguna Aricota, the basic condition is that meeting the power demand is to be given first priority. The reason for this is that with the water stored in Laguna Aricota the benefit from electric power will be far greater than the benefit produced by irrigation regardless of the method of operation.

8.2 HYDROELECTRIC POWER SCHEME

8.2.1 DEVELOPMENT SCALE

(1) Regulating Pond

Since the existing Aricota No. 1 and No. 2 are power stations having the character of meeting base load, it is desirable for Aricota No. 3 Power Station planned this time to be made to meet peak load from the standpoint of operation of the power system. In order to make Aricota No. 3 a peaking power station it is necessary to have a regulating pond capable of storing the power generation discharge from Aricota No. 2 Power Station.

In view of the fact that the firm discharge of Aricota No. 2 Power Station after completion of the Laguna Aricota water supply scheme will be approximately 3 m^3/sec, and that in order to serve as a peaking power station it must be possible for peak load power generation of about 4 to 6 hours to be done, although it will depend also on the maximum discharge, a regulating capacity of approximately 200,000 m^3 would be necessary.

However, the Rio Curibaya downstream of Aricota No. 2 Power Station has an average river-bed gradient of about 1/22 and is steep, while in addition, the river-bed deposits are thick and the river width is large. Taking these into consideration there is no suitable place for providing a regulating pond with capacity mentioned before.

In addition, to make the peak discharge of Aricota No. 3 Power Station larger than the maximum discharge of 4.6 $m^{3/sec}$ of the existing Aricota No. 2 Power Station will result in the necessity for a reregulating pond to be provided, and this would be a factor to increase cost.

As a result of consideration of the above, it was decided that a regulating pond which is able to regulate daily would not be provided for Aricota No. 3 Power Station.

However, in order to prevent the effect of runoff variation produced by Aricota No. 2 Power Station meeting load variations and directly affecting Aricota No. 3 Power Station, there will be a necessity for a pond to absorb runoff variation. Besides the above, when there is outage of the No. 2 Power Station, the power generating discharge from the No. 1 Power Station will run down the Rio Curibaya. Consequently, an intake structure will be needed for this water to be effectively utilized. This point was considered and it was planned for a small-scale dam to be constructed.

(2) Maximum Discharge

As previously mentioned, Aricota No. 3 Power Station will be directly connected to the tailrace of the existing Aricota No. 2 Power Station. In effect, Aricota No. 3 Power Station will be a facility completely subordinate to the No. 2 and No. 1 power stations. Since no residual flow can be seen at all in the Rio Curibaya at the intake dam site, drawing water from the river is inconceivable. Consequently, the maximum available discharge of Aricota No. 3 Power Station will be the same 4.6 m^3/sec as the No. 1 and No. 2 power stations.

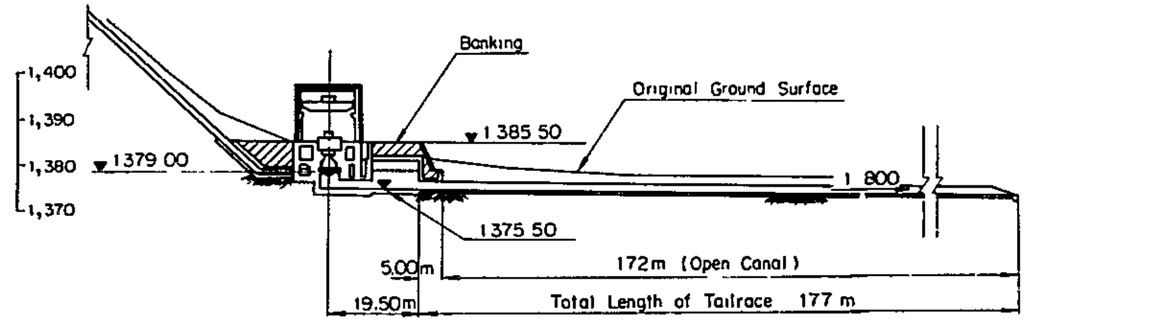
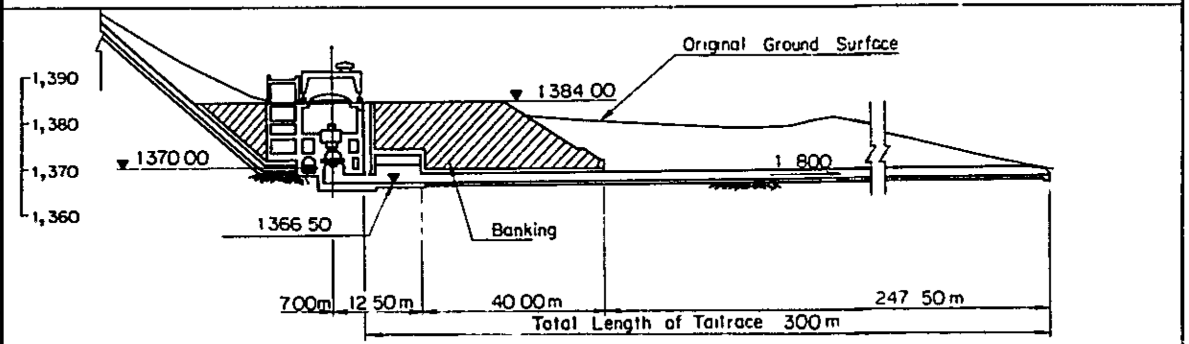
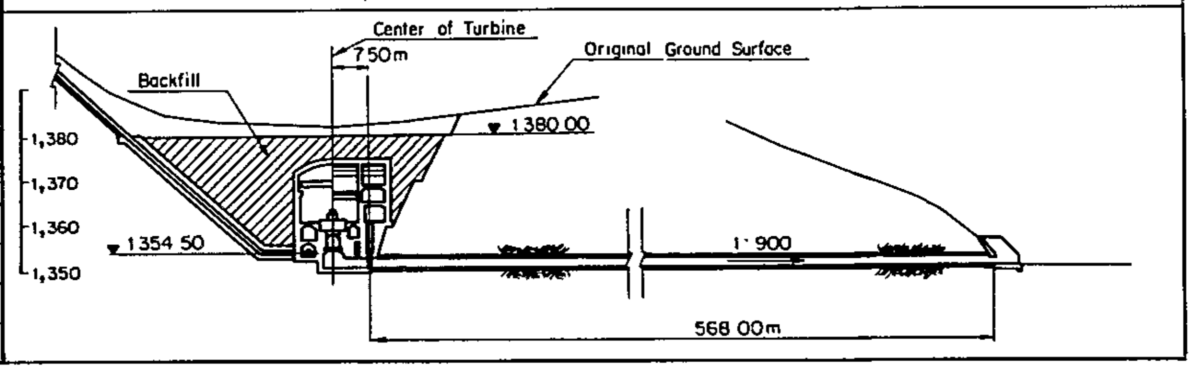
(3) Installed Capacity

Aricota No. 3 Power Station will be a non-pressure conduit-type power station using a Pelton turbine, and the head will be the height differential between the water level at the head tank and the center of the turbine. The water level at the head tank will be determined by the intake water level at the intake dam and the hydraulic gradient of the headrace tunnel.

The existing Aricota No. 2 Power Station is provided with a Pelton turbine. In general, with this type of turbine, structurally it is said that the turbine efficiency will not be affected if a head of 2 m is secured between the turbine center and the discharge water level. Therefore, considering the hydraulic gradient of the tailrace and wave height due to wind, a freeboard of 0.5 m was added to the above 2 m, and the intake water level was made EL. 1,749.5 m, 2.5 m down from the elevation of the turbine center.

In deciding the elevation of the turbine center, the topography and geology of the power station site were considered, and upon carrying out a comparison study by varying the discharge water level within practicable limits to effectively utilize head to the maximum limit as shown in Table 8-1, the optimum alternative was selected. As a result, the elevation of the capacity center was made EL. 1,370 m, and the installed capacity 13,400 kW.

Table 8-1 Comparison Table of Installed Capacity (Aricota No.3 P/S)

Case	Item			Profile
I	Head Tank Water Level	m	1,739 10	
	Centerline of Turbine	m	1,379 00	
	Gross Head	m	360 10	
	Effective Head	m	348 00	
	Installed Capacity	kW	13,000	
	Annual Energy Production	10^6 kWh	69 0	
	Construction Cost	10^3 US$	29,000	
	Construction Cost per kW	US$/kW	2,230	
	Construction Cost per kWh	US$/kWh	0 420	
II	Head Tank Water Level	m	1,739 10	
	Centerline of Turbine	m	1,370 00	
	Gross Head	m	369 10	
	Effective Head	m	357 00	
	Installed Capacity	kW	13,400	
	Annual Energy Production	10^6 kWh	70 8	
	Construction Cost	10^3 US$	29,400	
	Construction Cost per kW	US$/kW	2,200	
	Construction Cost per kWh	US$/kWh	0.415	
III	Head Tank Water Level	m	1,739 10	
	Centerline of Turbine	m	1,354 50	
	Gross Head	m	384 60	
	Effective Head	m	372 50	
	Installed Capacity	kW	14,000	
	Annual Energy Production	10^6 kWh	73 9	
	Construction Cost	10^3 US$	31,500	
	Construction Cost per kW	US$/kW	2,250	
	Construction Cost per kWh	US$/kWh	0.426	

8.2.2 OPERATION AND CONTROL OF ARICOTA POWER STATIONS

The waterway system of the Aricota No. 1 to No. 3 power stations, as shown in Fig. 8-1, does not have any regulating ponds, while long times are required for the water pumped up at the pumping station to reach the individual power stations. Therefore, it is diff- ficult for operation following the load of the Aricota System without waste discharge. However, it is thought power station control with minimizing waste discharge will be possible if the techniques of (1) to (3) below are employed.

(1) Setting of Waterway Conditions Necessary for Control

Arrival time of flow from pumping station to Aricota No. 1 Power Station, T_1 ; approx. 45 min.

Arrival time of flow from Aricota No. 1 Power Station to No. 2 Power Station, T_2 ; approx. 45 min.

Arrival time of flow from Aricota No. 2 Power Station to No. 3 Power Station, T_3 ; approx. 75 min.

(2) Control of Power Station

The most economical method of power station control would requires waste discharge to be minimized. According to this conditions, rather than operating the individual power stations by alloting base load or peak load, it will be more economical to operate the power stations according to the inflow from upstream, controlling the pump-up quantity in correspondence with the load.

As an example, the pump-up quantities corresponding to forecast load curves for 1992 are shown in Fig. 8-2 (1) and (2).

Controlling pump-up quantities as shown in the figures based on the results, it will be possible to hold waste discharge to a minimum. However, it is thought necessary for the following considerations to be given in application of this control method.

a) The control method above requires a load forecast for the system. In case the accuracy of the load forecast is not good, it will be necessary for more water than that corresponding to the load to be pumped up and be prepared for an increase in load while carrying out waste discharge.

b) In case waste discharge is produced for load adjustment, it will be easier to implement control carrying out waste discharge at only a specific power station. There will be no difference economically at whichever power station the waste discharge is made, but in consideration of ease of control, it is thought better to select the No. 3 power station.

c) When there is a sudden increase in the load on the system during peak hours, it is not possible to increase the loads on all of the power stations simultaneously by merely increasing the pump-up quantity because of the lags in arrival times of flow in the waterway system. Therefore, by providing an intake dam downstream of the tailrace of the No. 2 power station and through control by a gate to increase load rapidly at the No. 3 power station in step with the No. 1, it will be possible to follow the load on the system.

(3) Capacity of No. 3 Power Station Regulating Pond

The capacity of the pond required for the control described in (2).c) was calculated based on the conditions below.

a) The discharge quantity is taken to be a quantity which can meet a sudden increase in load of 75% to 100% at the No. 3 power station.

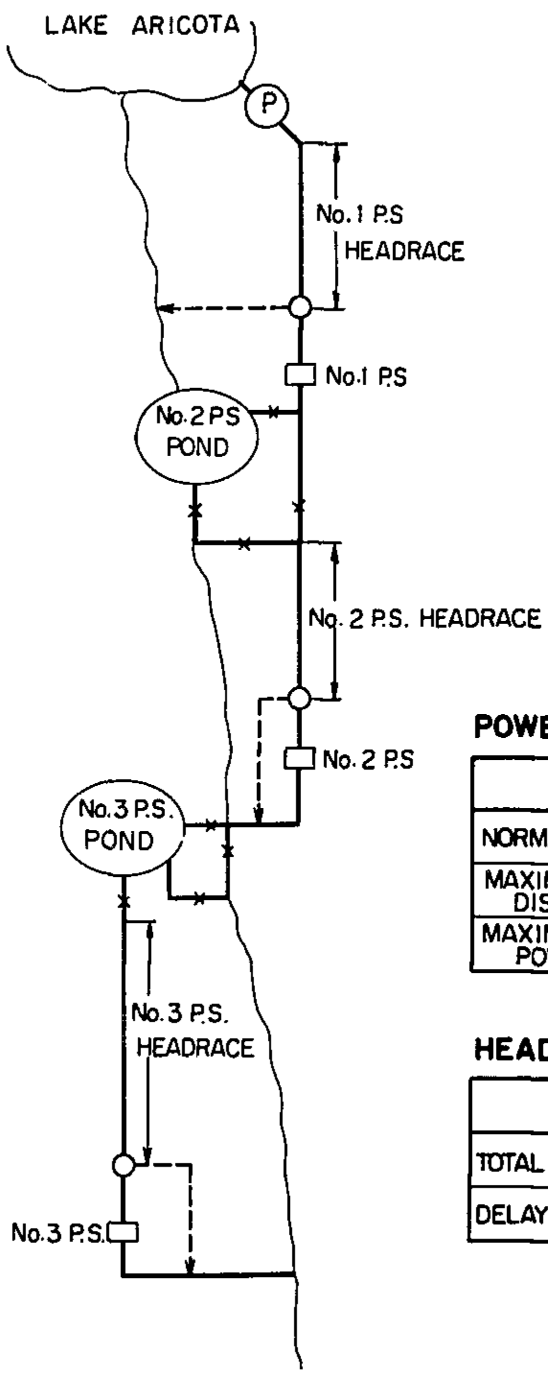
$$4.6\ m^3/sec \times (1.0 - 0.75) = 1.15\ m^3/sec$$

b) The duration of discharge from the regulating pond is put as 90 minutes, the time required for pump-up at the pumping station to be increased then flowing down to the regulating pond.

$$\text{Capacity ; } 1.15\ m^3/sec \times 90\ min \times 60sec \doteqdot 6,000\ m^3$$

Based on the above, the capacity of the regulating pond for the No. 3 power station was made 6,000 m^3.

Fig. 8-1 Schematic Plan of Aricota Powér System



LEGEND :

- ——— OPEN CHANNEL OR TUNNEL
- ～ RIVER CURIBAYA
- (P) PUMPING STATION
- ○ HEADTANK
- □ POWER STATION
- × GATE OR OVERFLOW GATE
- - - -> SPILLWAY OR TUNNEL FROM HEADTANK

POWER STATION

	No. 1 P.S.	No. 2 P.S	No. 3 P.S.
NORMAL HEAD	617.10 m	311.8 m	357 m
MAXIMUM DISCHARGE	4 6 m^3/s	4.6 m^3/s	4.6 m^3/s
MAXIMUM POWER	23.8 MW	11.9 MW	13.4 MW

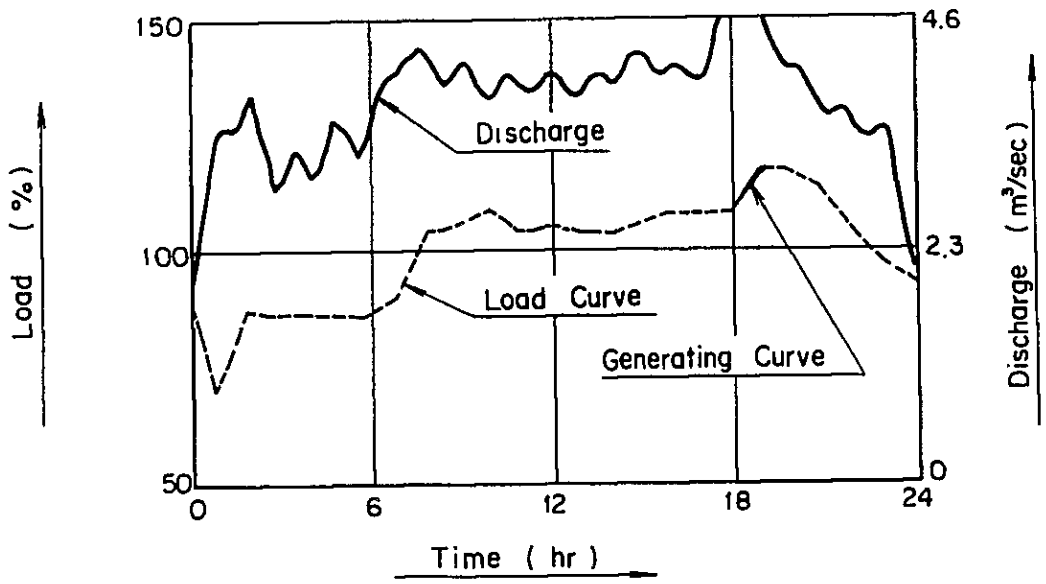
HEADRACE

	No. 1 P.S.	No. 2 P.S.	No. 3 P.S.
TOTAL LENGTH	4 700 m	4 900 m	7 000 m
DELAY TIME	45 min	45 min	75 min

Fig. 8-2(1) **Estimated Daily Load Curve in Aricota System and Discharge Control Curve at the Pumping Station**

Case-1, Estimated from the daily load curve on the day which the system demand recorded the maximum of the year during the last five years, 1978-1982

(1) Control interval of the station is 15 minutes



(2) Control interval of the station is 60 minutes

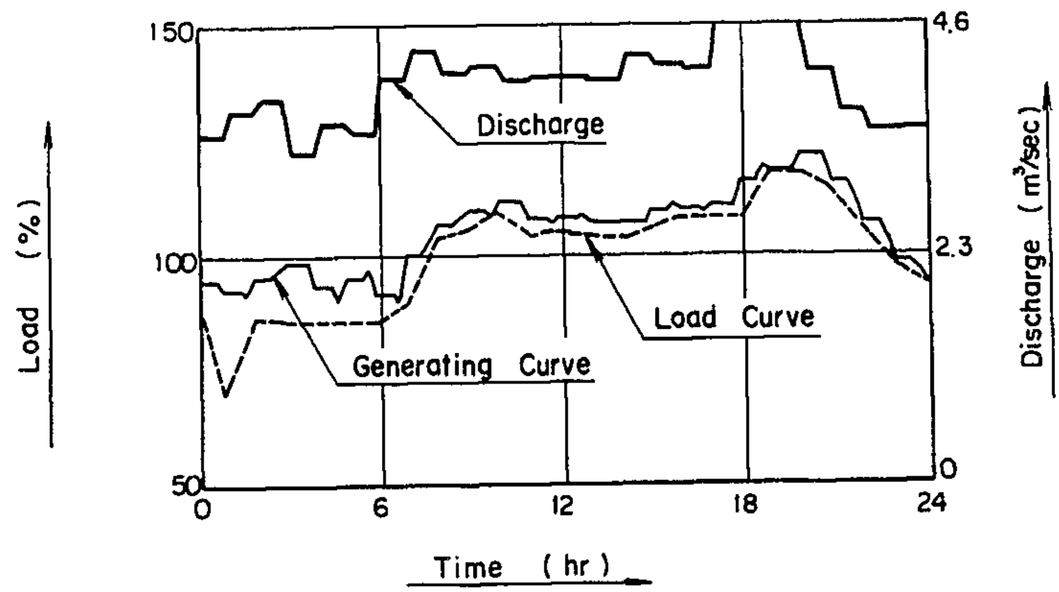
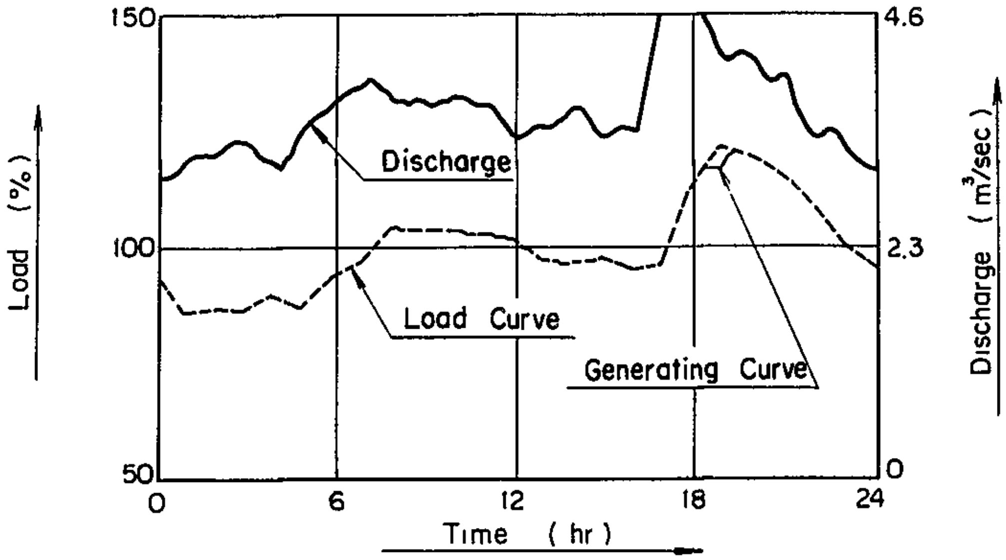


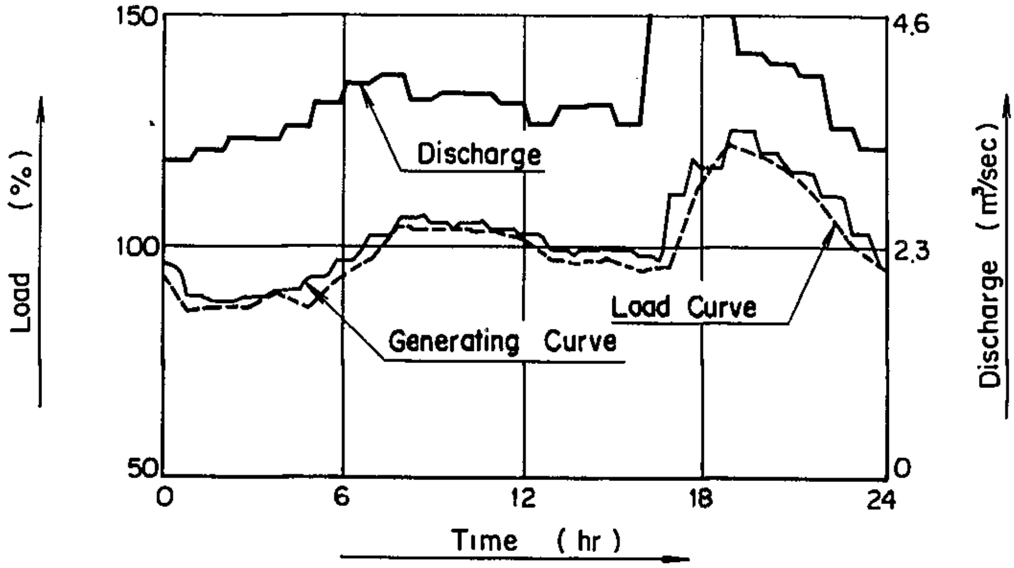
Fig. 8-2(2) Estimated Daily Load Curve in Aricota System and Discharge Control Curve at the Pumping Station

Case-2 , Estimated from the average daily load curve during the month on June in 1982

(1) Control interval of the station is 15 minutes



(2) Control interval of the station is 60 minutes



8.2.3 CIVIL STRUCTURES

(1) Intake Dam

The intake dam will be provided in the form of directly connecting to the tailrace of Aricota No. 2 Power Station. In this vicinity, there are two large canyons merging with the Rio Curibaya from both the right and left banks of the river, but these canyons normally do not have water flows and are dry. However, since there is a possibility of debris flow being produced by rainfall, the dam axis was selected upstream of these canyons to avoid any debris flow. The intake dam site has thick river-bed deposits close to 30 m down to the foundation rock, and permeability is very high.

Consequently, in selection of the dam type, since it was permissible for the scale of the dam to be made small, and in considertion of the depth to bedrock and the physical properties of river-bed deposits, it was decided that a fill-type dam with facing member should be constructed. Concrete or asphalt concrete are conceivable as materials for the facing member and asphalt concrete which is superior in the capability to follow uneven settlement and which has a low coefficient of permeability was selected.

As shown in Fig. 8-4, it is to be a fill-type with facing member dam with the height of 5 m and crest length of 56 m, and asphalt concrete facing is to be provided at the entire bottom and slope surfaces.

A spillway cum sand flush is to be provided at the right-bank side of the dam with a roller gate of 1.50 m x 2.75 m for a discharge capacity of 10 m^3/sec.

(2) Intake

The intake, as shown in Fig. 8-5, is to be provided facing the spillway at the right bank side of the dam. It is to be a vertical intake structure with a screen provided at the front, and a maximum of 4.6 m^3/sec is to be taken in. The intake is to be equipped with two roller gates of 3.00 m x 1.00 m for maintenance and control of the headrace tunnel. All of the intake water will be power generation discharge water of Aricota No. 2 Power Station, and since it is thought there will be little sedimentation, a settling basin will not be provided.

(3) Headrace Tunnel

The headrace tunnel, as shown in Fig. 8-6, is a non-pressure tunnel with length of about 7.2 km to be provided along the right bank of the Rio Curibaya, and the maximum capacity is to be 4.6 m^3/sec.

The geology in the upstream part of the headrace tunnel route consists with rocks such as dacite, andesite and a rhyolitic pyroclastic rock, which are hard although joints are developed. The middle and downstream parts are comprised of tuff breccia. It is though that none of the rocks will pose much of an obstacle to excavation. The tunnel route was selected considering the geological conditions, cover of the tunnel, locations of work adits, and the work schedule.

In consideration of economy and workability at construction stage for the maximum capacity of 4.6 m^3/sec, and performances at the existing Aricota power stations, the tunnel is to be 1.90 m in height and 1.70 m in width, with a cross section semi-circular at the top and rectangular at the bottom, and the gradient is to be 1/800.

Lining with concrete or shotcrete is to be provided depending on the geology. Where the tunnel crosses the canyon merging with the Rio Curibaya at the right bank at a point 300 m downstream from the intake, it is to be made a reinforced concrete culvert structure.

(4) Head Tank

The head tank, as shown in Fig. 8-7, is to be provided by excavating the mountain slope in consideration of the topographical and geological conditions. The length is to be 36 m with an effective width of 4.5 m, and the effective capacity is to be 500 m^3 with which the power station can be operated safely for 1.5 minutes at the maximum discharge of 4.6 m^3/sec. Besides a sand flush gate equipped near the end of the head tank, a screen and a roller gate of 1.80 x 1.80 m are to be provided at the intake bay immediately before the penstock orifice. A side-overflow type spillway is also provided adjointing to the head tank so as to safely release the maximum available discharge during complete load shutdown at the power station.

(5) Penstock and Spillway Pipeline

The penstock, as shown in Fig. 8-8, is to be a exposed type in consideration of the topographical and geological conditions of the site, workability at construction, and economy. As a result of surface reconnaissances, it is estimated that foundation rock exists within 6 m from the ground surface, while the deposits at the surface layer as seen by the topography and condition of deposition will have little mobility, are in block form and are stable. The average gradient of the mountain slope is 23 degrees and it is thought that it will pose no problem in particular for erection of the penstock. The penstock is to be a single line, with inside diameters of 1.40 to 0.90 m, and the length is approximately 820 m.

The spillway pipeline is for safely releasing water overflowing the head tank, and in consideration of the topography of this site, it is to be installed in parallel to the penstock. The inside diameter of the spillway pipeline is 0.90 m, and the length is about 810 m. Water flowing down from the head tank is to have its energy dissipated by a stilling basin provided adjacent to the power station, then released to the Rio Curibaya.

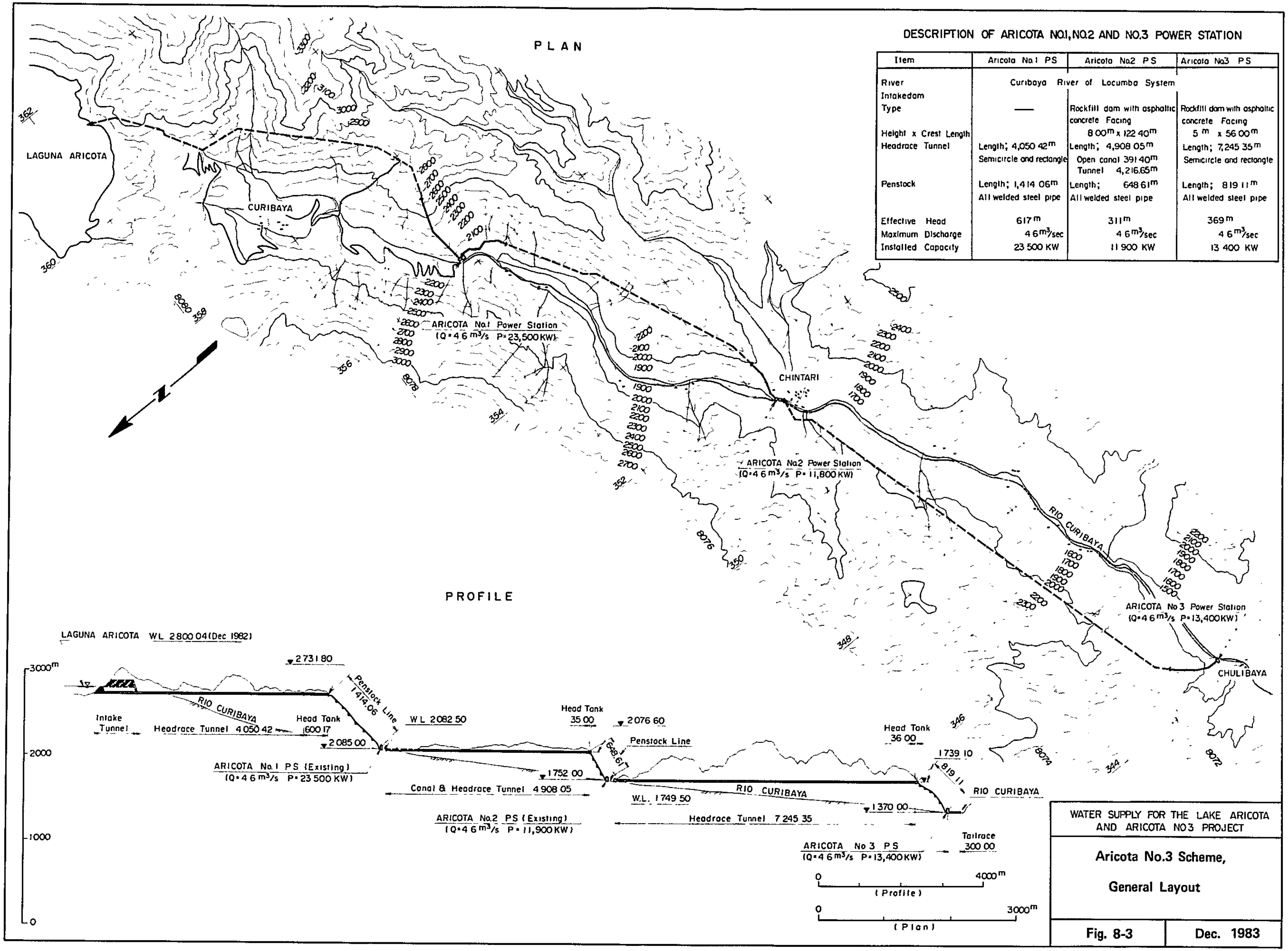
(6) Powerhouse and Switchyard

The powerhouse and switchyard, as shown in Figs. 8-9 and 8-10 are to be a surface-type powerhouse and outdoor switchyard in consideration of topographical and geological conditions, and of economy.

As described under the item, 8.2.2, "Optimum Scale," in order to effectively utilize the head in view of economy, the turbine center elevation has been lowered approximately 10 m from the present ground level and the powerhouse will be of semi-underground type structure. Consequently, delivery of equipment is to be achieved by a gantry crane provided on the ground.

(7) Tailrace

The tailrace is to be a cobble stone concrete lined trapezoidal-shape open canal of bottom width of 2.00 m, and top width of 4.00 m, the gradient being 1/800 and the length 300 m.



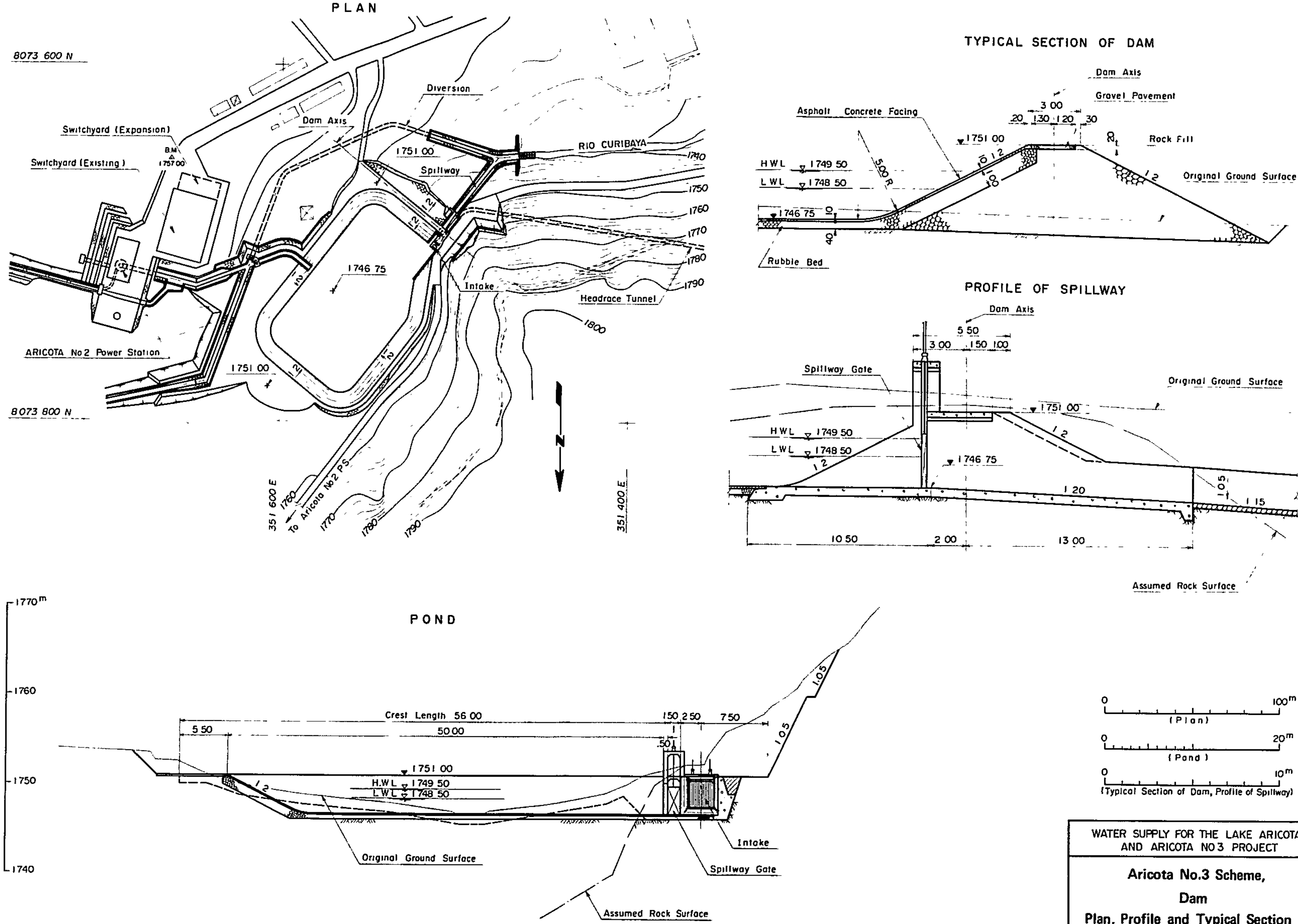
DESCRIPTION OF ARICOTA NO.1, NO.2 AND NO.3 POWER STATION

Item	Aricota No.1 PS	Aricota No.2 PS	Aricota No.3 PS
River	Curibaya River of Locumba System		
Intakedam			
Type	—	Rockfill dam with asphaltic concrete Facing	Rockfill dam with asphaltic concrete Facing
Height x Crest Length		8 00m x 122 40m	5 m x 56 00m
Headrace Tunnel	Length; 4,050 42m Semicircle and rectangle	Length; 4,908 05m Open canal 391 40m Tunnel 4,216.65m	Length; 7,245 35m Semicircle and rectangle
Penstock	Length; 1,414 06m All welded steel pipe	Length; 648 61m All welded steel pipe	Length; 819 11m All welded steel pipe
Effective Head	617m	311m	369m
Maximum Discharge	4 6m³/sec	4 6m³/sec	4 6m³/sec
Installed Capacity	23 500 KW	11 900 KW	13 400 KW

WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO3 PROJECT

Aricota No.3 Scheme, General Layout

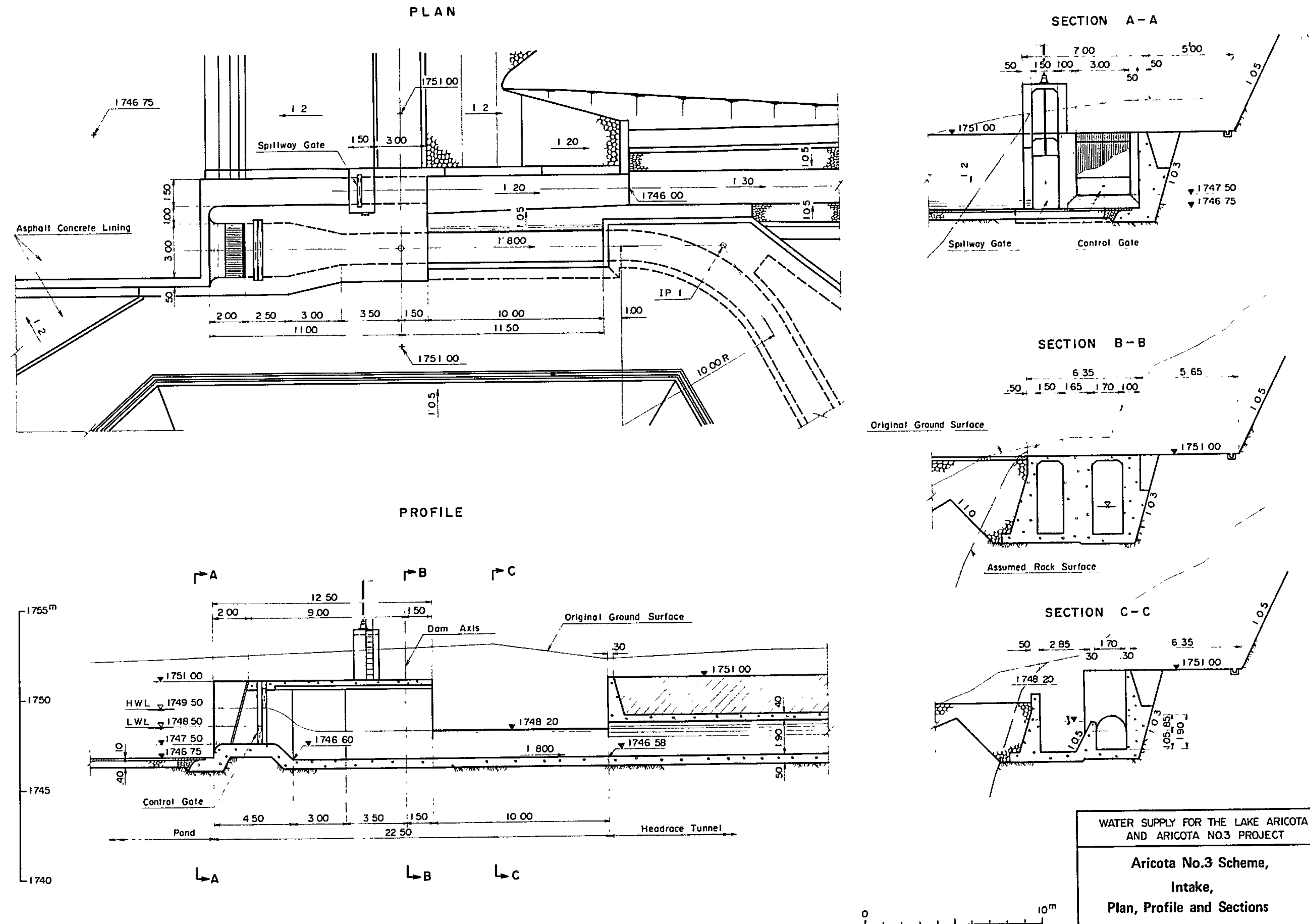
Fig. 8-3 | **Dec. 1983**



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT

Aricota No.3 Scheme,
Dam
Plan, Profile and Typical Section

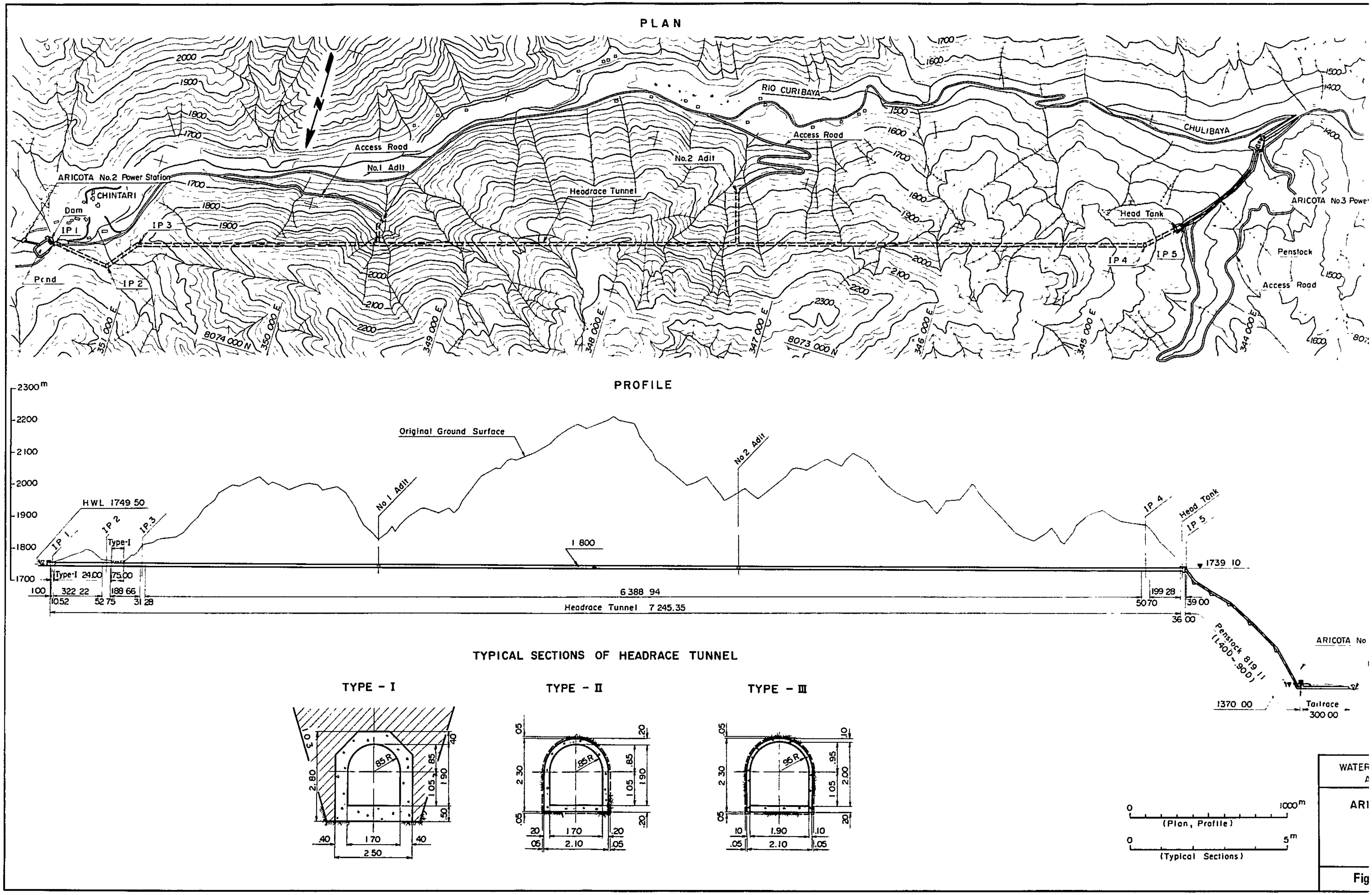
Fig. 8-4	**Dec. 1983**



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT

Aricota No.3 Scheme, Intake, Plan, Profile and Sections

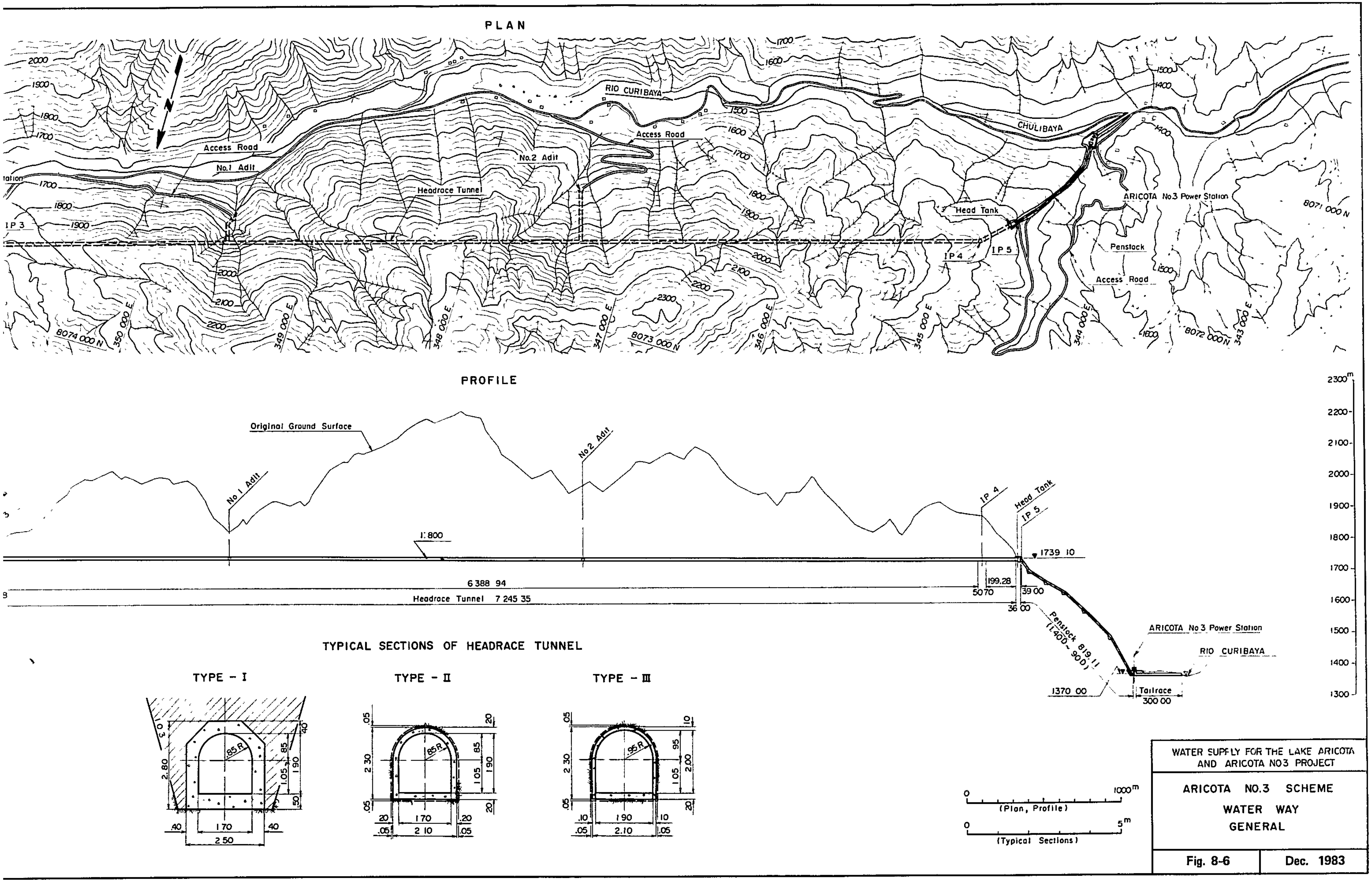
Fig. 8-5	Dec. 1983



WATER
A
ARI

Fig

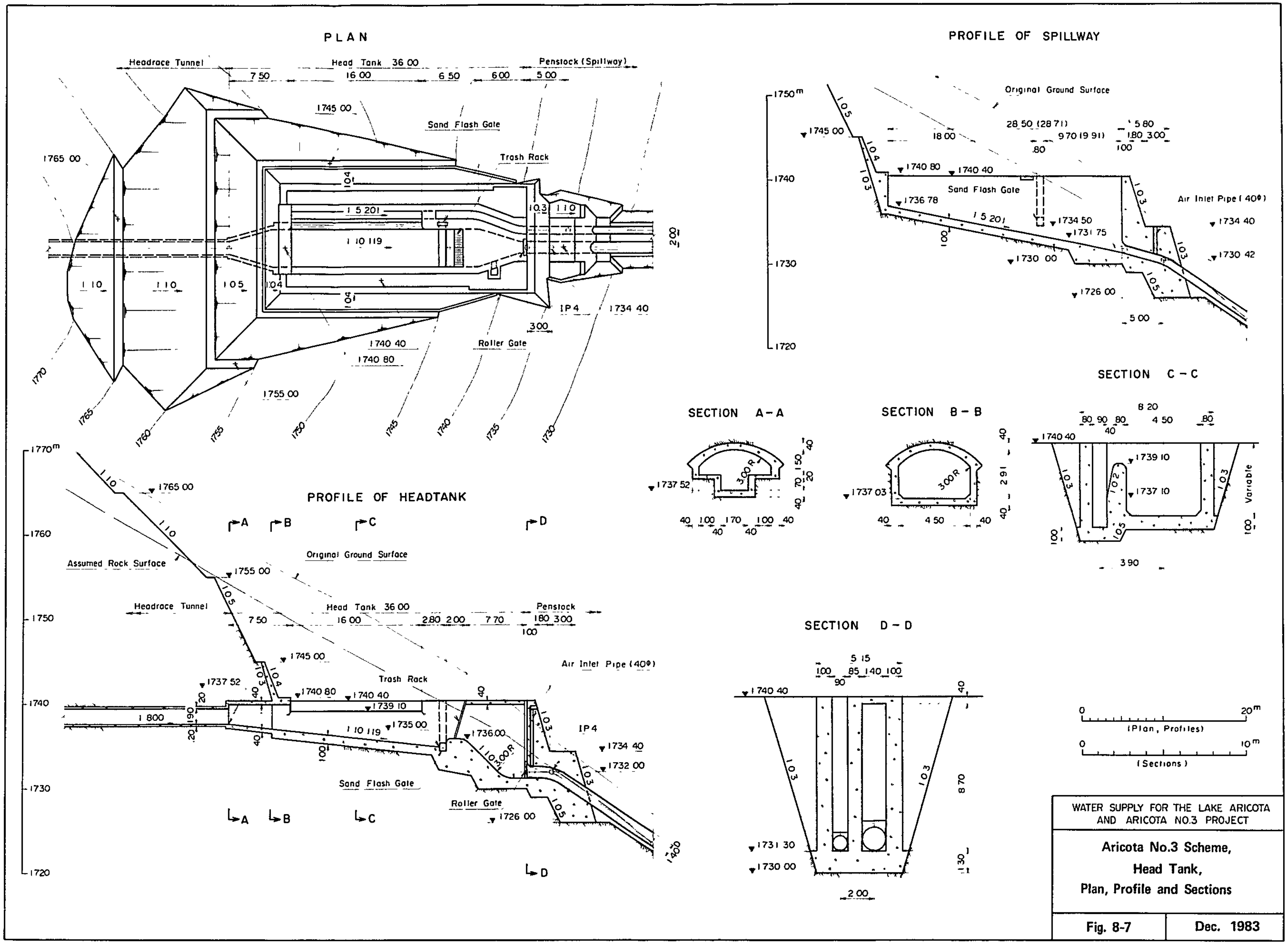
V



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT

ARICOTA NO.3 SCHEME
WATER WAY
GENERAL

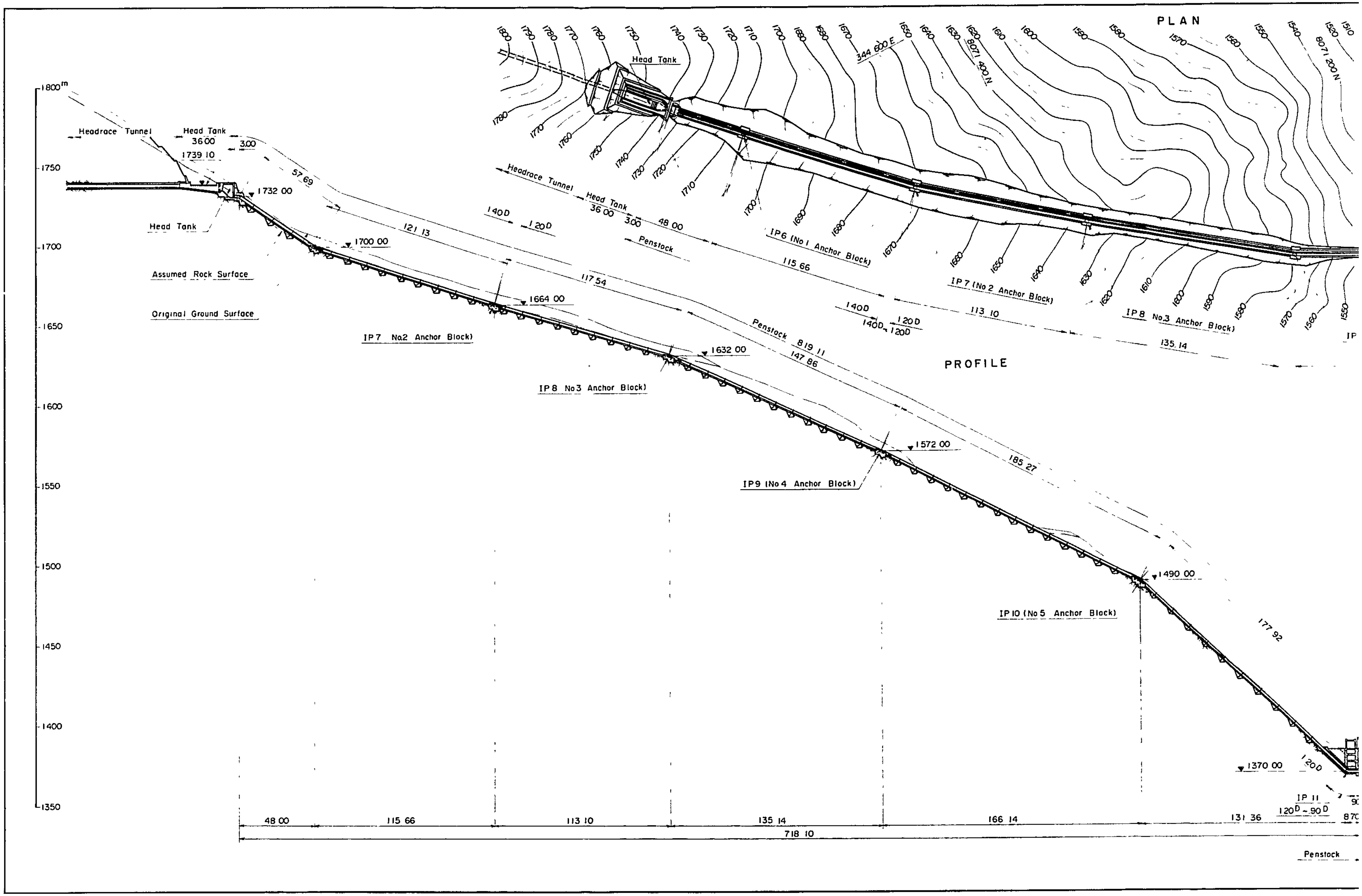
Fig. 8-6 Dec. 1983

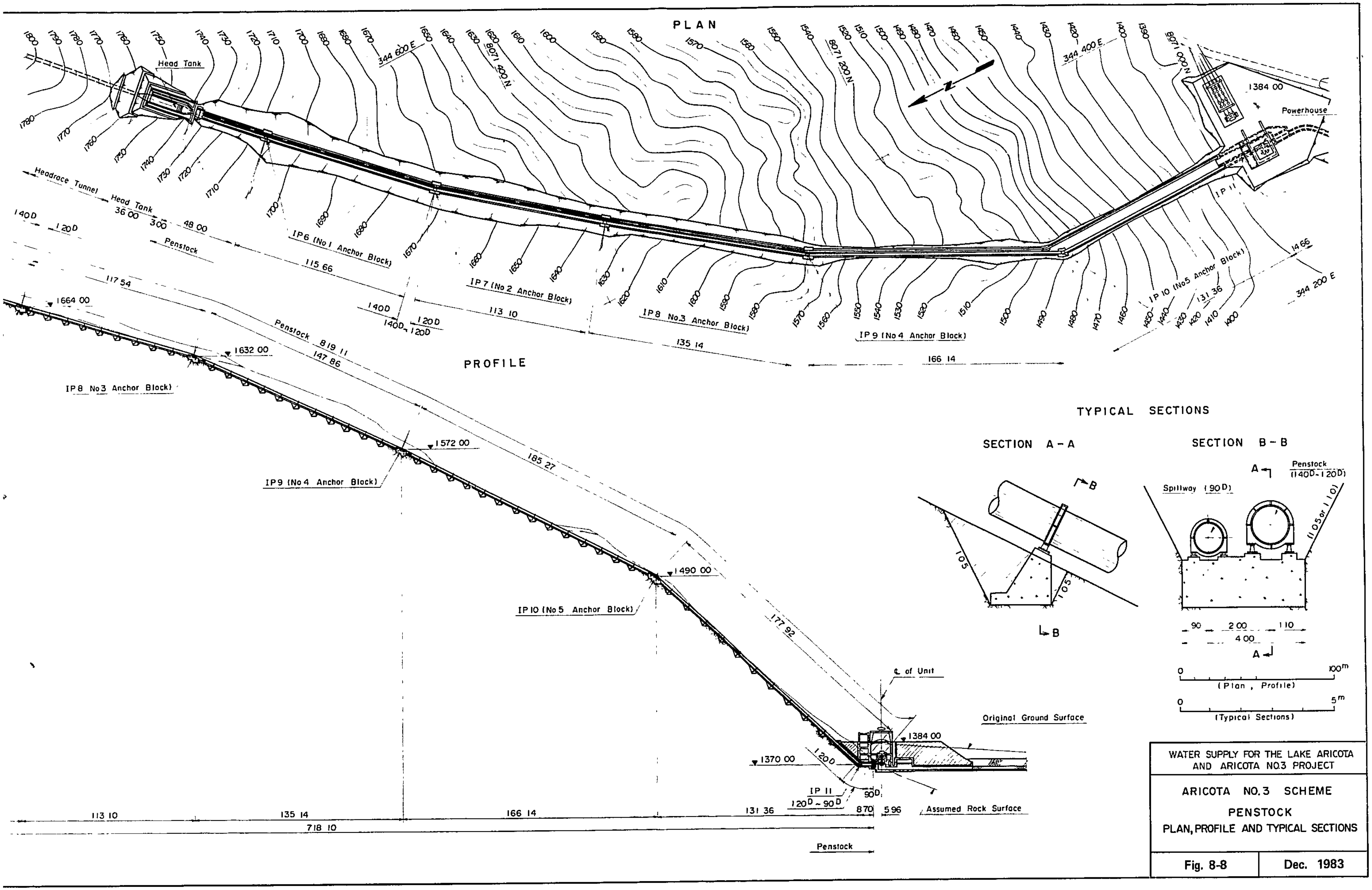


WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT

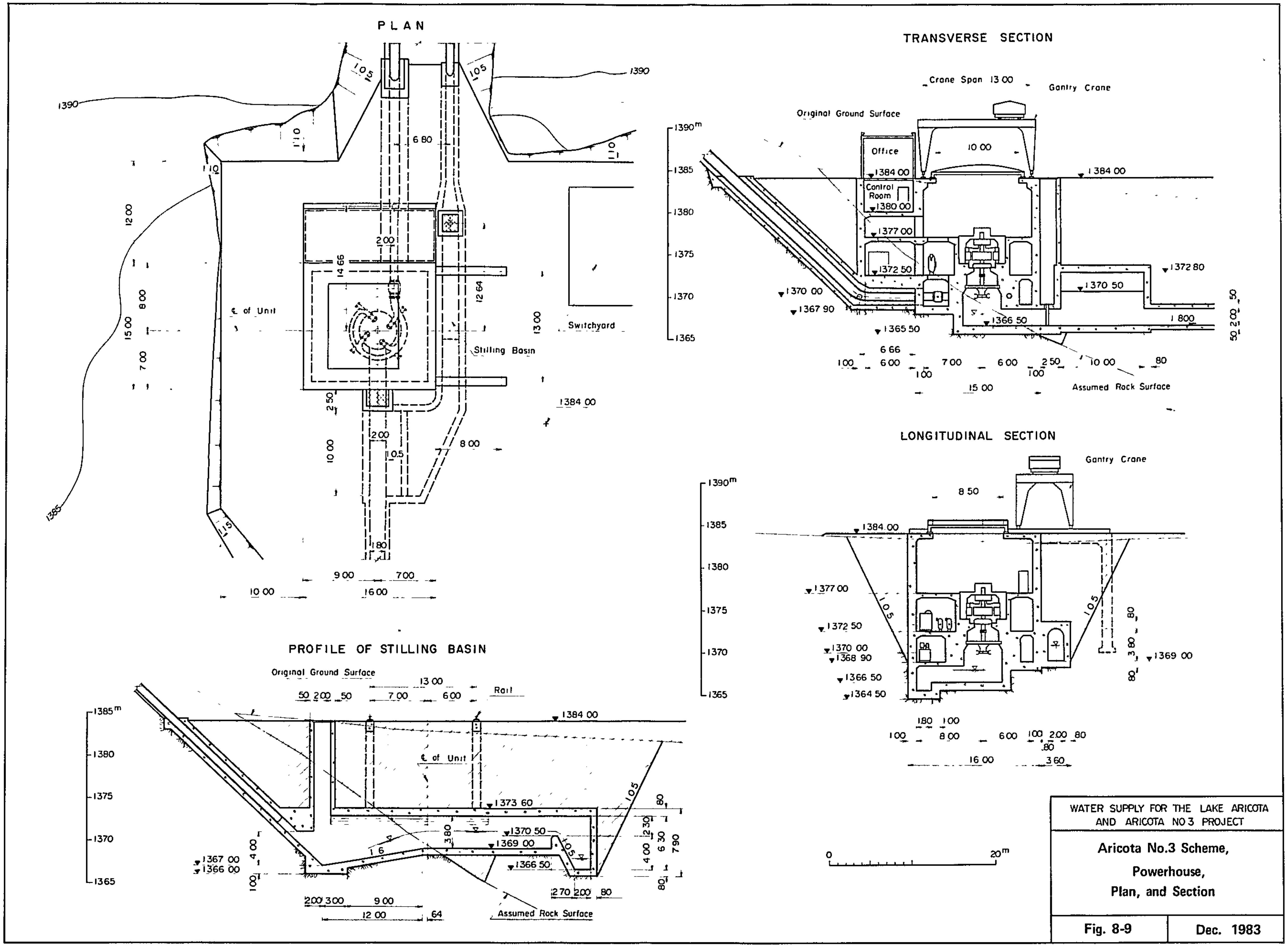
Aricota No.3 Scheme, Head Tank, Plan, Profile and Sections

Fig. 8-7	Dec. 1983


PLAN
PROFILE
Headrace Tunnel
Head Tank
36 00
3.00
1739 10
57 69
1732 00
Head Tank
1700 00
121 13
Assumed Rock Surface
Original Ground Surface
140D
120D
117 54
1664 00
IP 7 No.2 Anchor Block)
1632 00
IP 8 No 3 Anchor Block)
Penstock 819 11
147 86
1572 00
IP9 (No 4 Anchor Block)
185 27
1490 00
IP 10 (No 5 Anchor Block)
177 92
1370 00
120D
IP 11
120D ~ 90D
48 00
115 66
113 10
135 14
166 14
131 36
718 10
Penstock
Headrace Tunnel
Head Tank
36 00
3.00
48 00
Penstock
IP6 (No 1 Anchor Block)
115 66
IP 7 (No 2 Anchor Block)
113 10
140D
140D ~ 120D
120D
IP 8 No.3 Anchor Block)
135.14
344 600 E
8071 400 N
8071 200 N
1800
1750
1700
1650
1600
1550
1500
1450
1400
1350



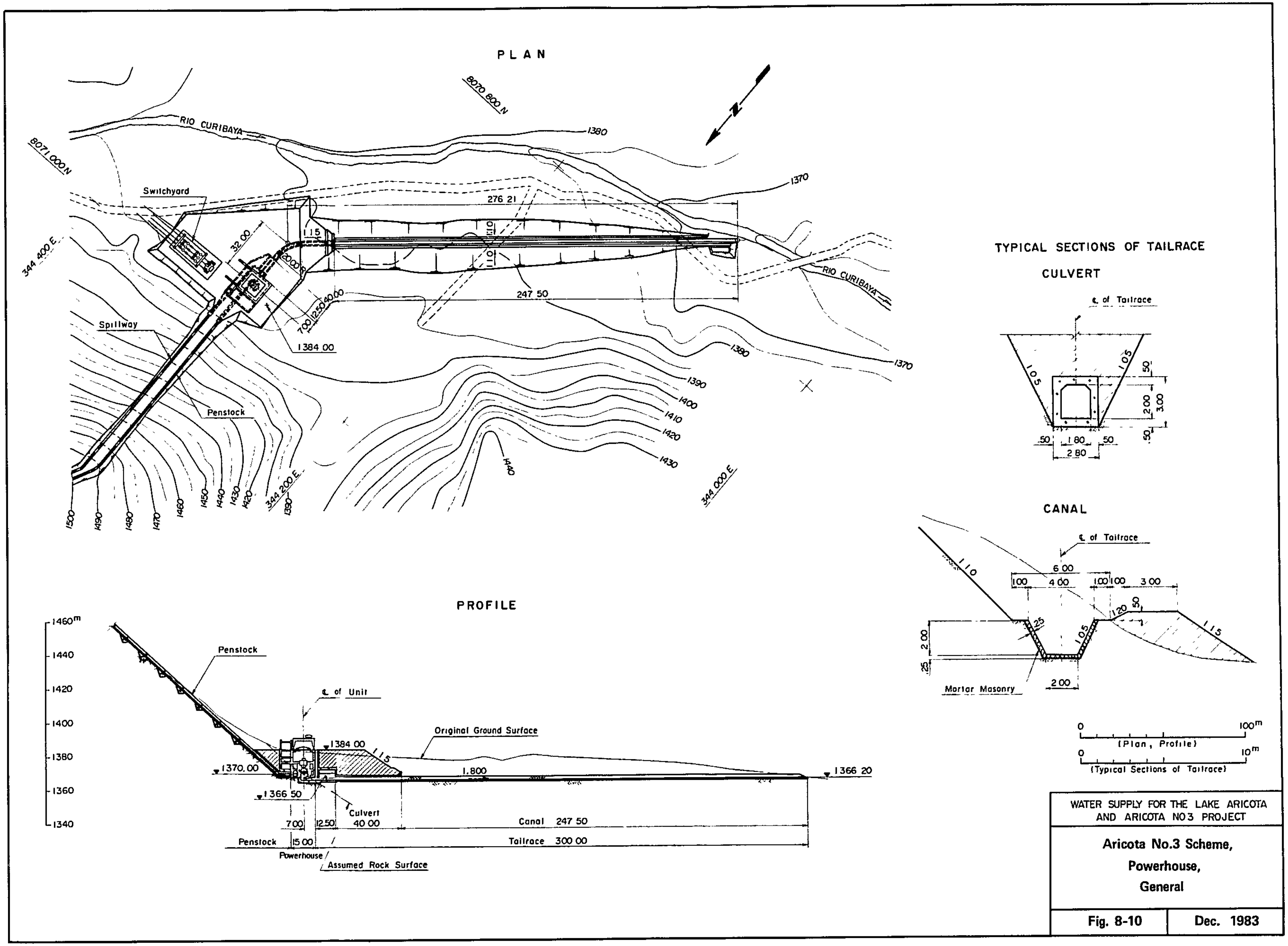
PLAN
Head Tank
Headrace Tunnel
Head Tank
36 00
3 00
48 00
140D
120D
Penstock
115 66
IP6 (No 1 Anchor Block)
IP7 (No 2 Anchor Block)
IP8 No.3 Anchor Block)
IP9 (No 4 Anchor Block)
IP 10 (No5 Anchor Block)
IP 11
Powerhouse
1384 00
344 600 E
344 400 E
344 200 E
8071 400 N
8071 200 N
8071 000 N
140D
140D - 120D
120D
113 10
135 14
166 14
131 36
14 66
117 54
PROFILE
1664 00
1632 00
1572 00
1490 00
1384 00
1370 00
Penstock 819 11
147 86
185 27
177 92
IP8 No3 Anchor Block)
IP9 (No 4 Anchor Block)
IP10 (No5 Anchor Block)
℄ of Unit
Original Ground Surface
Assumed Rock Surface
12 00
IP 11
120D ~ 90D
90D
113 10
135 14
166 14
131 36
8 70
5 96
718 10
Penstock
TYPICAL SECTIONS
SECTION A - A
SECTION B - B
Penstock (140D - 120D)
Spillway (90D)
(1 05 or 1 10)
1 05
90
2 00
1 10
4 00
0
100m
(Plan, Profile)
0
5m
(Typical Sections)
WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT
ARICOTA NO.3 SCHEME
PENSTOCK
PLAN, PROFILE AND TYPICAL SECTIONS
Fig. 8-8
Dec. 1983



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO.3 PROJECT

Aricota No.3 Scheme, Powerhouse, Plan, and Section

Fig. 8-9	Dec. 1983



WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO3 PROJECT

Aricota No.3 Scheme, Powerhouse, General

Fig. 8-10	Dec. 1983

8.2.4 ELECTRICAL EQUIPMENT

(1) Aricota No. 3 Power Station

a) Main Equipment

Aricota No. 3 Power Station will have an effective head of 357.0 m and maximum discharge of 4.6 m^3/sec, and selection of the turbine type is limited to a Pelton turbine. The number of main equipment units, in view of economy, system capacity, and unit capacities at Aricota No. 1 and No. 2 Power Stations, was judged to pose no problem with respect to the effect on the power system during faulting even if it were to be only one unit, and was therefore set at one unit. As for the selection between vertical-shaft and horizontal-shaft type, a vertical-shaft machine was selected in view of economy. Table 8-2 gives a comparison between vertical shaft and horizontal shaft.

(i) A horizontal-shaft machine, compared with a vertical-shaft one, makes it easier to overhaul of the turbine and generator, but it is necessary for the turbine center elevation to be raised approximately 1.0 m, and effective head is reduced.

(ii) In case of a horizontal-shaft design, there would be a choice between an one-runner, two-nozzle Pelton turbine, and a two-runner, four-nozzle Pelton turbine, but in the case of the present scheme, a one-runner, two-nozzle will have a speed of 360 rpm based on the critical specific speed of the turbine. As a result, increase in price will occure in the case of larged size and weight of the turbine and generator. Furthermore, a two-nozzle Pelton turbine is inferior to a four-nozzle Pelton turbine in partial load characteristics. A horizontal-shaft, two-runner, four-nozzle Pelton turbine would have the same speed as a vertical-shaft four-nozzle Pelton turbine of 514 rpm, but the turbine weight will become large compared with a vertical-shaft machine, and the price will be increased.

(iii) The horizontal-shaft machine generally result in larger dimensions for the powerhouse in comparison with a vertical-shaft machine. In the case of the present scheme, a horizontal-shaft, one-runner, two-nozzle Pelton turbine and a horizontal-shaft, two-runner, four-nozzle Pelton turbine compared with a vertical-shaft machine, will respectively have powerhouse floor areas approximately 100 m^2 and 150 m^2 larger.

Table 8-2 Comparative Table of Pelton Turbine

Type of Turbine	Vertical shaft 4-nozzles pelton turbine (VP-1R4N)	Horizontal shaft 2-nozzles pelton turbine (HP-1R2N)	Horizontal shaft 4-nozzles double wheel pelton turbine (HP-2R4N)
Effective head (m)	357.0	356.0	356.0
Maximum power Discharge (m^3/sec)	4.6	4.6	4.6
Capacity of Turbine (kW)	13,900	13,900	13,900
Revolving speed (rpm)	514	360	514
Output (kVA)	15,000	15,000	15,000
Construction lost (10^3US$)			
Electrical equipment	5,750	6,180	5,833
Civil structure	804	1,144	1,019
Total	6,554	7,324	6,852

Based on the above results, it was decided that a vertical-shaft, four-nozzle Pelton turbine with output of 13,900 kW and a speed of 514 rpm is adopted.

The generator is to be a 3-phase, AC, synchronous generator with output of 15,000 kVA. Regarding the generator load factor, since the load factors of Aricota No. 1 and No. 2 Power Stations are 0.85 (lagging) and reactive power for adjusting voltage of the system will be adequate with supply from these two power stations, the generator load factor of Aricota No. 3 Power Station was made 0.9 (lagging). As voltage of the generator, 11,000 V, which is the optimum in view of the output of the generator, was adopted. One main transformer which have a capacity of 15,000 kVA, and outdoor type, 3-phase, oil-immersed transformer for step-up from the generator voltage of 11,000 V to the transmitting voltage of 138,000 V is to be installed at the outdoor switchyard.

b) Outdoor Switchyard and Control System

The switchyard is to be of an outdoor type, provided close to the power station. The transmission line is to have a voltage of 138 kV with a single circuit, and with a length of approximately 8 km, is to connect to a bus on the 138 kV side of Aricota No. 2 Power Station.

As the control system, a one-man control system with an operator resident at the power station at all times is to be adopted.

Further, the design is to be such that it will be possible for remote control from Aricota No. 2 Power Station to be done in the future.

The general specifications of the electrical equipment of Aricota No. 3 Power Station are given in Table 8-3.

(2) Expansion of Switchyard at Aricota No. 2 Power Station

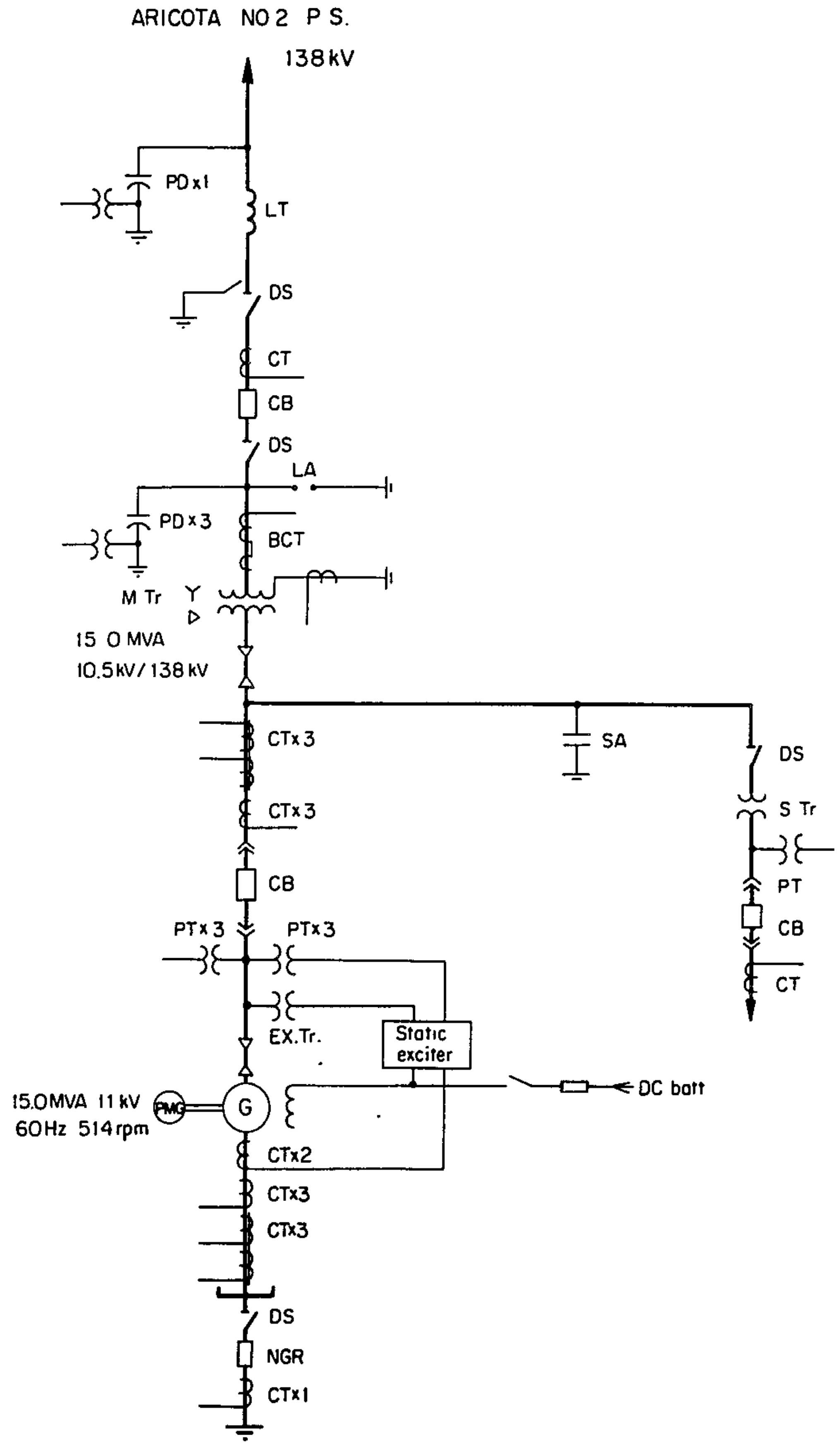
The 138 kV, 1 cct transmission line from Aricota No. 3 Power Station will connect to a 138 kV bus at Aricota No. 2 Power Station. Since the outdoor switchyard at Aricota No. 2 Power Station has had space for expansion of 138 kV and 1 cct secured from before, a 138 kV switchgear of one circuit is to be installed in this space.

Further, since the electric power of Aricota No. 3 Power Station is to be connected to a 138 kV bus, expansion of a 10.5 kV/66 kV/138 kV interconnecting transformer as installed at Aricota No. 2 Power Station will not be considered.

The single line diagram of Aricota No. 3 Power Station is shown in Fig. 8-11, and the single line diagram (expansion) of Aricota No. 2 Power Station in Fig. 8-12.

Table 8-3 Dimension of Electrical Equipment of Aricota No.3 Power Station

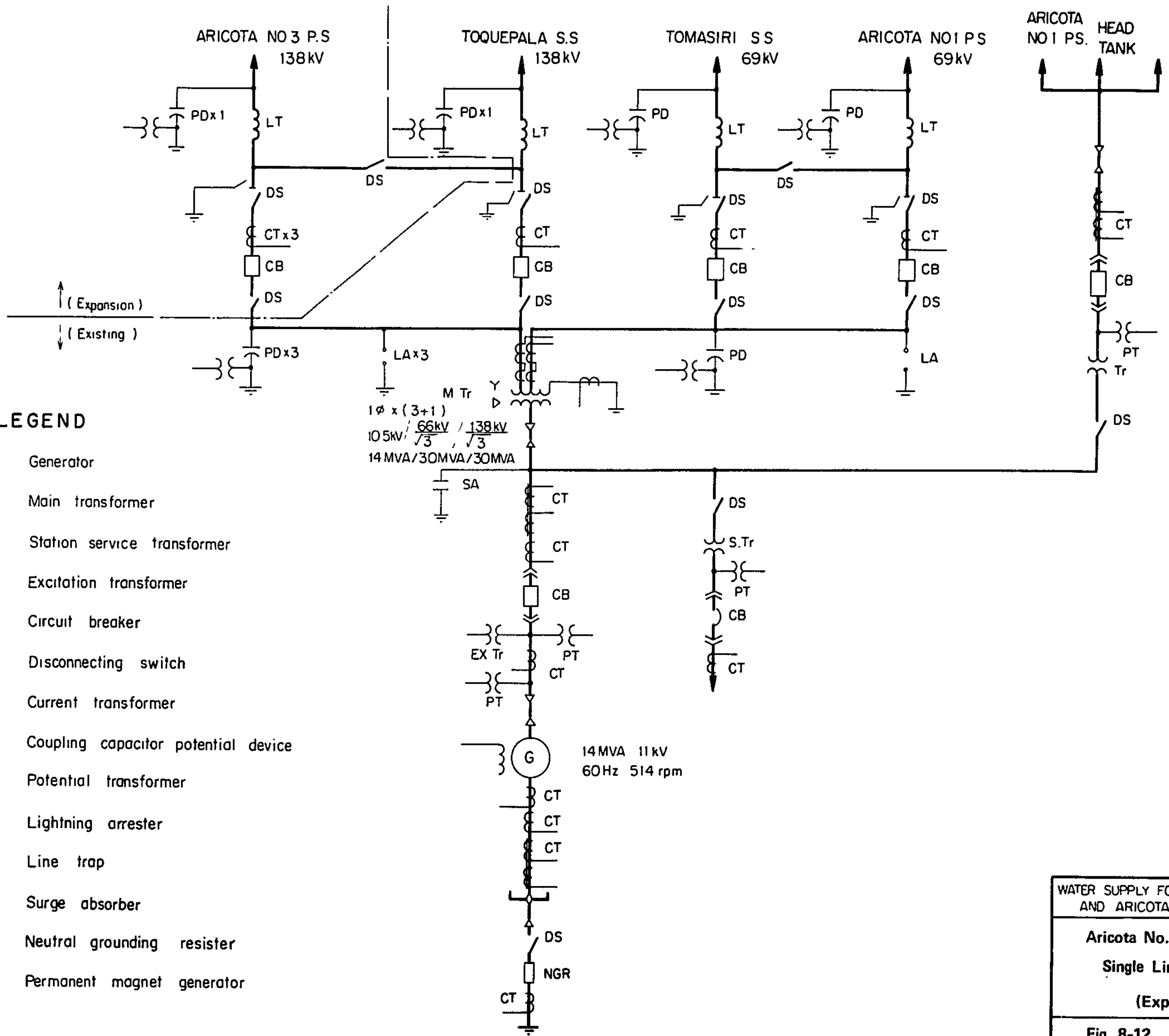
Installed Capacity	13,400 kW
Turbine	
Type	Vertical shaft 4-nozzles, pelton turbine
Number of Unit	1
Effective Head	357.0 m
Maximum Power Discharge	4.6 m^3/sec.
Revolving Speed	514 r.p.m.
Generator	
Type	3-phase, alternating current synchronous generator
Number of Unit	1
Output	15,000 kVA
Voltage	11,000 V
Power Factor	0.9 (lagging)
Frequency	60 Hz
Main Transformer	
Type	Outdoor, 3-phase, Oil immersed Transformer
Number of Unit	1
Capacity	15,000 kVA
Voltage	10.5 kV/138 kV
Number of Circuit of Transmission Line	1 CCT



LEGEND

G : Generator
M Tr : Main transformer
S Tr : Station service transformer
E Tr : Excitation transformer
CB : Circuit breaker
DS : Disconnecting switch
CT : Current transformer
PD : Coupling capacitor potential device
PT : Potential transformer
LA : Lightning arrester
LT : Line trap
SA : Surge absorber
NGR : Neutral grounding resister
PMG : Permanent magnet generator

WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT	
Aricota No.3 Power Station **Single Line Diagram**	
Fig. 8-11	Dec 1983



LEGEND

G	Generator
M Tr	Main transformer
S Tr.	Station service transformer
E.Tr.	Excitation transformer
CB	Circuit breaker
DS	Disconnecting switch
CT	Current transformer
PD	Coupling capacitor potential device
PT	Potential transformer
LA	Lightning arrester
LT	Line trap
SA	Surge absorber
NGR	Neutral grounding resister
PMG	Permanent magnet generator

WATER SUPPLY FOR THE LAKE ARICOTA AND ARICOTA NO 3 PROJECT

Aricota No.2 Power Station

Single Line Diagram

(Expansion)

Fig. 8-12 Dec. 1983

8.2.5 POWER TRANSMISSION AND TELECOMMUNICATION FACILITIES

(1) Transmission Line

In order to connect the energy produced at Aricota No. 3 Power Station with the SPCC-Aricota Power System in the south-western region of Peru, a 138 kV, 1 cct transmission line of 8 km in length was planned between the Aricota No. 3 and Aricota No. 2 power stations. Details are given in Chapter 9, "Power Transmission Plan and Power System Analysis."

(2) Telecommunication Facilities

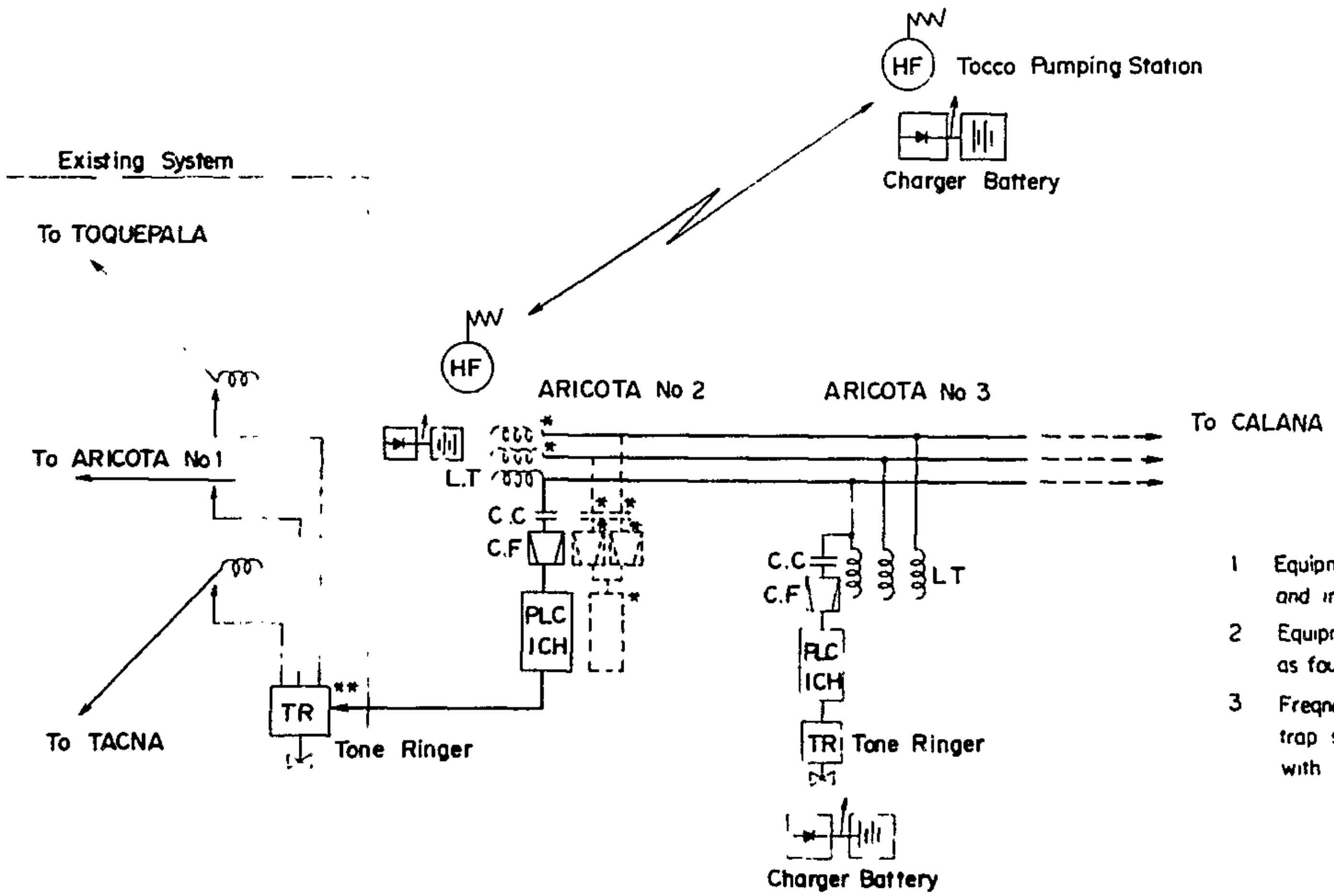
A party line telephone circuit telephone is structured connecting the eight stations including Aricota No. 1 Power Station, Aricota No. 2 Power Station, Toquepala Substation, Toquepala Switching station, Ilo Substation, and Tacna Substation, and Aricota No. 3 Power Station is to be incorporated in the above line to comprise a load dispatching telephone circuit. However, since Calana Substation being separately prepared by ELECTROPERU is scheduled to be incorporated in this party-line telephone circuit, it is necessary for further detailed investigations to be made at the time of detailed design.

Power line carrier (PLC) channels would be used as the transmission line between Aricota No. 2 and No. 3 Power Stations, but since Calana Substation planned separately by ELECTROPERU is to be connected to this transmission line, it is scheduled for phase-to-phase PLC channels to be structured between Aricota No. 2 Power Station and Calana Substation. Therefore, a PLC channel of an earth-return system is to be structured between the Aricota No. 2 and No. 3 power stations for a phase not to be used in the transmission line between Aricota No. 2 Power Station and Calana Substation. This system structure is as shown in Fig. 8-13.

It will be necessary for the frequency of PLC transmission to be examined at the time of detailed design. The structure of the party-line telephone circuit is as shown in Fig. 8-14.

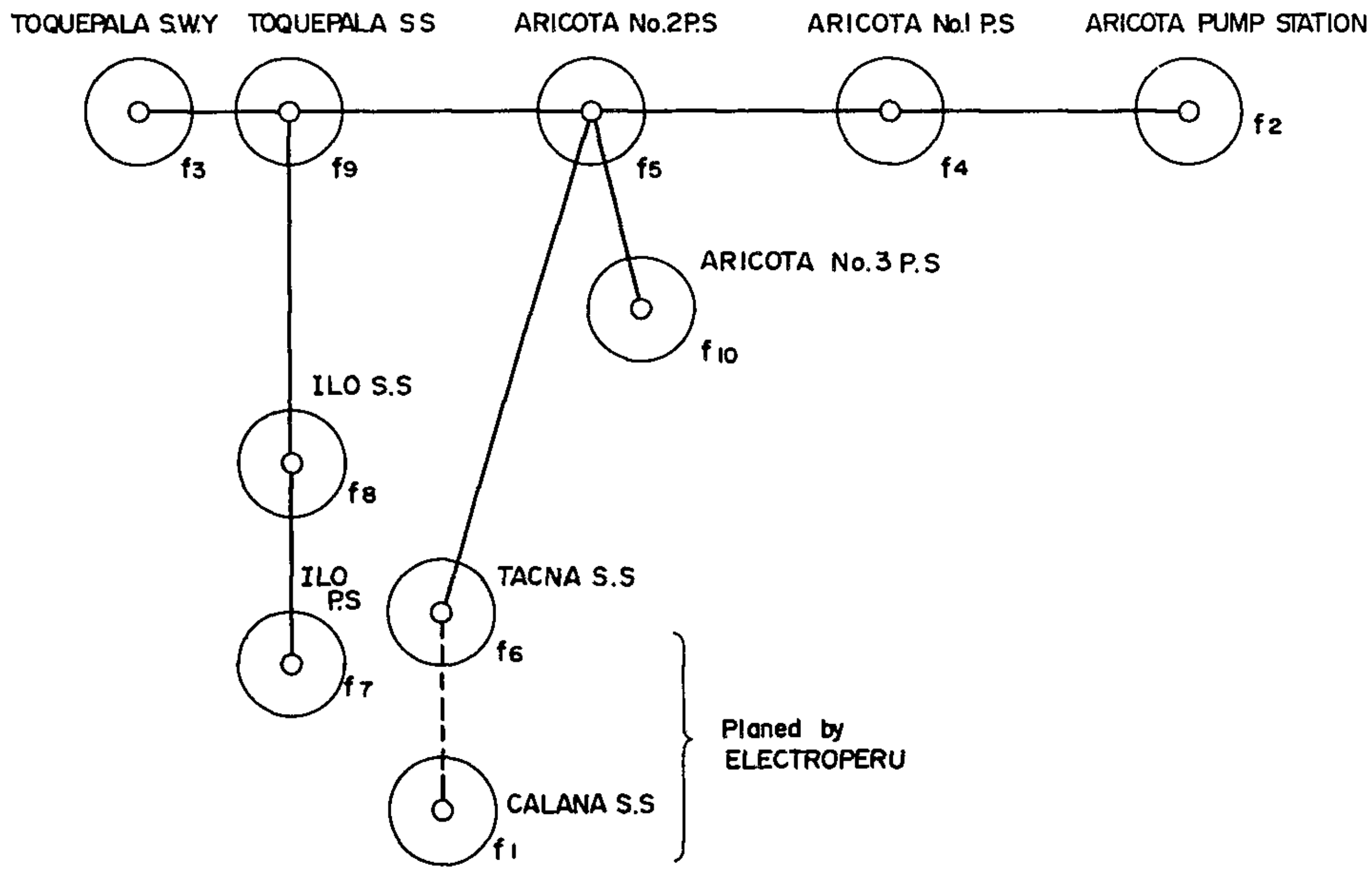
The equipment required for structuring the PLC circuit will be as shown in the following:

Fig. 8-13 Communication System for Aricota No.3 Project



1 Equipments marked (*) shall be supplied and installed by ELECTROPERU
2 Equipment marked (**) shall be replaced as four direction branching type
3 Freqnency characteristics of each line trap shall be fixed after consultation with ELECTROPERU

Fig. 8-14 Tone Ringer Network



8.3 RESERVOIR OPERATION (LAGUNA ARICOTA)

8.3.1 BASIC CONDITIONS

In order to most effectively use the inflow into Laguna Aricota after completion of the scheme, it will be necessary to examine the operation method for utilizing the reservoir functions of the lake.

Conceptually, the greatest benefit should be produced by making the total artificial intake to be approached to the total inflow as much as possible. In effect, it is a mode of operation in which evaporation from the lake surface and seepage from the lake bottom will be suppressed as much as possible within limits to meet demand. This will be more effective if operation is done at a low water level in view of the results obtained from analyses in Chapter 5, "Hydrology."

The method of operating Laguna Aricota for evaluation of this Project was established in view of the above points.

(1) Scope of Reservoir Operation Water Level

The time-dependent variation in lake water level up to the present had shown a trend only of decline because of intake of water supply scheme excess of the inflow. However, after completion of the water supply scheme, the water level will fluctuate up and down within the scope of aiming at stabilization of supply. The storage capacity required at that time will be the accumulation of the difference between the intake quantity required from the aspect of demand and the actual inflow.

Fig. 8-15 is the mass curve of the inflow of the 15-year period from 1966 to 1980 as one cycle. Using this figure it is possible to find the relation between the dependable intake during the 15-year period and the necessary storage volume at that time.

Fig. 8-16 shows the relation between dependable runoff and storage capacity in case the low water level is put at EL. 2,740 m. Based on these figures, the capacity at which a constant and maximum intake can be secured during the 15 years irrespective of wet or dry years will be approximately 200 x 10^6 m^3. To set the capacity above this only serves to raise the average water level. As a result by increasing the guantity of evaporation and seepage, the effective intake is to be decreased.

Accordingly, the range of operation to be used in evaluating the plans for Laguna Aricota is taken to be that of low water level at 2,740 m and high water level at 2,774 m corresponding to the effective capacity of 200 x 10^6 m^3.

(2) Intake Method

Intake from Laguna Aricota is at present being done by pump-up. Until this Project is completed the lake water level will recede successively so new intake tunnels are being planned or constructed.

However, as previously stated, the water level of this lake will of necessarily fluctuate up and down. Therefore, it is desirable for facilities such as intake tunnels to be expanded for the interim period to be held to as small a number as possible.

In view of the above, the reference elevation of the intake was set at 2,760 m in calculating operation of the scheme. It was decided that the existing pump-up facilities would be utilized in the range of 2,760 to 2,740 m below the reference elevation. For intake above 2,760 m, an intake tower would be provided and direct intake carried out.

Fig. 8-15 Mass Curve of Lake Aricota (1966 ~ 1980)

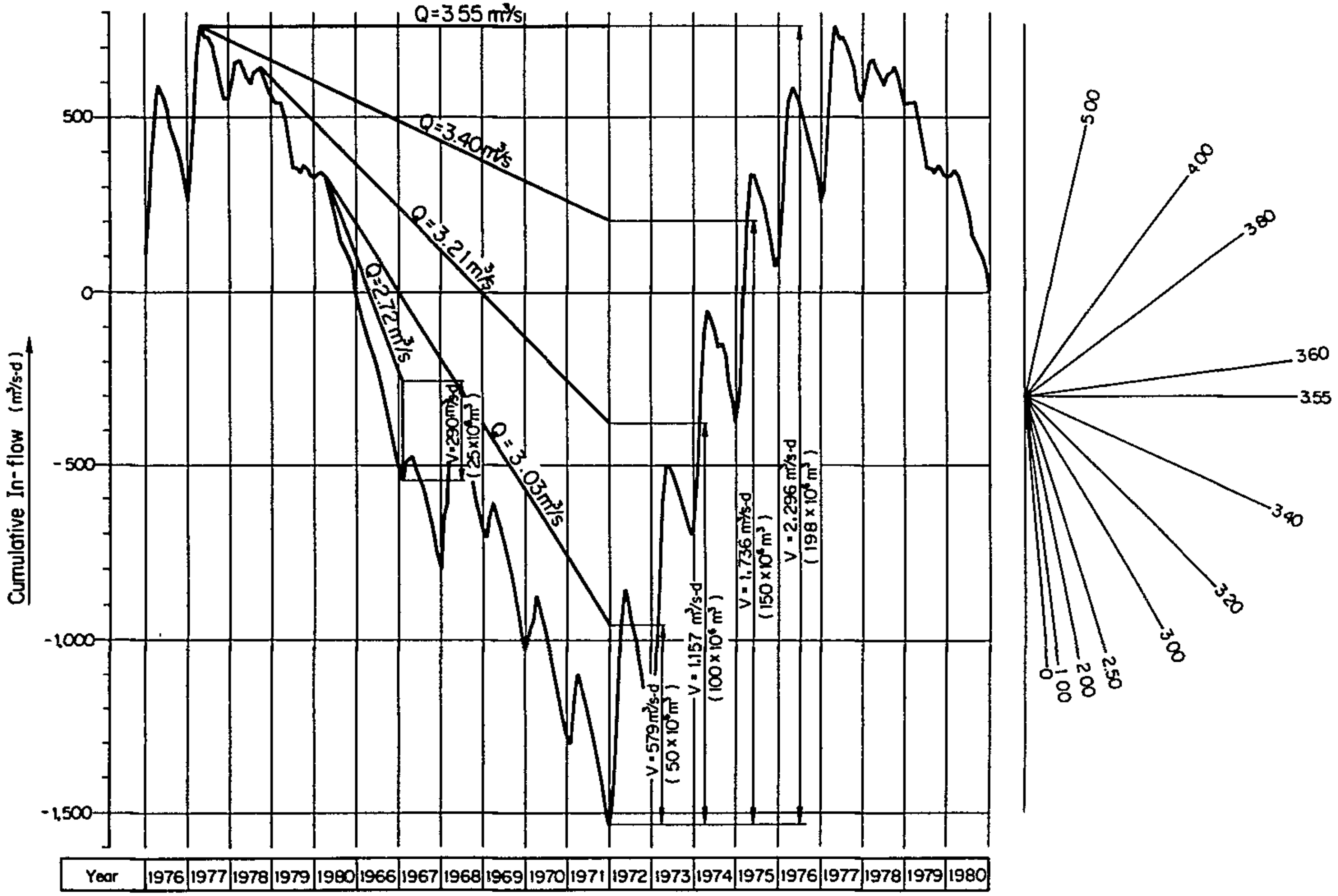
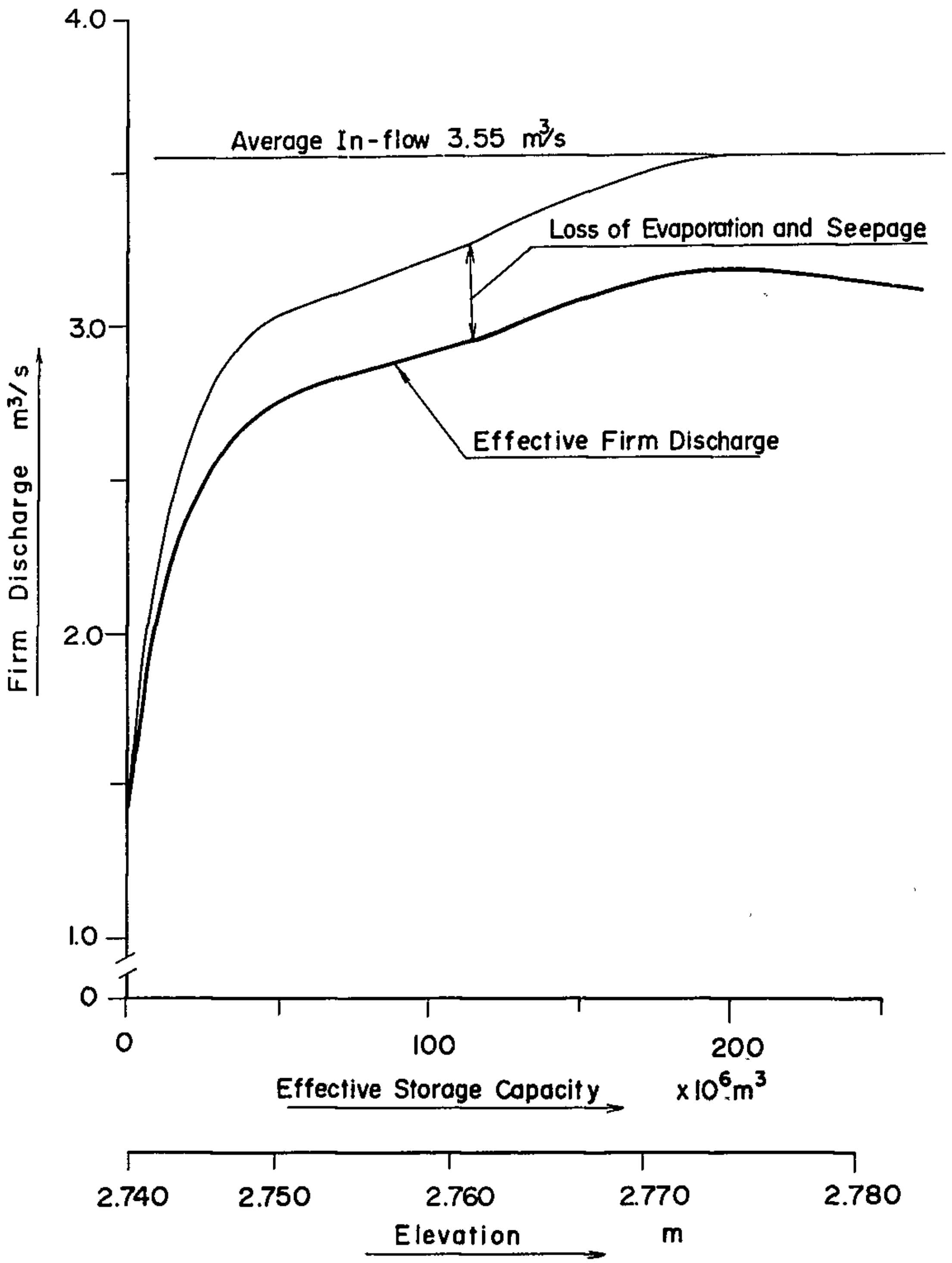


Fig. 8-16 Relation between Firm Discharge and Effective Storage Capacity



8.3.2 CALCULATIONS

The greatest of the benefits to be derived from this Project is that from electric power. Therefore, it was decided that meeting power demand would be given priority in operating Laguna Aricota.

According to the monthly energy demands of the Aricota System for the period from 1978 to 1982, the range of variation from the average monthly demand during a year was roughly ±10%. Months with greater variations amounted to approximately 7% for the entire period, with the minimum value -20% and the maximum +18% (see Fig. 8-17). Based on this, it was considered that the monthly average intake would not vary too much in comparison with the annual, and as the normal discharge, a constant figure was adopted for the whole year.

However, since in reality prediction of inflow is extremely difficult, it is necessary to impose restrictions on intake for a storage volume at a certain time. In these calculations, the period of intake restriction was taken to be within 2% and the available intake at that time not less than the own flow of Aricota were used as the conditions, and simulations were made.

The intake quantities according to water level are shown below.

EL. 2,740 - 2,745 m	2.0 m^3/sec
2,745 - 2,765 m	3.0 m^3/sec
2,765 - 2,770 m	4.0 m^3/sec

The results of reservoir operation calculations for Laguna Aricota made for the above conditions are shown in Fig. 8-18.

According to the results of calculations it will be possible for intake of 3.0 m^3/sec or more to be done 98.3% of the period, while intake quantities at high water level would be comparatively stable.

Fig. 8-17 Rate of Fluctuation of Monthly Demand in Aricota Power System

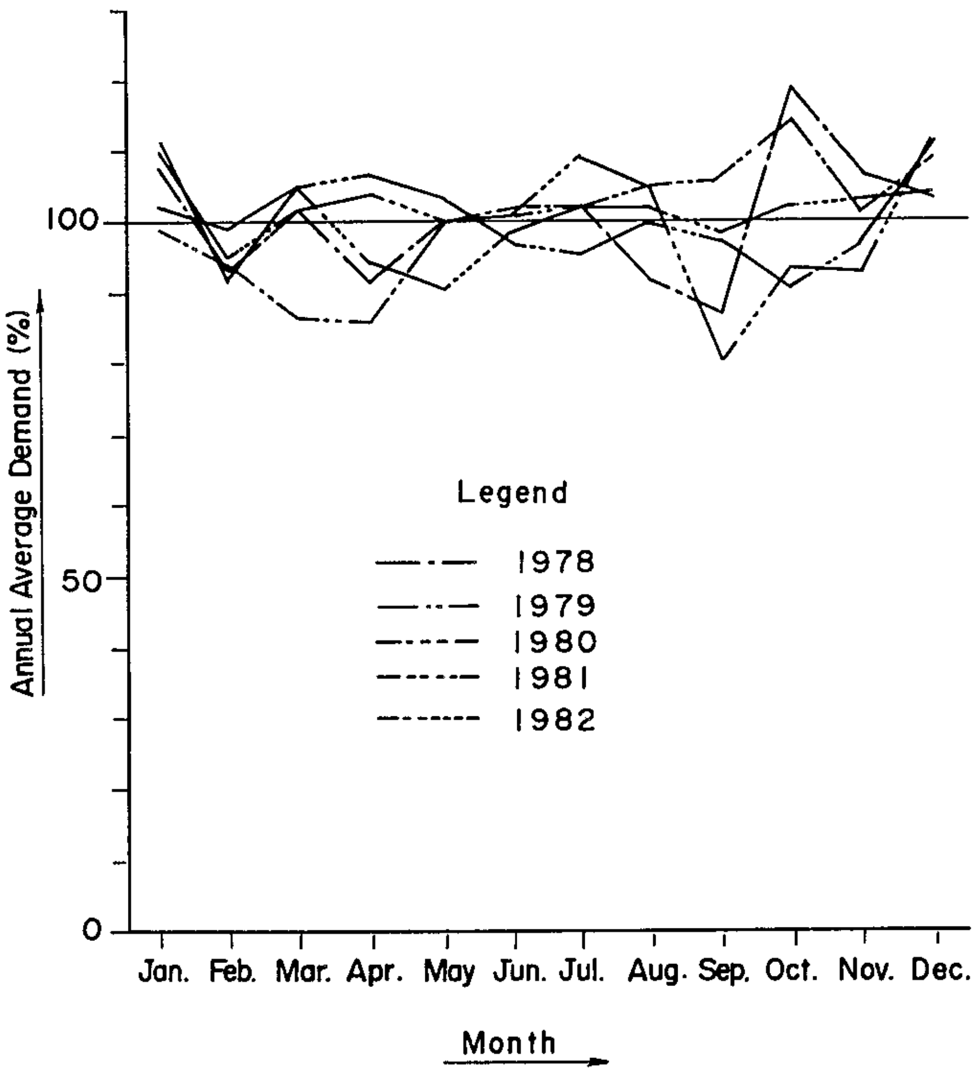
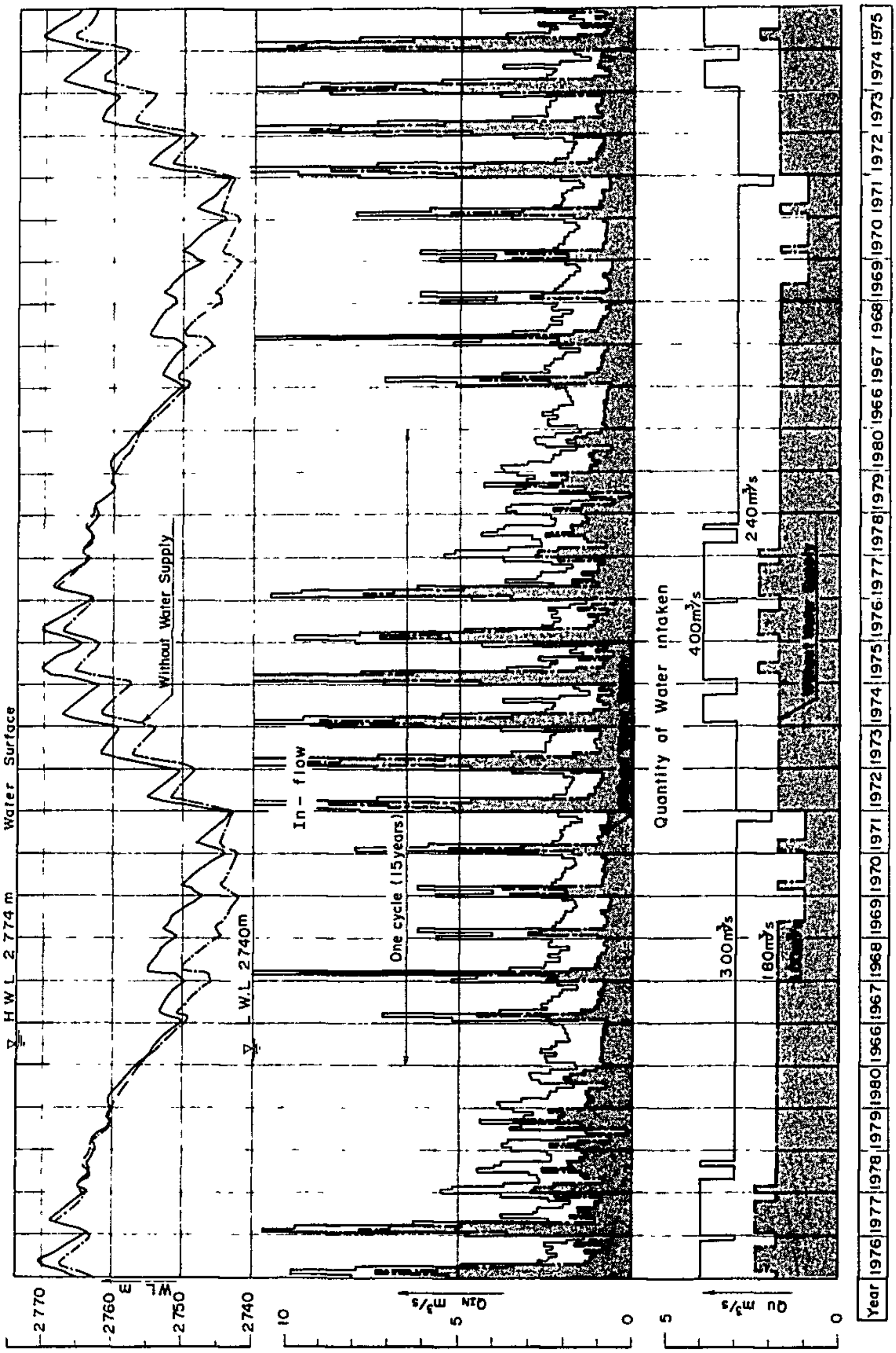


Fig. 8-18 Reservoir Operation of Lake Aricota



8.4 AVAILABLE ENERGY PRODUCTION

The electric power benefit of this scheme will be from the increased production at the existing Aricota No. 1 and No. 2 Power Stations due to water supply and the energy production at Aricota No. 3 Power Station to be newly constructed.

Regarding the former, the difference between the energy production from the reservoir operation and the pump-up energy requirement in case this scheme did not exist and these values in case the scheme were to be carried out, that is the incremental electric energy, was taken as the benefit. For the latter, the electric energy in case of carrying out water supply scheme is carried out was taken into account.

The pump-up energy requirements in Aricota Pumping Station and Tocco Pumping Station were based on the end of transmission and the transmission loss was set at 5%.

It was assumed that the irrigation discharge between Laguna Aricota and the outlet of the No. 3 power station would all be consumed in the section, and return flow water was not taken into account in calculations.

The values are given below.

Section	Discharge
Pumping Station - No. 1 PS	0.14 m^3/sec
No. 1 PS - No. 2 PS	0.15 m^3/sec
No. 2 PS - No. 3 PS	0.20 m^3/sec
Total	0.49 m^3/sec

The results of calculations of electric energy in the various power stations and pumping stations are given in Tables 8-5 to 8-10.

The available energy production quantities used for benefit calculations are as shown in Table 8-4.

Table 8-4 Available Energy Production

Unit: GWh

Station	With Project	Without Project	Net Value
Energy Production			
No. 1 & No. 2	204.97	104.63	100.34
No. 3	70.82	-	70.82
Sub-total	275.79	104.63	171.16
Pumping Energy			
Aricota	1.59	1.11	0.48
Tocco	15.67	-	15.67
Subtotal	17.26	1.11	16.15
Total	258.53	103.52	155.01

Table 8-5 Energy Production of Aricota No.1 & No.2 P/S

(with Project)
Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
67	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
68	15.93	14.91	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	188.00
69	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
70	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
71	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	10.25	9.93	10.25	170.67
72	15.93	14.91	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	188.00
73	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
74	15.93	19.52	21.61	20.93	21.61	20.93	21.61	21.61	20.93	15.93	15.40	15.93	231.94
75	15.93	19.52	21.61	20.93	21.61	20.93	21.61	21.61	20.93	21.61	20.93	21.61	248.83
76	21.61	20.23	21.61	20.93	21.61	20.93	21.61	21.61	20.93	21.61	20.93	15.93	249.54
77	21.61	19.52	21.61	20.93	21.61	20.93	21.61	21.61	20.93	21.61	20.93	21.61	254.51
78	21.61	19.52	21.61	20.93	15.93	15.40	15.93	15.93	20.93	15.93	15.40	15.93	215.18
79	15.93	14.39	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	187.48
1980	15.93	14.91	15.93	15.40	15.93	15.40	15.93	15.93	15.40	15.93	15.40	15.93	188.00
Average	17.07	16.25	17.82	17.24	17.44	16.88	17.44	17.44	17.24	16.69	16.15	16.31	204.97

Table 8-6 Energy Production of Aricota No.3 P/S

(with Project)
Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
67	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
68	5.47	5.12	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.61
69	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
70	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
71	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	3.29	3.19	3.29	57.96
72	5.47	5.12	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.61
73	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
74	5.47	6.93	7.67	7.45	7.67	7.43	7.67	7.67	7.43	5.47	5.30	5.47	81.65
75	5.47	6.93	7.69	7.45	7.70	7.45	7.70	7.70	7.43	7.67	7.43	7.67	88.31
76	7.67	7.20	7.70	7.45	7.70	7.45	7.70	7.67	7.43	7.67	7.43	5.49	88.56
77	7.67	6.93	7.69	7.45	7.70	7.45	7.70	7.67	7.43	7.67	7.43	7.67	90.46
78	7.67	6.93	7.67	7.43	5.47	5.30	5.47	5.47	7.43	5.47	5.30	5.47	75.08
79	5.47	4.94	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
80	5.47	5.12	5.47	5.30	5.47	5.30	5.47	5.47	5.30	5.47	5.30	5.47	64.43
Average	5.91	5.66	6.21	6.02	6.06	5.87	6.06	6.06	6.01	5.76	5.59	5.62	70.82

Table 8-7 Pumping Energy of Aricota Pumping Station

(with Project)
Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	0.13	0.12	0.15	0.16	0.17	0.17	0.19	0.21	0.22	0.24	0.25	0.28	2.29
67	0.29	0.24	0.21	0.19	0.21	0.21	0.23	0.24	0.25	0.28	0.29	0.31	2.95
68	0.29	0.25	0.15	0.15	0.16	0.16	0.19	0.20	0.21	0.23	0.24	0.26	2.49
69	0.27	0.22	0.28	0.22	0.24	0.25	0.28	0.29	0.30	0.34	0.35	0.39	3.37
70	0.35	0.31	0.29	0.30	0.33	0.33	0.36	0.38	0.39	0.43	0.45	0.48	4.40
71	0.47	0.36	0.36	0.36	0.39	0.39	0.42	0.45	0.45	0.31	0.31	0.32	4.59
72	0.42	0.30	0.20	0.14	0.16	0.18	0.20	0.22	0.24	0.27	0.28	0.28	2.89
73	0.22	0.10	0	0	0	0	0	0	0	0.01	0.03	0.02	0.38
74	0	0	0	0	0	0	0	0	0	0	0	0	0
75	0	0	0	0	0	0	0	0	0	0	0	0	0
76	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	0	0	0	0	0	0
78	0	0	0	0	0	0	0	0	0	0	0	0	0
79	0	0	0	0	0	0	0	0	0	0	0	0	0
1980	0	0	0	0	0.01	0.02	0.05	0.06	0.06	0.07	0.09	0.11	0.47
Average	0.16	0.13	0.11	0.10	0.11	0.11	0.13	0.14	0.14	0.15	0.15	0.16	1.59

Table 8-8 Pumping Energy of Tocco Pumping Station

(with Project)
Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	0.70	1.24	1.39	1.10	1.55	1.19	1.15	1.12	0.77	0.94	0.98	1.12	13.25
67	1.38	2.17	2.40	2.02	1.20	1.36	1.25	1.28	0.99	1.02	0.66	1.23	16.96
68	2.40	1.86	1.79	1.50	1.32	1.31	0.99	1.33	1.16	0.97	1.40	1.37	17.40
69	1.37	2.07	2.02	1.16	1.55	1.19	1.08	1.08	1.05	0.84	0.84	1.00	15.25
70	1.88	1.66	2.16	1.23	1.21	1.05	1.05	0.95	0.78	0.79	0.76	1.00	14.52
71	1.65	2.17	2.40	1.17	1.00	0.91	1.06	0.98	0.71	0.65	0.76	1.29	14.75
72	2.40	2.25	2.40	2.29	0.84	0.68	0.70	0.67	0.65	0.66	0.95	1.49	15.98
73	2.40	2.17	2.40	1.60	1.55	1.36	1.31	1.22	1.22	1.23	1.16	1.15	18.77
74	2.40	2.17	2.40	1.75	1.23	1.43	1.21	2.40	0.96	0.55	0.50	0.77	17.77
75	2.40	2.17	2.40	1.26	1.71	1.31	1.25	1.00	0.75	0.61	0.75	1.35	16.96
76	2.40	2.07	2.07	1.16	1.18	0.98	0.89	0.92	1.23	0.63	0.52	0.71	14.76
77	0.97	2.16	2.40	1.15	1.22	1.67	1.41	1.03	0.81	0.65	1.32	1.38	16.17
78	2.40	1.90	1.25	1.63	1.20	1.26	1.22	1.00	0.78	0.67	0.86	0.91	15.08
79	1.71	1.07	1.80	1.05	1.17	1.04	1.15	1.51	0.83	0.89	0.79	0.89	13.90
1980	2.18	2.11	1.74	1.00	0.95	0.82	0.84	0.83	0.76	0.99	0.76	0.54	13.52
Average	1.91	1.95	2.07	1.40	1.26	1.17	1.10	1.15	0.90	0.81	0.87	1.08	15.67

Table 8-9 Energy Production of Aricota No.1 & No.2 P/S

(without Project)

Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	9.12	8.24	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.40
67	9.12	8.24	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.40
68	9.12	8.54	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.70
69	9.12	8.24	9.12	8.83	9.12	4.44	4.59	4.59	4.44	4.59	4.44	4.59	76.11
70	4.59	4.14	9.12	8.83	4.59	4.44	4.59	4.59	4.44	4.59	4.44	4.59	62.95
71	4.59	8.24	9.12	8.83	4.59	4.44	4.59	4.59	4.44	4.59	4.44	4.59	67.05
72	9.12	8.54	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.70
73	9.12	8.24	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.40
74	9.12	8.24	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.40
75	9.12	8.24	9.12	12.13	12.54	12.13	9.12	9.12	8.83	9.12	8.83	9.12	117.42
76	9.12	11.73	12.54	12.13	12.54	12.13	12.54	12.54	12.13	9.12	8.83	9.12	134.47
77	9.12	11.32	12.54	12.13	12.54	12.13	12.54	12.54	12.13	12.54	8.83	9.12	137.48
78	12.54	11.32	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	113,90
79	9.12	8.24	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	107.40
1980	9.12	8.54	9.12	8.83	9.12	8.83	9.12	9.12	8.83	9.12	8.83	9.12	7.70
Average	8.74	8.67	9.58	9.49	9.20	8.67	8.67	8.67	8.39	8.44	7.95	8.22	104.63

Table 8-10 Pumping Energy of Aricota Pumping Station

(without Project)
Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1966	0.08	0.08	0.10	0.11	0.12	0.12	0.13	0.14	0.15	0.16	0.17	0.18	1.51
67	0.19	0.17	0.16	0.16	0.18	0.18	0.19	0.20	0.20	0.22	0.23	0.24	2.32
68	0.24	0.22	0.17	0.17	0.18	0.19	0.20	0.21	0.21	0.23	0.24	0.26	2.52
69	0.27	0.23	0.25	0.25	0.28	0.14	0.15	0.15	0.15	0.16	0.15	0.16	2.33
70	0.15	0.13	0.26	0.27	0.15	0.14	0.15	0.15	0.15	0.16	0.15	0.16	2.01
71	0.16	0.25	0.26	0.26	0.14	0.14	0.14	0.15	0.14	0.15	0.15	0.16	2.09
72	0.27	0.21	0.16	0.14	0.15	0.15	0.17	0.17	0.18	0.19	0.20	0.21	2.20
73	0.18	0.11	0.07	0.05	0.05	0.06	0.07	0.08	0.08	0.10	0.10	0.10	1.05
74	0.07	0.02	0	0	0	0	0	0.01	0.01	0.03	0.03	0.04	0.20
75	0.02	0	0	0	0	0	0	0	0	0	0	0	0.02
76	0	0	0	0	0	0	0	0	0	0	0	0	0
77	0	0	0	0	0	0	0	0	0	0	0	0	0
78	0	0	0	0	0	0	0	0	0	0	0	0	0
79	0	0	0	0	0	0	0	0	0	0	0	0	0
1980	0	0.01	0.01	0.02	0.03	0.03	0.04	0.05	0.05	0.05	0.06	0.07	0.41
Average	0.11	0.09	0.09	0.09	0.08	0.08	0.08	0.09	0.09	0.10	0.10	0.11	1.11