INFRASTRUCTURE SURVEY REPORT

FOR THE HUANZALA MINE

IN THE REPUBLIC OF PERU

Vol. II

FEBRUARY 1984

JAPAN INTERNATIONAL COOPERATION AGENCY

TOKYO JAPAN

Vol. II CONTENTS

List of Fig. and Table

CHAPTER 1

POWER GENERATION SCHEME

CHAPTER 1 POWER GENERATION SCHEME

Photo 1-1-1
Proposed Dam Site seen from Upstream Side

Photo 1-1-2 Rapid Stream of Rio San Juan at the Upstream Edge of Huallanca

Photo 1-1-3
Rio San Juan running in Huallanca

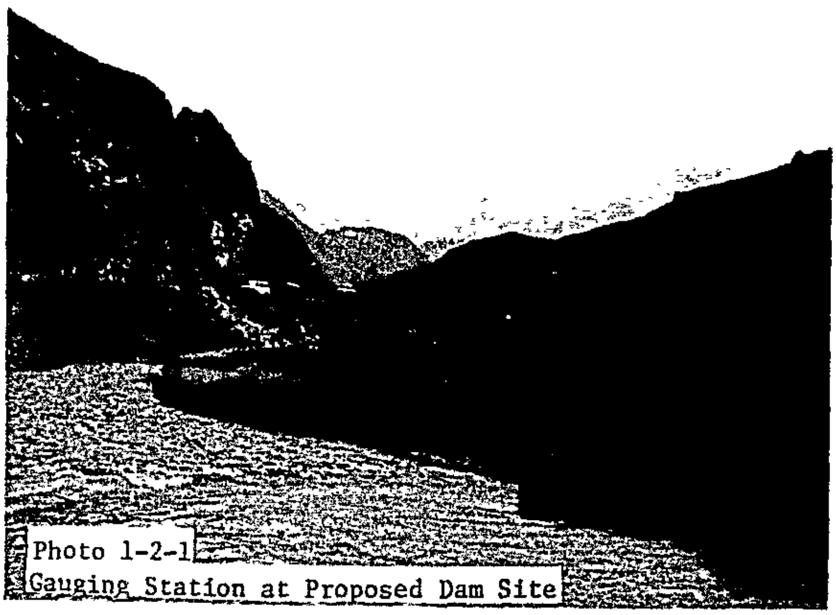


Photo 1-2-1
Gauging Station at Proposed Dam Site

Photo 1-2-2
San Juan Bridge (washed away by flood in March 1981)

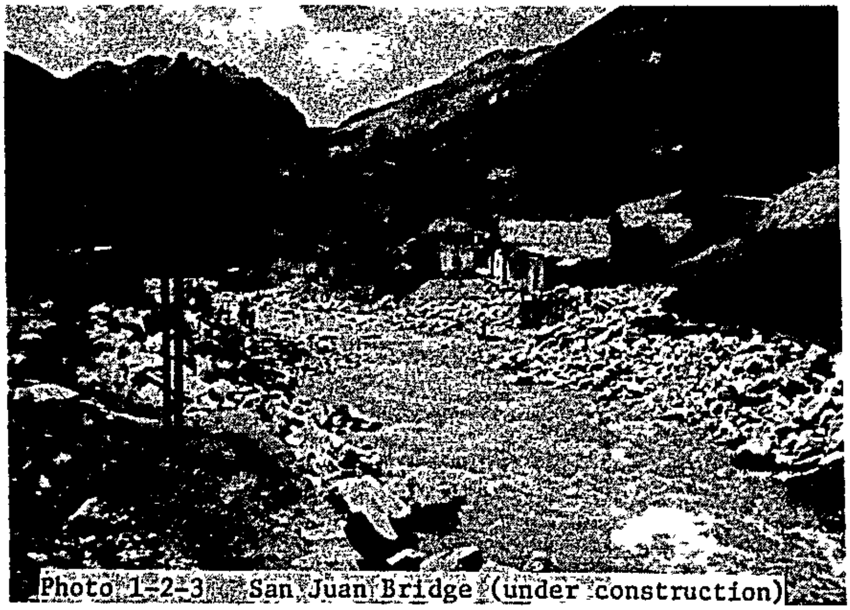


Photo 1-2-3 San Juan Bridge (under construction)

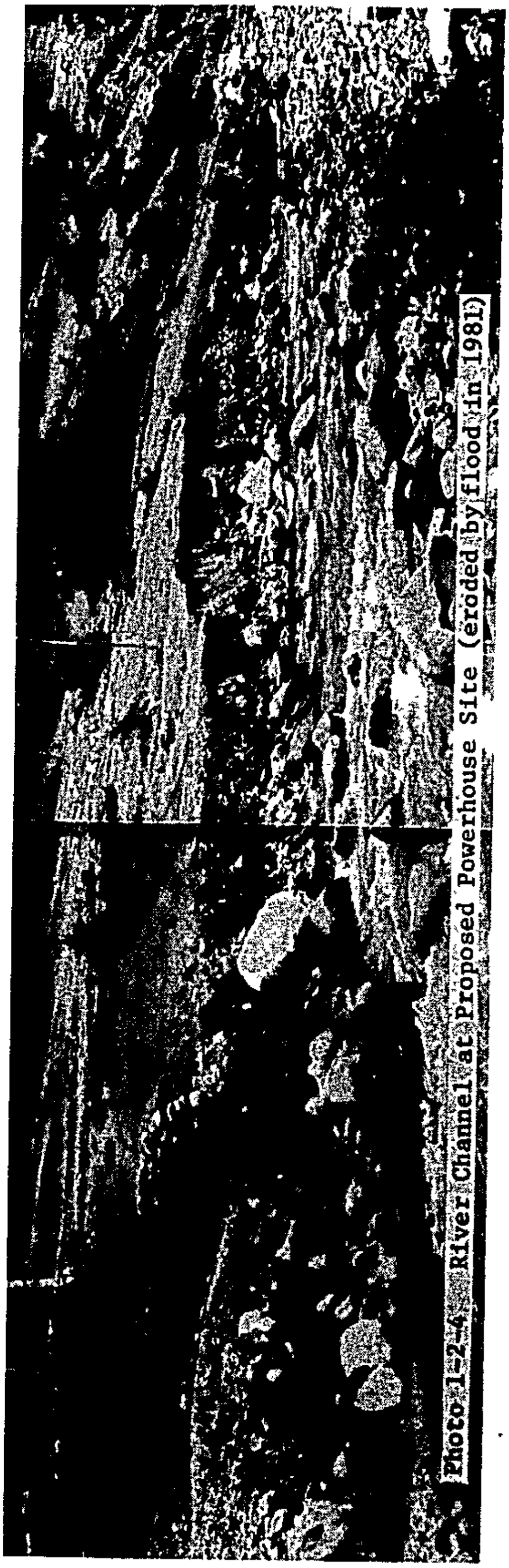


Photo 1-2-4 River Channel at Proposed Powerhouse Site (eroded by flood in 1981)

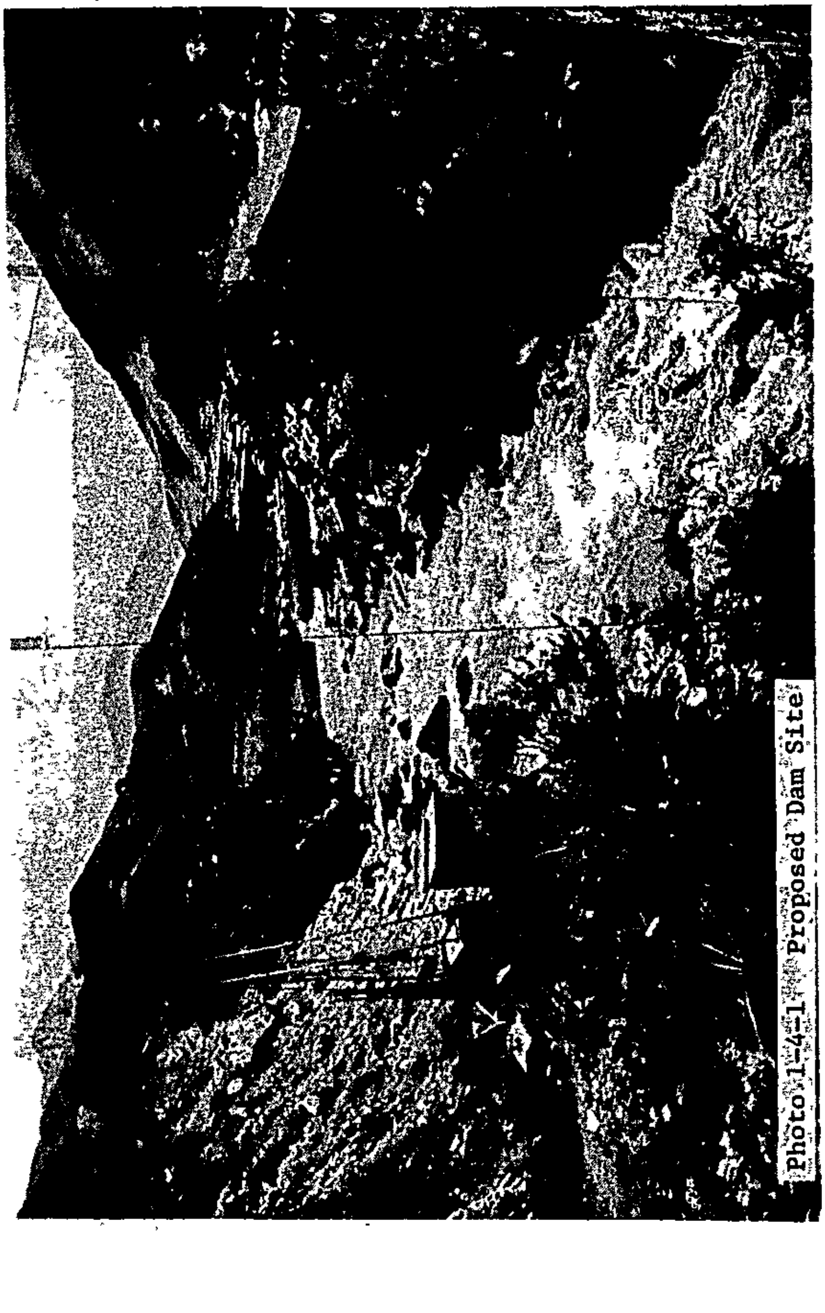


Photo 1-4-1 Proposed Dam Site

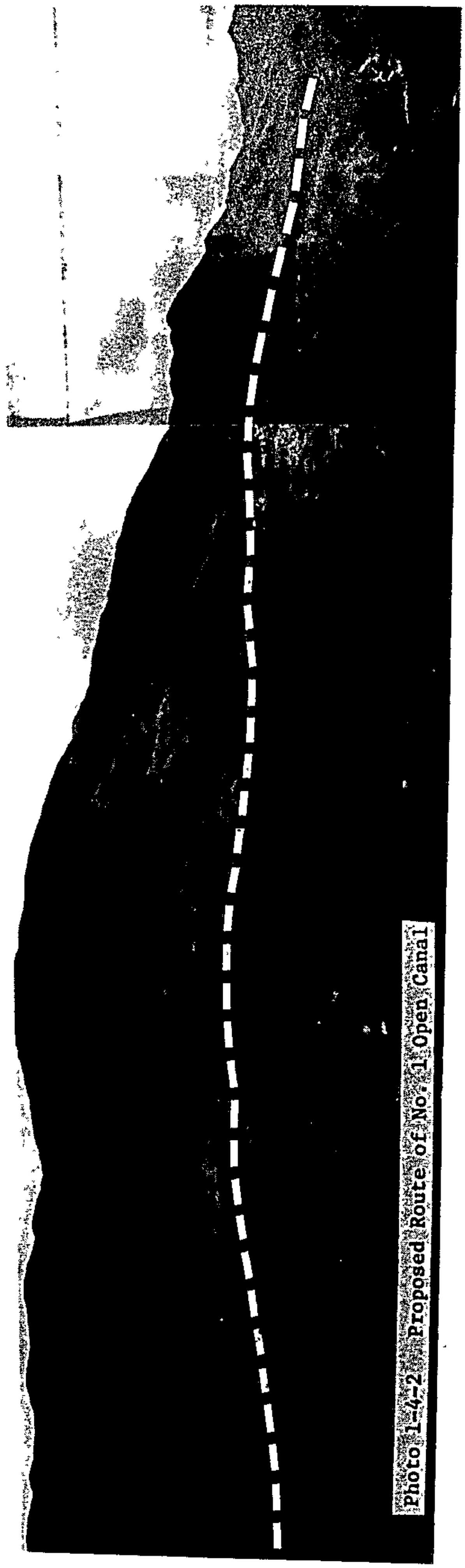


Photo 1-4-2 Proposed Route of No. 1 Open Canal

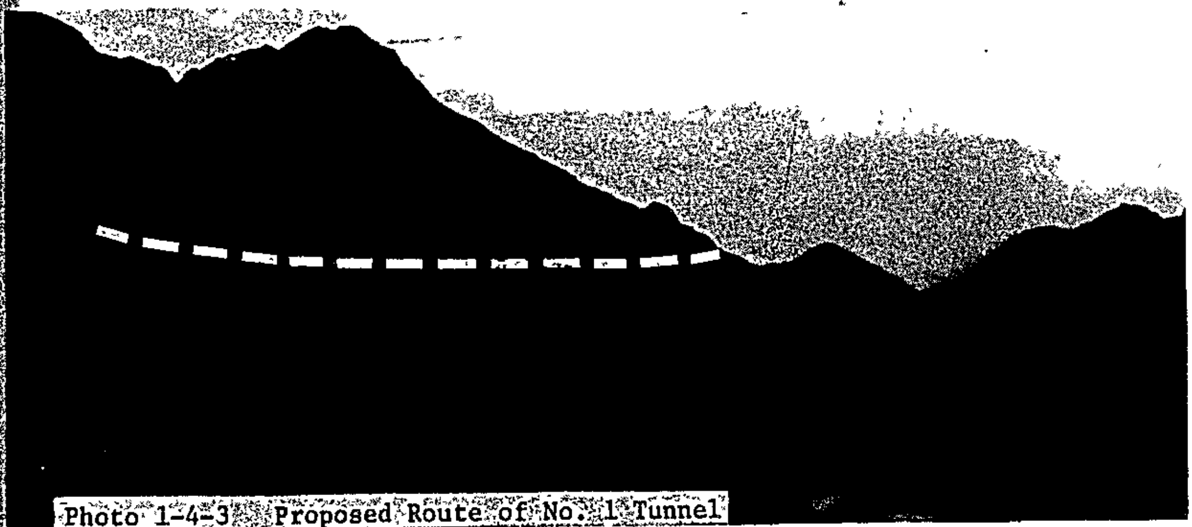


Photo 1-4-3 Proposed Route of No. 1 Tunnel

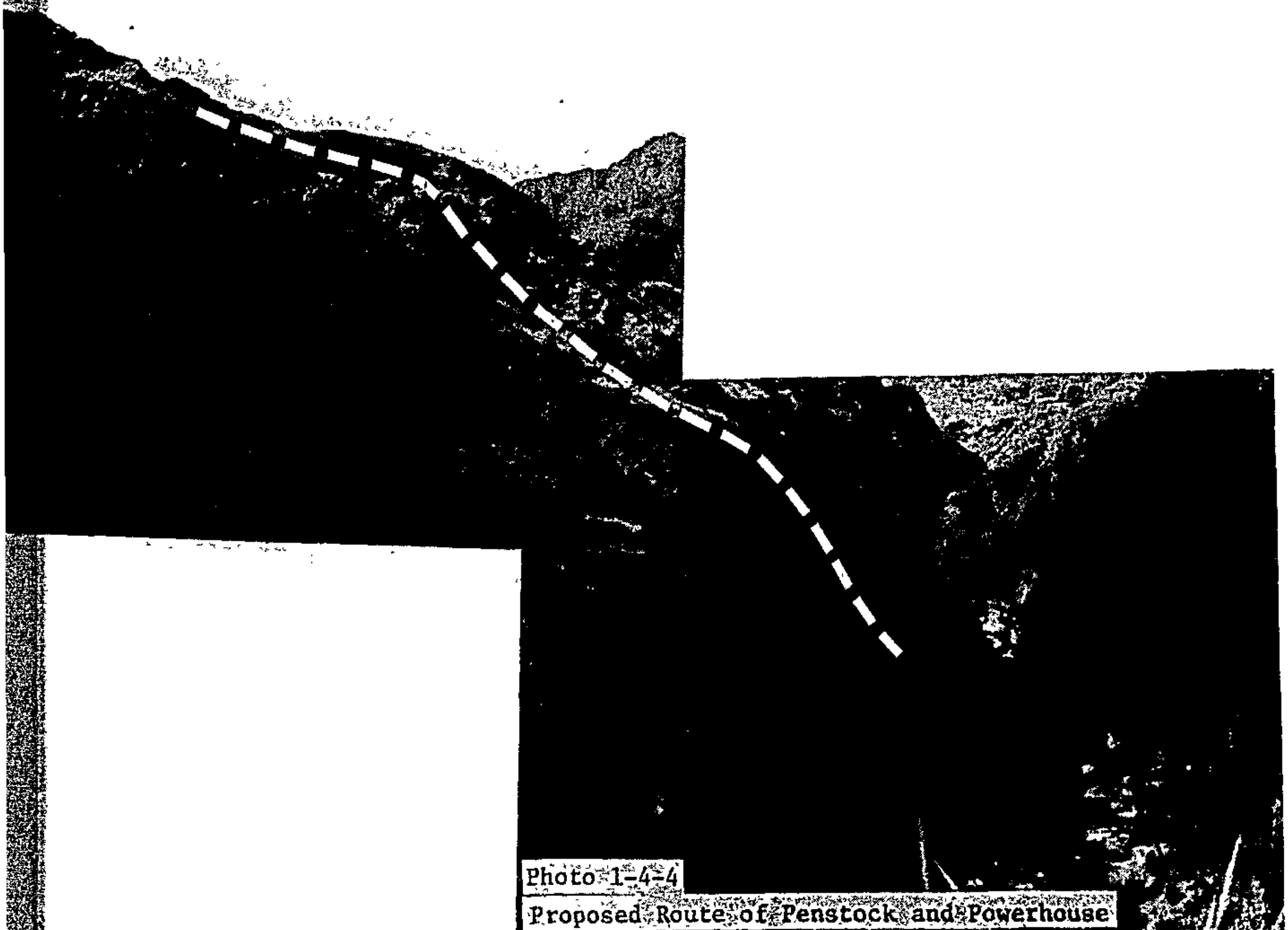


Photo 1-4-4
Proposed Route of Penstock and Powerhouse

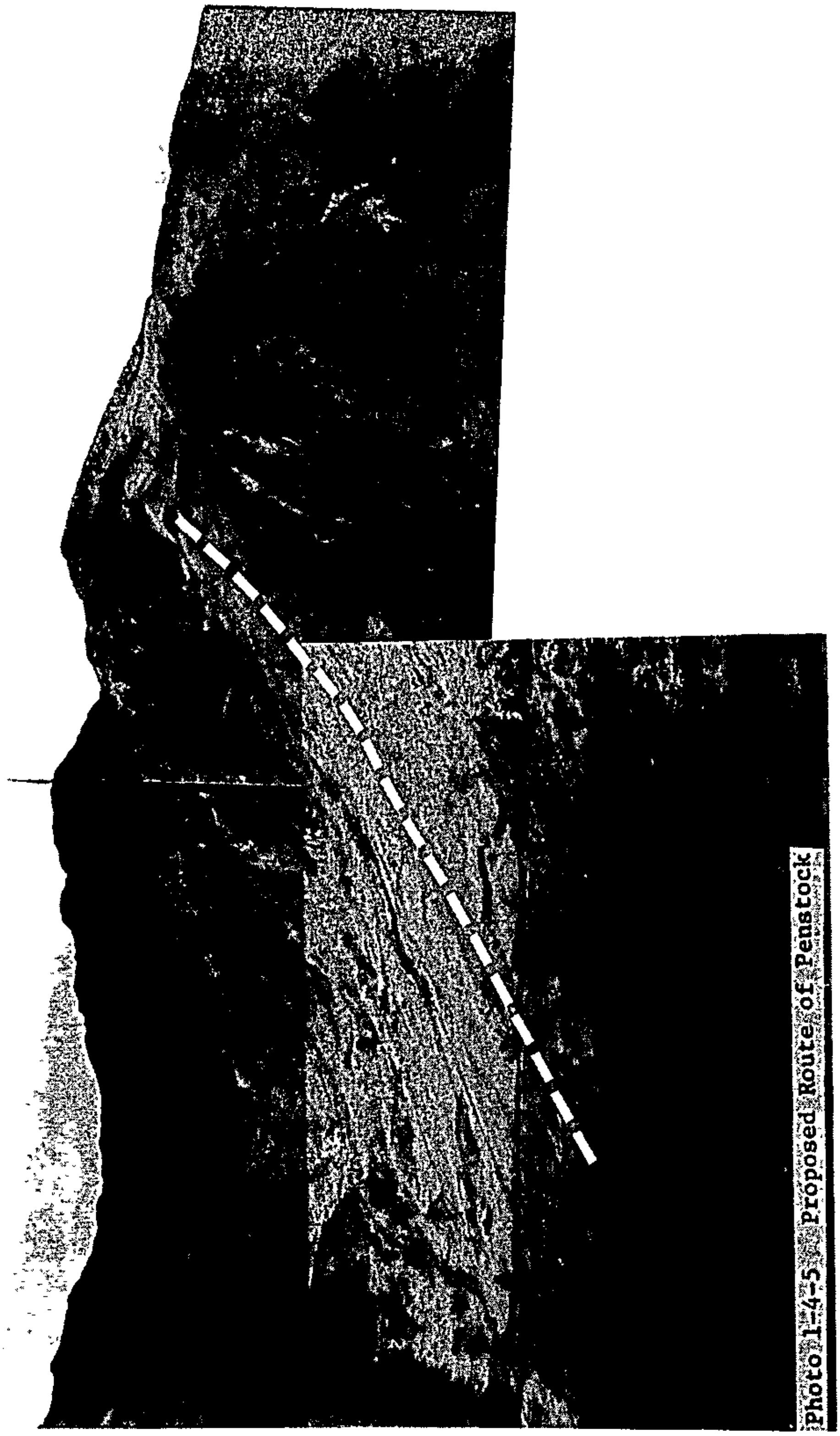


Photo 1-4-5 Proposed Route of Penstock

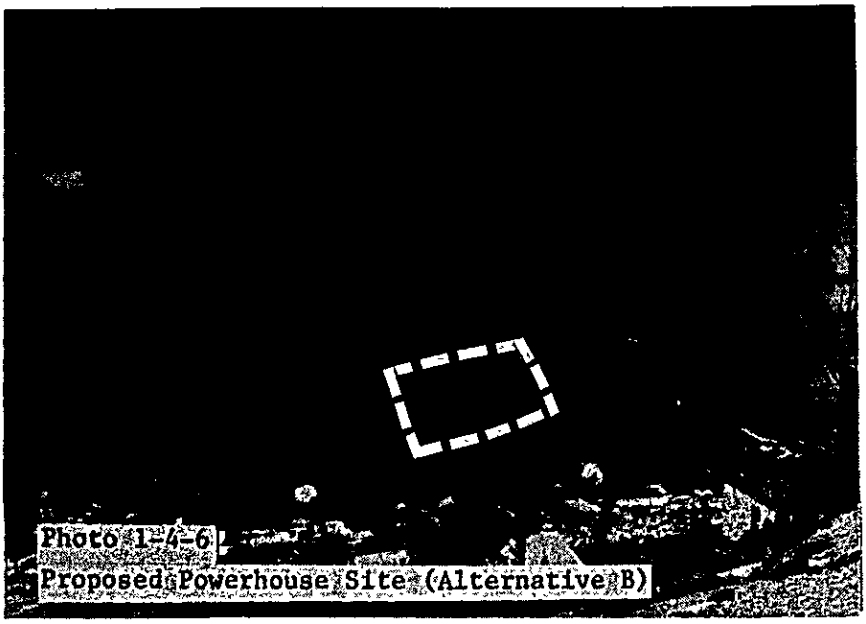


Photo 1-4-6
Proposed Powerhouse Site (Alternative B)

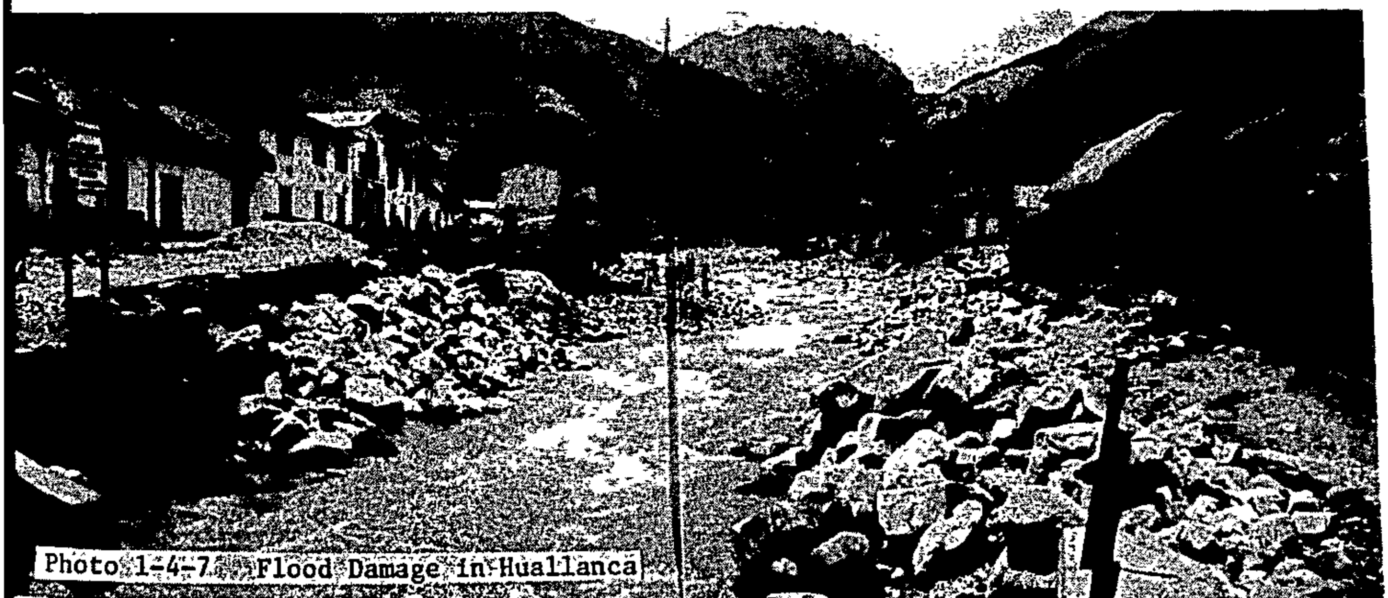


Photo 1-4-7 Flood Damage in Huallanca

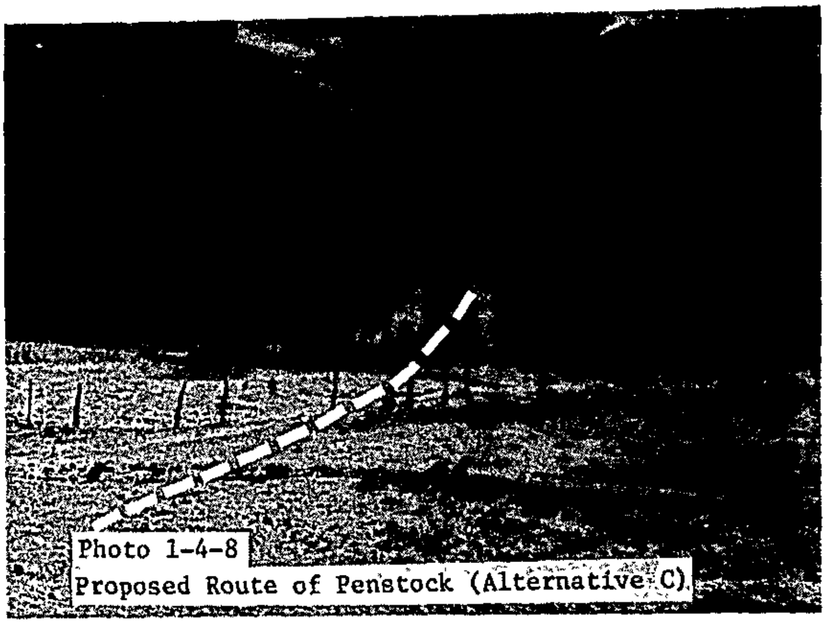


Photo 1-4-8
Proposed Route of Penstock (Alternative C)

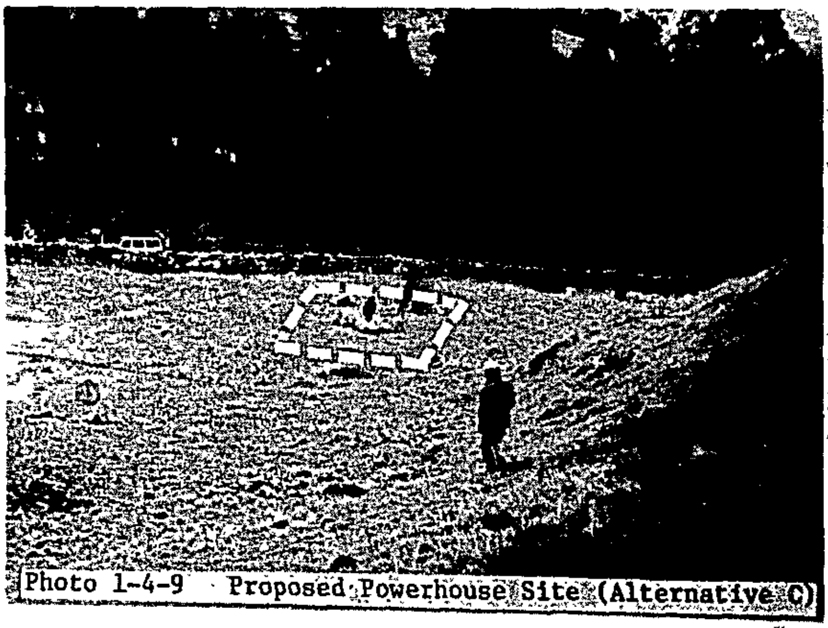


Photo 1-4-9 Proposed Powerhouse Site (Alternative C)

CHAPTER 1 POWER GENERATION SCHEME

1.1 Topography of Rio San Juan Drainage Basin

The Rio San Juan is located on the eastern slope of the Cordillera Chanpi Janca, one of the Los Andes in the central part of Peru, in which numerous peaks exceeding 5,000 m in height are strung. Fed from snowcaps of such mountains, the Rio San Juan runs north down a ravine, joins with the Rio Santa Rosa at Huallanca on the way. The Rio San Juan changes its course to the northeast to become the Rio Vizcara, passes through La Union, merges with the Rio Maranon at Quivilla. Then it becomes one of the headwaters of the Rio Amazonas. The project area in general is a mixture of plateau-like mountain-land and rugged terrain, and with vegetation consisting of lichens and shrubbery, it appears there is not much capacity for water retention.

The Rio San Juan greatly changes the conditions of its stream flow in the vicinity of Arequipa Bridge (Pte. Arequipa). The whole area upstream of Pte. Arequipa, as shown in Photo. 1-1-1, is a broad marshland with flood plains on both sides of the river. The flow is a gentle one of gradient about 1/100, contrasted to which, on the downstream side, it is a swift stream to the upstream edge of Huallanca (immediately upstream of the Electro Peru power station) with an average river gradient of about 1/20. The river width becomes narrower the more that Huallanca is approached, and the river banks increasingly steep. (Photo. 1-1-2)

On the other hand, the flow again becomes gentle on entering Huallanca town, and the average river gradient to several kilometers downstream is around 1/50. (Photo. 1-1-3)

In view of the above, this area possesses ideal topographical conditions for hydro-electric power generation scheme. As a natural consequence, the intake is planned at the starting point of the abovementioned section of swift flow with the powerhouse at a location bypassing this swift portion.

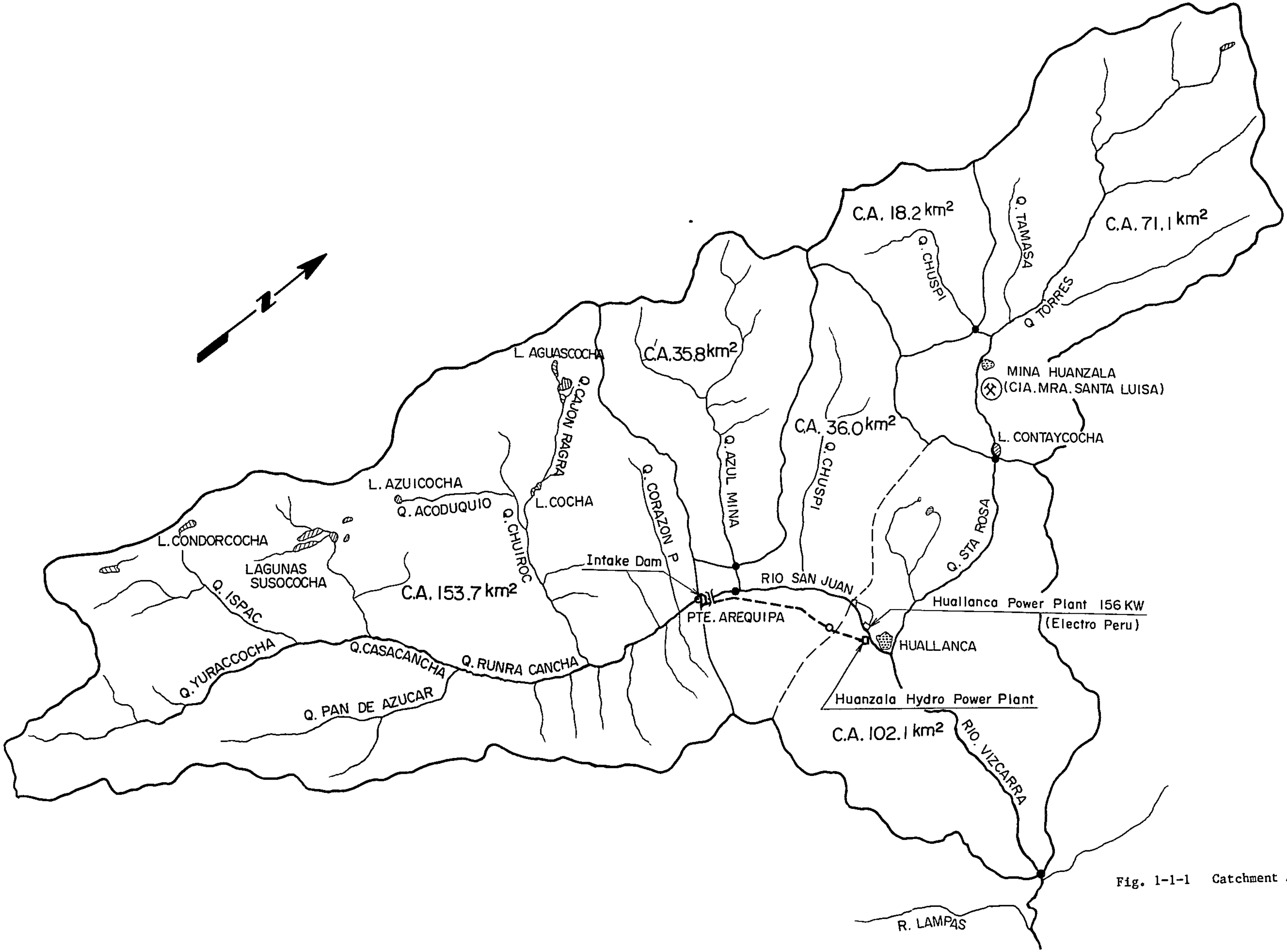


Fig. 1-1-1 Catchment Area

1.2 Hydrological Data

1.2.1 Stream Flow Gauging

Stream flow gauging in the vicinity of the Rio San Juan is being made at the sites below, and there are no sites other than these which are being observed independently by Santa Luisa. (Photos 1-2-1, 1-2-2)

Site (Stream)	Month & Year Gauging Begun	Catchment Area (sq.km)
Rio San Juan	January 1981	153.7
Q. Azul Mina	January 1981	35.8
Rio Viscarra	September 1981	380.9
Rio Torres	July 1979	89.3
Q. Chuspi	November 1979	18.2

At all of these sites, the frequencies of observations are about once a week, and the data are not complete since the periods of observation are short.

At runoff data for the project site, the data of the Rio San Juan which are closest to the dam site and where the observation facilities and measurement are reliable, will be used. The observation records for this site are available for the three years of 1981, 1982, and 1983 (up to October 22). Fortunately, at the Andes mountain system, the rainy season and the dry season are fairly distinct, with the rainfall in the dry season very little, being 30 mm/month to 40 mm/month. Runoff in the dry season has a strong tendency to be governed by the quantity of melt from glaciers rather than rainfall, and variation in runoff is very gradual. Accordingly, even if gauging is done at intervals of about one week, it may be considered that runoff data between measurements are reliable. Runoff records prepared augmenting missing portions are as shown in Tables 1-2-1, 1-2-2 and 1-2-3.

1.2.2 Precipitation

Precipitation observations have been carried out at Huallanca from 1964 to 1976 and at Huanzala from 1979 to 1982. On arranging the records, the monthly precipitations will be as shown in Figs. 1-2-1 and 1-2-2. The annual mean precipitation is 1,150 mm at Huallanca, and 970 mm at Huanzala, so that it would not be said this area has extremely little precipitation on an annual basis. By month, however, rainfall in the dry season from May to November is about 40 mm at Huallanca and about 30 mm at Huanzala for an extremely low figure compared with the annual. Therefore it cannot be looked forward to for large-volume intake to be made.

1.2.3 Design Flood Discharge

Fig. 1-2-3 shows the flood water level at the power station site estimated from the marks left by the flood of March 1981. Due to this flood a masonry bridge at Huallanca having a considerable history was washed away and the magnitude of flood is estimated to be a scale occurring once in several tens of years. Estimating the flood discharge at that time from the flood water level, the cross section and river gradient as shown on Fig. 1-2-3, it was 81 cu.m/sec, which converted to the discharge at the dam site, was 50 cu.m/ sec. To be on the conservative estimation, a design flood discharge 100 cu.m/sec double the above-mentioned figure was considered. This amount of 100 cu.m/sec is the extreme limit that can pass the Pte. Arequipa site immediately downstream of the dam. (Photos. 1-2-2, 1-2-3 and 1-2-4)

The design flood is estimated by the following method.

Manning Formula

$$V = \frac{1}{n} \cdot R^{3/2} I^{1/2}$$

$$Q = AV = 17.5\ m^2 \times 4.6\ m/sec = 81\ m^3/sec$$

where

Q: Discharge (m^3/sec)

A: Sectional area of flow = 17.5 (m^2)

V: Velocity of flow (m/sec)

n: Coefficient of roughness = 0.05

R: Hydraulic mean depth (m)

I: Slope = $\frac{1}{22}$

Fig. 1-2-3 Cross Section of River at Power Station (Scale 1:200)

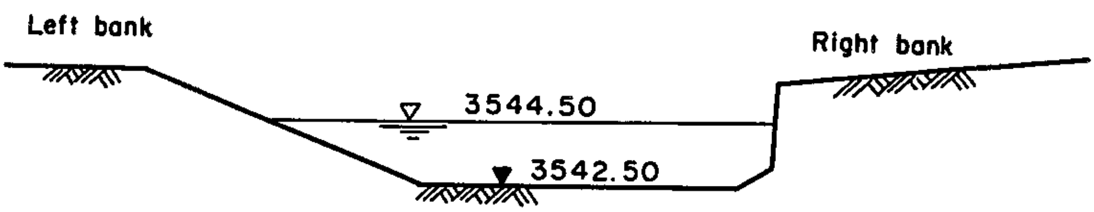
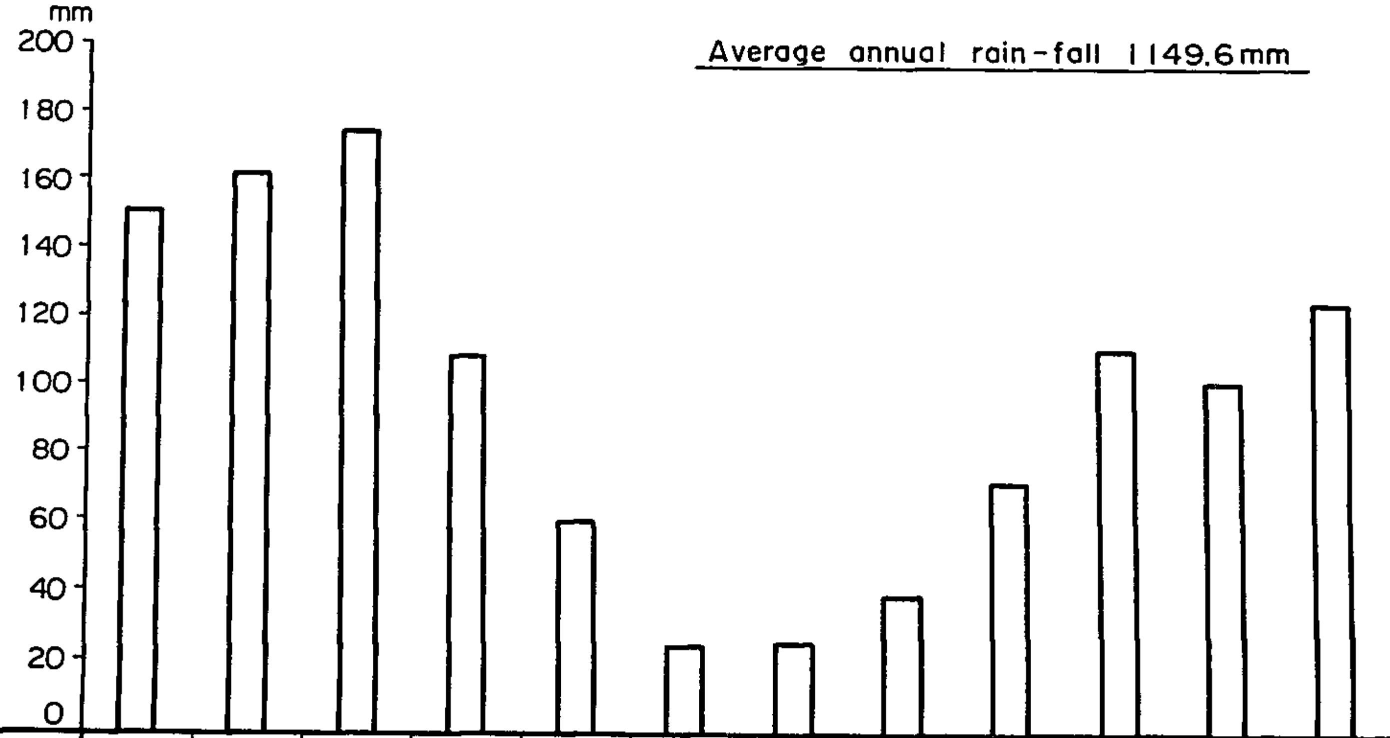
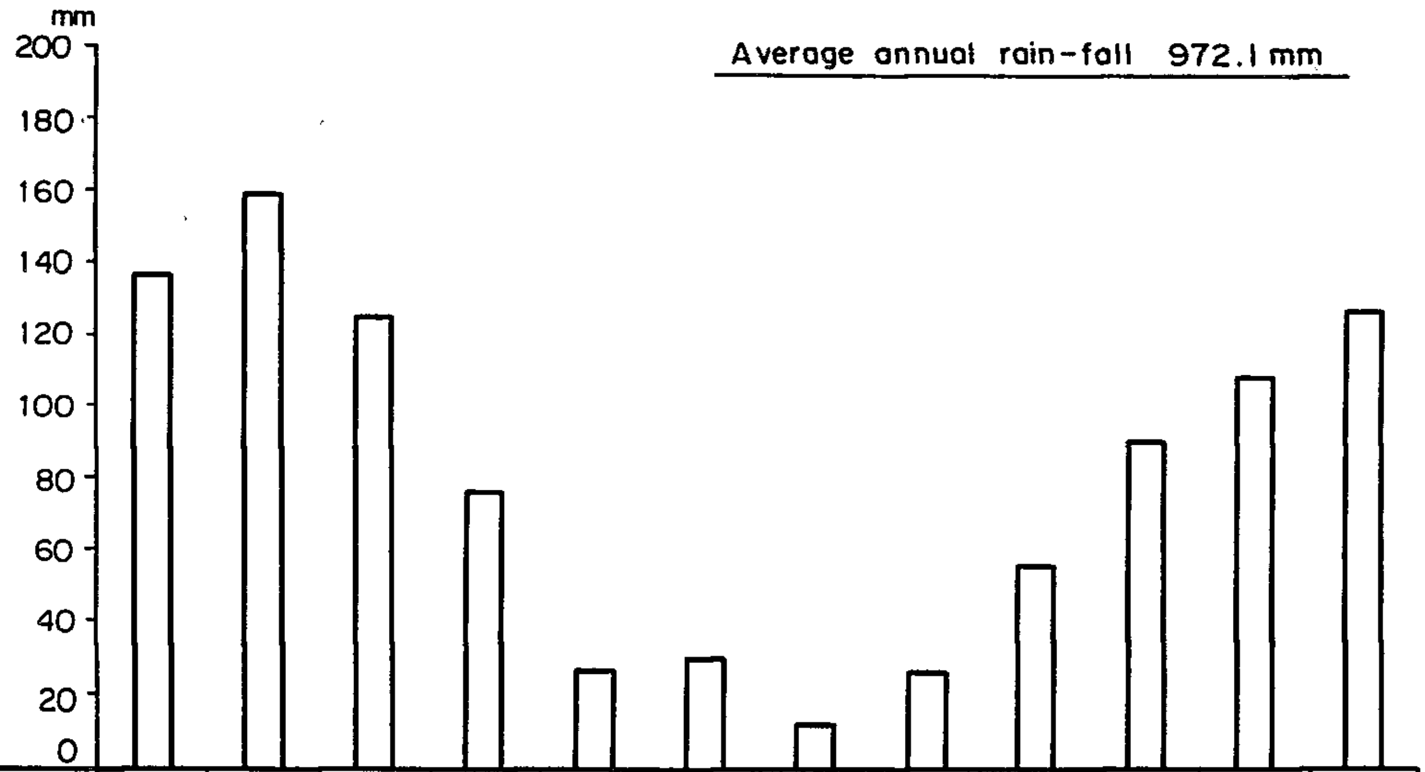


Fig. 1-2-1 Average of Monthly Rainfall in 1964 - 1976 (Huallanca)



MONTH	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dic.
MAXIMUM	212.3	232.7	252.4	142.9	119.0	56.8	58.3	84.1	121.0	171.1	163.6	166.0
MINIMUM	84.0	112.0	117.0	69.7	12.1	2.0	9.1	9.0	26.2	61.0	60.8	63.5
Maximum dayly rain-fall	31.5	48.2	28.0	34.0	42.0	15.0	32.0	28.4	17.5	25.4	32.2	31.0

Fig. 1-2-2 Average of Monthly Rainfall in 1979 - 1982 (Huanzala)



MONTH	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dic.
MAXIMUM	173.5	241.5	152.5	123.0	44.0	79.5	39.5	60.5	94.5	125.0	151.0	141.5
MINIMUM	92.0	59.0	84.5	37.0	9.0	2.0	0.0	1.0	23.0	43.0	61.0	99.0
Maximum dayly rain-fall	20.0	18.0	26.5	24.0	9.0	12.5	9.5	20.0	14.0	25.5	14.5	17.0

Table.1-2-1 Natural Run-off of Rio San Juan at Dam Site (1981)

(m3/s)

day	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1	2.30	10.83	10.92	4.71	3.62	2.49	1.10	0.95	0.79	1.92	7.75	12.39
2	2.30	10.83	9.87	4.67	3.58	2.45	1.01	0.94	0.76	1.92	7.53	12.08
3	3.06	10.83	8.81	4.63	3.54	4.42	0.91	0.93	0.74	1.97	7.31	11.77
4	3.81	10.83	7.76	4.60	3.51	2.38	0.95	0.92	0.71	2.02	7.09	11.47
5	4.57	10.82	6.70	4.56	3.47	2.34	0.99	0.92	0.76	2.07	6.86	11.16
6	5.33	10.82	5.65	4.52	3.43	2.31	1.03	0.91	0.82	2.12	6.64	10.85
7	6.09	11.21	5.61	4.49	3.40	2.27	1.07	0.90	0.87	2.16	6.42	10.54
8	6.84	11.59	5.58	4.45	3.36	2.23	1.11	1.08	0.93	2.21	6.20	10.23
9	7.60	11.98	5.54	4.41	3.32	2.20	1.15	1.26	0.98	2.26	5.98	9.93
10	7.32	12.37	5.50	4.38	3.29	2.16	1.19	1.44	1.04	2.31	5.76	9.62
11	7.04	12.76	5.47	4.34	3.25	2.13	1.15	1.62	1.09	2.24	7.85	9.31
12	6.76	13.14	5.43	4.31	3.22	2.09	1.11	1.80	1.14	2.18	9.94	9.20
13	6.47	13.53	5.40	4.27	3.18	2.05	1.07	1.98	1.19	2.11	12.03	9.09
14	6.19	13.49	5.36	4.23	3.14	2.02	1.03	2.16	1.24	2.04	11.26	8.98
15	5.91	13.46	5.32	4.20	3.11	1.98	0.99	2.18	1.29	1.98	10.50	8.86
16	5.63	13.42	5.29	4.16	3.07	1.94	0.95	2.20	1.34	1.91	9.73	8.75
17	5.21	13.39	5.25	4.12	3.03	1.91	0.91	2.22	1.39	2.45	8.96	8.64
18	4.79	13.35	5.21	4.09	3.00	1.87	0.92	2.24	1.44	2.98	8.19	8.53
19	4.37	13.32	5.18	4.05	2.96	1.83	0.94	2.26	1.50	3.52	7.42	8.19
20	3.95	13.28	5.14	4.01	2.92	1.80	0.95	2.28	1.55	4.06	6.66	7.85
21	3.53	13.24	5.10	3.98	2.89	1.76	0.97	2.30	1.61	4.60	7.65	7.50
22	3.11	13.21	5.07	3.94	2.85	1.73	0.98	2.10	1.67	5.13	8.65	7.16
23	2.69	13.17	5.03	3.90	2.82	1.69	1.00	1.90	1.72	5.67	9.64	6.82
24	3.85	13.14	5.00	3.87	2.78	1.65	1.01	1.70	1.78	6.03	10.64	6.48
25	5.02	13.10	4.96	3.83	2.74	1.62	1.00	1.50	1.83	6.39	11.63	6.13
26	6.18	13.07	4.92	3.80	2.71	1.58	0.99	1.30	1.89	6.75	12.63	5.79
27	7.35	13.03	4.89	3.76	2.67	1.48	0.99	1.10	1.90	7.11	13.62	5.45
28	8.51	11.98	4.85	3.72	2.63	1.39	0.98	0.90	1.90	7.47	13.31	5.11
29	9.68		4.81	3.69	2.60	1.29	0.97	0.87	1.91	7.83	13.00	4.76
30	10.84		4.78	3.65	2.56	1.20	0.96	0.85	1.91	8.19	12.70	4.42
31	10.84		4.74		2.52		0.96	0.82		7.97		4.08
Ave.	5.71	12.47	3.06	4.18	3.07	2.01	1.01	1.50	1.32	3.86	9.12	8.42

Table.1-2-2 Natural Run-off of Rio San Juan at Dam Site (1982)

(m^3/s)

day	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1	3.93	9.94	3.98	6.35	6.07	1.56	0.98	0.93	1.16	2.46	5.11	3.51
2	3.78	9.70	3.94	6.47	5.52	1.55	0.97	0.94	1.18	2.55	4.77	3.38
3	3.63	9.47	3.90	6.59	4.96	1.53	0.96	0.95	1.21	2.63	4.44	3.45
4	3.48	9.24	3.87	6.71	4.40	1.52	0.95	0.96	1.20	2.72	4.10	3.53
5	3.32	9.00	3.83	6.84	3.84	1.51	0.93	0.97	1.20	2.80	3.76	3.60
6	3.17	8.77	3.80	6.96	3.29	1.49	0.92	0.98	1.19	2.89	3.42	3.68
7	3.02	8.53	3.76	7.08	2.73	1.48	0.91	0.98	1.18	2.97	3.09	3.75
8	2.87	8.30	3.73	7.20	2.62	1.47	0.90	0.98	1.17	3.00	2.75	3.83
9	2.99	8.07	3.69	7.32	2.51	1.46	0.89	0.98	1.17	3.03	2.42	3.90
10	3.11	7.83	3.66	7.44	2.39	1.44	0.88	0.98	1.16	3.06	3.72	3.98
11	3.24	7.60	3.62	7.16	2.28	1.43	0.87	0.97	1.15	3.09	5.02	4.05
12	3.36	7.37	3.59	6.87	2.17	1.38	0.86	0.97	1.16	3.12	6.32	4.13
13	3.48	7.13	4.80	6.59	2.06	1.33	0.85	0.97	1.18	3.15	7.61	4.20
14	3.60	6.90	6.01	6.30	1.94	1.28	0.84	0.96	1.19	3.18	8.91	4.11
15	3.73	6.66	7.22	6.02	1.83	1.23	0.84	0.95	1.20	3.21	10.21	4.02
16	4.22	6.43	8.43	5.73	1.81	1.18	0.83	0.94	1.22	3.24	11.51	3.93
17	4.70	6.20	9.64	5.45	1.79	1.17	0.83	0.93	1.23	3.27	10.78	3.85
18	5.19	5.96	10.85	5.18	1.77	1.15	0.82	0.92	1.24	3.30	10.04	3.76
19	5.68	5.73	12.06	4.91	1.74	1.14	0.82	0.91	1.26	3.33	9.31	3.67
20	6.16	5.49	11.57	4.65	1.72	1.12	0.81	0.90	1.27	3.36	8.57	3.58
21	6.65	5.26	11.09	4.38	1.70	1.11	0.82	0.92	1.28	3.39	7.84	3.49
22	7.13	5.03	10.60	4.11	1.69	1.09	0.83	0.94	1.29	3.42	7.11	3.42
23	7.62	4.79	10.12	3.84	1.67	1.08	0.84	0.96	1.30	3.45	6.37	3.31
24	8.12	4.56	9.63	4.24	1.66	1.06	0.85	0.98	1.31	3.78	5.64	3.22
25	8.63	4.32	9.15	4.64	1.65	1.05	0.86	0.99	1.32	4.12	4.90	3.14
26	9.13	4.09	8.66	5.04	1.64	1.03	0.87	1.01	1.51	4.45	4.17	3.05
27	9.63	4.05	8.17	5.43	1.62	1.02	0.88	1.03	1.70	4.78	4.04	2.96
28	10.14	4.02	7.69	5.83	1.61	1.01	0.89	1.05	1.89	5.11	3.91	2.87
29	10.64		7.20	6.23	1.60	1.00	0.90	1.08	2.08	5.45	3.78	3.08
30	10.41		6.72	6.63	1.59	0.99	0.91	1.10	2.27	5.78	3.64	3.28
31	10.17		6.23		1.57		0.92	1.13		5.44		3.49
Ave.	5.64	6.80	3.63	5.94	2.43	1.26	0.88	0.98	1.33	3.53	5.91	3.59

Table.1-2-3 Natural Run-off of Rio San Juan at Dam Site (1983)

(m^3/s)

day	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1	3.69	4.49	6.36	4.13	4.11	1.78	2.50	1.20	1.11	1.09		
2	3.90	4.33	6.52	3.65	4.11	2.09	2.90	1.15	1.23	1.17		
3	4.10	4.18	6.68	3.65	3.70	1.83	2.90	1.10	1.34	1.24		
4	4.31	4.04	5.43	9.00	2.90	2.90	2.00	1.06	1.45	1.32		
5	4.51	3.90	8.75	2.62	2.90	4.10	2.30	1.00	1.56	1.39		
6	4.72	3.76	6.20	3.39	2.90	5.10	2.20	1.06	1.68	1.46		
7	4.92	3.63	10.00	10.26	4.30	3.90	3.30	1.10	1.79	1.54		
8	5.18	3.49	8.75	4.13	2.90	2.70	2.70	1.10	1.90	1.61		
9	5.43	3.35	6.20	5.43	2.09	3.30	2.09	1.10	2.02	1.68		
10	5.69	3.21	6.20	4.13	1.78	6.15	2.30	1.06	2.13	1.76		
11	5.95	3.38	8.75	3.65	1.78	4.93	2.00	1.06	2.24	1.83		
12	6.20	3.55	7.47	4.42	2.09	4.71	2.15	1.06	2.35	1.91		
13	6.46	3.72	7.21	6.20	1.78	4.11	2.10	1.03	2.47	1.98		
14	6.71	3.89	7.47	4.30	1.78	5.10	2.05	1.10	2.58	2.05		
15	6.97	4.06	5.43	2.30	1.78	6.15	2.01	1.03	2.44	2.13		
16	6.84	4.23	7.99	5.70	1.78	5.10	1.96	0.96	2.31	2.20		
17	6.70	4.40	10.52	8.00	1.78	4.71	1.91	0.96	2.17	2.28		
18	6.57	4.57	7.99	6.40	1.78	5.50	1.86	0.96	2.03	2.35		
19	6.43	4.74	8.75	5.90	1.90	4.41	1.82	1.06	1.90	2.42		
20	6.30	4.90	11.60	4.90	2.09	3.50	1.77	1.00	1.77	2.50		
21	6.16	5.06	10.52	4.90	2.50	3.50	1.72	1.03	1.63	2.57		
22	6.03	5.23	8.50	5.90	2.50	3.10	1.67	1.03	1.56	2.64		
23	5.88	5.39	10.52	2.90	2.09	2.50	1.63	1.03	1.49			
24	5.72	5.55	6.45	2.90	1.78	3.90	1.58	1.03	1.43			
25	5.57	5.71	3.39	5.90	1.78	3.50	1.53	1.03	1.36			
26	5.41	5.87	3.46	5.70	1.19	4.10	1.48	1.03	1.29			
27	5.26	6.03	3.52	4.50	1.78	3.70	1.43	1.00	1.22			
28	5.11	6.20	3.59	2.90	1.42	5.10	1.39	1.00	1.16			
29	4.95		3.65	2.30	1.23	4.41	1.34	1.00	1.09			
30	4.80		5.17	2.90	1.60	2.50	1.29	1.03	1.02			
31	4.64		5.68		1.60		1.24	1.00				
Ave.	5.52	4.46	4.10	4.77	2.25	3.95	1.97	1.04	1.72			

1.3 Runoff Duration Curve

The catchment area of the intake dam site is 153.7 sq.km. Since runoff measurement is done immediately upstream of the dam, while the catchment area is practically the same, it may be considered that the records of the runoff gauging site on the Rio San Juan in effect represent the discharge conditions at the dam site.

The stream regimen at the above site is shown in Table 1-3-1 and the runoff duration curves of the site in Fig. 1-3-1 and Tables 1-3-2 to 1-3-5. According to these data, the runoff in 1981 and 1983 were roughly similar in trend, but those in 1982 was an extremely small discharge in comparison. The stream regimen during the period of 30 days of lowest water was 0.71 - 0.97 cu.m/sec in 1981, 0.81 - 0.92 cu.m/sec in 1982, and 0.96 - 1.10 cu.m/sec in 1983. There were no great differences in discharges between the individual years and the stream regimens were relatively stable. The number of days on which the maximum power discharge of 2.2 cu.m/sec could be available for the hydro-electric power station was 246 days in 1981, 224 days in 1982, and 248 days in 1983 to indicate slight differences.

In case of determining the scale of the power station based on the runoff duration curves, there would be a problem if the data were to be inadequate. The scale of power generation of this Project, however, is determined on the basis of the load forecast. Only the amount of supplemental firing of diesel generating facilities in the dry season is estimated from the runoff duration curves, and for such purposes it is judged that the presently available discharge data are adequate.

Therefore, the average for a 3-year period is used for estimating the runoff duration curve to be used in the study on power generation scheme.

Runoff Data at Rio San Juan

(Unit: cu.m/sec)

Year	Max. Discharge	95 day Discharge	185 day Discharge	275 day Discharge	355 day Discharge	Min. Discharge
1981	13.62	6.75	3.76	1.90	0.90	0.71
1982	12.06	5.11	3.29	1.20	0.85	0.81
1983	-	-	-	1.83	1.03	0.96
Average	12.54	5.93	3.53	1.64	0.93	0.83

Fig. 1-3-1 Flow-duration Curve of Rio San Juan

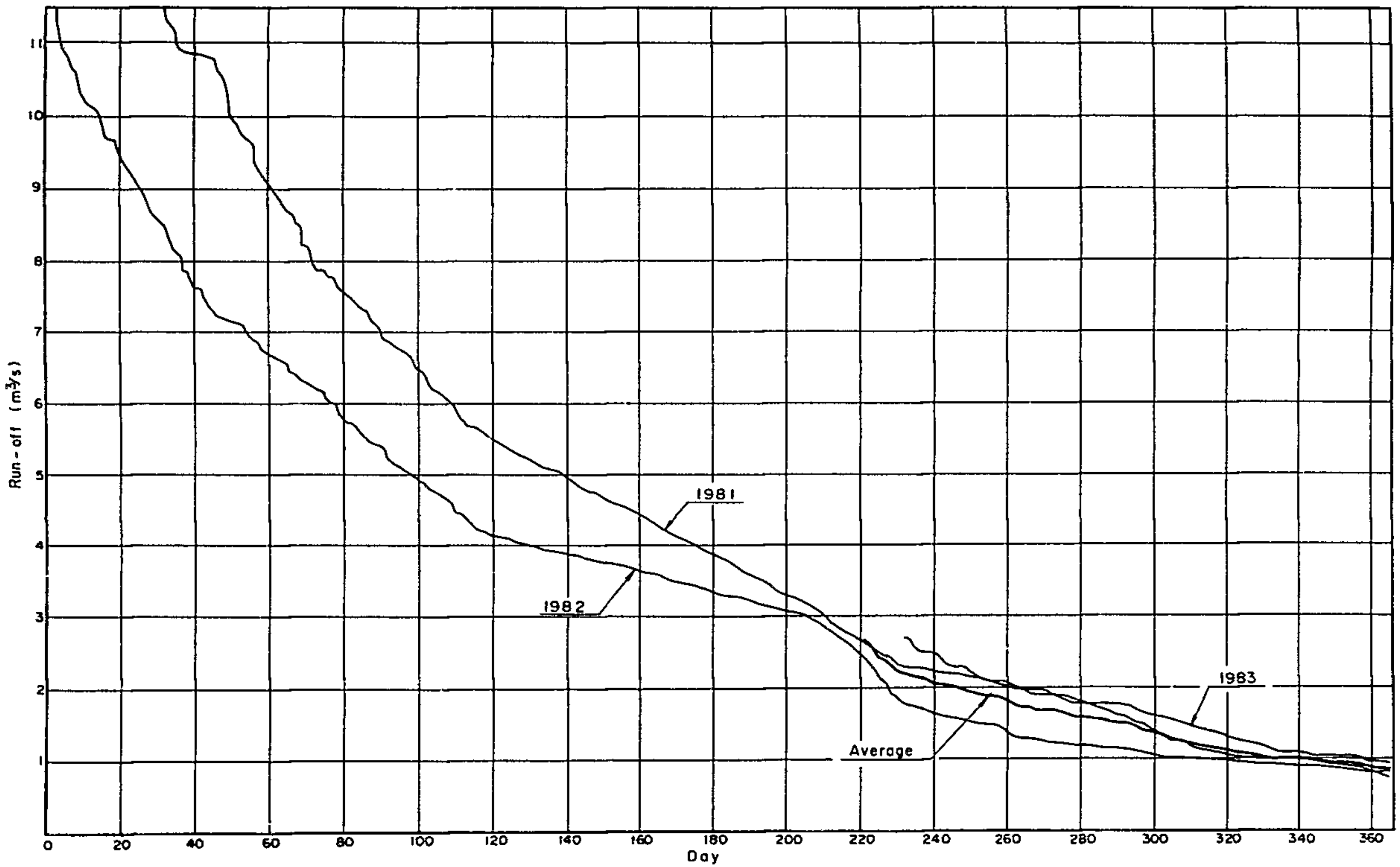


Table.1-3-1 Flow-duration table of Rio San Juan (1981)

(m^3/s)

	1	2	3	4	5	6	7	8	9	10
0	13.62	13.53	13.49	13.46	13.42	13.39	13.35	13.32	13.31	13.28
10	13.24	13.21	13.17	13.14	13.14	13.10	13.07	13.03	13.00	12.76
20	12.70	12.63	12.39	12.37	12.08	12.03	11.98	11.98	11.77	11.63
30	11.59	11.47	11.26	11.21	11.16	10.92	10.85	10.84	10.84	10.83
40	10.83	10.83	10.83	10.82	10.82	10.64	10.54	10.50	10.23	9.94
50	9.93	9.87	9.73	9.68	9.64	9.62	9.31	9.20	9.09	8.98
60	8.96	8.86	8.81	8.75	8.65	8.64	8.53	8.51	8.19	8.19
70	8.19	7.97	7.85	7.85	7.83	7.76	7.75	7.65	7.60	7.53
80	7.50	7.47	7.42	7.35	7.32	7.31	7.16	7.11	7.09	7.04
90	6.86	6.84	6.82	6.76	6.75	6.70	6.66	6.64	6.48	6.47
100	6.42	6.39	6.20	6.19	6.18	6.13	6.09	6.03	5.98	5.91
110	5.79	5.76	5.67	5.65	5.63	5.61	5.58	5.54	5.50	5.47
120	5.45	5.43	5.40	5.36	5.33	5.32	5.29	5.25	5.21	5.21
130	5.18	5.14	5.13	5.11	5.10	5.07	5.03	5.02	5.00	4.96
140	4.92	4.89	4.85	4.81	4.79	4.78	4.76	4.74	4.71	4.67
150	4.63	4.60	4.60	4.57	4.56	4.52	4.49	4.45	4.42	4.42
160	4.41	4.38	4.37	4.34	4.31	4.27	4.23	4.20	4.16	4.12
170	4.09	4.08	4.06	4.05	4.01	3.98	3.95	3.94	3.90	3.87
180	3.85	3.83	3.81	3.80	3.76	3.72	3.69	3.65	3.62	3.58
190	3.54	3.53	3.52	3.51	3.47	3.43	3.40	3.36	3.32	3.29
200	3.25	3.22	3.18	3.14	3.11	3.11	3.07	3.06	3.03	3.00
210	2.98	2.96	2.92	2.89	2.85	2.82	2.78	2.74	2.71	2.69
220	2.67	2.63	2.60	2.56	2.52	2.49	2.45	2.45	2.38	2.34
230	2.31	2.31	2.30	2.30	2.30	2.28	2.27	2.26	2.26	2.24
240	2.24	2.23	2.22	2.21	2.20	2.20	2.18	2.18	2.16	2.16
250	2.16	2.13	2.12	2.11	2.10	2.09	2.07	2.05	2.04	2.02
260	2.02	1.98	1.98	1.98	1.97	1.94	1.92	1.92	1.91	1.91
270	1.91	1.91	1.90	1.90	1.90	1.89	1.87	1.83	1.83	1.80
280	1.80	1.78	1.76	1.73	1.72	1.70	1.69	1.67	1.65	1.62
290	1.62	1.61	1.58	1.55	1.50	1.50	1.48	1.44	1.44	1.39
300	1.39	1.34	1.30	1.29	1.29	1.26	1.24	1.20	1.19	1.19
310	1.15	1.15	1.14	1.11	1.11	1.10	1.10	1.09	1.08	1.07
320	1.07	1.04	1.03	1.03	1.01	1.01	1.00	1.00	0.99	0.99
330	0.99	0.99	0.98	0.98	0.98	0.97	0.97	0.96	0.96	0.95
340	0.95	0.95	0.95	0.94	0.94	0.93	0.93	0.92	0.92	0.92
350	0.91	0.91	0.91	0.90	0.90	0.87	0.87	0.85	0.82	0.82
360	0.79	0.76	0.76	0.74	0.71					

max = 13.62 95day = 6.75 185day = 3.76

275day = 1.90 355day = 0.90 min = 0.71

Table.1-3-2 Flow-duration table of Rio San Juan (1982)

(m^3/s)

	1	2	3	4	5	6	7	8	9	10
0	12.06	11.57	11.51	11.09	10.85	10.78	10.64	10.60	10.41	10.21
10	10.17	10.14	10.12	10.04	9.94	9.70	9.64	9.63	9.63	9.47
20	9.31	9.24	9.15	9.13	9.00	8.91	8.77	8.66	8.63	8.57
30	8.53	8.43	8.30	8.17	8.12	8.07	7.84	7.83	7.69	7.62
40	7.61	7.60	7.44	7.37	7.32	7.22	7.20	7.20	7.16	7.13
50	7.13	7.11	7.08	6.96	6.90	6.87	6.84	6.72	6.71	6.66
60	6.65	6.63	6.59	6.59	6.47	6.43	6.37	6.35	6.32	6.30
70	6.23	6.23	6.20	6.16	6.07	6.02	6.01	5.96	5.83	5.78
80	5.73	5.73	5.68	5.64	5.52	5.49	5.45	5.45	5.44	5.43
90	5.26	5.19	5.18	5.11	5.11	5.04	5.03	5.02	4.96	4.91
100	4.90	4.80	4.79	4.78	4.77	4.70	4.65	4.64	4.56	4.45
110	4.44	4.40	4.38	4.32	4.24	4.22	4.20	4.17	4.13	4.12
120	4.11	4.11	4.10	4.09	4.05	4.05	4.04	4.02	4.02	3.98
130	3.98	3.94	3.93	3.93	3.91	3.90	3.90	3.87	3.85	3.84
140	3.84	3.83	3.83	3.80	3.78	3.78	3.78	3.76	3.76	3.76
150	3.75	3.73	3.73	3.72	3.69	3.68	3.67	3.66	3.64	3.63
160	3.62	3.60	3.60	3.59	3.58	3.53	3.51	3.49	3.49	3.48
170	3.48	3.45	3.45	3.42	3.42	3.42	3.39	3.38	3.36	3.36
180	3.33	3.32	3.31	3.30	3.29	3.28	3.27	3.24	3.24	3.22
190	3.21	3.18	3.17	3.15	3.14	3.12	3.11	3.09	3.09	3.08
200	3.06	3.05	3.03	3.02	3.00	2.99	2.97	2.96	2.89	2.87
210	2.87	2.80	2.75	2.73	2.72	2.63	2.62	2.55	2.51	2.46
220	2.42	2.39	2.28	2.27	2.17	2.08	2.06	1.94	1.89	1.83
230	1.81	1.79	1.77	1.74	1.72	1.70	1.70	1.69	1.67	1.66
240	1.65	1.64	1.62	1.61	1.60	1.59	1.57	1.56	1.55	1.53
250	1.52	1.51	1.51	1.49	1.48	1.47	1.46	1.44	1.43	1.38
260	1.33	1.32	1.31	1.30	1.29	1.28	1.28	1.27	1.26	1.24
270	1.23	1.23	1.22	1.21	1.20	1.20	1.20	1.19	1.19	1.18
280	1.18	1.18	1.18	1.17	1.17	1.17	1.16	1.16	1.16	1.15
290	1.15	1.14	1.13	1.12	1.11	1.10	1.09	1.08	1.08	1.06
300	1.05	1.05	1.03	1.03	1.02	1.01	1.01	1.00	0.99	0.99
310	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.97	0.97	0.97
320	0.97	0.97	0.96	0.96	0.96	0.96	0.95	0.95	0.95	0.94
330	0.94	0.94	0.93	0.93	0.93	0.92	0.92	0.92	0.92	0.91
340	0.91	0.91	0.90	0.90	0.90	0.89	0.89	0.88	0.88	0.87
350	0.87	0.86	0.86	0.85	0.85	0.84	0.84	0.84	0.83	0.83
360	0.83	0.82	0.82	0.82	0.81					

max = 12.06   95day = 5.11   185day = 3.29

275day = 1.20   355day = 0.85   min = 0.81

Table.1-3-3 Flow-duration table of Rio San Juan (1983)

(m³/s)

	1	2	3	4	5	6	7	8	9	10
0										
10										
20										
30										
40										
50										
60										
70										
80										
90										
100										
110										
120										
130										
140										
150										
160										
170										
180										
190										
200										
210										
220					2.64	2.62	2.58	2.57	2.50	2.50
230	2.50	2.50	2.50	2.50	2.47	2.44	2.42	2.35	2.35	2.31
240	2.30	2.30	2.30	2.30	2.28	2.24	2.20	2.20	2.17	2.15
250	2.13	2.13	2.10	2.09	2.09	2.09	2.09	2.09	2.09	2.05
260	2.05	2.03	2.02	2.01	2.00	2.00	1.98	1.96	1.91	1.91
270	1.90	1.90	1.90	1.86	1.83	1.83	1.82	1.79	1.78	1.78
280	1.78	1.78	1.78	1.78	1.78	1.78	1.78	1.78	1.78	1.78
290	1.77	1.77	1.76	1.72	1.68	1.68	1.67	1.63	1.63	1.61
300	1.60	1.60	1.58	1.56	1.56	1.54	1.53	1.49	1.48	1.46
310	1.45	1.43	1.43	1.42	1.39	1.39	1.36	1.34	1.34	1.32
320	1.29	1.29	1.24	1.24	1.23	1.23	1.22	1.20	1.19	1.17
330	1.16	1.15	1.11	1.10	1.10	1.10	1.10	1.10	1.09	1.09
340	1.06	1.06	1.06	1.06	1.06	1.06	1.03	1.03	1.03	1.03
350	1.03	1.03	1.03	1.03	1.03	1.02	1.00	1.00	1.00	1.00
360	1.00	1.00	0.96	0.96	0.96					

max = 95day = 6.52 185day = 3.89

275day = 1.83 355day = 1.03 min = 0.96

Table.1-3-4 Flow-duration table of Rio San Juan (Ave.)

(m^3/s)

	1	2	3	4	5	6	7	8	9	10
0										
10										
20										
30										
40										
50										
60										
70										
80										
90										
100										
110										
120										
130										
140										
150										
160										
170										
180										
190										
200										
210										
220		2.64	2.53	2.51	2.44	2.40	2.36	2.32	2.26	2.22
230	2.21	2.20	2.19	2.18	2.16	2.14	2.13	2.10	2.09	2.07
240	2.06	2.06	2.05	2.04	2.03	2.01	1.98	1.98	1.96	1.95
250	1.94	1.92	1.91	1.90	1.89	1.88	1.87	1.86	1.85	1.82
260	1.80	1.78	1.77	1.76	1.75	1.74	1.73	1.72	1.69	1.69
270	1.68	1.68	1.67	1.66	1.64	1.64	1.63	1.60	1.60	1.59
280	1.59	1.58	1.57	1.56	1.56	1.55	1.54	1.54	1.53	1.52
290	1.51	1.51	1.49	1.46	1.43	1.43	1.41	1.38	1.38	1.35
300	1.35	1.33	1.30	1.29	1.29	1.27	1.26	1.23	1.22	1.21
310	1.19	1.19	1.18	1.17	1.16	1.16	1.15	1.13	1.13	1.12
320	1.11	1.10	1.08	1.08	1.07	1.07	1.06	1.05	1.04	1.03
330	1.03	1.03	1.01	1.00	1.00	1.00	1.00	0.99	0.99	0.98
340	0.97	0.97	0.97	0.97	0.97	0.96	0.95	0.94	0.94	0.94
350	0.94	0.93	0.93	0.93	0.93	0.91	0.90	0.90	0.88	0.88
360	0.87	0.86	0.85	0.84	0.83					

max = 95day = 6.13 185day = 3.65

275day = 1.64 355day = 0.93 min = 0.83

1.4 Power Generation Scheme

1.4.1 Installed Capacity

In the case of a power station to be incorporated into an ordinary electric power system, the maximum power discharge and installed capacity are generally determined so that the power generating cost will be cheapest. In general, the maximum power discharge is selected at around a 95-day discharge, which in the case of this power station site corresponds approximately to 6 cu.m/sec.

However, the private power station for Huanzala Mine will not be incorporated in any other system. Moreover, there is the special situation that existing diesel facilities will be used when water power will be insufficient. Therefore, the installed capacity of this power station was decided not on the basis of the above-mentioned principle, but the load forecast for the service area projected. In essence, the output of the power station contemplated to be built is 4,200 kW and the maximum power discharge is 2.2 cu.m/sec (corresponding to 230-day discharge). (Refer to Vol. 1, Section 6-1)

1.4.2 Comparisons of Powerhouse Sites

(1) General

As described in Section 1.1, "Topography of Rio San Juan Drainage Basin," the point immediately upstream of Pte. Arequipa is optimum as the site for intake facilities, and any alternative is inconceivable. (Photo. 1-4-1)

For the powerhouse site, comparisons were made of three alternatives, A (upstream site), B (midstream site), and C (downstream site), the special notes about these alternatives being as follows:

Alternative A:

This was a plan formulated by Santa Luisa on their own basis, which became the origin of this Project. The tailrace is planned upstream of the intake of the existing Electro Peru power station. The chief aim is that the existing power station would not be affected at all.

Alternative B:

This was formulated as an alternative plan for A. Since the powerhouse site is at the upstream edge of Huallanca, it is expected that the town will ask the revetments lost in the flood of 1981 to be restored. Because of such circumstances, Santa Luisa had been looking upon this plan as being difficult to realize.

Also, since the location of the tailrace is downstream of the existing intake of Electro Peru power station, it was feared that operation of the above-mentioned power station in the dry season would be affected.

Alternative C:

Being a proposal to use the greatest head, this was taken up as a plan for a high rate of operation with the minimum amount of water. However, similarly to Alternative B, this proposal would affect operation of the existing Electro Peru power station in the dry season.

It was confirmed by JICA Survey Team that there is no promising alternative other than the above three. In carrying out the field investigations, detailed examinations were made for the individual alternatives determining locations of the structures actually required. The features of the individual alternatives clarified as a result of investigations are described below. The intake site is the same for all alternatives, so that the features described here will be those excepting the intake site.

(2) Alternative A (Upstream site)

The headrace is comprised of 3,000 m of open canals (No. 1 open canal, 1,900 m; No. 2 open canal, 1,100 m) and 1,400 m of headrace tunnels (No. 1 tunnel, 1,000 m; No. 2 tunnel, 400 m).

The No. 1 open canal is to be located at the gentle slope of the right bank which shows a great contrast with the steep mountain-side on the left bank, and construction will be very easy. On going downstream 1,900 m, the degree of steepness increases slightly, and boulders of glacial origin are deposited (Photo 1-4-2). This part will be the first half of the No. 1 tunnel. The topography becomes even more rugged at the latter half of the No. 1 tunnel and a large ridge is cut through to make up the total length of 1,000 m of the tunnel. (Photo. 1-4-3)

The No. 2 open canal will appear at a comparatively gentle slope between the above-mentioned ridge and the next ridge, but compared with the No. 1 open canal route the slope is steep and the route having a length of 1,100 m will cross small-scale landslide areas and collapse areas of glacial deposits so that the degree of difficulty in construction is increased. The No. 2 tunnel of length of 400 m will go through the next large ridge to reach the head tank site.

Both No. 1 and No. 2 tunnels will be constructed driving through hard bedrock consisting of alternations of quartzite and slate, and concrete lining will be unnecessary except for portals and parts close to the ground surface of valley topography.

The head tank will be situated at a ridge of gentle slope where taluses are deposited. It is expected that the taluses are comparatively thick. The type of the head tank is to be such that it has a tall retaining wall concurrently serving as the tunnel portal at its back, and capacity is to be secured by increasing the area of the tank bottom.

As stated in Section 2.3 "Geological Structure and Collapse Area", there exists a fault across Rio San Juan in the downstream of the head tank from a place at EL. 3,780 m on the right bank to a place on the left bank about 50 m upstream of the intake of the existing Electro Peru hydro-power station. The fault forms a kind of long and narrow valley and narrow ridges on each side of the fault, where weathered alternating beds of slate and quatzite are exposed.

If the powerhouse is designed to build on the right bank at the just upstream of the existing intake, the penstock lines will be installed at valley or ridges of the said fault. If the penstock line is designed avoiding the said topography, the powerhouse must be considerably removed to the upstream resulting in decreasing head and generating capability.

In Alternative A, consequently, the upper portion of penstock from the head tank to the road is located at the fault valley where construction works seem to be not so difficult. The lower portion of it after the road to the powerhouse is designed at the fault ridge of the downstream side.

However, there will be serious problem if the penstock line is located on the fault zone. Special construction methods such as cable crane, etc. will be required and considerable quantity of the exposed rock will be excavated. Construction costs will be finally increased. (Photo 1-4-4)

The powerhouse will be at a location approximately 100 m down from the departmental road crossing the penstock at the opposite bank immediately upstream of the existing Electro Peru power station. This site corresponds to the end of a gully, and taluses at the contact portion with the river have been washed away by a flood to form a vertical slope of a height of 7 m. In order to develop a lot for the powerhouse here, large-scale earthmoving work and construction of a retaining wall and revetments will be necessary. Consequently, the construction cost will be increased. In addition to the fact that the access road to the powerhouse must be constructed from the vicinity of the powerhouse site of Alternative B, since construction must be done cutting the steep mountainsides of ravines, the construction cost will be increased from this aspect also.

(3) Alternative B (Midstream site)

The No. 1 open canal, No. 1 tunnel, and No. 2 open canal would be the same as in Alternative A. The No. 2 tunnel of length of 650 m would be constructed through the east side of the next large ridge from the head tank location of Alternative A to reach the site of the head tank in Alternative B.

The head tank would be located close to the ridge of a slope having a gradient of approximately 30 deg. For the reasons that it is provided at mid-height of a slope so that a large lot cannot be secured, and that it is close to a ridge where outcrops are seen here and there. Since the bedrock is laid in shallow place, the head tank is of cylindrical type.

At the slope of the penstock, the bedrock consists of alternations of slate and quartzite with the strike of the bedding dipping in the opposite direction from the dip of the slope overlying which there are taluses and terrace deposits. It may be said that the slope is comparatively stable. The deposits contain angular debris and rounded gravels, and ample bearing capacity can be expected. While the gradient of the slope becomes gentler the lower down on the slope so that the topography is ideal for a penstock.

The gradient from the departmental road to the head tank is gentle at 20 to 30 deg. and it is possible to provide an access road, the ease of construction being the greatest in comparison with the other alternatives. (Photo. 1-4-5)

The powerhouse would be located at the opposite bank from the existing Electro Peru power station and an access road can easily be connected from the existing departmental road. However, the tailrace would be located at the upstream edge of the town, and in consideration of harmony with the residents, the cost of restoration of revetments lost due to flood is to be included as part of the construction cost. (Photos. 1-4-6, 1-4-7)

(4) Alternative C (Downstream site)

The No. 2 tunnel of 900 m would bore through the ridge of the head tank of Alternative B, and connect to the No. 3 open canal 100 m in length at the upper part of the gully of the penstock in Alternative B. The No. 3 open canal then reaches the No. 3 tunnel at the next ridge, which subsequently reaches the head tank site at the eastern slope of this ridge.

The No. 3 tunnel also runs through an alternation of quartzite and slate, the bedrock being harder than at any other tunnel route.

With regard to the head tank, a type identical to that in Alternative B is conceivable based on the topographical conditions. Because of the geological condition of the supporting basement, it is necessary to examine a proposal to provide the head tank widening the waterway inside the No. 3 tunnel.

The upper part of the penstock would be installed on top of a bedding plane of quartzite exposed over a length of approximately 180 m, the slope being steep at about 40 deg. Because of this reason, it is not possible to provide an access road, and the construction cost will be increased with respect to both foundation work and installation work.

The lower part has a comparatively gentle slope, and it is possible for an access road to be provided, but the length is long, and in addition, the topography has much relief. In addition to a large volume of earthwork being necessary in order to be able to install the penstock line, there is a risk of debris produced by collapse of the slope in the vicinity of EL. 3,900 m sweeping away the penstock and a countermeasure will be required.

Furthermore, practically all of the lower part is being used for pasture with scatterred houses, and it is expected there will be difficult problems with respect to utilization of the pasture after installation of the penstock. The length of the penstock is 835 m, the longest in all of the proposals. (Photo. 1-4-8)

The powerhouse is to be located by the side of the departmental road at the downstream edge of Huallanca, is easily approached. There will be no adverse effects on the town. (Photo. 1-4-9)

(5) Conclusions

As studied in Section 5.4, the installed capacity of Huanzala Power Station required from the standpoint of demand and supply balance is 4,200 kW. On selecting the maximum power discharge so that the maximum output would be 4,200 kW in each alternative. The results are as shown in Table 1-4-1.

After the particulars and the topographical and geological features at the respective site are considered, the construction costs of the individual alternatives are computed as shown in Table 1-4-2.

According to these studies, the maximum power discharges of the various alternatives are 2.6 cu.m/sec, 2.2 cu.m/sec, and 2.0 cu.m/sec, respectively. Since the cross-sectional area of the headrace is unchanged in all alternatives, there are no differences in the unit construction costs per meter. Therefore, the difference in construction costs is governed by the length of the headrace, and the construction conditions determined by the topographical and geological conditions of the head tank, penstock and powerhouse.

Alternative A has a short length in the headrace but the civil works cost of the penstock is high, while the topographies of both the powerhouse and powerhouse access road are adverse and comparatively costly.

In Alternative C, since both headrace and penstock are longer than in the other alternatives the construction cost will be the highest, and the result is that Alternative B shows the lowest construction cost.

On evaluation of the construction costs of the three alternatives by unit cost per annual energy production for the sake of convenience, the results are 107.8 yen/kWh for Alternative A, 92.8 yen/kWh for Alternative B, and 102.6 yen/kWh for Alternative C. Since Alternatives B and C will affect operation of the Electro Peru power station in the dry season, the amount of reduction in the power generation must be taken into account in the evaluation. That is, after start-up of Huanzala Power Station, the water for power generation at the ElectroPeru power station will be only the runoff from the residual catchment area (71.8 sq.km) downstream of the intake of Huanzala Power Station. As seen from the duration curve given in Section 1.3, the catchment area of the Huanzala Power Station intake is 153.7 sq.km, and the extreme dry-season runoff is 0.83 cu.m/sec. The extreme dry-season runoff of the above-mentioned residual catchment area will be 0.38 cu.m/sec.

Since the maximum power discharge of the Electro Peru Power Station is 0.7 cu.m/sec (maximum output 156 kW), the station can at present demonstrate 100% of its capacity even in the dry season. But after start-up of Huanzala Power Station, there will be a shortage of 0.32 cu.m/sec at maximum, and power generation will be subject to restriction. Calculated by the runoff duration curve, the period during which power generation will be restricted will be 75 days and the electric energy loss will be approximately 78 MWh/yr. (At present, the Electro Peru power station is operated only during the nighttime, but this calculation is on the basis of available energy production.)

Regarding Alternative B and C, if the construction costs per kWh are reevaluated deducting the energy loss of the Electro Peru power station from the annual available energy production, the results will be 93.0 yen/kWh for Alternative B and 102.8 yen/kWh for Alternative C, and the superiority of Alternative B remains intact.

As a result of the above studies the site in Alternative B was selected for the powerhouse site of Huanzala Power Station in this Project.

1.4.3 Turbine and Generator Types and Number of Units

The types of turbine and generator and the number of units were decided upon consideration of the conditions below.

(a)	Hydraulic conditions:	Effective head 242 m, maximum power discharge 2.2 cu.m/sec, extreme minimum discharge 0.83 cu.m/sec
(b)	Power generation type:	Runoff-river type
(c)	Development objective:	Private generating plant for the Mine and electric power supply for public use in the region
(d)	Electric power system:	Independent power system consisting of parallel operation with Huanzala Mine Diesel Power Station
(e)	Position in electric power system :	As main power station in Huanzala Electric Power System, frequency adjustment and voltage regulation of the system
(f)	Load condition :	Maximum power demand forecast for system (2006) 5,600 kW, minimum power demand forecast (1987 holiday) 830 kW

(1) Turbine Type

A horizontal-shaft, 1-runner, 2-nozzle Pelton turbine was adopted taking into consideration the above-mentioned conditions.

(2) Generator Type

The generator to be coupled to the turbine is a horizontal-shaft, 3-phase, alternating current, synchronous generator, and excitation is by a brushless system because of simplification of maintenance.

(3) Number of Turbine and Generator Units

The number of turbine and generator units conceivable for this case is either one or two units. Since the turbiine type adopted was a horizontal-shaft, l-runner, 2-nozzle Pelton, it was decided to adopt the one unit proposal based on reasons described below:

(a) A two-nozzle Pelton turbine is capable of two-nozzle operation at 50% to 100% load, and one-nozzle operation at 50% to 20% load. Reduction in efficiency is small and it is possible to cope with a wide range of load variation. (See Figs. 1-4-5 and 1-4-6)

(b) With a horizontal-shaft, l-runner, 2-nozzle Pelton turbine, large-scale repair work (once annually or biannually) such as runner replacement can be done with 10 to 12 hours

(c) The case of a single unit, compared with two units, is approximately 20% cheaper in turbine, generator, and appurtenant equipment costs.

(d) A single unit would be somewhat less reliable than two units in case there should be trouble such as faulting. However, the accident rate of equipment of this class is low according to recent statistics.

Table 1-4-1 Dimensions

Item	Unit	Case A	Case B	Case C
Intake Water Level (E.L.)	m	3,802.5	3,802.5	3,802.5
Head Tank Water Level (E.L.)	"	3,796.5	3,796.5	3,794.5
Center of Turbine (E.L.)	"	3,585.0	3,547.3	3,508.0
Normal Head	m	211.5	249.2	286.5
Effective Head	m	206.0	242.0	273.0
Maximum Discharge	m^3/s	2.6	2.2	2.0
Output	KW	4,200	4,200	4,200
Annual Energy Production	KWh	$30,941 \times 10^3$	$32,187 \times 10^3$	$33,828 \times 10^3$

Table 1-4-2. Construction Cost Comparison for Huanzala Hydro-Power Project

UNIT 10^6 ¥

ITEM	CASE A		CASE B		CASE C	
	Cost	Remarks	Cost	Remarks	Cost	Remarks
1. Civil Works						
(1) Intake	133	Dam. Intake. Sand Basin	133	Dam. Intake. Sand Basin	133	Dam. Intake. Sand Basin
(2) Headrace Canal	243	3,000m	243	3,000m	251	3,100m
(3) Headrace Tunnel	345	1,400m	406	1,650 m	664	2,700 m
(4) Head Tank	147	Head Tank 320 m^3 Spillway 530 m	119	Head Tank 280 m^3 Spillway 374 m	169	Head Tank 240 m^3 Spillway 670 m
(5) Penstock	521	Installation 680 m Cable Crane 900 m	239	Installation 664 m	394	Installation 835 m Cable Crane 250 m
(6) Powerhouse	216		116		96	
(7) Miscellaneous	120	Road 4000 m	128	Road 4500m Protection Wall 1200m	108	Road 6500m
TOTAL	1725		1384		1815	
2. Electric Works						
(1) Electrical Equipment	920	Pelton Turbine 4450kW A.C. Generator 5200kVA. 1 unit	920	Pelton Turbine 4450kW A.C. Generator 5200kVA. 1 unit	920	Pelton Turbine 4450kW A.C. Generator 5200kVA. 1 unit
(2) Huanzala Substation	66	Outdoor Substation 5200 kVA	66	Outdoor Substation 5200 kVA	66	Outdoor Substation 5200 kVA
(3) Transmission Line	99	33kV. 1cct. 12km	96	33kV. 1cct. 10km	99	33kV. 1cct. 12 km
TOTAL	1085		1082		1085	
3. Other Works						
(1) Compensation	60		70		100	
(2) Engineering Fee	280		280		280	
(3) Administration Cost	100		100		100	
(4) Contingency	86	Civil Cost x 5%	70	Civil Cost x 5%	90	Civil Cost x 5%
TOTAL	526		520		570	
GRAND TOTAL	3336		2986		3470	

※ Exchange rate 1 U$ = 230円

Table.1-4-3 Energy Production at Huanzala P.S. CASE A

(KWH)

	1	2	3	4	5	6	7	8	9	10
0	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
10	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
20	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
30	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
40	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
50	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
60	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
70	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
80	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
90	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
100	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
110	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
120	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
130	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
140	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
150	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
160	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
170	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
180	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
190	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
200	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
210	102038	102038	102038	102038	102038	102038	102038	102038	102038	102038
220	102038	102038	99291	98506	95759	94189	92619	91050	88695	87125
230	86733	86340	85948	85555	84770	83985	83593	82416	82023	81238
240	80846	80846	80453	80061	79668	78883	77706	77706	76921	76529
250	76136	75351	74959	74566	74174	73782	73389	72997	72604	71427
260	70642	69857	69465	69072	68680	68287	67895	67502	66325	66325
270	65932	65932	65540	65148	64363	64363	63970	62793	62793	62400
280	62400	62008	61615	61223	61223	60831	60438	60438	60046	59653
290	59261	59261	58476	57298	56121	56121	55336	54159	54159	52981
300	52981	52197	51019	50627	50627	49842	49449	48272	47880	47487
310	46702	46702	46310	45917	45525	45525	45132	44347	44347	43955
320	43563	43170	42385	42385	41993	41993	41600	41208	40815	40423
330	40423	40423	39638	39246	39246	39246	39246	38853	38853	38461
340	38068	38068	38068	38068	38068	37676	37283	36891	36891	36891
350	36891	36498	36498	36498	36498	35713	35321	35321	34536	34536
360	34144	33751	33359	32966	32574					

* TOTAL 30940700

Table.1-4-4 Energy Production at Huanzala P.S. CASE B

(KWH)

	1	2	3	4	5	6	7	8	9	10
0	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
10	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
20	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
30	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
40	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
50	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
60	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
70	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
80	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
90	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
100	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
110	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
120	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
130	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
140	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
150	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
160	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
170	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
180	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
190	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
200	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
210	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
220	101429	101429	101429	101429	101429	101429	101429	101429	101429	101429
230	101429	101429	100968	100507	99584	98662	98201	96818	96357	95435
240	94974	94974	94513	94052	93591	92669	91286	91286	90364	89903
250	89442	88520	88059	87597	87136	86675	86214	85753	85292	83909
260	82987	82065	81604	81143	80682	80221	79760	79299	77916	77916
270	77455	77455	76994	76533	75610	75610	75149	73766	73766	73305
280	73305	72844	72383	71922	71922	71461	71000	71000	70539	70078
290	69617	69617	68695	67312	65929	65929	65007	63623	63623	62240
300	62240	61318	59935	59474	59474	58552	58091	56708	56247	55786
310	54864	54864	54403	53942	53481	53481	53020	52097	52097	51636
320	51175	50714	49792	49792	49331	49331	48870	48409	47948	47487
330	47487	47487	46565	46104	46104	46104	46104	45643	45643	45182
340	44721	44721	44721	44721	44721	44260	43799	43338	43338	43338
350	43338	42877	42877	42877	42877	41955	41494	41494	40571	40571
360	40110	39649	39188	38727	38266					

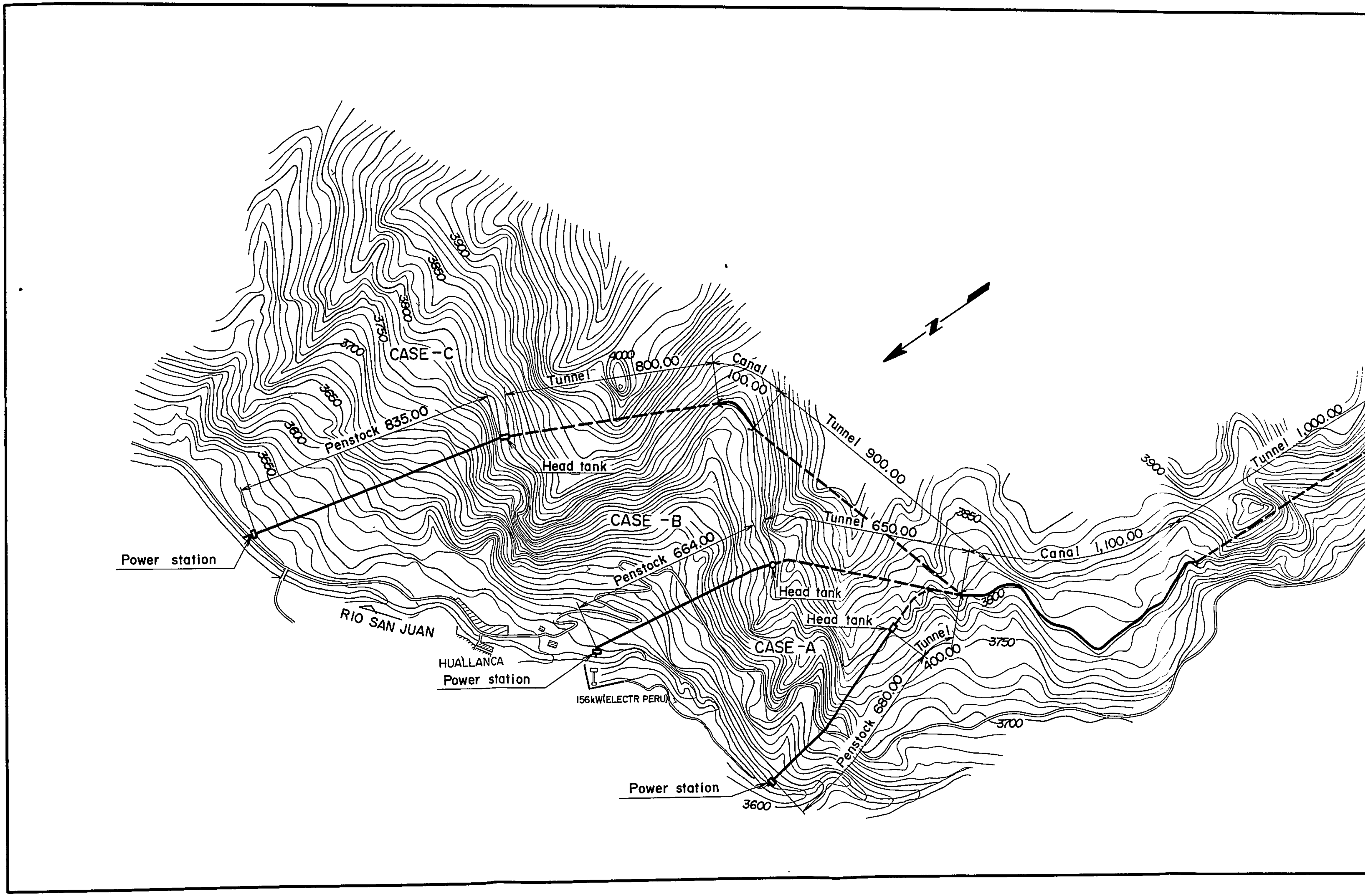
* TOTAL 32187000

Table.1-4-5 Energy Production at Huanzala P.S. CASE C

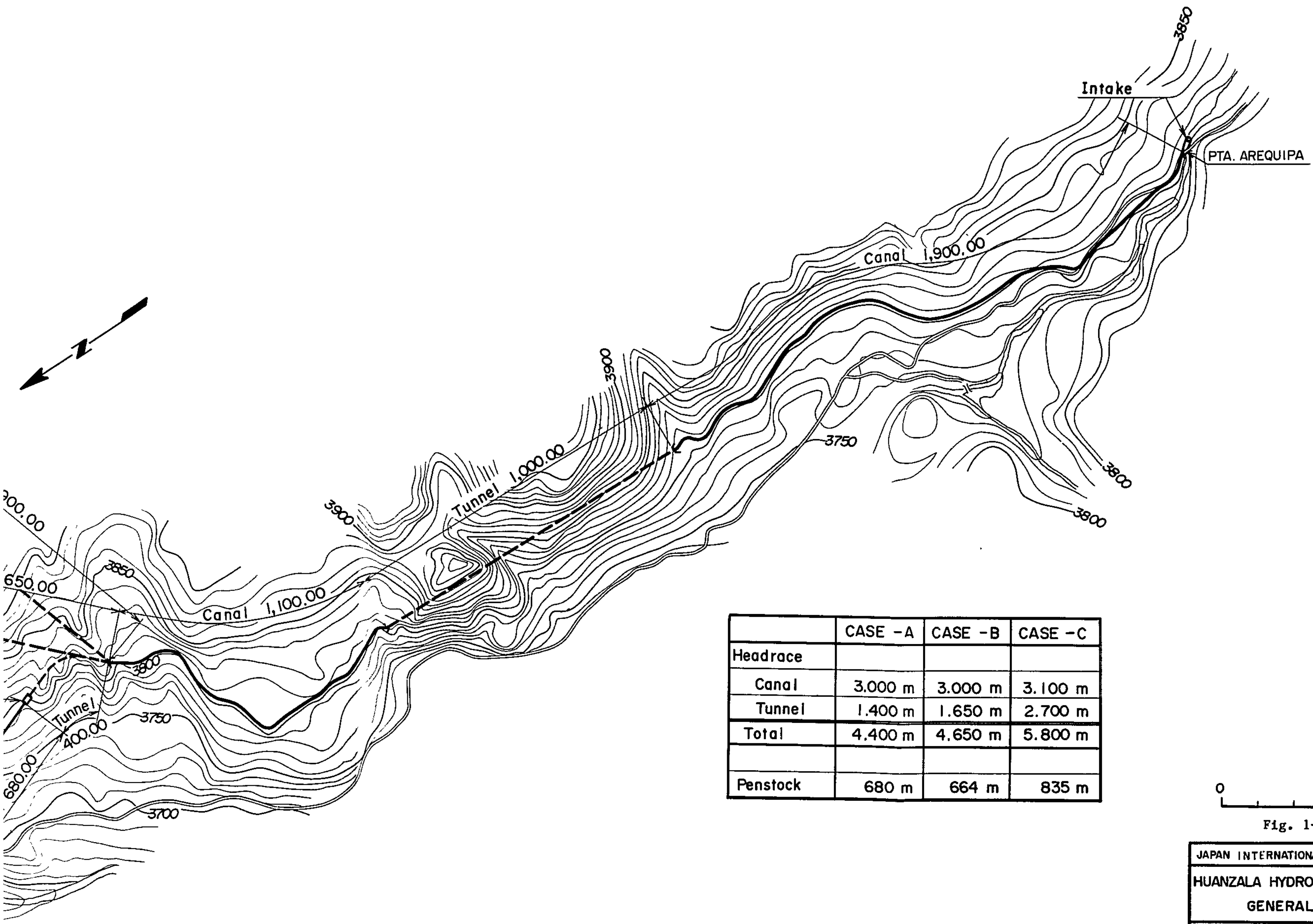
(KWH)

	1	2	3	4	5	6	7	8	9	10
0	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
10	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
20	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
30	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
40	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
50	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
60	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
70	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
80	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
90	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
100	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
110	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
120	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
130	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
140	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
150	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
160	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
170	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
180	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
190	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
200	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
210	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
220	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
230	104020	104020	104020	104020	104020	104020	104020	104020	104020	104020
240	104020	104020	104020	104020	104020	104020	102979	102979	101939	101419
250	100899	99859	99339	98819	98299	97778	97258	96738	96218	94658
260	93618	92577	92057	91537	91017	90497	89977	89457	87897	87897
270	87376	87376	86856	86336	85296	85296	84776	83216	83216	82696
280	82696	82176	81655	81135	81135	80615	80095	80095	79575	79055
290	78535	78535	77495	75934	74374	74374	73334	71774	71774	70213
300	70213	69173	67613	67093	67093	66052	65532	63972	63452	62932
310	61892	61892	61372	60851	60331	60331	59811	58771	58771	58251
320	57731	57211	56171	56171	55651	55651	55130	54610	54090	53570
330	53570	53570	52530	52010	52010	52010	52010	51490	51490	50970
340	50450	50450	50450	50450	50450	49929	49409	48889	48889	48889
350	48889	48369	48369	48369	48369	47329	46809	46809	45769	45769
360	45249	44728	44208	43688	43168					

* TOTAL 33828800



CASE-C
CASE-B
CASE-A
Tunnel 800.00
Canal 100.00
Tunnel 900.00
Penstock 835.00
Head tank
Power station
Tunnel 650.00
Canal 1,100.00
Tunnel 1,000.00
Penstock 664.00
Head tank
Head tank
Tunnel 400.00
Penstock 680.00
Power station
Power station
RIO SAN JUAN
HUALLANCA
156kW(ELECTR PERU)
N



	CASE -A	CASE -B	CASE -C
Headrace			
Canal	3.000 m	3.000 m	3.100 m
Tunnel	1.400 m	1.650 m	2.700 m
Total	4.400 m	4.650 m	5.800 m
Penstock	680 m	664 m	835 m

Fig. 1-4-1

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT

GENERAL PLAN

EPDC International Ltd. TOKYO JAPAN

D.R.; SUBMITTED;
T.R.; RECOMMENDED;
C.K.; APPROVED;

— — —

LOCATION	DATE	DESCRIPTION	BY
REVISION			

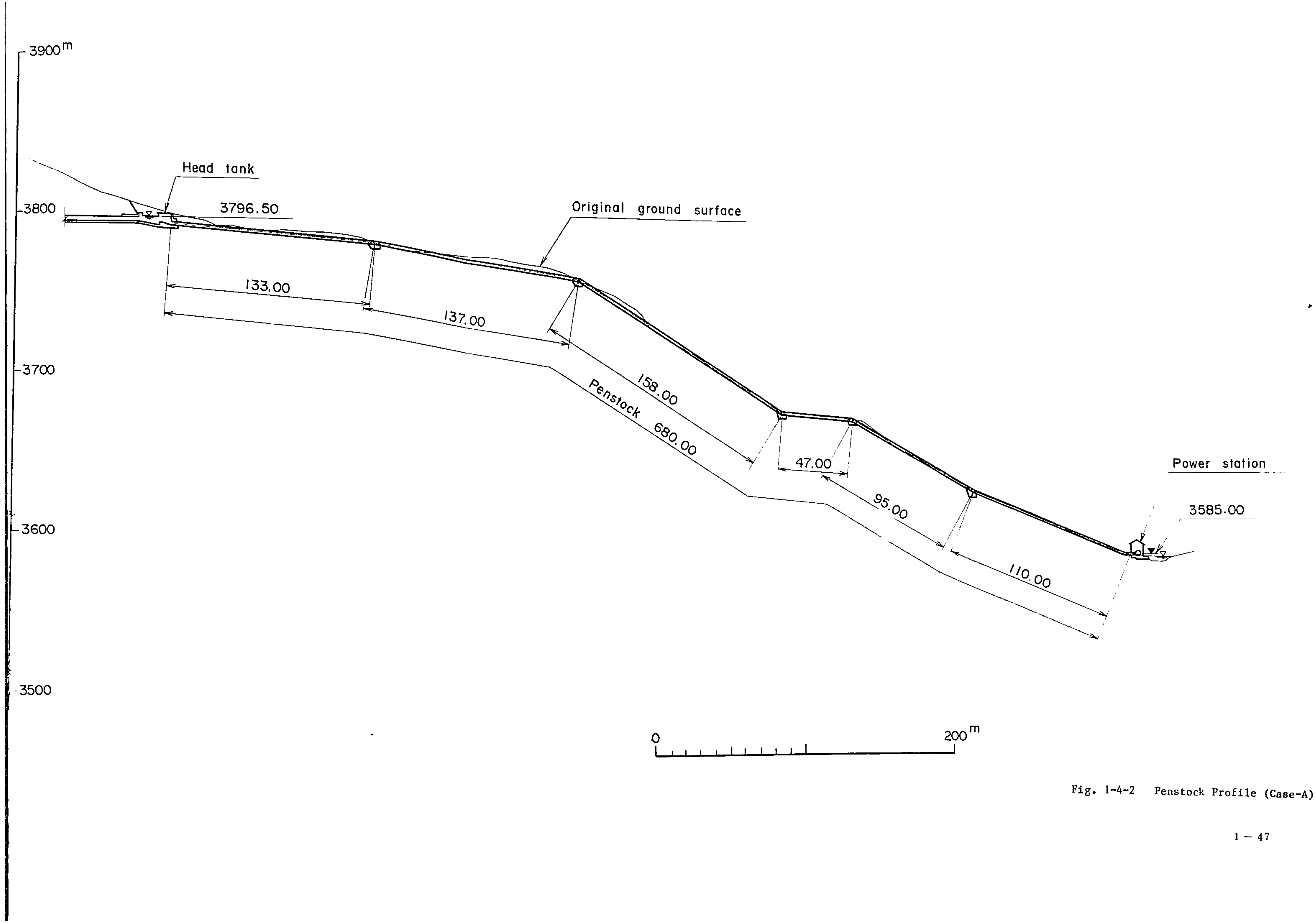


Fig. 1-4-2 Penstock Profile (Case-A)

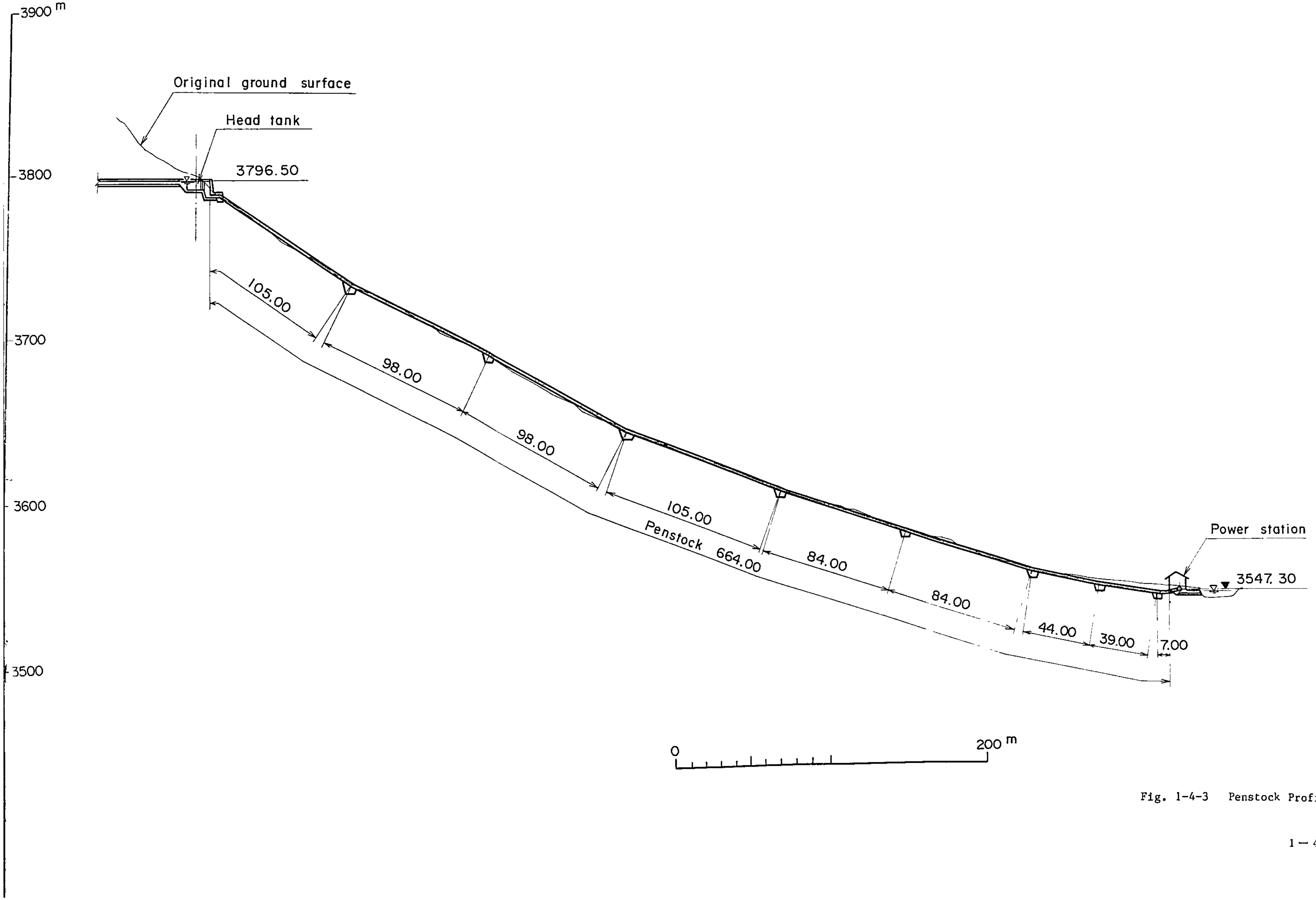


Fig. 1-4-3 Penstock Profile (Case-B)

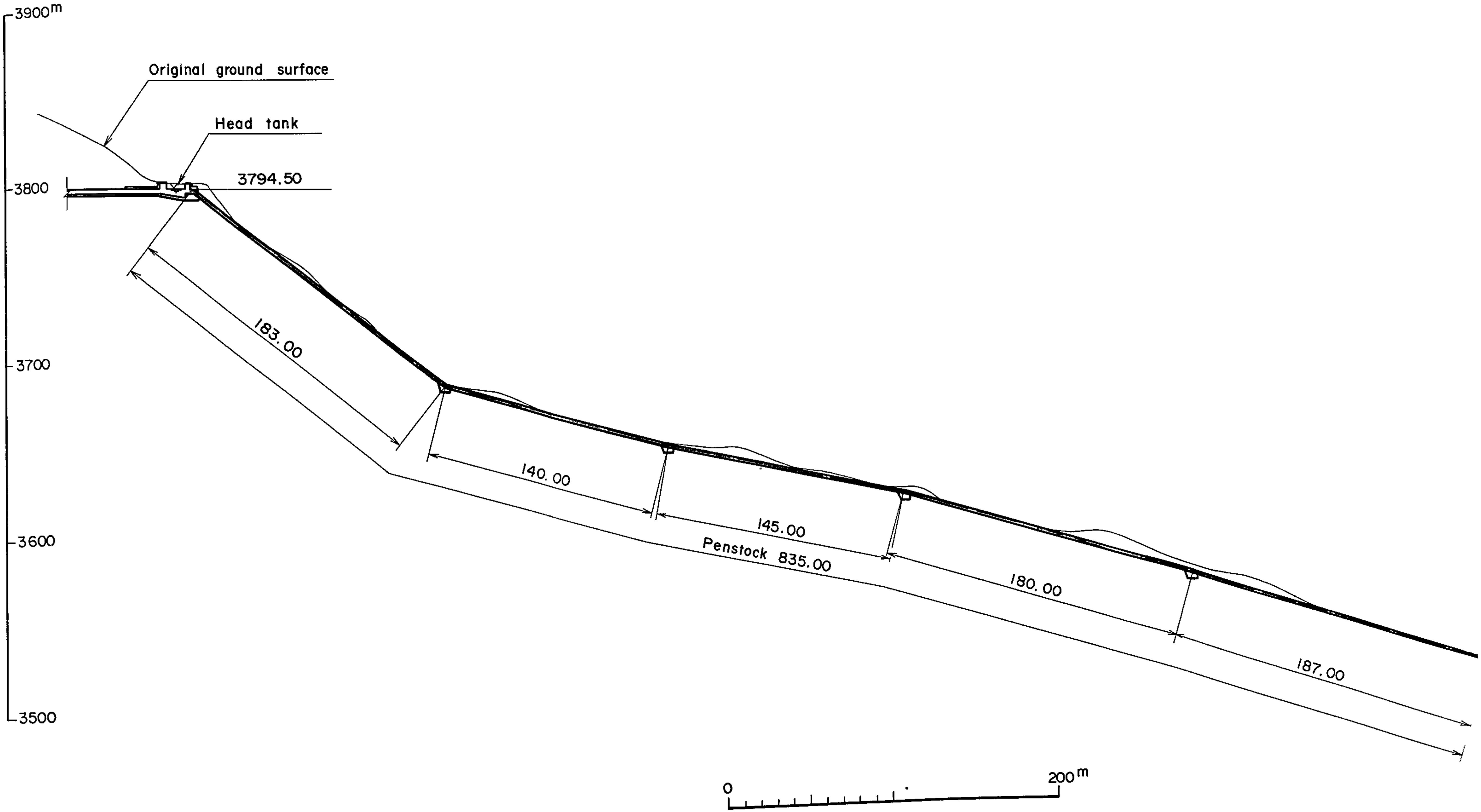

3900m
3800
3700
3600
3500
Original ground surface
Head tank
3794.50
183.00
140.00
145.00
180.00
187.00
Penstock 835.00
0
200m

Fig.

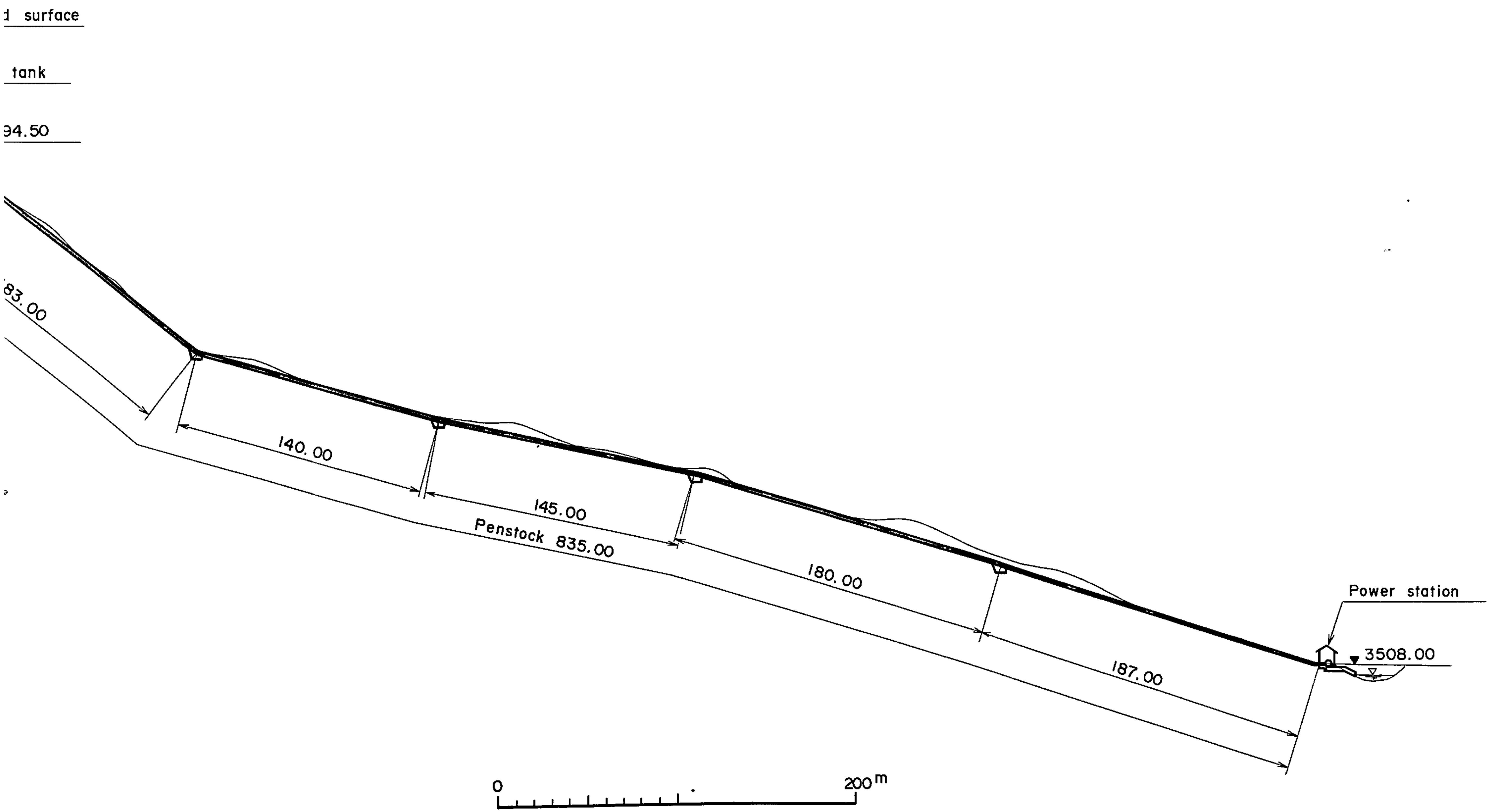


Fig. 1-4-4 Penstock Profile (Case-C)

Fig. 1-4-5 Efficiency Curve of Pelton Turbine (Alt. 1 unit)

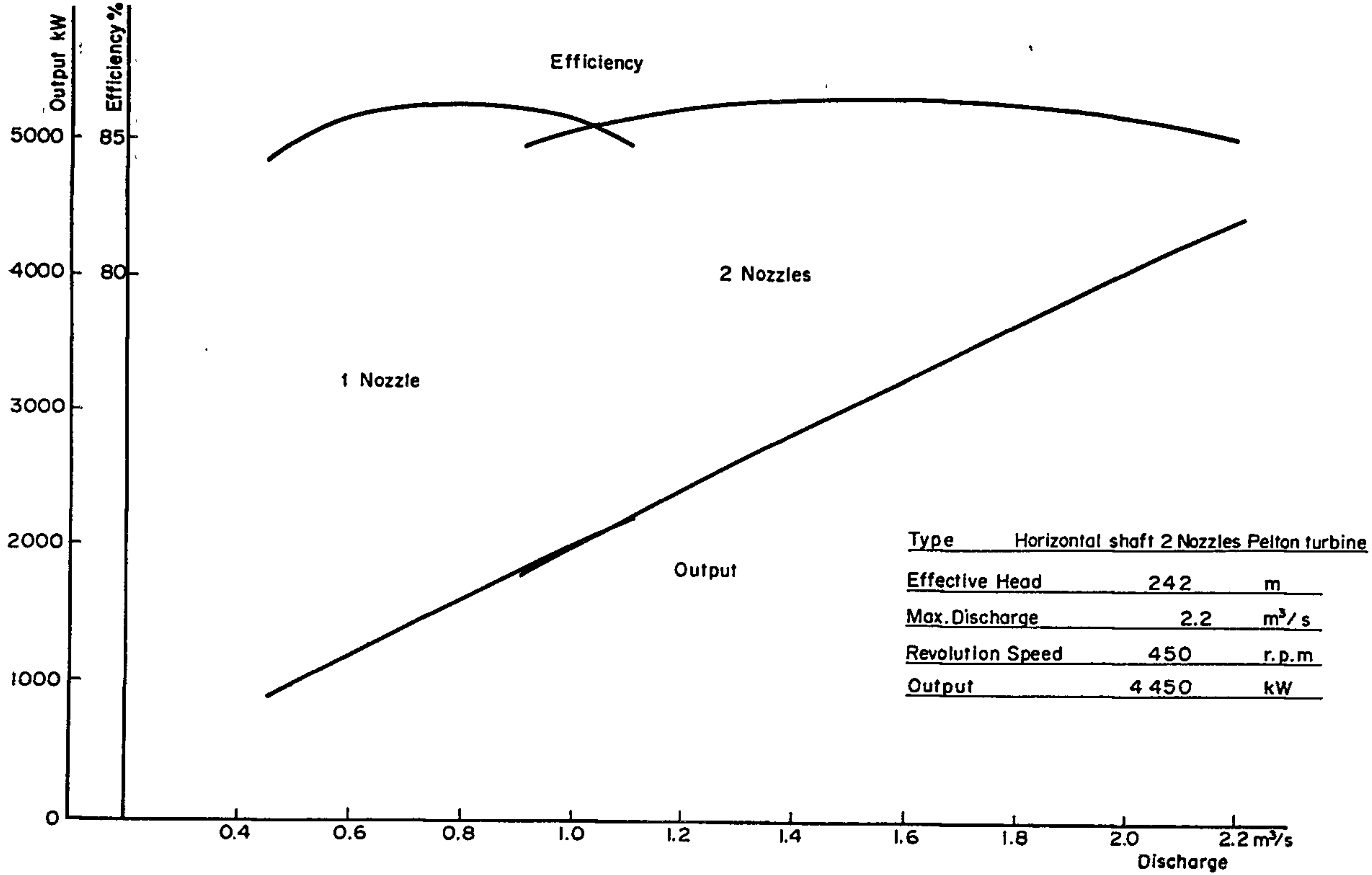
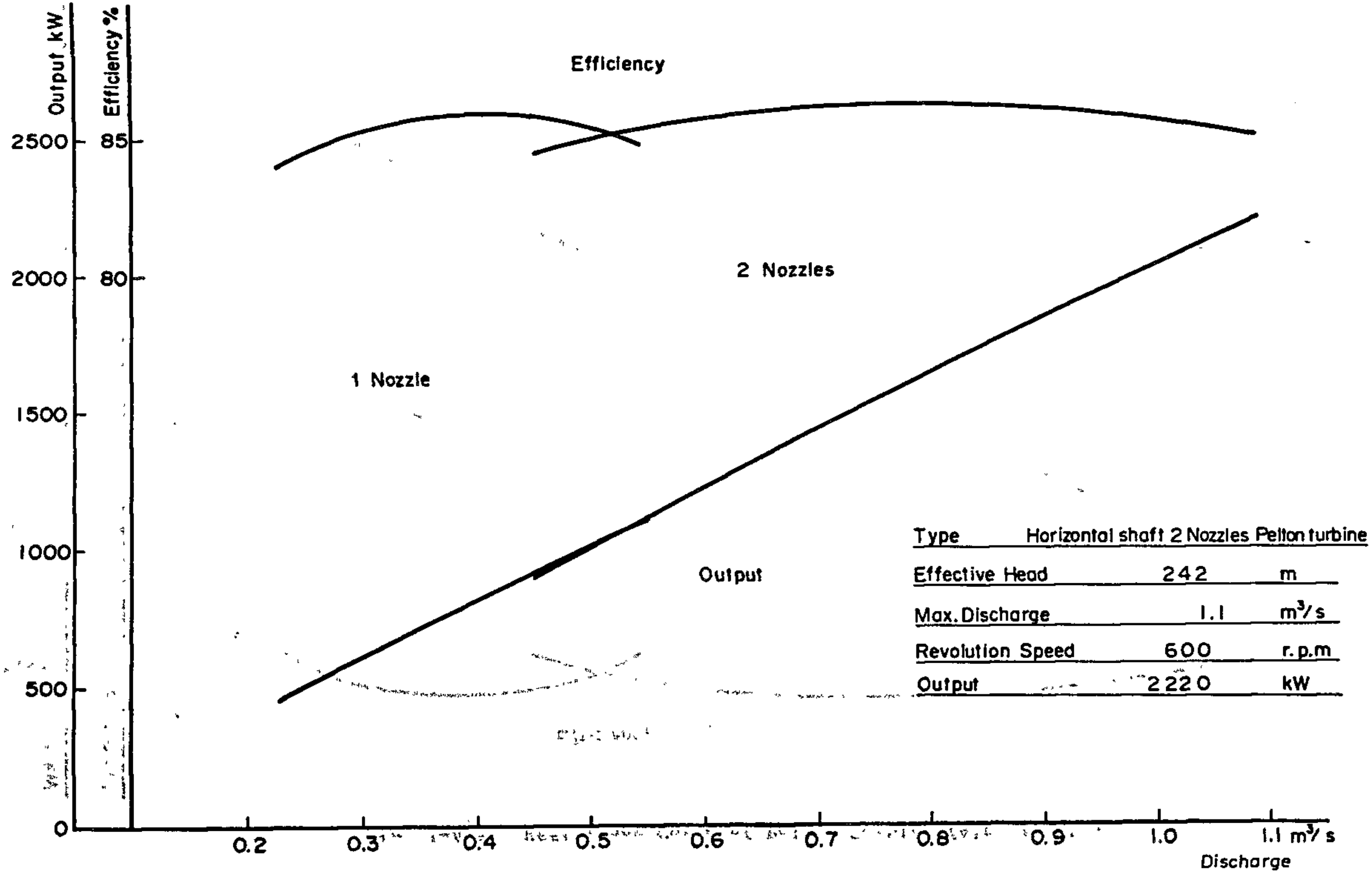


Fig. 1-4-6 Efficiency Curve of Pelton Turbine (Alt. 2 units)



CHAPTER 2

TOPOGRAPHY AND GEOLOGY

CHAPTER 2 TOPOGRAPHY AND GEOLOGY

Photo 2-2-1 Cracks developed in Bed of Quartzite

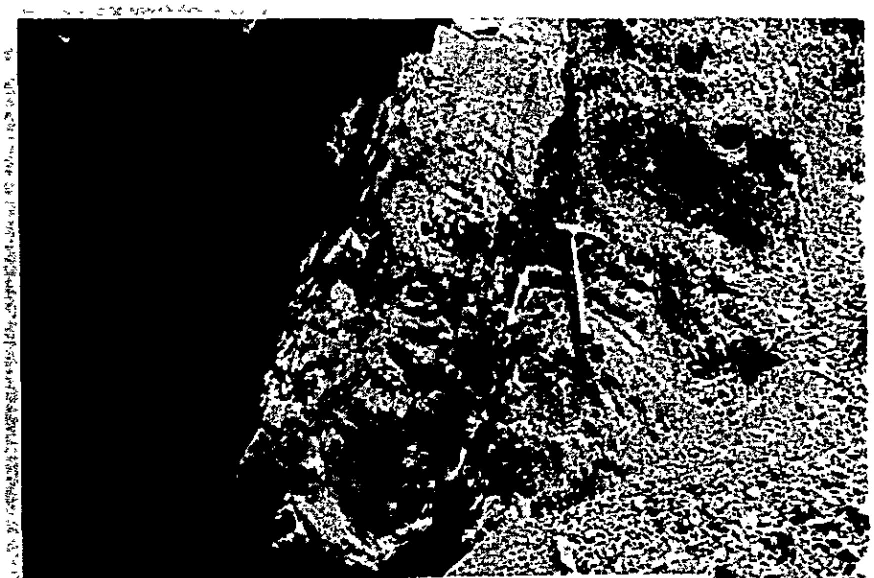


Photo 2-2-2 Weathered Slate and Quartzite

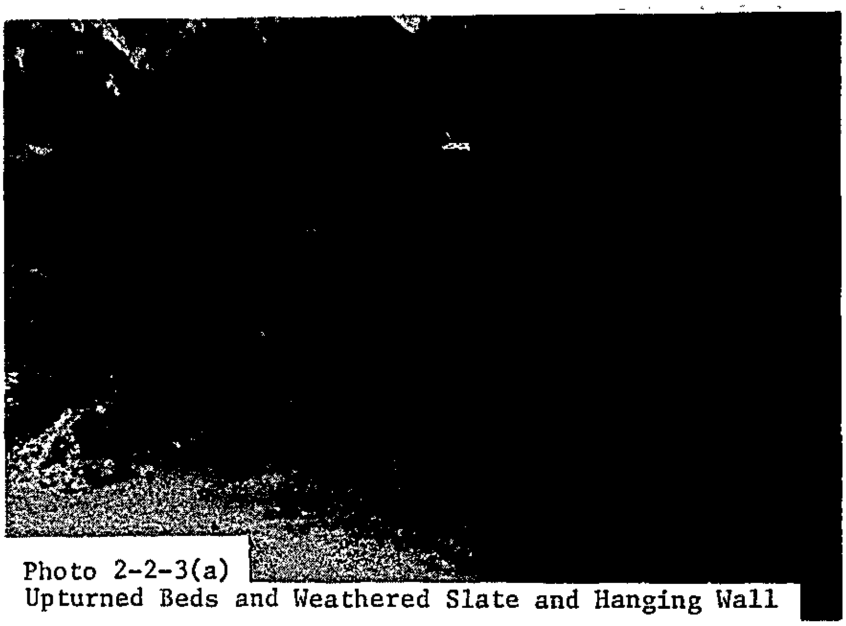


Photo 2-2-3(a)
Upturned Beds and Weathered Slate and Hanging Wall

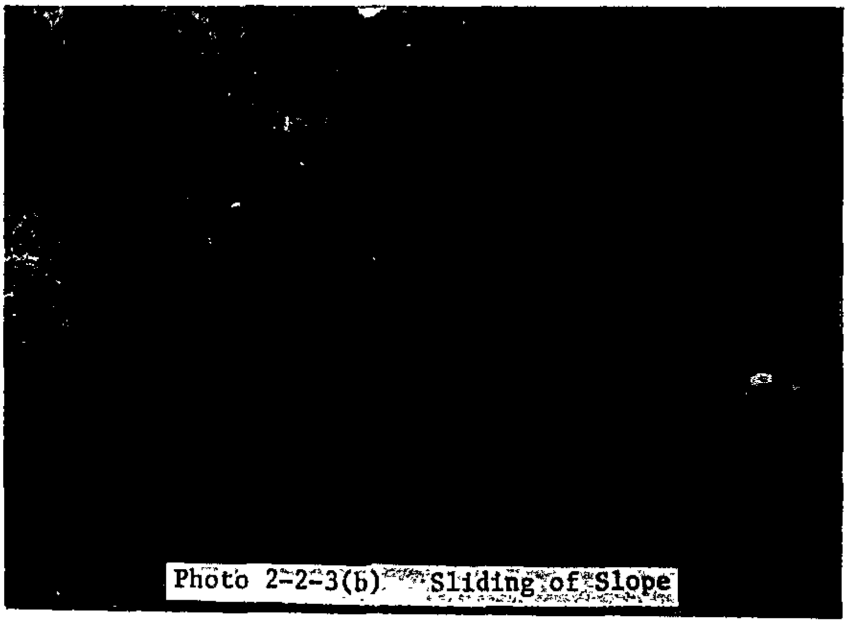


Photo 2-2-3(b) Sliding of Slope

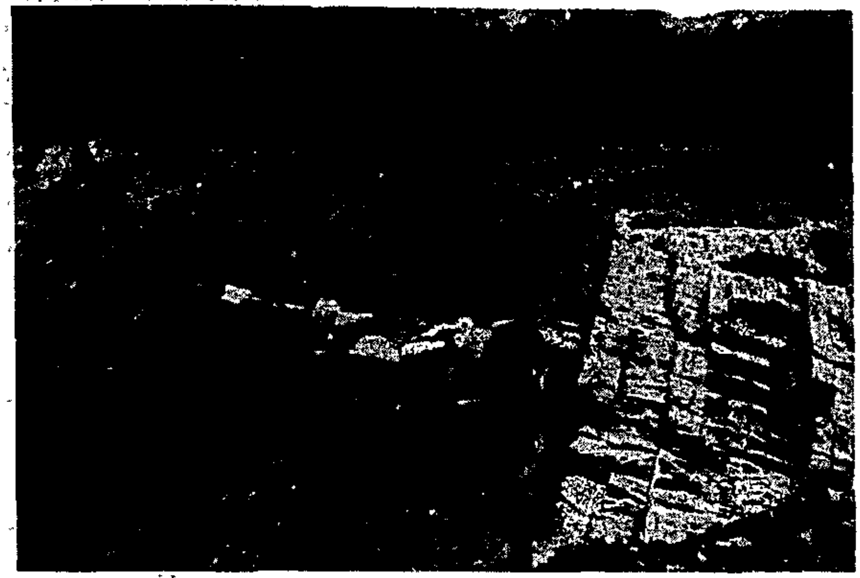


Photo 2-2-3(c) Alternating Beds of Slate and Quartzite

Photo 2-2-4 Glacial Deposit of Quartzite

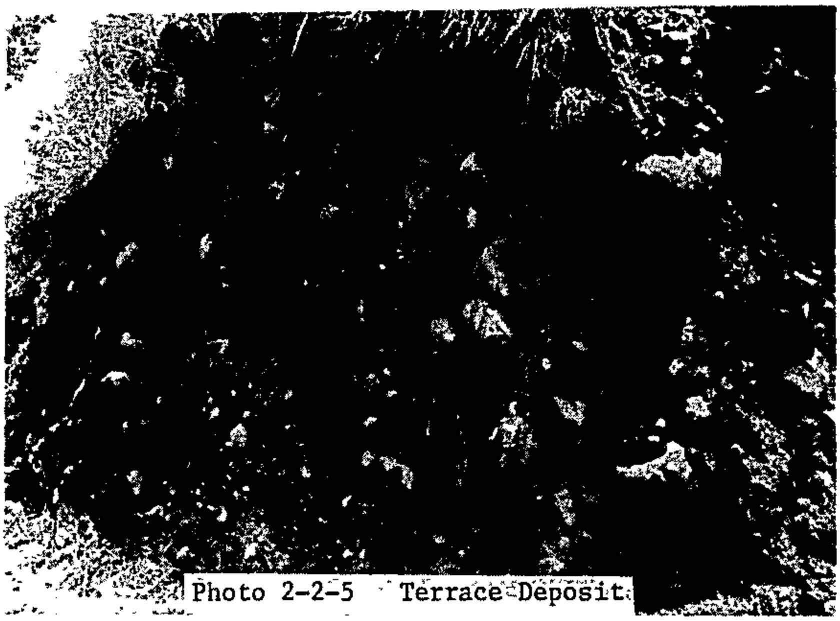


Photo 2-2-5 Terrace Deposit

Photo 2-2-7 Talus Deposit

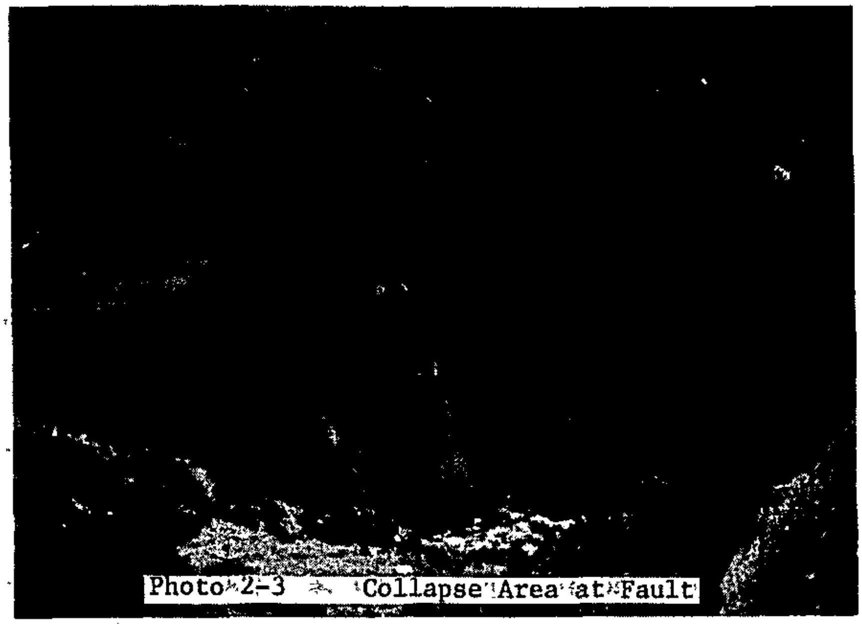


Photo 2-3 Collapse Area at Fault

CHAPTER 2 TOPOGRAPHY AND GEOLOGY

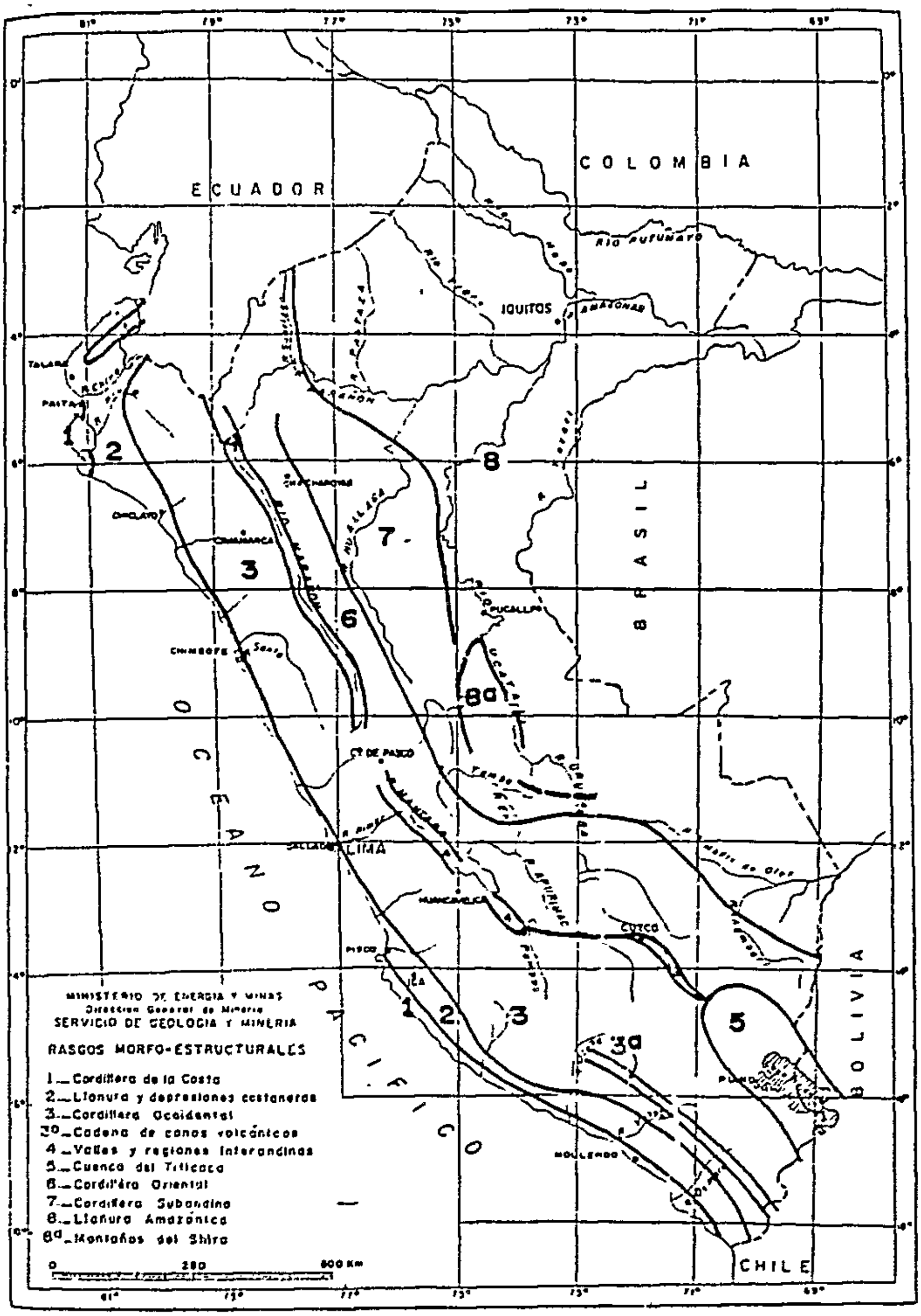
2.1 Outline of Topography

The Andes mountain system consists of the three mountain ranges of Cordillera Occidental, Cordillera Oriental and Cordillera Sub-Andia. Of these, the Cordillera Occidental is composed of Mesozoic and Tertiary rocks and is the highest range in the Andes mountain system. Mt. Huascaran in this range is the tallest peak in Peru (6,768 m) and is permanently capped by snow.

The Cordillera Occidental rises high up as the divide between the Pacific Ocean and Atlantic Ocean sides. The project site is in the drainage basin of the Rio San Juan which springs from the Cordillera Occidental and is located at around 3,500 to 4,000 m. The Rio San Juan flows in the north-northeast direction, joins with the Rio Santa Rosa at the town of Huallanca to become the Rio Viscarra, and feeds the Atlantic Ocean via the Rio Maranon and the Rio Amazonas. The Rio San Juan upstream from the vicinity of Puente Arequipa has a large width and the gradient of the river bed is gentle, but the downstream part is steep and the width of the river become narrowed down.

Since the Rio San Juan flows in a direction perpendicular to the strike of beds, the mountainsides of both banks of the river are steeply sloped. However, glacial deposits and terrace deposits are well-exposed at the right-bank side, and these are distributed to form gentle slopes. The directions of valleys and ridges within the area surveyed reflect the geological structure well and are formed from southeast to northwest. Valleys are formed to roughly coincide with synclinal axis of fold structures, and ridges with anticlinal axis.

Fig. 2-1-1 Classification of Landform of Peru



(Geology of Peru)

2.2 Outline of Geology

Sedimentary rocks of the Lower Cretaceous System Goyllarisquizga Group are distributed in the vicinity of the project site.

Cretaceous rocks are extensively distributed throughout Peru, and it is said that 75% of Mesozoic rocks are Cretaceous.

The Goyllarisquizga Group may be divided into the Chimu Formation, Santa Formation, Carhuaz Formation, Pariahuanca Formation, and Pariatambo Formation. These strata comprise fold structures having been subjected to severe folding accompanying the orogenic movement of the Andes.

The formation at the project site corresponds to the Chimu Formation and is comprised of quartzite, slate and alternating beds of quartzite and slate.

The Chimu Formation shows a belt-like distribution in the northwest-southeast direction with alternating anticlines and synclines in the Rio San Juan drainage basin. There are unconsolidated glacial deposits, terrace deposits, fan deposits, and talus deposits covering the slate and quartzite in the project site. Stratigraphic sequence in the project site are as given in Table 2-2-1 below.

Table 2-2-1 Geological Formation in the Project Area

Age		Formation	Rock-facies	Remarks
Cenozoic	Quarternary	Talus deposit	Talus deposit	Consisted of angular debris, clayey soil and sandy soil
		Fan deposit	Fan deposit	Consisted of fan gravel with rounded gravel of 0.3 - 1.0 m in size
		Terrace deposit	Terrace deposit	Consisted of clayey soil and sandy soil with rounded gravel of 0.1 - 0.3 m in size
		Gracial deposit	Gracial deposit	Large boulder app. 2 m in size of quartzite contained
Mesozoic	Cretaceous	Chimu formation	Quartzite Slate Alternating beds of quartzite and slate	Distributed in the direction of NW - SE. Anti-cline and syn-cline alternated

2.2.1 Quartzite (Ss)

Quartzite is distributed around the No. 1 tunnel and the No. 1 open canal. It presents a white to milky white color and is comprised of fine to medium quartz grains. It has a strike of N30°-40°W and a dip of 60°-70° northeast-southwest reflecting the anticlinal-synclinal structure. Cracks have been formed in the quartzite parallel to the bedding planes at intervals of 10 to 60 cm. Although the cracks can be broken open with strong blows of a hammer, they are more or less tight.

Rock fragments give light metallic sounds when tapped with a hammer and are of a hardness that they can be broken by strong blows. Slate is intercalated in the quartzite. There are cases when the slate at outcrops has been subjected to weathering to become fragmented and will crumble under a hammer, but in the vicinity of the junction between the Azulmina and the Rio San Juan fresh slate is exposed, and this is of a hardness that it can be broken by a strong blow of a hammer. (Photo 2-2-1)

2.2.2 Slate (Sl)

Slate has a strike of N30°W and dips 50°-60° to the southwest. The color is black. Slate and veryfine sandstone are rarely interbedded.

Slate at the ground surface has been subjected to weathering and is fragmented so that a hammer will stick into it, while pieces of rock are readily broken into small fragments when struck by a hammer. The rock is relatively soft compared with quartzite and is susceptible to erosion, and gentle slopes of talus are formed. (Photo 2-2-2)

2.2.3 Alternating Beds of Quartzite and Slate

The alternation of beds are divided into three units and shown on a geological map.

(a) Alternating Beds of Quartzite and Slate, 1 (Al_1)

This unit is distributed at the southwest side along a fault. The strike of the beds is N25°-30°W, with dip being 50°-60° to the southwest. The alternation of beds are in thicknesses of approximately 1 m, but there are occasionally cases of quartzite 3 m to 4 m in thickness being interposed. Cracks have been formed in the quartzite parallel to the bedding plane. Pieces of the rock can be broken with strong hammer blows. The slate is slightly phyllitic and has easily detached cracks parallel to the bedding plane. Slate close to the fault plane has been subjected to weathering and a hammer will readily stick into the rock. Furthermore, upturning of beds due to faulting can be confirmed suggesting disturbance of this unit along the fault. (Photo 2-2-3(a))

(b) Alternating Beds of Quartzite and Slate, 2 (Al_2)

The alternation of beds are in thicknesses of 30 to 50 cm. The distribution is at the northeast side of the fault with the strike of the beds N25°-35°W and dip 55°-70° to the southwest. There is a trend seen of the dip becoming steeper with increasing proximity to the fault. Bedding planes show close contact, but are readily detached. Especially, at places where the slope of the surface coincides with the dip of the bedding plane, slate and quartzite are seen to have been detached from bedding planes in blocks. (Photo 2-2-3(b))

(c). Alternating Beds of Quartzite and Slate, 3 (Al_3)

This unit comprises the mountain mass south of Huallanca. The alternation of beds are in thicknesses of approximately 3 to 4 m. They have a strike of N30°-50°W and dip 50°-75° to the southwest or northeast, and repetitions of anticlines and synclines are seen.

Cracks parallel to the bedding plane are seen in the quartzite at intervals of 30 to 90 cm, and pieces of the rock can be broken with strong blows of a hammer. Cracks are mostly tight. The slate is slightly phyllitic and there are cases when thin layers of quartzite and intercalated in lens form. Quartzite and slate have tight contact and are strongly bonded to each other. (Photo 2-2-3(c))

2.2.4 Glacial Deposit (Md)

Distributions of glacial deposits are seen at the right bank upstream from the confluence with the Azulmina and in the vicinity of the No. 2 open canal. Both are mainly of boulders (angular debris 2 to 3 m in size) of fresh quartzite with matrix of clay. Also, boulders of quartzite are scattered in monadnock form to show signs of transport by glacier. (Photo 2-2-4)

2.2.5 Terrace Deposit (Te)

Terrace deposits may be seen distributed along the present river. They are composed of fresh rounded gravels 10 to 30 cm in size with the matrix being sandy soil or clayey soil. Further, in the vicinity of the penstock in Alternative B, there is distribution of terrace deposits in a manner to cling to the slope up to the vicinity of EL. 3,700 m, and it is shown that the river existed at a level higher than at present in the past. (Photo 2-2-5)

2.2.6 Fan Deposit (Fa)

Fan deposits consist chiefly of rounded gravels 30 cm in size, but there are boulders of one meter at times. These fan deposits were derived principally from quartzite. Since the valleys in which these deposits are found have large runoffs, it is judged that boulders released during floods were accummulated.

2.2.7 Talus Deposit (Dt)

Talus deposits consist mainly of slate and quartzite debris with the matrix chiefly clayey soil. Talus deposits are developed in the area of slate at the southwest side of the fault and in the area of quartzite through which the No. 1 open canal passes.

The talus deposits in the slate area have high contents of clayey soil and the debris of slate are mostly of 10-cm size, whereas in case of the quartzite area, angular debris of fresh quartzite 1 m in size abound. Talus deposits are also extensively distributed in the vicinity of the proposed penstock route of Alternative C with the ground surface utilized for agriculture and stock raising. (Photo 2-2-7)

2.3 Geological Structure and Collapse Area

At·the project site, there are four anticlinal axis and synclinal axis at the upstream and downstream sides, respectively, with a fault as the boundary. The axis extend in the northwest-southeast direction with valley topographies at synclinal axis locations and ridge topographies at anticlinal axis locations, and a fold structure is reflected in the topography. The spacings between axis are 70 to 100 m where close, and 1.3 to 1.5 km where distant. Anticlinal and synclinal axis come close to each other in the vicinities of the outlet of the No. 1 waterway tunnel and the head tank of Alternative C, and beds are bent at steep angles. The lengths of the anticlinal and synclinal axis are unknown. Axial planes are estimated to be vertical.

A fault exists running northwest-southeast in the vicinity of the penstock in Alternative A to parallel the direction of the fold axis. The strike of the fault plane is N35°W with the dip 70° to the southwest. Relatively speaking, the hanging wall (upstream side) has dropped, and in the vicinity of the head tank in Alternative A a difference in level is recognized in the topography also. A fault valley is formed on the fault line, and at the left-bank side (hanging wall) the beds are upturned and weathering is advanced. Slate weathered to GL -10m has been confirmed by core drilling (PA-1). Upturning of beds is seen along the Rio San Juan at the opposite bank also, where there are small collapses on the fault. It is thought the degree of fracturing which accompanies faulting is higher at the hanging wall side than the foot wall side. Although the properties of the fracture zone are unknown, it may be estimated that slate and quartzite are considerably deteriorated.

Regarding collapses seen at the project site, at "Nagareban"* where strikes and dips of beds and the slope coincide, places where beds have become detached from the bedding planes to result in collapses may be confirmed. Slopes of "Nagareban" may be said to be unstable from the standpoint of topography also.

In the vicinity of EL. 3,800 m near the starting point of the No. 1 open canal, there is a minor collapse of glacial deposits seen to have been caused by small springing of water.

Collapses are all of small scale. (Photo 2-3)

* Nagareban: The condition of the dip of beds being parallel or close to parallel with the slope of the ground surface. Landslides and collapses are liable to occur.

2.4 Classification of Rock Masses

There have been many proposals made from the past regarding methods of classifying rock masses from engineering or mechanical standpoints, but due to intertangling of complex natural factors, it is difficult for simple quantification to be done. Accordingly, there is nothing unified, and the situation is that various classification methods have been proposed.

The relationships between tapping diagnosis by hammer, which is a basic operation in rock mass investigations, and uniaxial compressive strengths are as shown in Table 2-4-1.

According to this table, it may be estimated that the quartzite in the project site has compressive strengths in the range of 1,000 to 2,000 kg/cm^2, and slate in the range of 500 to 1,000 kg/cm^2.

Next, according to the classification of rock masses of the Japanese Society of Soil Mechanics and Foundation Engineering, the quartzite may be classified as H_1-B and the slate as H_2-C, or H_6-E in case the rock is especially phyllitic and anisotropy is considered.

(Note) H_1-B: Maximum uniaxial compressive strength higher than 1,000 kg/cm^2. Average crack spacing 30 - 90 cm.

H_2-C: Maximum uniaxial compressive strength 500 - 1,000 kg/cm^2 with averaged uniaxial compressive strength not less than 50% of the maximum. Average crack spacing 10 - 30 cm with no filler in cracks.

H_6-E: Anisotropic rock with maximum uniaxial compressive strength 100 - 500 kg/cm^2. Average crack spacing less than 10 cm with filler in cracks.

With regard to elastic wave velocities in the project site they are estimated to be 4,000 - 4,500 m/sec in quartzite and 3,000 - 3,500 m/sec in slate. As for alternating beds of quartzite and slate velocity is estimated to be 3,000 - 4,000 m/sec.

2.5 Geology at Structure Sites

2.5.1 Intake and Sedimentation Basin

The quartzite existing at the left-bank side of the Rio San Juan has cracks parallel to bedding planes as well as cracks crossing the bedding planes perpendicularly or diagonally. The quartzite is divided into blocks by the cracks.

The cracks tend to be open with those with strikes and dips of N70°E/78°N, N45°E/42°NW, and N30°W/90° being predominant. The quartzite is of a hardness that pieces of rock can be obtained with strong blows of a hammer.

At the right-bank side of the Rio San Juan there are glacial deposits and slate. The slate has been subjected to erosion and exposure at the ground surface is localized. According to investigations by boring, the slate is considerably weathered and there are parts which have been subjected to argillic alterration. As for the glacial deposits, they are unconsolidated deposits mainly consisting of fresh quartzite debris. Consequently, measures against strongly weathered slate and unconsolidated deposits at the right-bank side will be necessary in constructing the intake and sedimentation basin.

A geological map of the intake vicinity is shown in Fig. 2-6-3, and a geological section of the damsite in Fig. 2-6-4.

2.5.2 Headrace

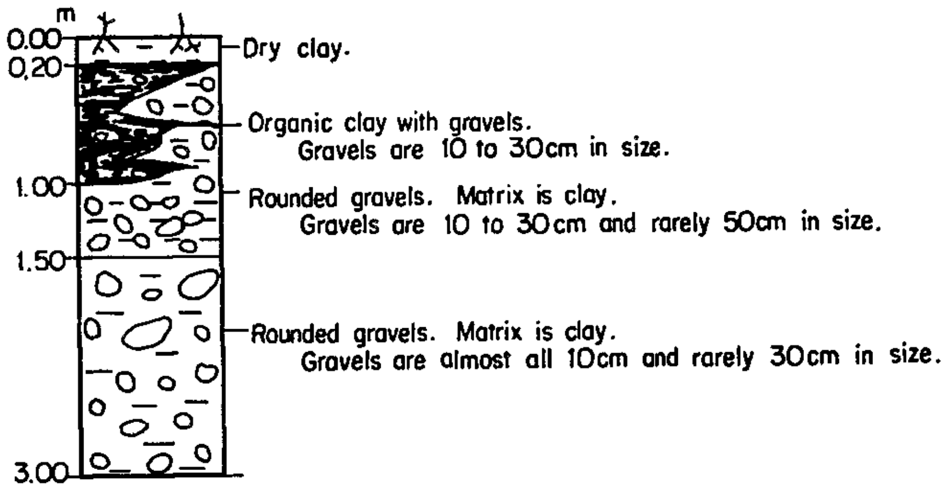
(1) Open Canal

Talus deposits, terrace deposits, and glacial deposits will be the objects of excavation. All of these are unconsolidated deposits and are distributed forming gradual slopes. Fresh, hard, angular debris of quartzite (1 - 2 m in size) are mixed in both talus deposits and glacial deposits. Particularly, at the No. 2 open canal, there are places where large debris of quartzite are concentrated in piles and these will require removal.

Excavated slopes are readily stabilized since most of the matrices are of clayey soils. As a whole, excavation will be at gentle slopes, and therefore, sliding of slopes due to excavation cannot occur. Glacial deposits have shown small collapses in the vicnity of the No. 2 open canal starting point, and therefore, care will be needed in crossing the open canal.

The results of test pit along the canal are shown below as columnar sections.

Fig. 2-5-1 Test Pit 1 (Terrace deposits)



O Kind of gravels is quartzite.

Fig. 2-5-2

Test Pit 2 (Glacial deposits)

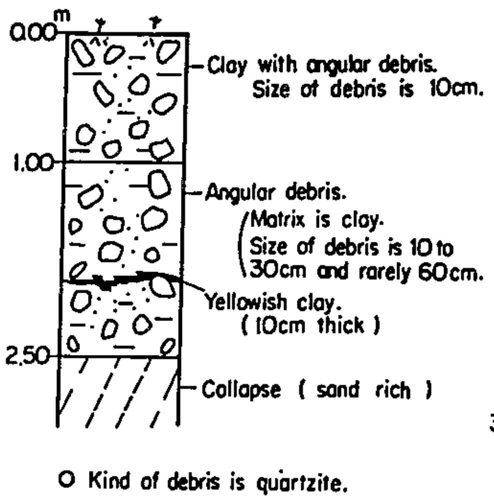


Fig. 2-5-4

Test Pit 4 (Glacial deposits)

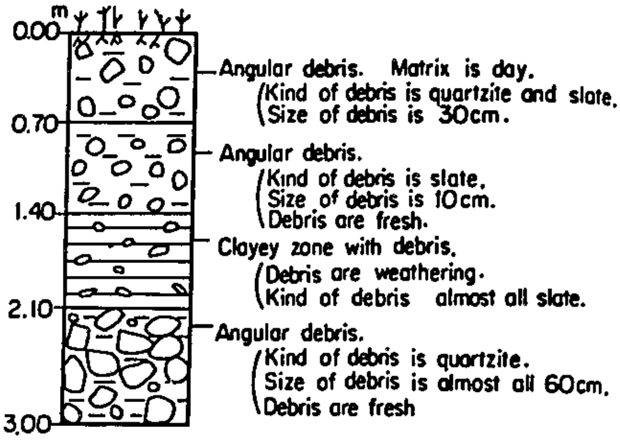


Fig. 2-5-3

Test Pit 3 (Talus deposits)

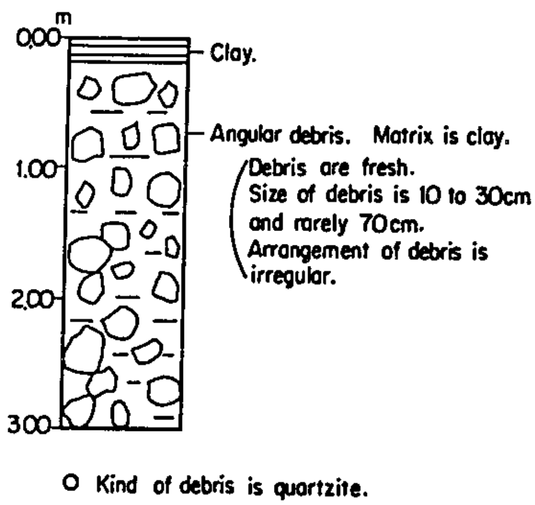
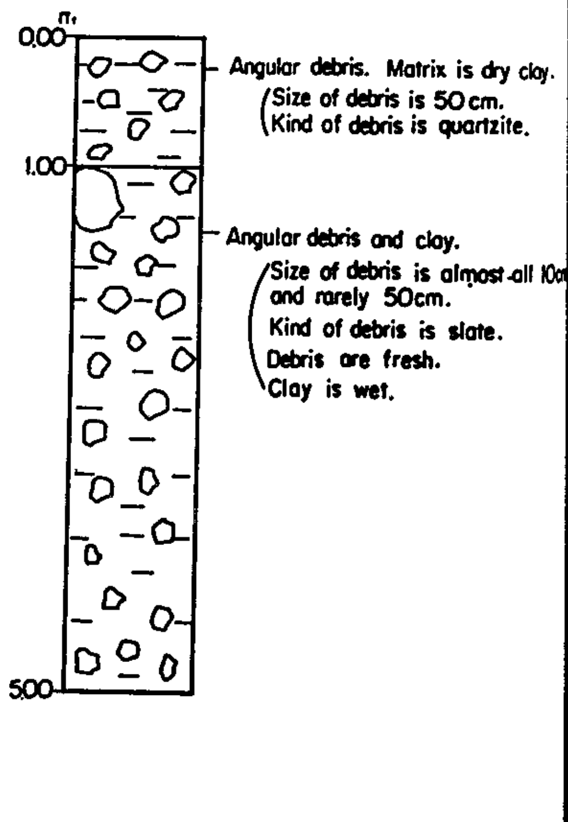


Fig. 2-5-5

Test Pit 5 (Talus deposits)



(2) Headrace Tunnel

(a) No. 1 Tunnel

The tunnel route is mainly of quartzite with occasional intercalations of slate. Bed-like cracks are developed in the quartzite. Because of this stick cores are not readily recovered in boring (T-1) and the rock quality designation (RQD (%) = Total Length of Cores 10 cm or More in Length/Total Length of Boring) is low.

Site walls are comparatively stable and uneven pressures are not readily experienced. However, slate at location of the synclinal axis have been subjected to deformation due to folding action and it is expected that deterioration of the rock will have occurred. The possibility also exists that surface water from the valley has infiltrated to accelerate deterioration of the rock, and it will be necessary to exercise care in excavation.

(b) No. 2 Tunnel

Alternative A (Upstream site): The route is comprised of slate and alternating beds of quartzite and slate. It is expected there will be reduction in strength due to weathering of slate at the two portals. Further, at the area of alternating beds of quartzite and slate, since the direction of the tunnel and the direction of strike of beds cross at an acute angle, the beds appearing at the site wall on the eastern side will dip toward the interior of the tunnel. Accordingly, the east side wall will be of "nagareban" form for a condition that the ground strata are prone to detachment from the bedding planes in block form, and it can be that blocks released in the future will dam up the waterway.

Alternative B (Midstream site): The objects of excavation would be slate and alternating beds of quartzite and slate. Since the tunnel will cross perpendicularly with the strike of beds, side walls will tend to be stable, and as a whole, uneven pressures are not liable to act. However, in the area where a fault is to be crossed, since it may be considered that deterioration of rock accompanying fracturing will have progressed more at the hanging wall of the fault than the foot wall, care will need to be exercised in case there is springing of water. The vicinity of the fault may require lining with concrete.

Alternative C (Downstream site): The route will be comprised of alternating beds of quartzite and slate. Since excavation will be done in a manner to perpendicularly cross the direction of strike of beds, the side walls will be easily stabilized. The same considerations as for Alternative B will be needed for any area where a fault is to be crossed.

(c) No. 3 Tunnel

Alternative C (Downstream site): The route will be comprised of alternating beds of quartzite and slate. Since it will cross at an obtuse angle with the direction of strike of beds, side walls will be comparatively stable. Because beds are bent at sharp angles in the vicinity of the tunnel outlet where anticlinal and synclinal axis will be close by, a possibility exists that there are many cracks in the slate and quartzite.

2.5.3 Head Tank

Alternative A (Upstream site): This is an area where talus is deposited. The talus, according to observations at five points where test pits were excavated consists mainly of angular debris of slate with a matrix of clayey soil. Consequently, it is necessary for the foundation of the head tank to be placed on alternating beds of quartzite and slate. The thickness of the talus is unknown, and there may be a case of excavation being deep.

Alternative B (Midstream site): The head tank is located on the alternating beds of slate and quartzite. Since the dip of the alternating beds is against the topographical slope, there is scarce possibility for landslides or collapses to occur. The place of head tank seems to be stable in view of topography and geology.

Alternative C (Downstream site): This is located at the northeast wing of an anticline, at the slope of a "nagareban" where the strike and dip of beds coincide with the strike and dip of the slope. This is an area of alternating beds of quartzite and slate, and it is thought lining will not be necessary in excavation. Since the vicinity of the outlet of the head tank will be at a "nagareban" slope, there is a risk of collapse occurring due to sliding of beds. It may be noted that collapse of quartzite due to "nagareban" exists at a point at EL. 3,900 m.

2.5.4 Spillway

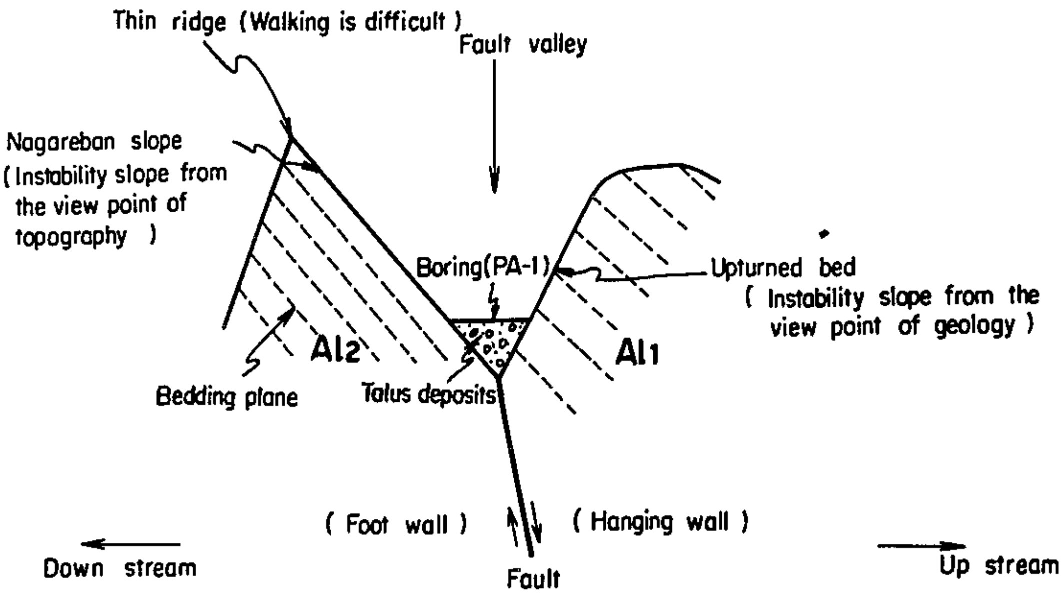
Talus deposits exist at the valley where the spillway of Alternative B (Midstream site) is projected. At the upper part of the right-bank slope of the valley there is a collapse, and it is indicated that debris had been supplied from there to the valley in the past. Consequently, it is estimated that talus deposits are deposited thickly, and it is necessary to exercise thorough care that the talus deposits will not slide due to construction of the spillway to cause damage to structures.

2.5.5 Penstock

Alternative A (Upstream site): A schematic section in Fig. 2-5-6 shows the cross section between the place of head tank and the road. The slope and the ridge in the downstream of the fault valley is regarded as so-called "Nagare-ban" and it seems to be unstable topography for constructing structure in the narrow ridge. The fault valley is rather suitable for structure. However it is expected that there would be deteriorated rock underlying the talus deposit because the deteriorated rock in the fault valley is selectively eroded to form the valley.

As a result of core drilling (PA-1), the existence of extremely soft and brittle slate was recognized. The slate laid above the fault at the left bank of Rio San Juan has been fragmented and small scale of collapses has occurred. It may be considered that slate which has become deteriorated and brittle continues along the fault to form unstable condition in views of topography and geology. Construction structure on such places should be avoided.

Fig. 2-5-6 Cross Section of Fault Valley



Alternative B (Midstream site): This is a slope which is ideal for installing a penstock. The basement alternating beds of quartzite and slate comprise an "Ukeban"* dipping in the opposite direction from the dip of the slope. Therefore, if the relation between the basement rock and the slope is examined, it may be said to be a comparatively stable slope. Talus deposits and terrace deposits cover the basements, and the foundation of the penstock is to be laid there. The talus deposits have a high content of clay, with a mixture of angular debris 10 to 30 cm in size, while the terrace deposits have a large mixture of rounded gravels of 10-cm size, and there are no special problems for the foundation of a penstock.

Next, the results of test pits made under Alternative B are shown as columnar sections.

* Ukeban: The dip of beds in a relation of inverse dip to the topographical slope. Antonym of "Nagareban."

Fig. 2-5-7

Test Pit, Pb-1 (Talus deposits)

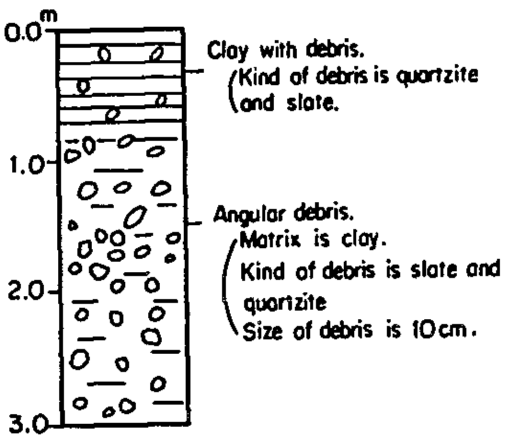
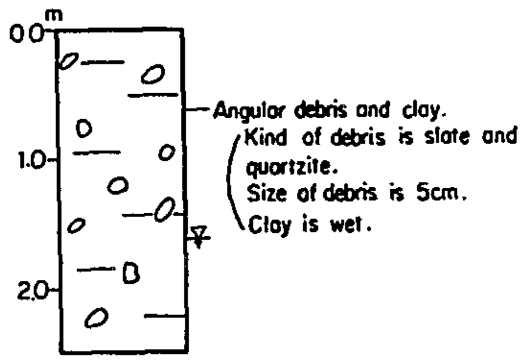


Fig. 2-5-8

Test Pit, Pb-2 (Talus deposits)



Alternative C (Downstream site): This is located at the slope of the northeast wing of the anticline, with bedding planes of quartzite directly exposed at the upper part of the penstock and the dip of the beds coinciding with the dip of the slope. Because of this, it is necessary to exercise care regarding the slope due to "Nagareban" when installing the penstock. In the vicinity of EL. 3,900 m, there exists a new collapse due to "Nagareban." Further, the greater part of the penstock passes a talus area. The thickness of the talus deposits, according to boring (PA-1), is more than 15 m. The talus deposits area is being utilized for stock raising since the slope is gentle, but it will require levelling because of undulations.

The geological sections of Alternatives A, B, and C for the penstock are shown in Fig. 2-6-5, Fig. 2-6-6 and Fig. 2-6-7.

2.5.6 Powerhouse

Alternative A (Upstream site): Topographically, the site is located in a valley where talus deposits are thickly deposited. According to investigations by core drilling (SA-1), the talus deposits have a high content of clayey soil as the matrix, and the thickness is greater than 15 m. The clayey soil is soft and unsuitable as bearing for the powerhouse, and more than 15 m of excavation will be necessary to reach bearing ground. Further, in the event the end portion of the talus deposits should be scoured during flooding of the Rio San Juan, the talus deposits at the Alternative A site will start sliding, and there is danger that damage will be inflicted on the sturcture. In view of the above, the power station site of Alternative A is unsuitable.

Alternative B (Midstream site) and Alternative C (Downstream site): Topographically, these sites are located on river terraces. According to observations by test pits, both of the sites have terrace deposits with high contents of fresh, rounded gravels of quartzite. Core drilling (SB-1, SC-1) shows these terrace deposits to have thicknesses of 10 to 12 m.

The terrace deposits at both the B and C sites do not pose any problems as supporting ground for the powerhouse.

The results of test pits at the powerhouse sites (Alternatives B and C) are shown as columnar sections.

Fig. 2-5-9
Test Pit, Sb-1 (Terrace deposits)

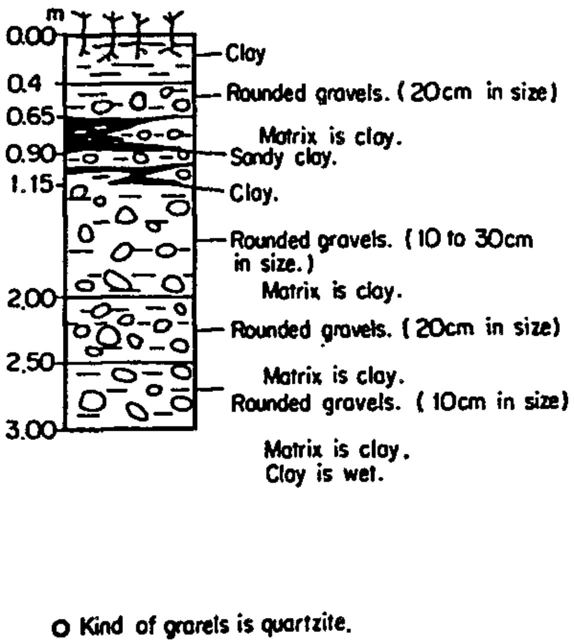
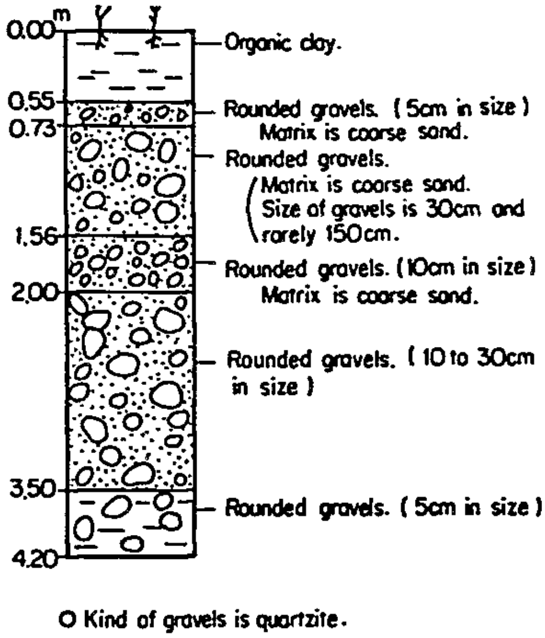
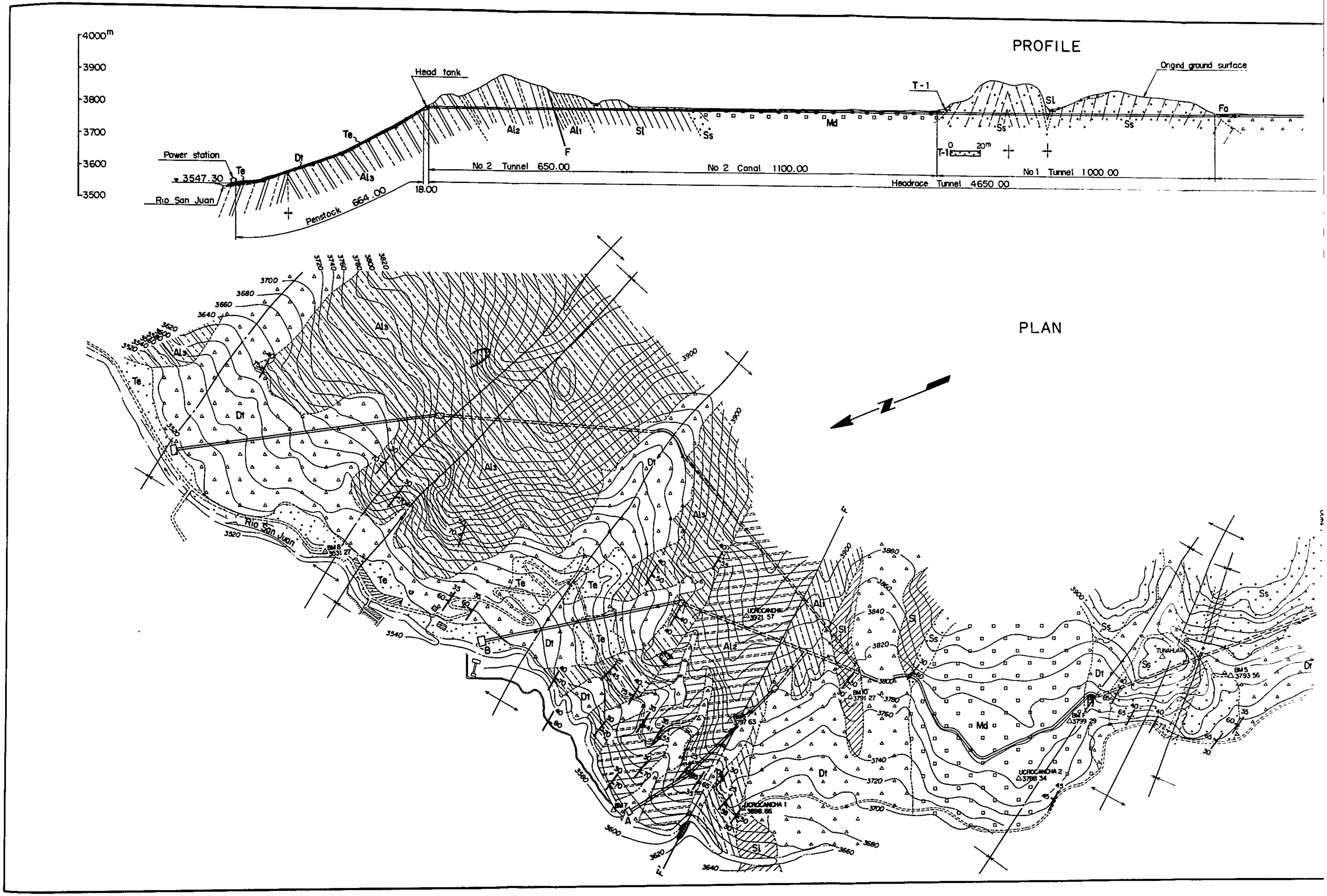


Fig. 2-5-10
Test Pit, Sc-1 (Terrace deposits



2.6 Geological Maps

Reference is made to the geological maps of Fig. 2-6-1 to Fig. 2-6-7.



PROFILE
PLAN
Head tank
Power station
3547.30
Rio San Juan
Penstock 664.00
18.00
No 2 Tunnel 650.00
No 2 Canal 1100.00
No 1 Tunnel 1000.00
Headrace Tunnel 4650.00
T-1
Original ground surface
Te
Dt
Al3
Al2
Al1
Sl
Ss
Md
Fa
F
Rio San Juan
UCROCANCHA 1
UCROCANCHA 2
TUNAHUAIN

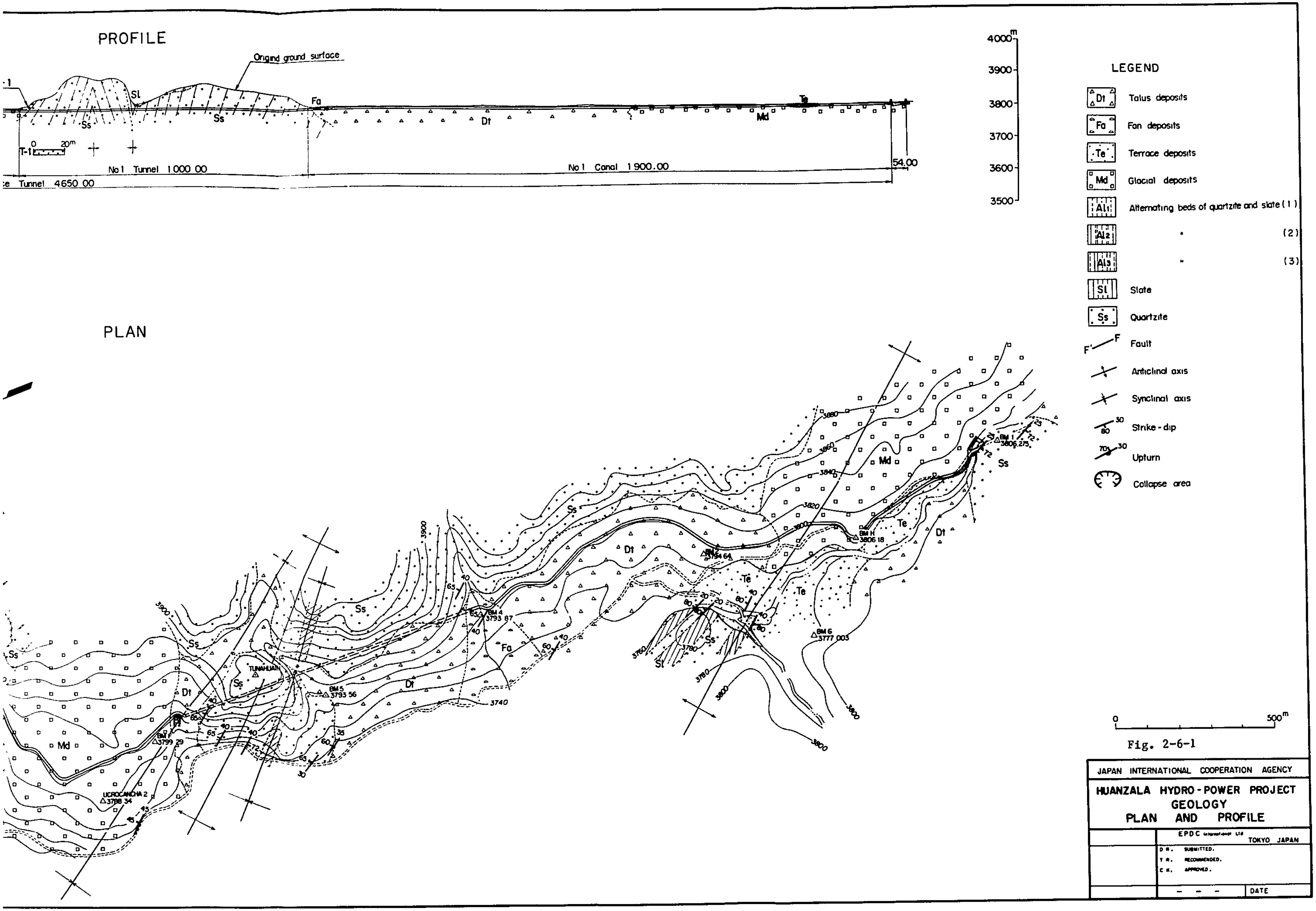


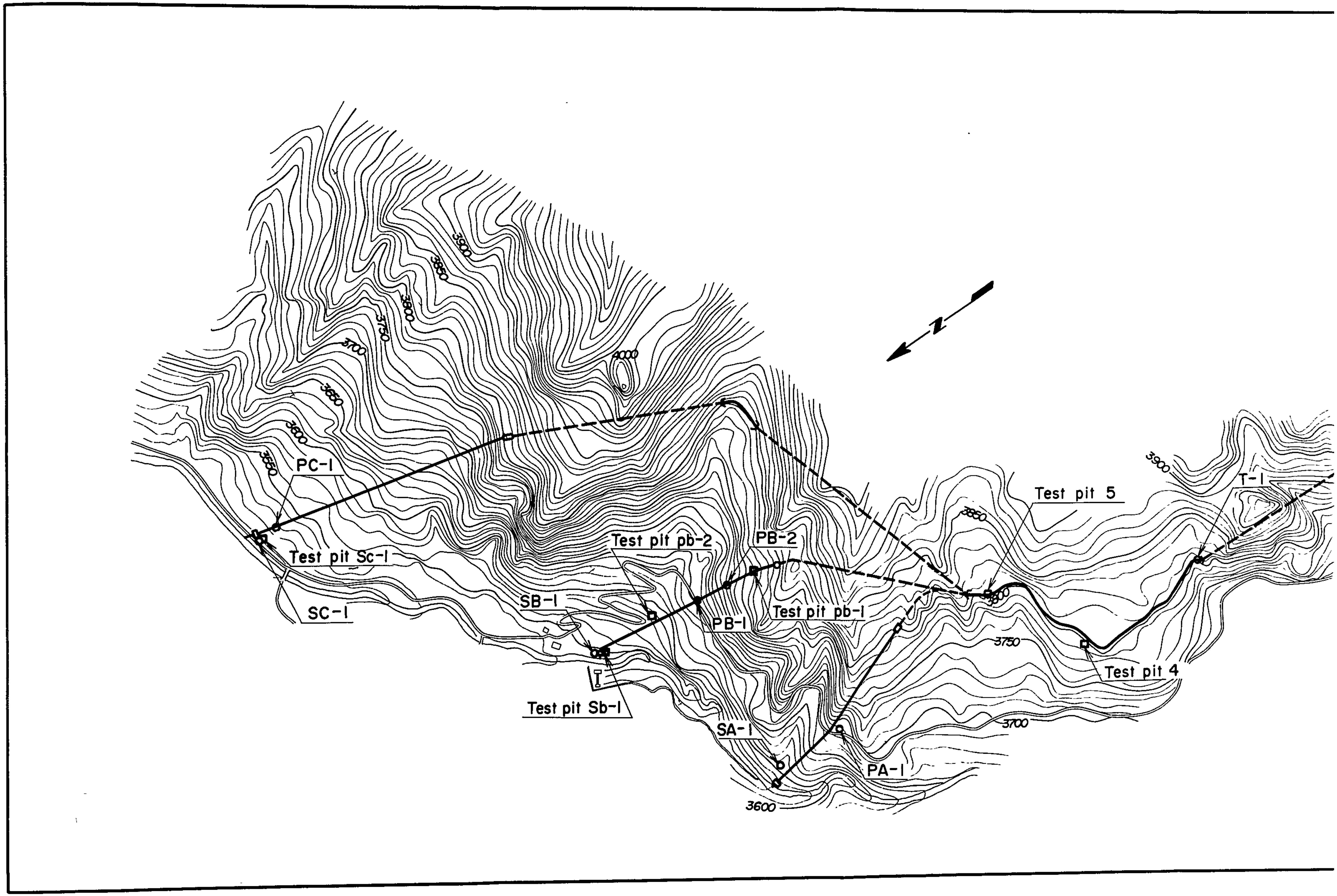
Fig. 2-6-1

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GEOLOGY
PLAN AND PROFILE

EPDC International Ltd TOKYO JAPAN

DR.	SUBMITTED.
TR.	RECOMMENDED.
CK.	APPROVED.
— — —	DATE



3900
3850
3800
3750
3700
3650
3600
3550
4000
N
PC-1
Test pit Sc-1
SC-1
Test pit pb-2
PB-2
SB-1
PB-1
Test pit pb-1
Test pit Sb-1
SA-1
PA-1
3600
3700
3750
Test pit 5
3850
3800
3900
T-1
Test pit 4

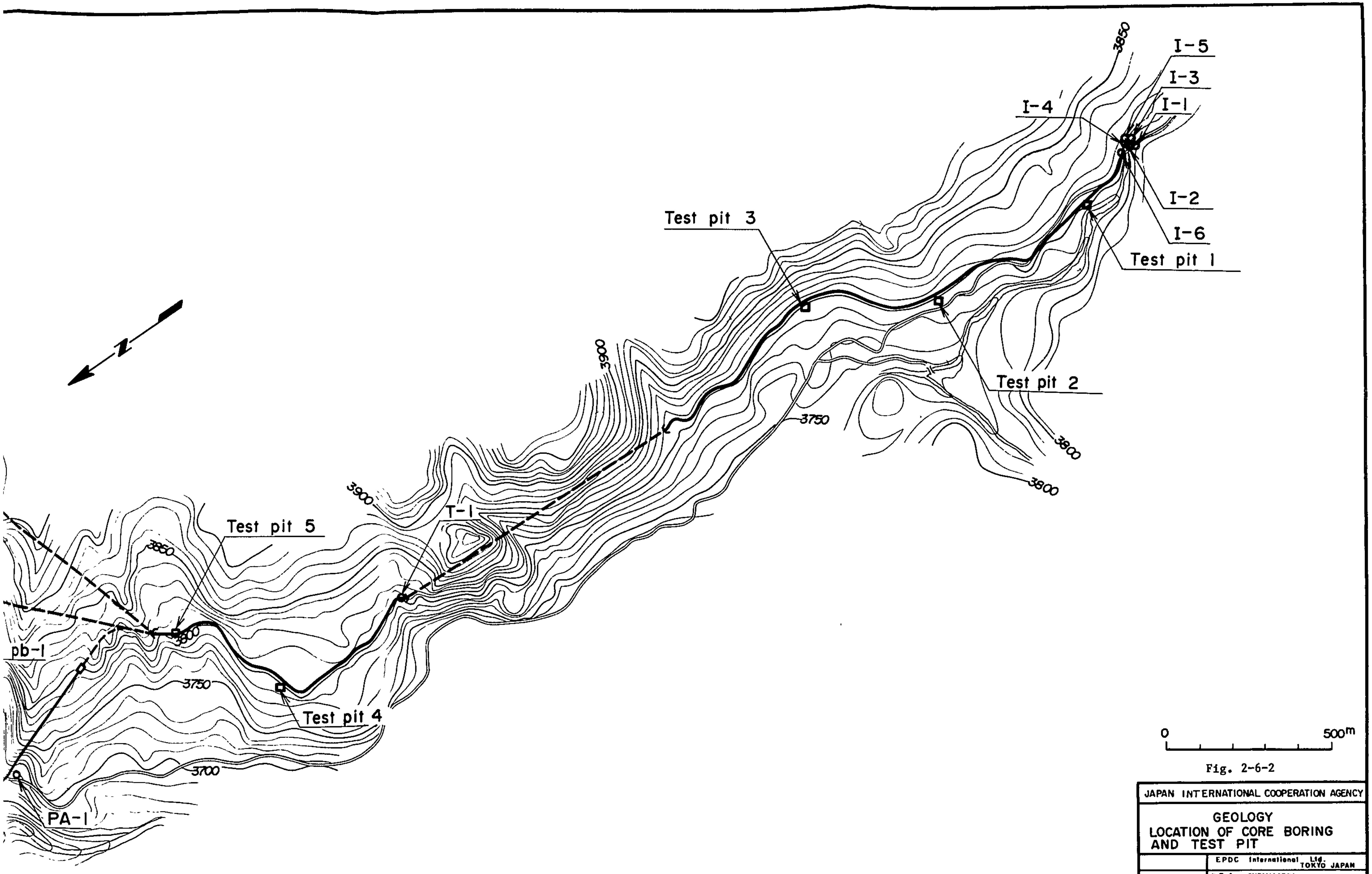


Fig. 2-6-2

JAPAN INTERNATIONAL COOPERATION AGENCY

GEOLOGY
LOCATION OF CORE BORING AND TEST PIT

EPDC International Ltd. TOKYO JAPAN

LOCATION	DATE	DESCRIPTION	BY
REVISION			

D.R.; SUBMITTED;
T.R.; RECOMMENDED;
C.K.; APPROVED;

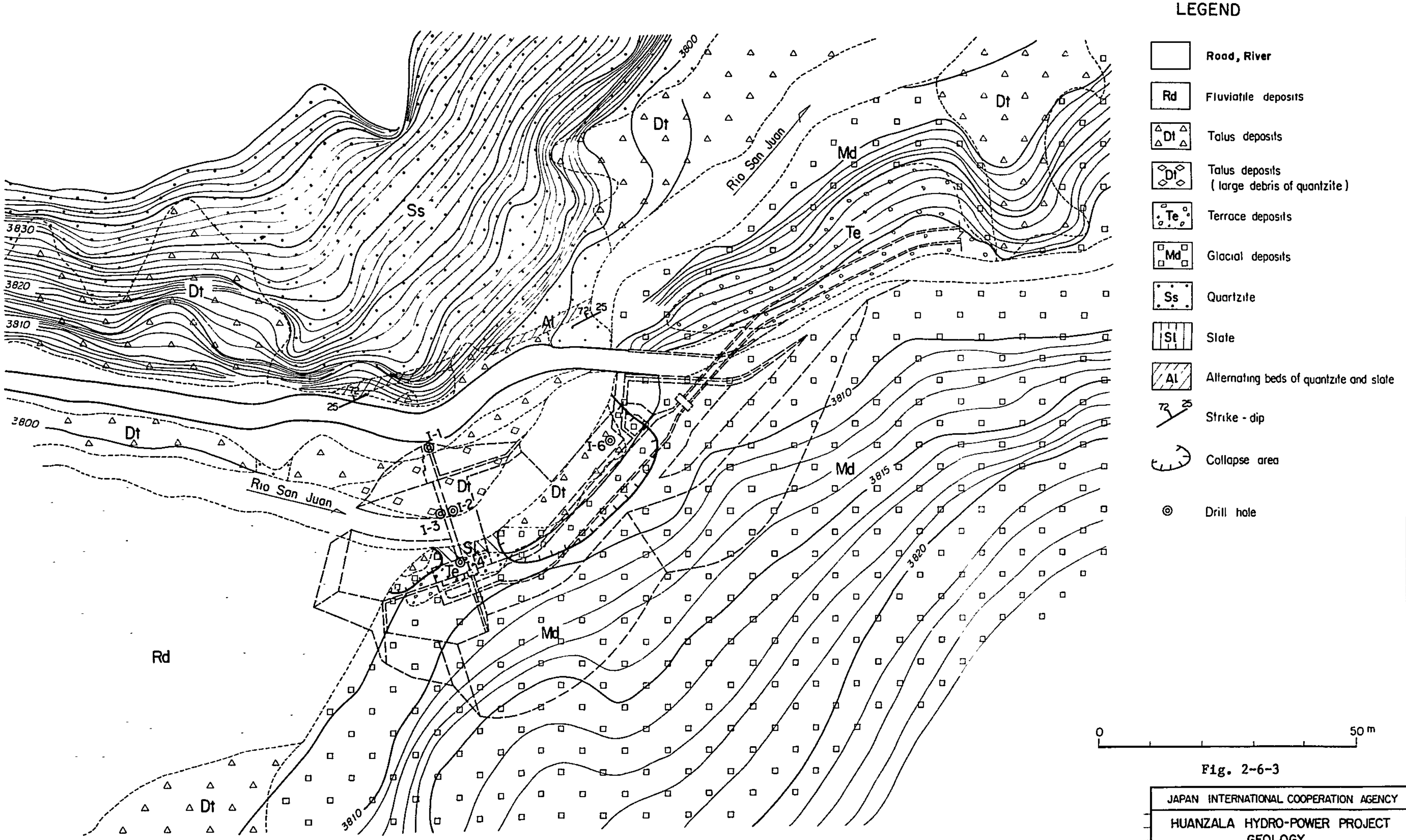


Fig. 2-6-3

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GEOLOGY
INTAKE AND SEDIMENTATION BASIN

EPDC International Ltd TOKYO JAPAN

LOCATION | DATE | DESCRIPTION | REVISION

SHEET NO. OF

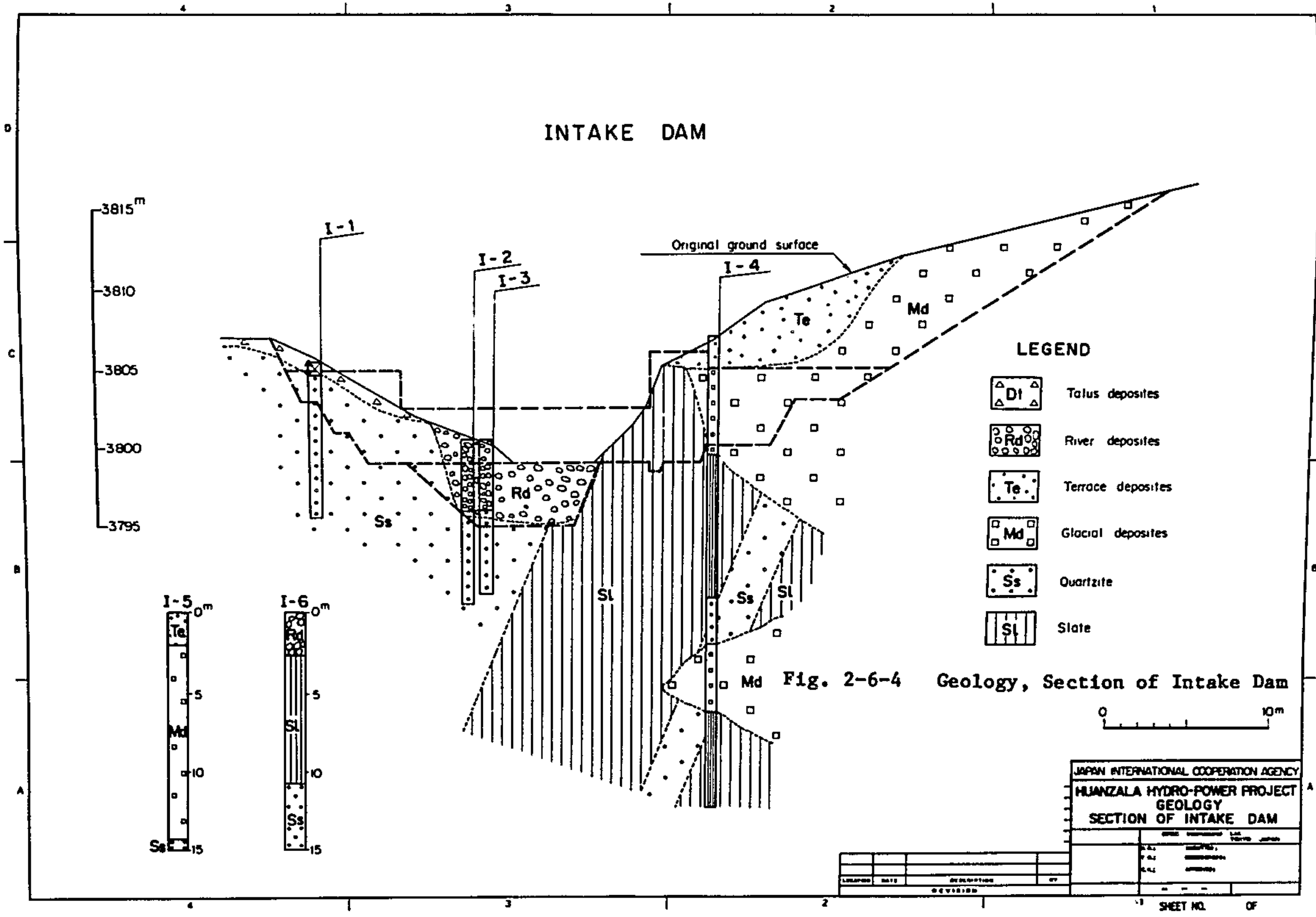


Fig. 2-6-4 Geology, Section of Intake Dam

PENSTOCK PROFILE (CASE-A)

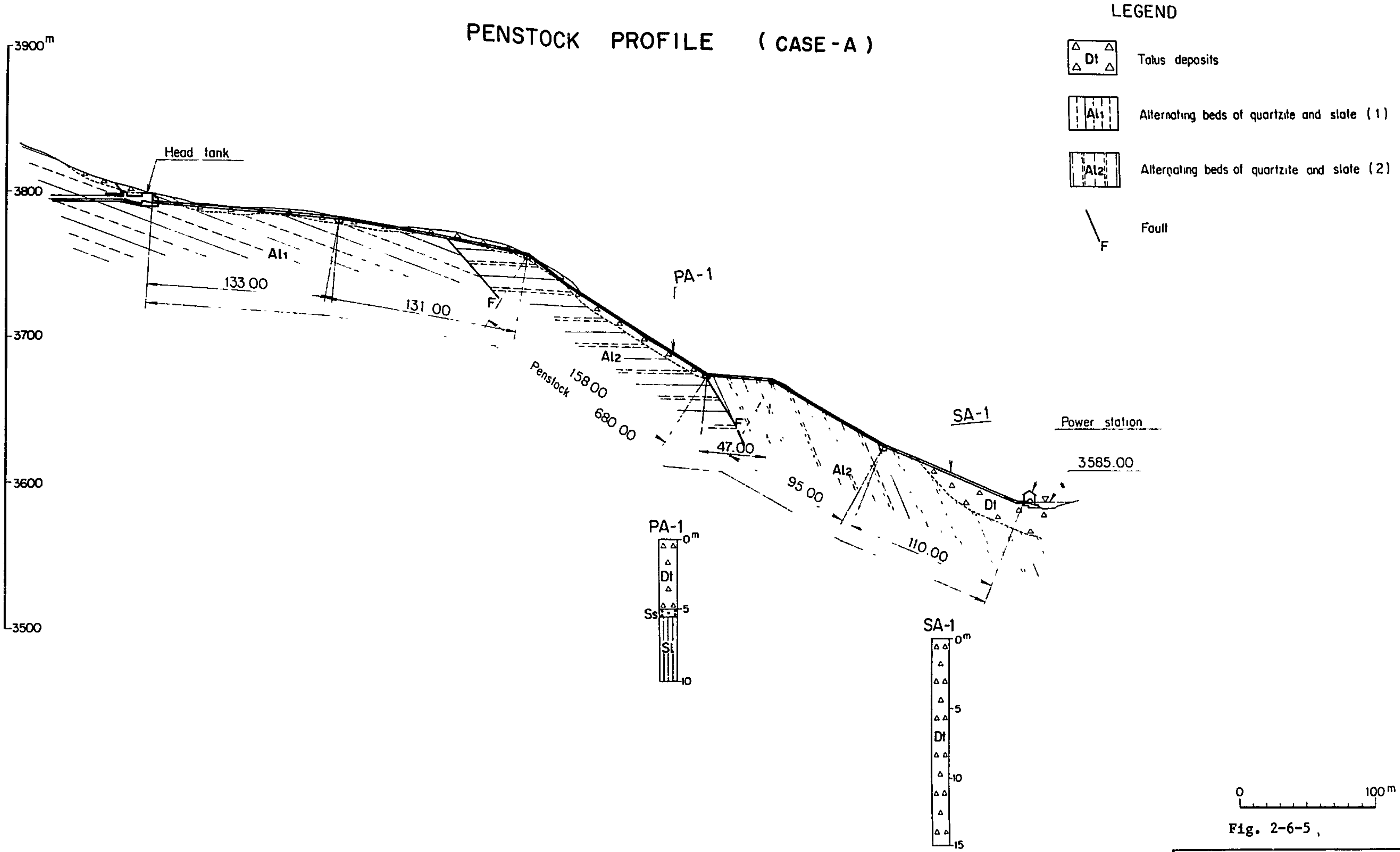


Fig. 2-6-5

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GEOLOGY
PENSTOCK (CASE A)

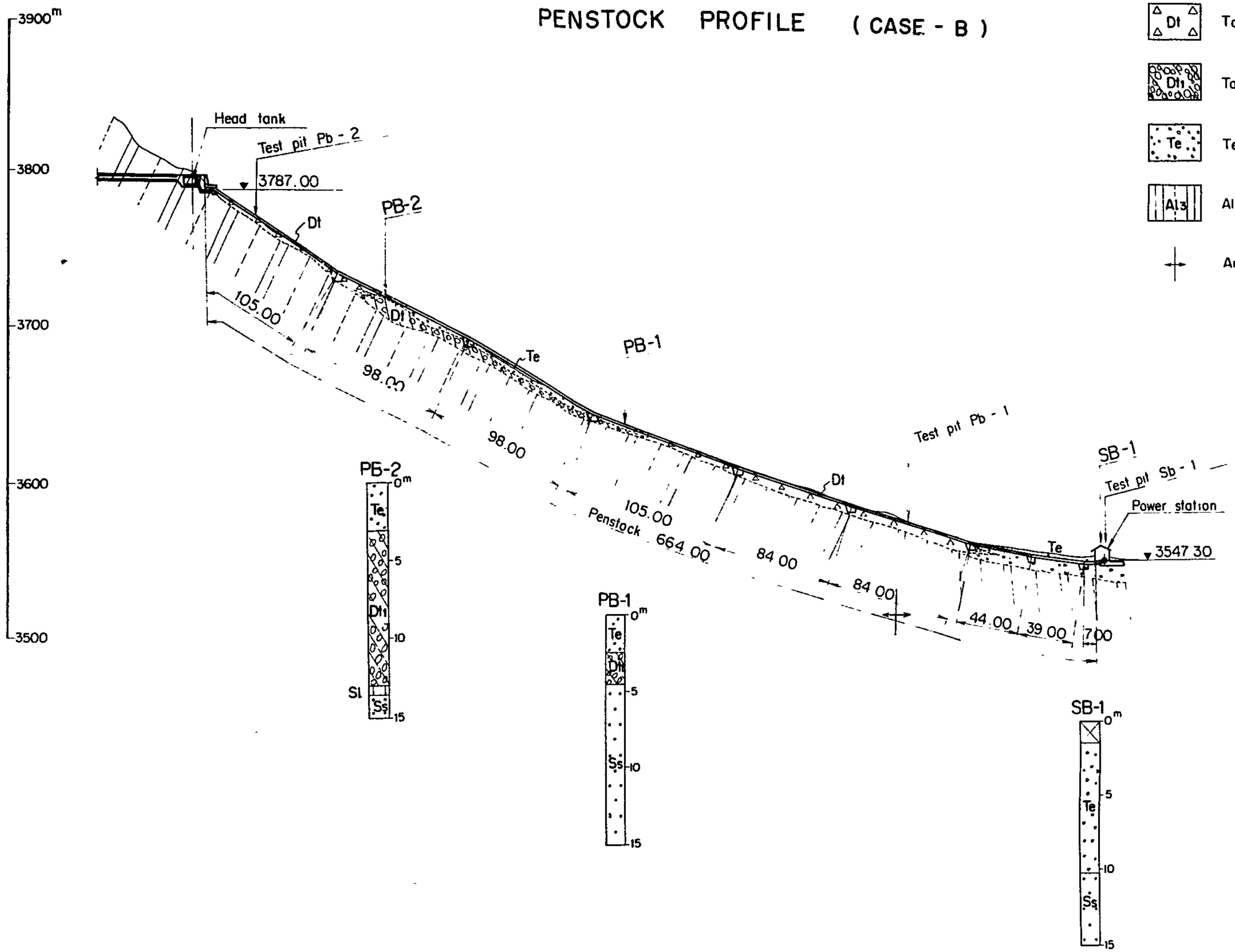
EPDC International Ltd TOKYO JAPAN

DR	SUBMITTED.	
TR	RECOMMENDED.	
CK	APPROVED	

LOCATION	DATE	DESCRIPTION	BY
REVISION			

SHEET NO OF

PENSTOCK PROFILE (CASE - B)



LEGEND

Dt	Talus deposits
Dt1	Talus deposits (1) (Large debris of quartzite and slate)
Te	Terrace deposits
Al3	Alternating beds of quartzite and slate (3)
+	Anticlinal axis

0 100m

Fig. 2-6-6

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GEOLOGY
PENSTOCK (CASE B)

EPDC International Ltd TOKYO JAPAN

D R	SUBMITTED
T R	RECOMMENDED
C K	APPROVED

LOCATION	DATE	DESCRIPTION	BY
REVISION			

SHEET NO OF

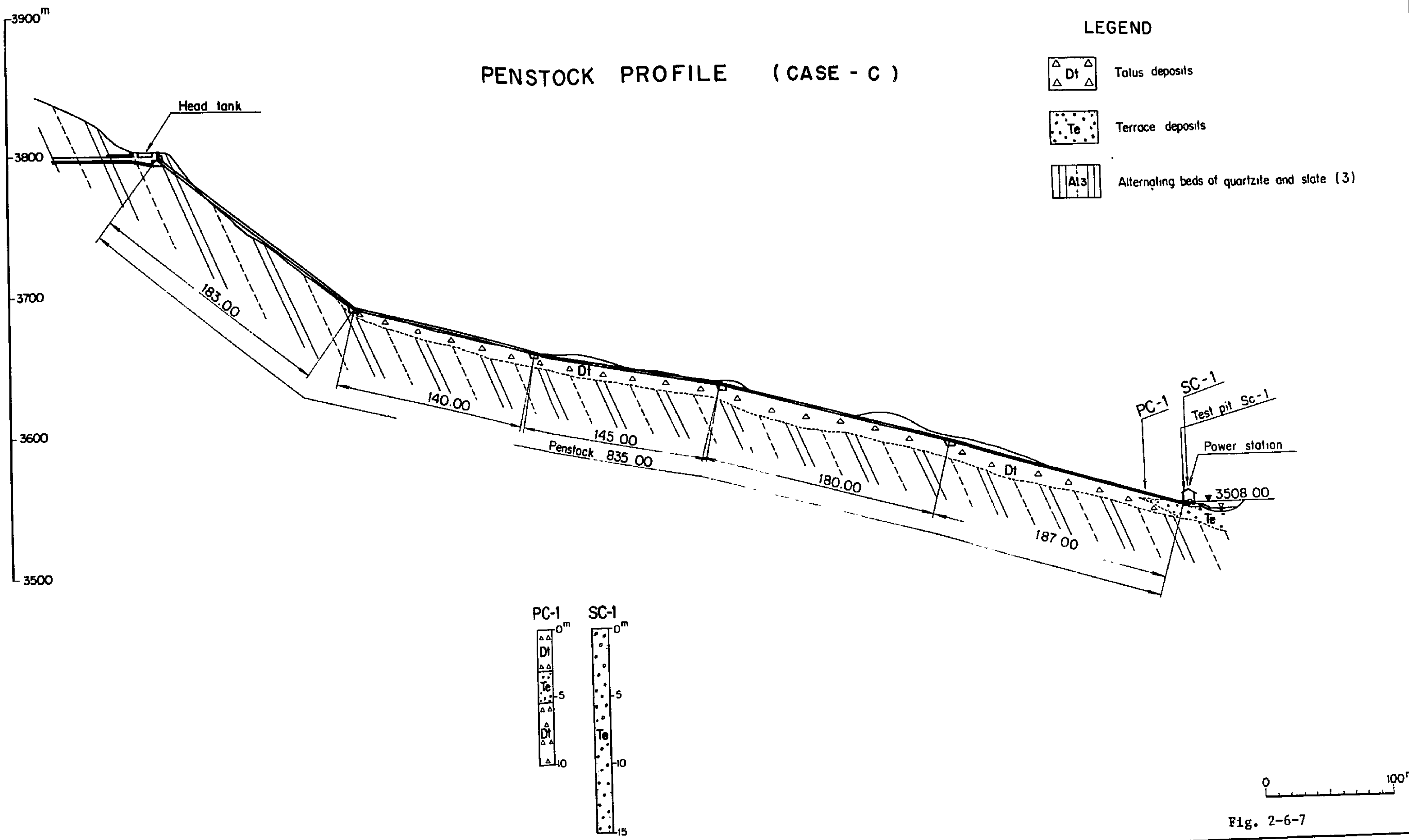


Fig. 2-6-7

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GEOLOGY
PENSTOCK (CASE C)

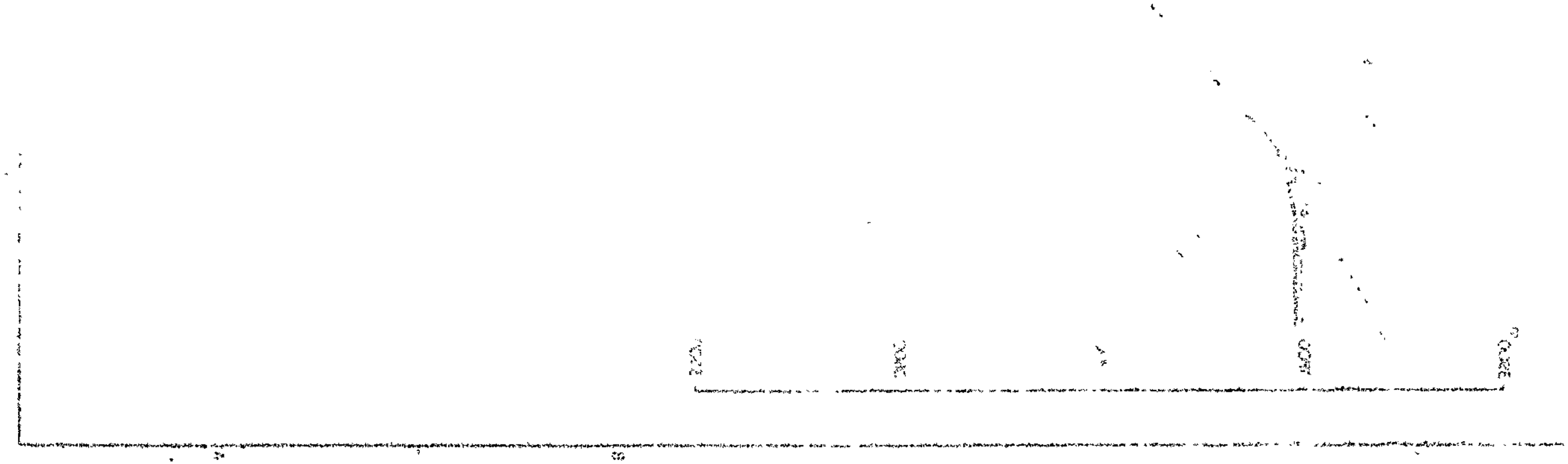
EPDC International Ltd TOKYO JAPAN

DR	SUBMITTED
TR	RECOMMENDED
CK	APPROVED

LOCATION	DATE	DESCRIPTION	BY

REVISION

SHEET NO OF



CHAPTER 3

PRELIMINARY DESIGN

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3.1 Intake Dam

3.1.1 Dam

The intake dam was selected about 40 m upstream from Pte. Arequipa. The stretch for approximately 1.0 km upstream from the dam site is a pampa of gentle gradient. The valley of the river at the dam site becomes narrow and rock is exposed at both banks. This location is optimum for the dam site from any standpoint--geology, topography, or ease of construction. Downstream of this, the valley width becomes slightly larger and the slopes at the two banks forming the valley are steep. There is a large height difference with the departmental road.

Downstream of this to the bridge, the right bank is comprised of a talus deposit with its base washed by flood to result in collapse.

Quartzite is exposed at the left bank of the dam site, while at the right bank there is a very slight amount of slate exposed near the river bed, above which there exist glacial deposits. Sand-gravel is deposited in a thickness of approximately 5.0 m at the river bed, the foundation of which is hard quartzite.

For the dam, the sand-gravel below the river bed is to be replaced by foundation concrete, upon which a gravity-type concrete dam of height of 3.5 m and length of approximately 25.0 m is to be constructed. The overflow section is to have a width of 15.0 m with no grate provided.

Because of steep river gradients, the rivers of Peru in the rainy season (flood season) show large sediment flows almost every day. Consequently, it is the general practice to provide dams with large sand flushing facilities in order not to impair intake efficiency. However, this dam site has a large pampa upstream and the river gradient is extremely gentle so that it is thought a sediment settling effect can be looked forward to and an especially large sand flush will not be provided. Furthermore, reinforcement with stone facing at the surfaces of the dam and overflow section will not be considered in particular.

The concrete of the dam including foundation concrete is to have cobbles mixed in to conserve concrete since it is possible for cobblestones to be collected in the vicinity.

The dam foundation, as a result of permeability tests at drillholes, has permeability of quartzite of 1×10^{-4} cm/sec, and of talus and weathered portions of slate (shale) at the right bank of 5×10^{-4} cm/sec, while the height of the dam is small and water pressure low so that special cut-off works will not be provided.

Downstream of the dam apron is such structure that cobblestones are laid out and interstices filled with concrete in order to prevent scouring of the river bed.

The depth of water when discharging a design flood of 100 cu.m/sec by the overflow length of 15.00 m and a sand flush of 2.00 m x 2.00 m was obtained by Cal. 3-1-1. As a result, the depth of water at the overflow crest (EL. 3,802.50 m) will be 2.00 m. The height of the non-overflow section is to be EL. 3,805.00 m considering an allowance of 0.50 m.

As a result of stability calculations by Cal. 3-1-2, both overturning and sliding, and bearing capacity of ground satisfy the stability conditions.

3.1.2 Intake Facilities

The intake is to be provided next to the right-bank side of the dam in the front and in a perpendicular direction, from which a maximum of 2.20 cu.m/sec is to be drawn and connected directly with the sedimentation basin. The orifice is to be such that sediment inflow will be prevented as much as possible. The inflow velocity at the orifice is to be 0.5 m/sec with inflow width 4.00 m. A trash rack is to be provided at the front. A regulating gate of 1.50 m x 1.20 m is to be provided at the entrance of the sedimentation basin for maintenance purposes.

3.2 Sedimentation Basin

The suspended sediment in flowing water will settle in the waterway to reduce the cross-sectional area of flow, with a part entering the penstock and turbine to cause equipment to be abraded. In order to prevent such occurrence, it is necessary for a sedimentation basin to be provided at a place as close as possible to the intake facilities to cause sediment in the flowing water to settle and be removed. The settling basin is to be provided at the right-bank side between the dam and Pte. Arequipa. Accordingly, since the dam, intake and sedimentation basin will be concentrated at one place, the conditions will be advantageous from the standpoints of both constructability and economy. This part is the location of collapse of a talus deposit so that the settling basin will fulfill the two roles of a revetment and a training wall.

Regarding the structure of the sedimentation basin, since the water storage area provided by the dam is large and a settling effect can be expected, the settling basin is to be a single pond, with the average velocity moreover being slightly high at 0.50 m/sec (ordinarily about 0.20 m/sec), the cross section being made as small as possible. From Cal. 3-2-1, the width is to be 3.50 m, the length 40.00 m, and depth 2.00 m (average). The side wall on the river-side would concurrently serve as a training wall downstream of the dam. In order to remove sediment, a sand flush gate of 1.00 m x 1.20 m is to be installed, while a spillway 25.00 m in width is to be provided on the river-side training wall for allowing water in excess of the maximum power discharge 2.20 cu.m/sec to overflow. A regulating gate of 1.20 m x 1.80 m is to be provided at the end of the settling basin for the purpose of maintenance of the headrace.

3.3 Headrace

The headrace is to be a non-pressurized waterway along the right bank of the Rio San Juan consisting of 3,000 m of open canal and 1,650 m of tunnel. The geology of the waterway route consists of taluses, terrace deposits and glacial deposits at portions of open canals. With regard to tunnels, the route of the No. 1 tunnel is mainly quartzite with occasional intercalation of slate, the rock being hard as a whole, and the route of the No. 2 tunnel has slate and slate-quartzite alternations, the rock being hard except for a fault zone in the vicinity of 150 m from the inlet.

Selection of the waterway route was made based on 1/5,000 topographical maps and field reconnaissances. Since the accuracies of the topographical maps were not so good, it is necessary for waterway lengths and other factors to be closely checked at the time of detail design. The routes of open canals will be laid conforming to the topography. Since the waterway is to run along slopes it will be economical if the cross section is made as small as possible. Accordingly, the cross section is to have a bottom width of 1.20 m, top width of 2.28 m, and depth of 1.80 m for a trapezoidal shape. The gradient is to be 1/1,000, the lengths 1,900 m for No. 1 and 1,100 m for No. 2, and the surfaces are to be stone-lined.

Where a road is crossed on the way, the canal is to become a culvert of 1.80 m in height, 1.20 m in width and 0.25 m in thickness, and where gullies are to be crossed the cross sections are to be rectangular, 1.80 m (width) x 1.20 m (height), with thickness of 0.20 m. Furthermore, since the route passes through grassland where livestock is grazing, it will be necessary for bridges and fences to be provided at places.

Tunnels are designed in the places where it is difficult for open canals to be constructed, the topography is steep and rugged, and large boulders are distributed. Since the maximum power discharge is small at 2.20 cu.m/sec, the cross section is the minimum from the standpoint of constructability. The form is of semi-circular top and rectangular bottom cross section of height of 2.50 m, width of 2.00 m. Thickness of invert concrete of 0.20 m, with the gradient 1/1,000 similarly to the open canals. Concrete lining is to be provided around the portals and elsewhere depending on the geology.

Hydraulic characteristics calculations of the various cross sections were made by Cal. 3-3-1, and the individual characteristics curves are shown in Fig. 3-3-1 through Fig. 3-3-5.

3.4 Head Tank and Spillway

The head tank, as shown in Fig. 3-10-5, was made of circular shape in consideration of topographical constraints.

The head tank, along with adjusting the difference between the penstock flow depending on load variation and the headrace flow, achieves final removal of sediment in the flowing water so that the turbine will not be damaged. The capacity required is a volume to supply water in a short period when load suddenly increases, corresponding to about a 2-minute supply at the rate of maximum power discharge.

The dimensions of the head tank are to be diameter of 9.00 m and height of 5.50 m. The effective capacity would be 280 cu.m, corresponding to 2 minutes of the maximum power discharge. A trash rack and regulating gate of 1.20 m x 1.20 m are to be provided at the intake bay immediately before the penstock orifice, and a sand flush gate at the sand flush.

It will be necessary to provide a spillway at the head tank capable of safely releasing the maximum power discharge at a water level within limits not to apply pressure to the top of the headrace when total load is shut off.

The spillway is to safely release the water overflowing from the head tank back into the river. Since this power station will carry out load-adjustment operation in the high-water season, it is thought that there will be overflow of water at all times. It is thought to lead the overflow water to the gully immediately upstream of the head tank site by tunnel, and down a slope by an open canal in consideration of the topography, constructability and economic efficiency of this site. The cross section would be that of a waterway bottom width of 0.60 m, top width of 1.50 m, and height of 1.50 m, the length from the head tank to the existing road being 370 m.

At the detail design stage, it may be necessary to complete an entire spillway route.

Calc 3-1-1 Spillway discharge capacity

a) General

Spillway discharge capacity is obtained by following formula.

$$Q = Q_1 + Q_2$$

where

Q : Spillway discharge (m^3/sec)

Q_1: Overflow discharge from spillway (m^3/sec)

Q_2: Outflow discharge through sandflush gate (m^3/sec)

b) Basic formula

Q_1 and Q_2 are calculated by the next formulas respectively.

◦ Overflow discharge from spillway

$$Q_1 = CB\,H^{3/2} \qquad (1)$$

$$Cd = 2.20 - 0.0416\,(Hd/w)^{0.990} \qquad (2)$$

$$C = 1.60 \times \frac{1 + 2a(H/Hd)}{1 + a(H/Hd)} \qquad (3)$$

where

Q_1: Overflow discharge (m^3/sec)

B : Width of crest (m)
B = 15.0 m

H : Water depth (m)

W : Height of crest (m)
W = 3.5 m

Hd: Design head (m)
Hd = 2.0 m

C : Variable coefficient of discharge

Cd: Coefficient of discharge
when H = Hd
Cd = 2.176

a : Constant

When H = Hd, C = Cd, a would be known by formulas (2) and (3).

a = 0.563

∘ Outflow discharge through sand flush gate

$$Q_2 = Cc\ a \cdot B \sqrt{\frac{2g\ (ho - Cca)}{1 - (Cca/ho)}} \qquad (4)$$

where

Q_2: Outflow discharge (m^3/sec)

Cc: Coefficient of convergence

a : Opening height of gate (m)

a = 2.0 m

ho: Upstream water depth (m)

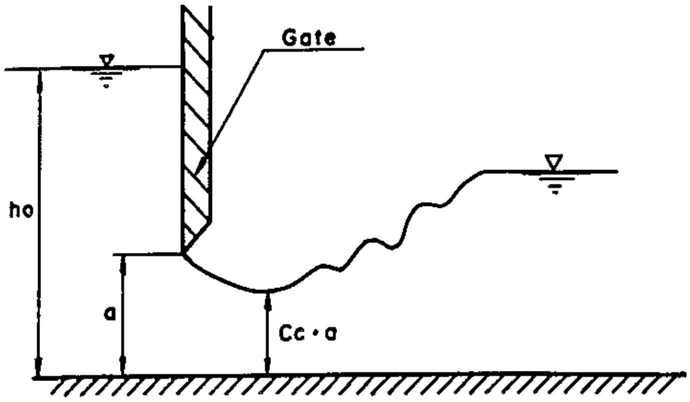
B : Width of sand flush gage

c) Results and conclusion

Results of calculations are shown in Tables 1-1, 1-2 and 1-3, respectively.

Discharge capacity curves are also shown in Fig. 1-2.

According to these results, design flood (100 m^3/sec) would be released safely with reservoir water level of 3,804.5 (m).



Coeffient of convergence

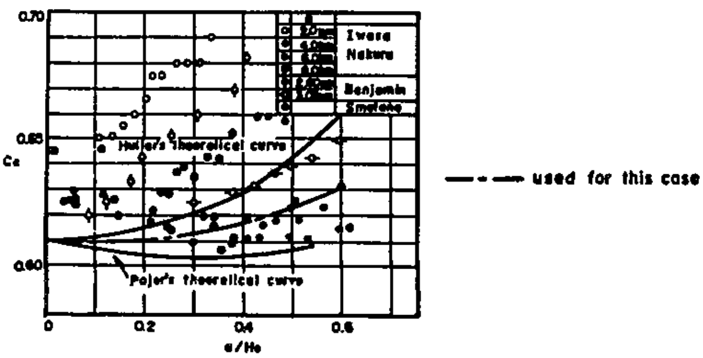


Table 3-1-1 Overflow discharge from Spillway

Water level (m)	Water depth H (m)	Coefficient of discharge C	Discharge Q (m3/sec)
3,802.5	0	0	0
3,803.0	0.5	1.797	9.53
3.803.5	1.0	1.951	29.27
3,804.0	1.5	2.075	57.18
3,804.5	2.0	2.176	92.32
3,805.0	2.5	2.261	134.06
3,805.5	3.0	2.333	181.84

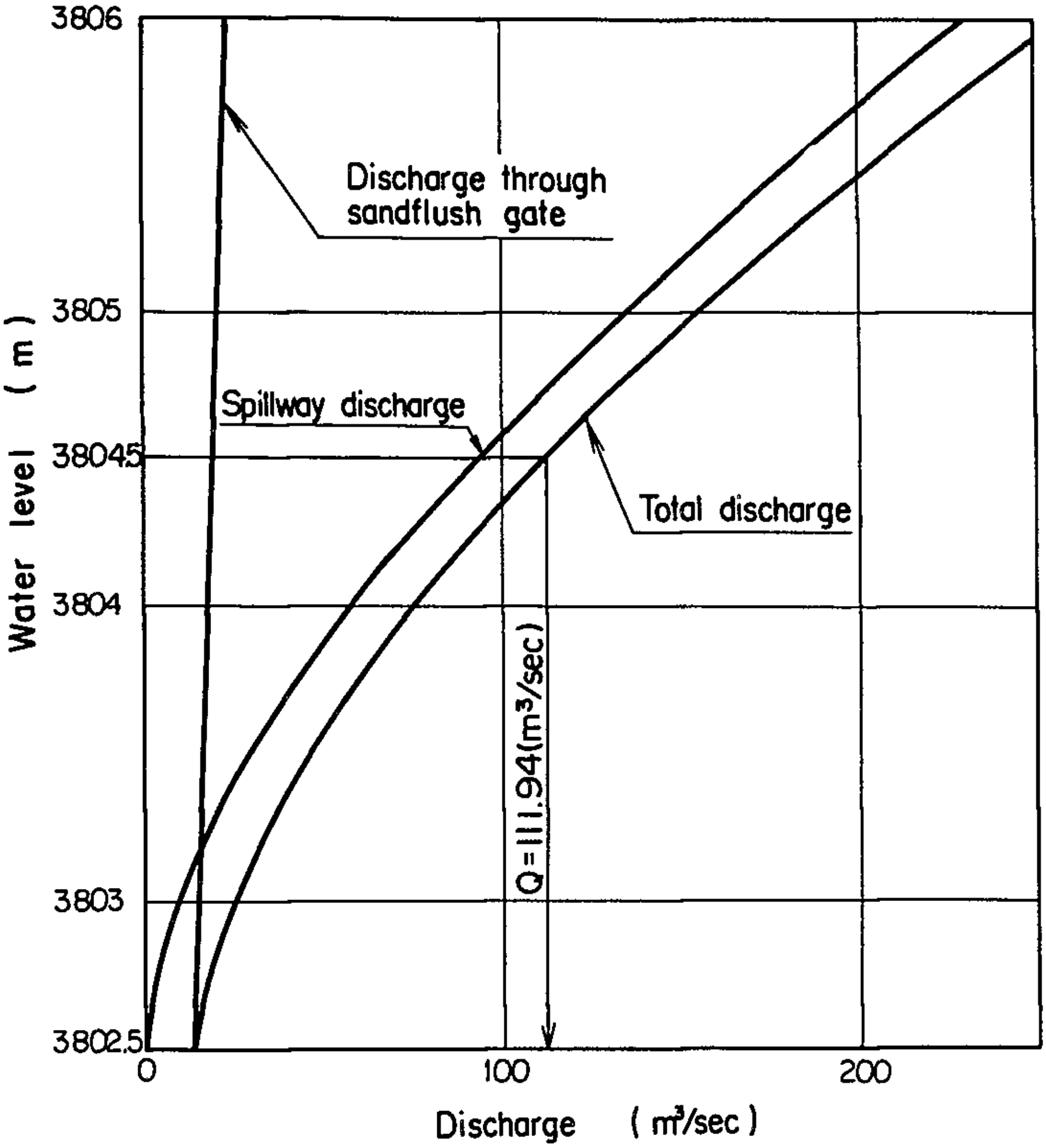
Table 3-1-2 Outflow discharge through sand flush gate

Water level (m)	Upstream water depth Ho m	a/Ho	Coefficient discharge C.v	Discharge Q m^3/sec
3,802.5	2.1	0.95	0.70	13.91
3,803.0	2.6	0.77	0.65	15.15
3,803.5	3.1	0.65	0.64	16.79
3,804.0	3.6	0.56	0.63	18.22
3,804.5	4.1	0.49	0.625	19.62
3,805.0	4.6	0.43	0.62	20.90
3,805.5	5.1	0.39	0.618	22.17

Table 3-1-3 Spillway capacity

Water level (m)	Discharge (m^3/sec)		Total discharge (m^3/sec)
	Spillway	Sand flush	
3,802.5	0	13.91	13.91
3,803.0	9.53	15.15	24.68
3,803.5	29.27	16.79	46.06
3,804.0	57.18	18.22	75.40
3,804.5	92.32	19.62	111.94
3,805.0	134.06	20.90	154.96
3,805.5	181.84	22.17	204.01

Fig. 3-1-1 Spillway Capacity



Calc 3-1-2 Stability analysis of intake dam

a) General

Stability analysis was performed on the typical section of intake dam under below mentioned conception.

- No tensile stress must be produced along the upstream face of dam in any conditions.
- Safety factor against sliding, obtained by following formula, is not less than 4.

$$Fs = \frac{IA + \mu \Sigma V}{\Sigma H}$$

where

Fs: Safety factor against sliding

I : Shearing strength of foundation rock (= 150 t/m^2)

A : Base area (m^2)

 : Coefficient of friction between rock and concrete (= 0.75)

V: Sum of vertical loads (t)

H: Sum of horizontal loads (t)

- Compressive stresses on foundation rock are not more than allowable value below.

Foundation rock = 200 t/m^2

b) Design loads

- Dead loads

Unit weight of concrete	γ_c = 2.3 t/m^3
Unit weight of water	γ_w = 1.0 t/m^3

- Seismic load

Seismic coefficient K = 0.10

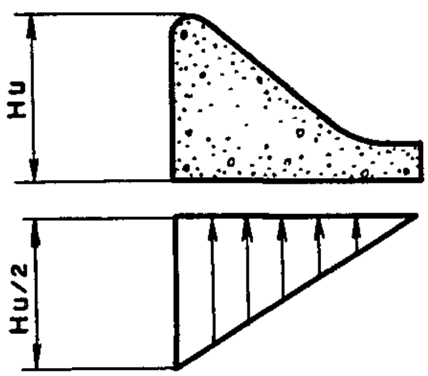
Seismic loads act in upstream direction.

- Hydrostatic pressure

. Hydrostatic pressure works on upstream and that of downstream is neglected.

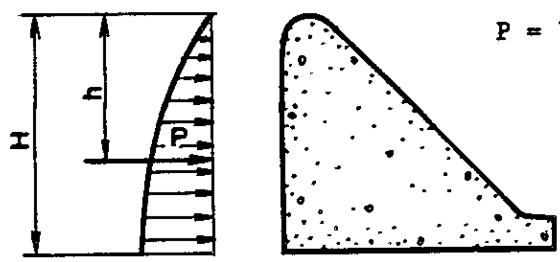
- Uplift

Distribution of uplift is assumed as below.



- Hydrodynamic pressure

Hydrodynamic pressure is assumed as follows:



$$P = 7/8\ \gamma_w K \sqrt{H \cdot h}\quad (t/m^2)$$

- Sedimentary pressure

Design level of sedimentation is E.L. 3,802.5 m.

$$Pe = Ce\gamma_s\ d\ (t/m^2)$$

where

Ce: Coefficient of sedimentary pressure (= 0.5)

$_s$: Unit weight of sediment (= 1.1 t/m^3)

d : Depth of sediment (m)

Note: Some values, which is impossible to assume without any experiments, are taken as same as those of Paucartambo II in Peru.

c) Result of calculations

- Normal condition

Load	V (t)	H (t)	x (m)	y (m)	M+ (t-m)	M- (t-m)
Dead L.	19.619	-	1.533	-	30.076	-
Hydrost. P.	-	6.125	-	1.167	7.148	-
Uplift	-8.750	-	1.667	-	-	14.586
Sediment P.	-	3.369	-	1.167	3.932	-
Total	10.869	9.494			41.156	14.586

Moment (+): Clockwise

$$F_{overturning} = \frac{\Sigma M+}{\Sigma M-}$$

$$= 2.82$$

$$x = \frac{\Sigma M}{\Sigma V}$$

$$= 2.445$$

$$e = 0.06 < B/6 = 0.83$$

Reaction forces

$$P = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right)$$

$$= \begin{cases} 2.32 \ (t/m^2) \\ 2.03 \end{cases} < 200 \ t/m^2$$

<u>OK</u>

$$F_{sliding} = \frac{IA + \mu\Sigma V}{\Sigma H}$$

$$= \frac{150 \times 1 + 0.75 \times 10.869}{9.494}$$

$$= 16.7 \quad > 4$$ OK

- Earthquake condition

Load	V (t)	H (t)	x (m)	y (m)	M+ (t-m)	M- (t-m)
Dead L.	19.619	-	1.533	-	30.076	-
Seismic L.	-	-1.962	-	0.974	-	1.911
Hydrost. P.	-	6.125	-	1.167	7.148	-
Uplift	-8.750	-	1.667	-	-	14.586
Hydrody. P.		-0.714	-	1.40	-	1.00
Sediment P.	-	3.369	-	1.167	3.932	-
Total	10.869	6.818			41.156	17.497

$$F_{overturning} = \frac{\Sigma M+}{\Sigma M-}$$

$$= 2.35$$

$$x = \frac{\Sigma M}{\Sigma V}$$

$$= 2.176$$

$$e = 0.32 \quad < \frac{B}{6}$$

Reaction forces

$$P = \frac{\Sigma V}{B} (1 \pm \frac{6e}{B})$$

$$= \begin{cases} 3.02 \ (t/m^2) \\ 1.33 \ (t/m^2) \end{cases} \quad < 200 \ t/m^2$$

OK

d) Conclusion

Stability analysis has done in normal and earthquake conditions briefly.

According to these results, intake dam is safely enough but now any properties of materials were not given. Therefore after investigating these properties, stability analysis should be done more in detail.

Fig. 3-1-2 Upstream View

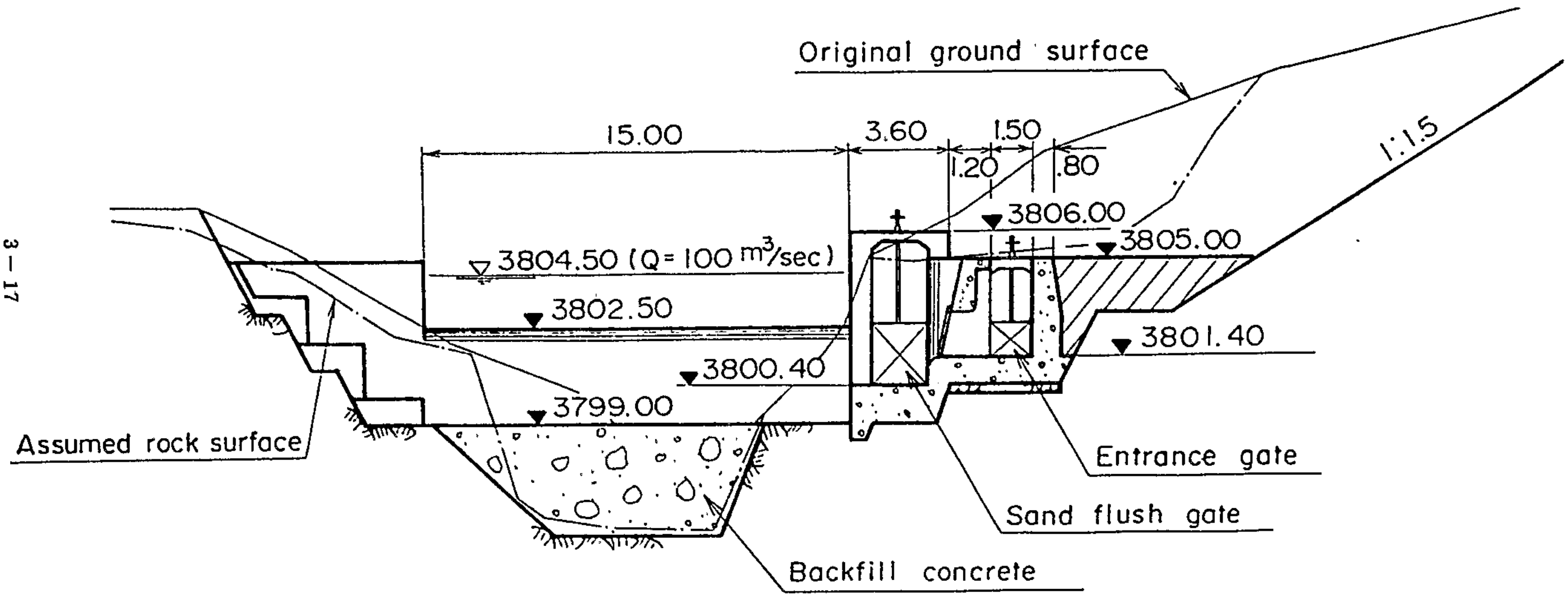
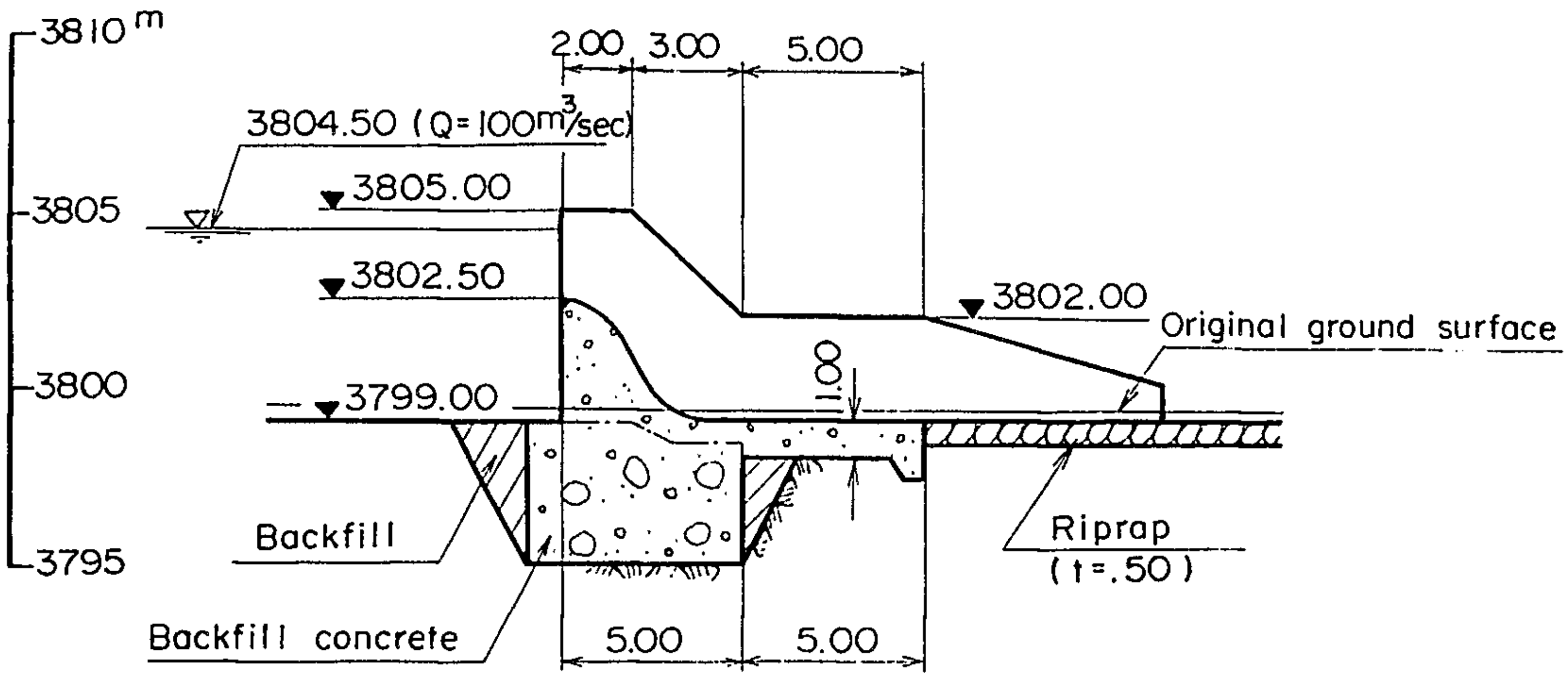


Fig. 3-1-3 Overflow Section



Calc 3-2-1 Design of sedimentation basin

a) Required length of sedimentation basin

$$L \geqq \frac{h}{V_g} u$$

where

L : Length of sedimentation basin (m)

h : Water depth (m)

u : Average velocity (m/sec)

V_g: Terminal settling velocity of finest sand particle required to be settled (cm/sec)

Terminal settling velocity of finest sand particle required to be settled is given in Table 3-2-1.

Table 3-2-1 Terminal settling velocity

d (mm)	Vg (cm/sec)
0.5	12.554
0.4	8.034
0.3	4.519
0.2	2.009
0.11	0.589
0.1	0.502

(These values were used for the design of Paucartambo II in Peru.)

In this study, average velocity was decided as 0.5 (m/sec) and finest sand particle required to be settled was chosen d = 0.2 (mm). Width of sedimentation basin was to be 3.5 (m) judging from topographical conditions.

$$u = 0.5 \text{ (m/sec)}$$

$$h = \frac{Q}{BU}$$

$$= \frac{2.2}{3.5 \times 0.5}$$

$$= 1.26 \text{ (m)}$$

$$\therefore L \geqq \frac{h}{V_g} u$$

$$= \frac{1.26}{2.009 \times 10^{-2}} \times 0.5$$

$$= 31.4 \text{ (m)}$$

L was decided as 40 (m) with some addition for safety.

b) Dimensions of sedimentation basin

Length	40.0 (m)
Width	3.5 (m)
Average velocity	0.5 (m/sec)
Average water depth	1.26 (m)
Min. particles to be settled	0.2 (mm)

Dimensions of sedimentation basin was decided as above.

Calc 3-3-1 Discharge capacity of canal (I)(II)(III) tunnel (I)(II)

a) Basic formula

Discharge capacity is given by the following formula.

$$Q = \frac{A}{n} R^{2/3} I^{1/2}$$

where

Q : Discharge (m^3/sec)

n : Coefficient of roughness (Table 3-3-1)

R : Hydraulic mean depth (m)

I : Slope $I = 1/1000$

Table 3-3-1 Coefficient of roughness

Condition	n
Mortar masonry	0.025
Concrete	0.014
Rock	0.035

b) Results

Discharge capacity curves are shown in Fig. 3-3-1 ∿ 3-3-5.

Fig. 3-3-1 Canal, Type-I

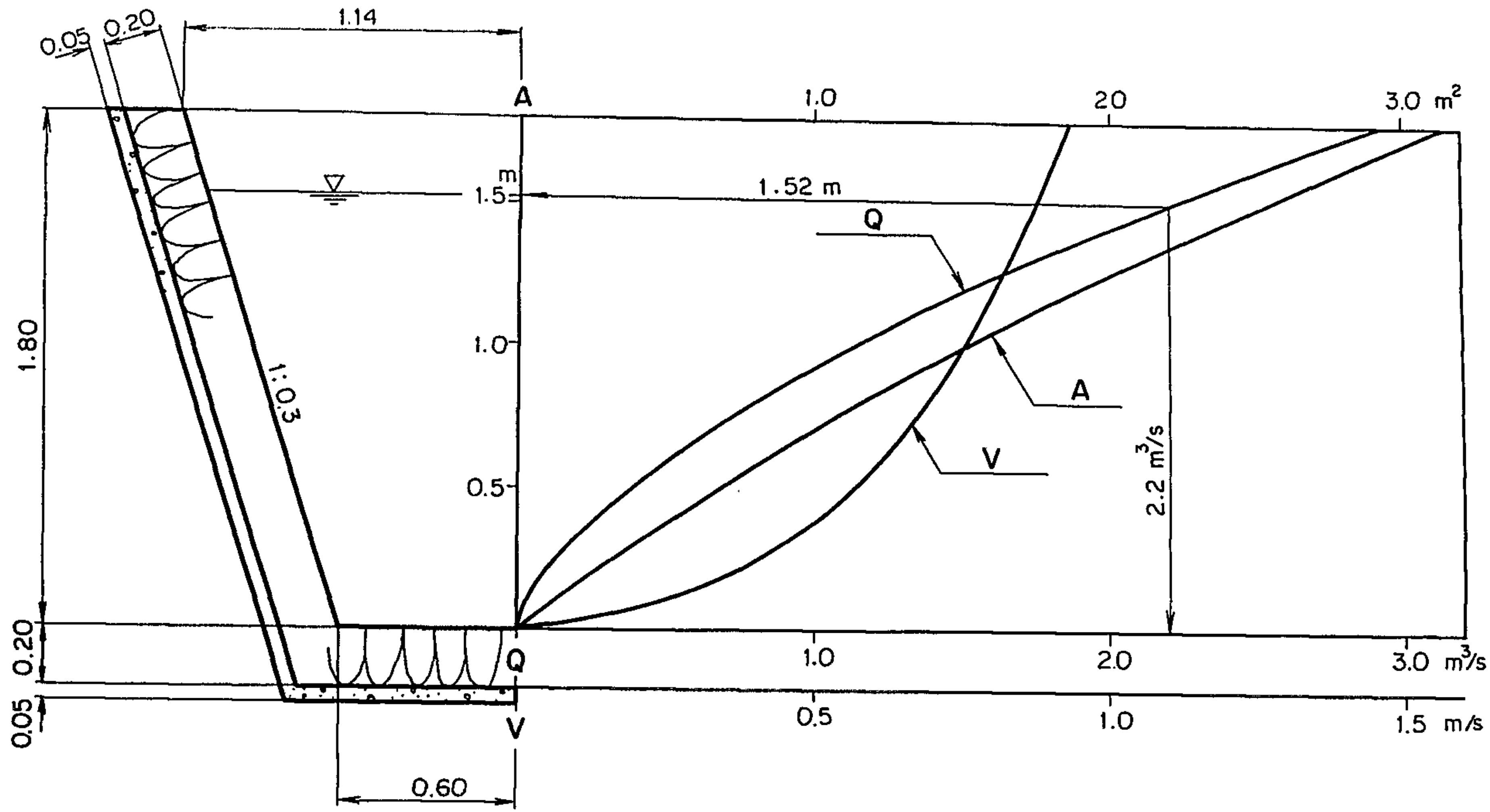


Fig. 3-3-2 Canal, Type-II

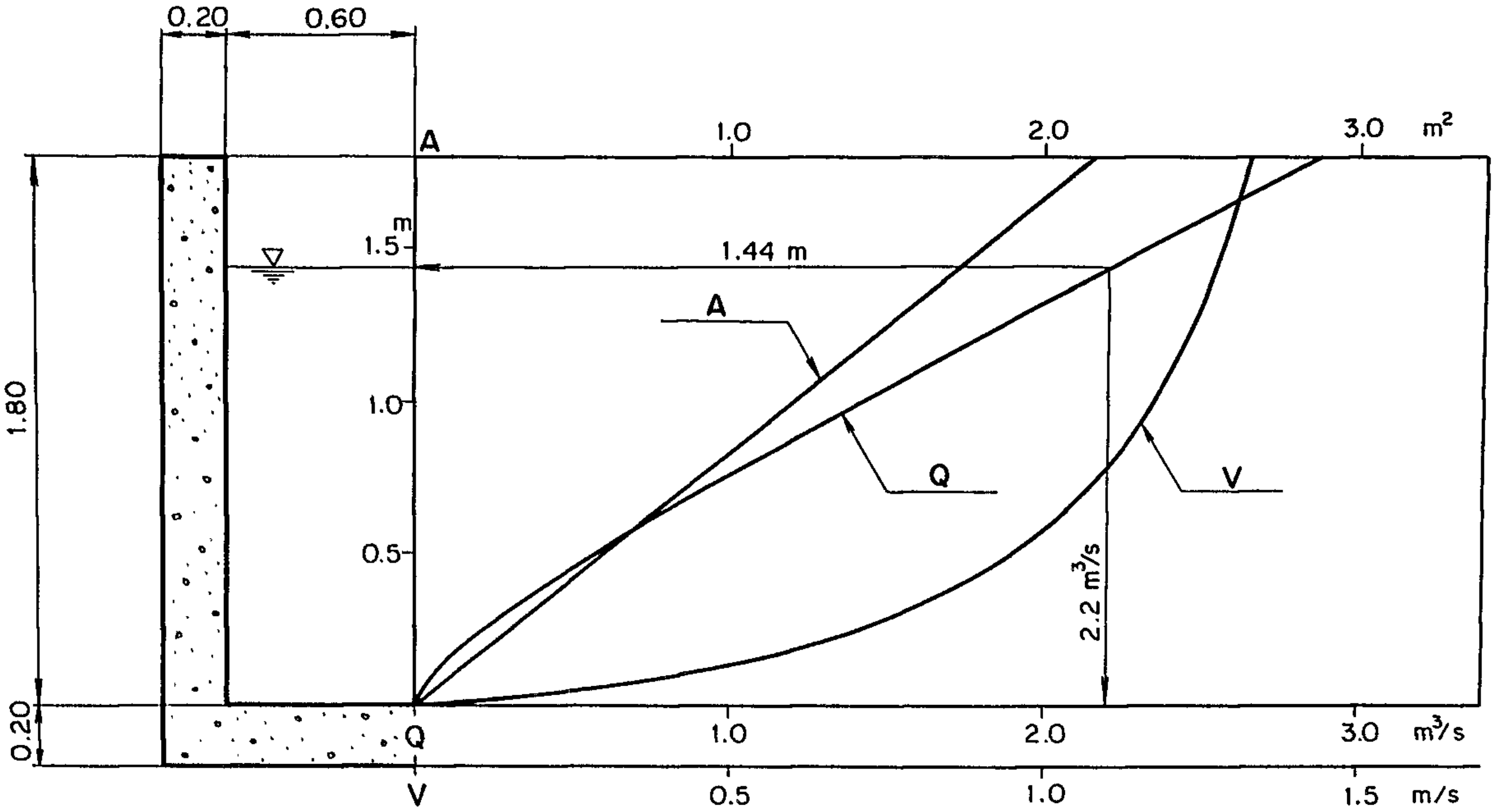


Fig. 3-3-3 Canal, Type-III

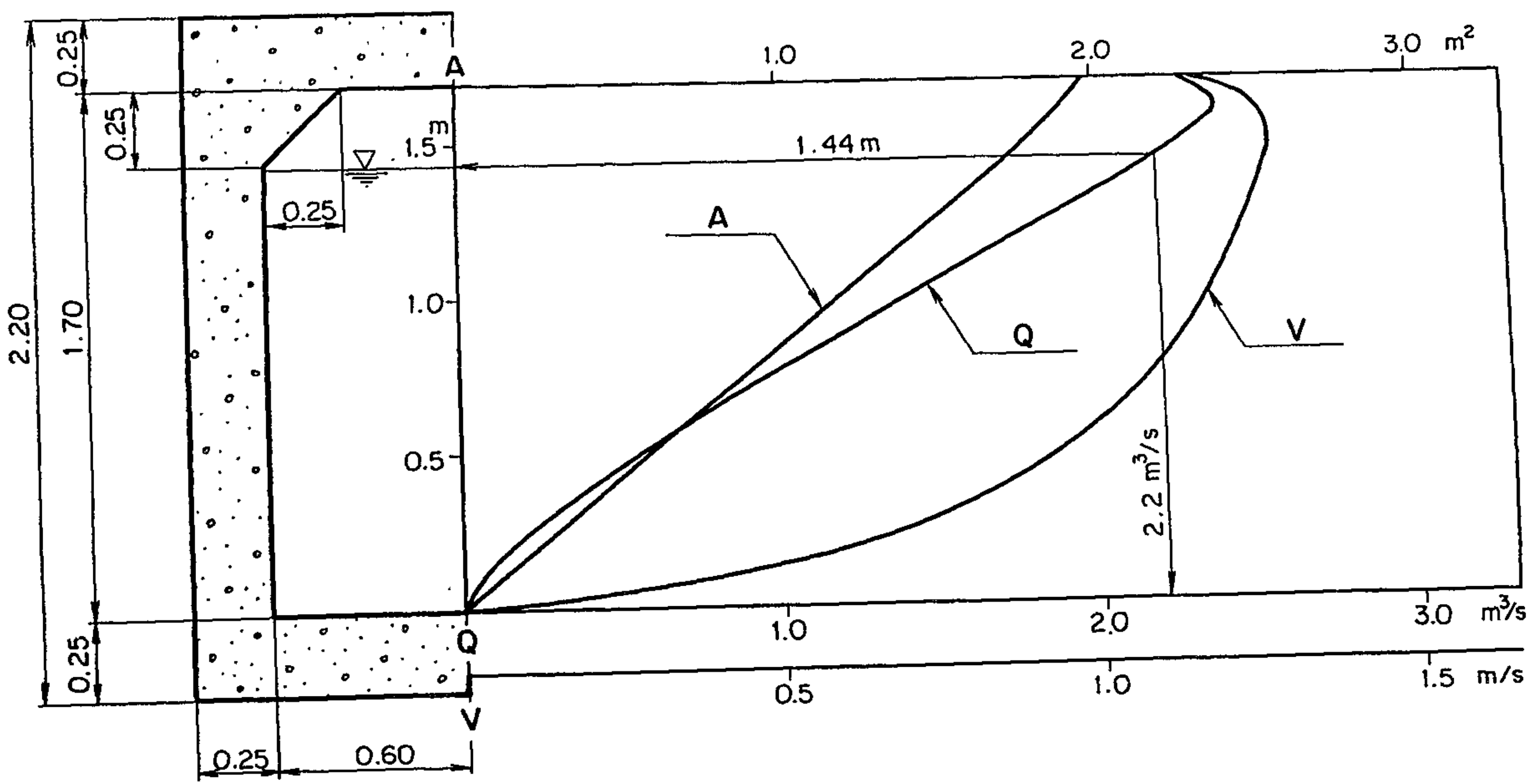


Fig. 3-3-4 Tunnel, Type-I

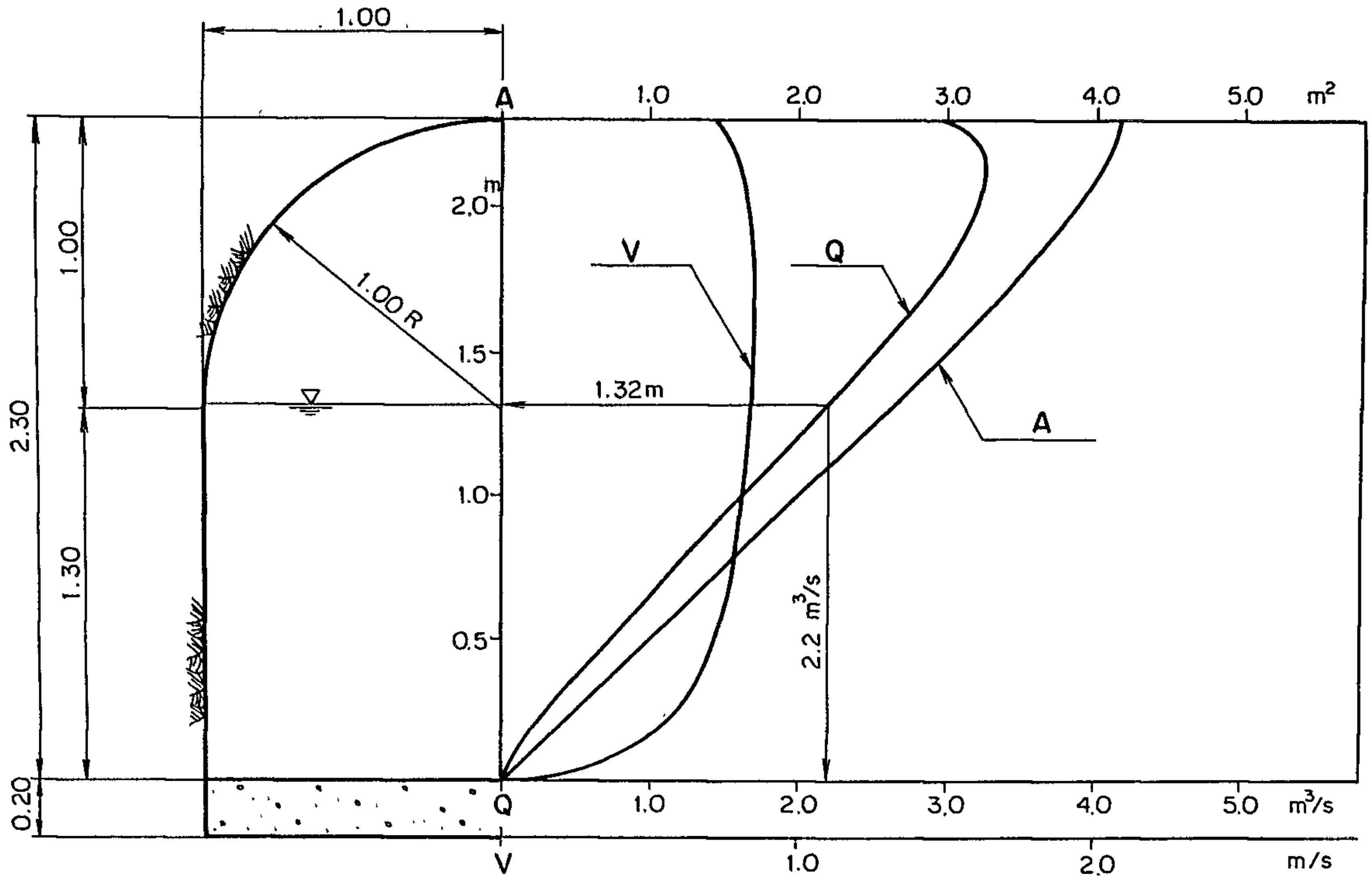
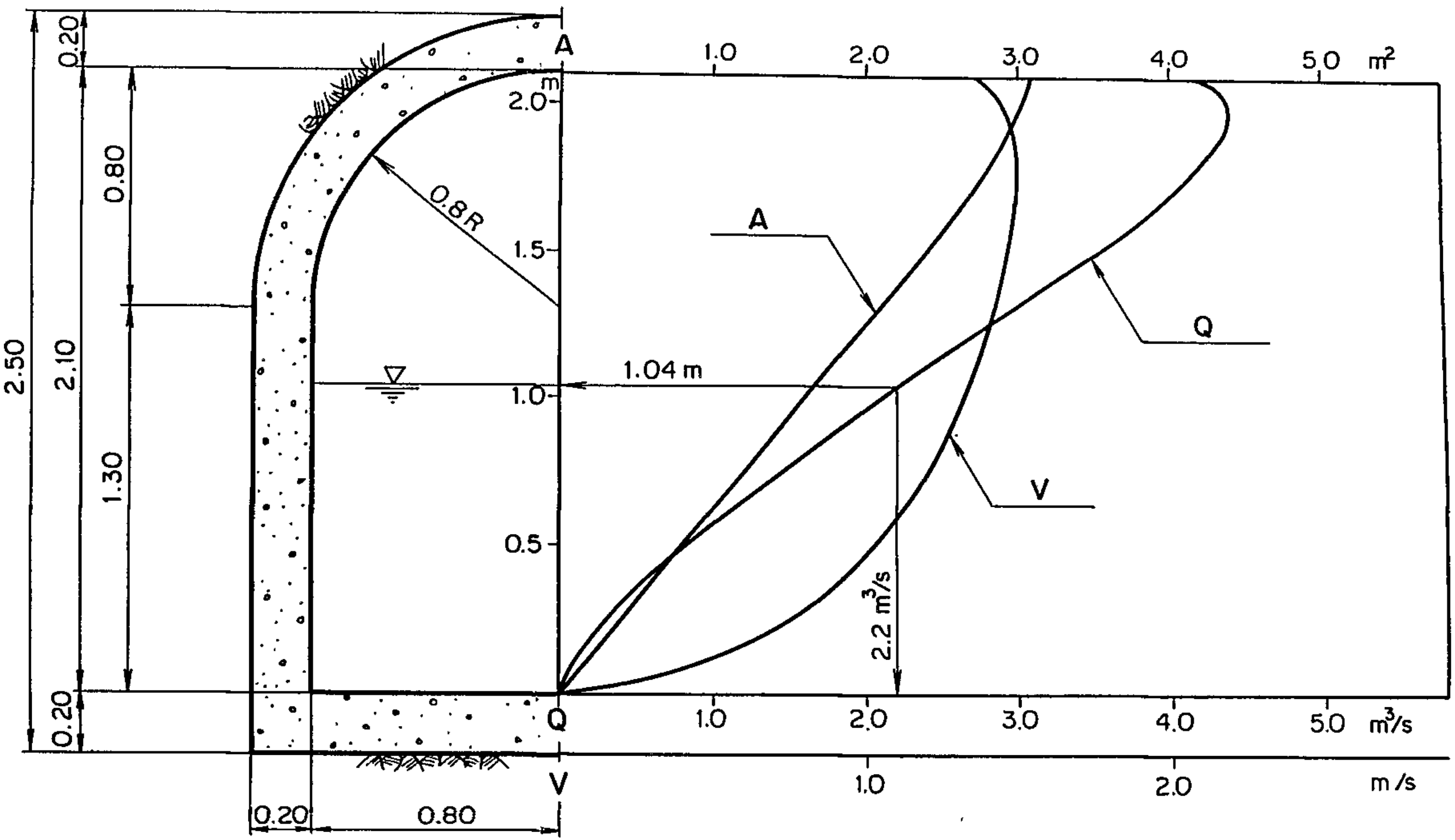


Fig. 3-3-5 Tunnel, Type-II



Calc 3-5-1 Water hammer

a) Basic formula

$$Hm = \frac{NHo}{2} + \frac{Ho}{2}\sqrt{N^2 + 4N} \qquad (1)$$

$$N = (\frac{LVo}{gTHo})^2 \qquad (2)$$

where

Hm: Max. water pressure rise (m)

Ho: Max. static head at turbine (= 249.2 m)

L : Length of penstock (= 665.5 m)

Vo: Mean velocity in penstock (m/sec)

g : Acceleration of gravity (= 9.8 m/sec^2)

T : Closing time (= 30 sec)

b) Results

- Case 1

  In this case, Hm was calculated by formula (1) and (2) with T = 30 (sec).

  Result is shown below.

  Hm = 7.033 (m)

  P = Hm/Ho

  = 2.82%

- Case 2

  In this case, Hm was assumed as 15% of max. static head at turbine.

  Hm = 249.2 x 0.15

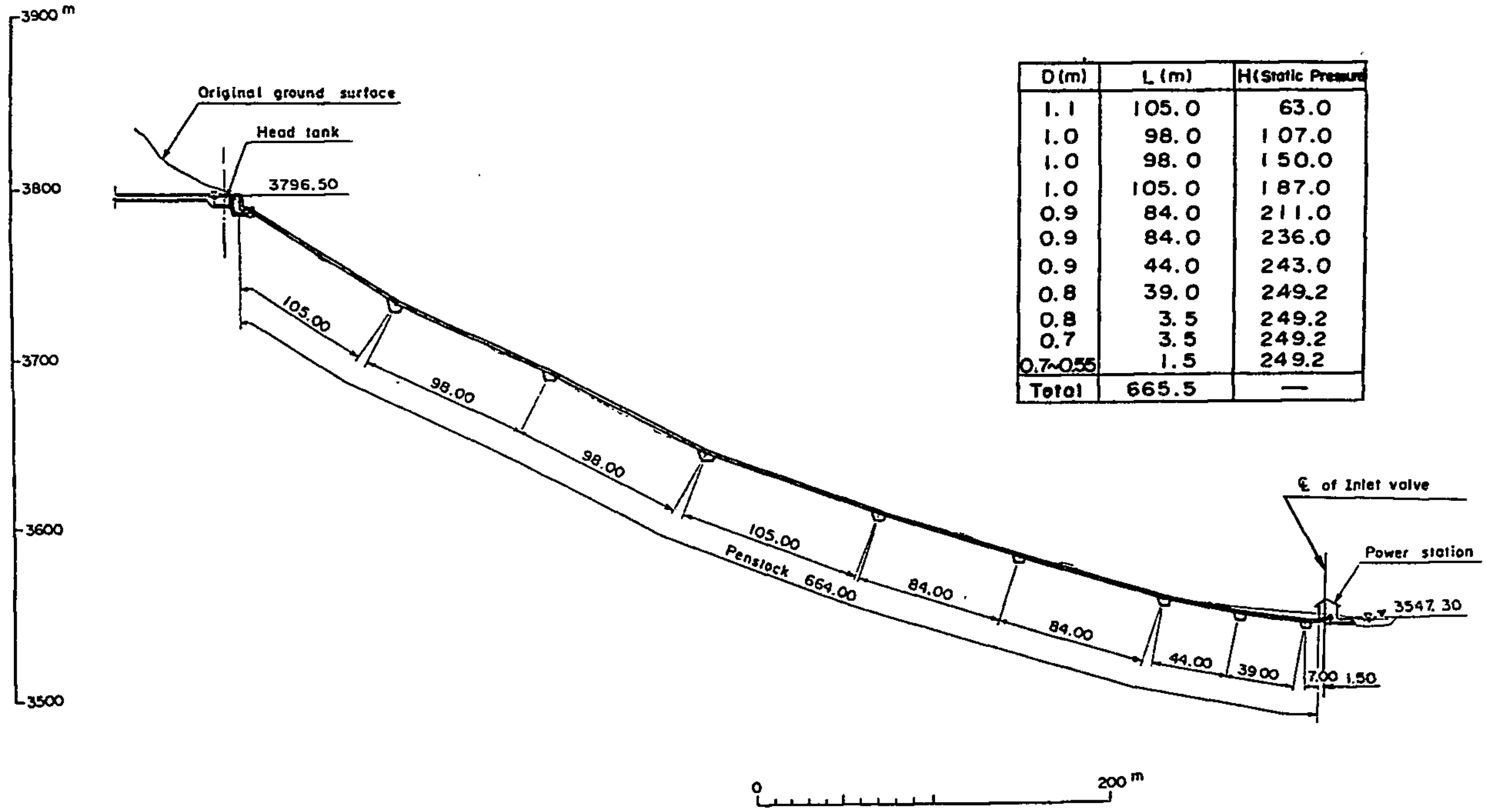
  = 37.38 (m)

  P = 15%

Comparison between two cases, case 2 was taken as a design head for designing penstock.

$$\frac{\text{Hm (Max. water pressure rise)} = 37.38\ (\text{m})}{\text{(at the center of turbine)}}$$

Fig. 3-5-1 **Penstock, Profile**



D (m)	L (m)	H (Static Pressure)
1.1	105.0	63.0
1.0	98.0	107.0
1.0	98.0	150.0
1.0	105.0	187.0
0.9	84.0	211.0
0.9	84.0	236.0
0.9	44.0	243.0
0.8	39.0	249.2
0.8	3.5	249.2
0.7	3.5	249.2
0.7~0.55	1.5	249.2
Total	665.5	—

Calc 3-5-2 Design of penstock

Diameter and thickness in each section were decided in this study

a) Head loss

Total head loss in penstock should be below 7.5 (m) considering power product, therefore diameter was decided to satisfy with this condition.

Generally head loss due to friction is over 90% of total head loss. According to this fact, head loss due to friction is only calculated in this study and total head loss is assumed with some addition.

- Basic formula

$$h_f = \frac{124.5\ n^2}{D^{4/3}} \times L \times \frac{Q^2}{2gA^2}$$

where

Q: Discharge (= 2.2 m^3/sec)

A: Area of penstock's section (m^2)

D: Diameter of penstock (m)

n: Coefficient of roughness (= 0.012)

L: Length (m)

- Result

Diameter of penstock in each section is given as follows:

D (m)	L (m)	Head loss (m)
		x Q^2
1.1	105.0	9.365×10^{-2}
1.0	98.0	1.453×10^{-1}
1.0	98.0	1.453×10^{-1}
1.0	105.0	1.556×10^{-1}
.90	84.0	2.184×10^{-1}
.90	84.0	2.184×10^{-1}
.90	44.0	1.144×10^{-1}
.80	39.0	1.901×10^{-1}
.80	3.5	1.706×10^{-2}
.70	3.5	3.477×10^{-2}
(.70 ∿ .55)	1.5	5.394×10^{-2}
Total	665.5	$1.387 \times Q^2$

$$\therefore h_{LOSS} = 1.387 \times 2.20^2 \text{ (cm)}$$

$$= 6.174 \text{ (m)}$$

Total head loss in penstock is assumed 7.2 m with 15% of additional.

$$\underline{H_{LOSS} = 7.2 \text{ (m)}}$$

b) Thickness of penstock

- Thickness of penstock

Thickness of penstock is obtained by next formulas.

$$t = \frac{PD}{2\sigma_a} + \varepsilon \qquad (1)$$

$$t \geqq \frac{D + 800}{400} \qquad (2)$$

where

- t : Thickness (mm)
- P : Internal water pressure
- D : Diameter
- σ_a: Allowable stress (= 1300 kg/m^2 SS41)
- ε : Corrosion allowance (ε = 1.5 mm)

Note: P was shown in chapter-5 (water hammer).

- Result

Result of calculations are shown below:

```
************************

  *** INPUT DATA ***
  N   D(M)      L(M)
  1  1.100   105.000
  2  1.000    98.000
  3  1.000    98.000
  4  1.000   105.000
  5  0.900    84.000
  6  0.900    84.000
  7  0.900    44.000
  8  0.800    39.000
  9  0.800     3.500
 10  0.700     3.500
 11  0.550     1.500

 ** WATER HAMMER **

DH=   7.033 (M)
DP=   2.82  (%)

**  DESIGN HEAD  **
------------------------
  N       DESIGN-H

  1        68.897 (M)
  2       118.402 (M)
  3       166.906 (M)
  4       209.804 (M)
  5       238.522 (M)
  6       268.240 (M)
  7       277.712 (M)
  8       286.102 (M)
  9       286.299 (M)
 10       286.495 (M)
 11       286.580 (M)
------        -----
```

```
** REQUIRED THICKNESS

  T(  1)=    4.75 (MM) *
  T(  2)=    6.05 (MM)
  T(  3)=    7.91 (MM)
  T(  4)=    9.56 (MM)
  T(  5)=    9.75 (MM)
  T(  6)=   10.78 (MM)
  T(  7)=   11.11 (MM)
  T(  8)=   10.30 (MM)
  T(  9)=   10.30 (MM)
  T( 10)=    9.21 (MM)
  T( 11)=    7.56 (MM)

* IS DECIDE BY MIN.-THIC
KNESS

** WEIGHT OF PENSTOCK **

  *** THICKNESS ***

  T(  1)=    5.00 (MM)
  T(  2)=    7.00 (MM)
  T(  3)=    8.00 (MM)
  T(  4)=   10.00 (MM)
  T(  5)=   10.00 (MM)
  T(  6)=   11.00 (MM)
  T(  7)=   12.00 (MM)
  T(  8)=   11.00 (MM)
  T(  9)=   11.00 (MM)
  T( 10)=   10.00 (MM)
  T( 11)=    8.00 (MM)
-----------------------
**  SIGMA(TENSILE)  **
     UNIT(KG/CM^2)

SGM(  1)=  1082.67
SGM(  2)=  1076.38
SGM(  3)=  1283.89
SGM(  4)=  1234.14
SGM(  5)=  1262.76<σsa=1300 kg
SGM(  6)=  1270.61
SGM(  7)=  1190.19
SGM(  8)=  1204.64
SGM(  9)=  1205.47
SGM( 10)=  1179.68
SGM( 11)=  1212.45
-----------------------
```

3.5 Penstock

The penstock, as shown in Fig. 3-10-7, is to be a surface type in consideration of the topographical and geological conditions, and ease of construction. As a result of surface reconnaissances, it was seen that talus and terrace deposits of thickness from 3 to 5 m exist overlying the basement slate and quartzite alternations. The deposits are cobbles and sand-gravel which are well-consolidated and are stable. Since the foundations of anchor blocks and saddles are not anchored on rock, care will need to be exercised in carrying out construction. The slope of the mountain is 30 deg at the upper part and 20 deg at the lower, and there will be no special problem in installation of the penstock.

The penstock is to be a single line of inside diameters 1.10 m to 0.55 m. The length is 664 m. Calculations were made of water hammer by Cal. 3-5-1 and head loss of the penstock and shell thickness by Cal. 3-5-2. As a result, the maximum water hammer was found to be 37.00 m at the turbine center, the head loss of the penstock 7.20 m, and the shell thickness 5 - 12 mm.

3.6 Power Station

3.6.1 Powerhouse

The powerhouse is to be constructed on a river terrace at the right bank of the Rio San Juan. This location is at the upstream opposite bank of Huallanca, immediately above the existing Electro Peru power station.

There is one unit of turbine-generator. A horizontal-shaft Pelton turbine is adopted, the powerhouse floor elevation and turbine center elevation are high at 3,546.5 m and 3,547.3 m, respectively, in comparison with design flood level of EL. 3,544 m, so that operation during flood will be possible.

Further, as shown in Fig. 3-10-9, since the turbine is of horizontal-shaft, the floor is single and the construction of the floor surface simple, resulting in a one-story structure. The dimensions of the building are width of 10.00 m, height of 8.30 m and length of 23.00 m. Footing beams and columns are made of reinforced concrete and runway girders of the crane of H-steel, and walls of concrete block.

The roof structure is to be of steel and corrugated steel sheet with skylights provided for lighting, while windows are to be clerestory for prevention of burglary and danger to outsiders.

The powerhouse foundation is a well-consolidated gravel layer. Since as a result of trial boring bedrock was not reached, considerations are given to the design of the building so that load will be distributed uniformly over a wide area. In construction work it is necessary for excavation to be done exercising care not to disturb the ground as much as possible. However, it is judged measures such as reinforcing treatment of the ground are not required.

3.6.2 Hydraulic Turbine

Huanzala Hydro-electric Power Station is a run-off-river type and its effective head will be 242 m and its maximum power discharge 2.2 cu.m/sec.

As stated in the section 1.4.3, this power station is to have one turbine-generator unit. A horizontal-shaft, 1-runner, 2-nozzle Pelton turbine is adopted according to the above conditions. Recently, as a result of a technological revolution regarding hydro-electric power generating equipment, it is no longer impossible for a Francis turbine to be used under the conditions mentioned. Therefore, comparisons were made of the characteristics of the two types, and as a result, the Pelton turbine is adopted for this Project, the reasons being as described below.

Comparison of Pelton and Francis Turbine

Item	Pelton Turbine	Francis Turbine
Hydraulic type of turbine	Impulse Turbine	Reaction Turbine
Rated output 1/	4,450 kW	4,480 kW
Rated revolution speed	450 r.p.m.	1000 1200 r.p.m.
Mechanism	Simple	Complicated
Efficiency	Max. 86% (Approx.) Not changeable at light load	Max. 87% (Approx.) Too much down at light loan
Affect by cavitation	Not so much	Serious
Abrasion for sand flow	Not so much	Serious
Lower limit of load 2/	20% (Approx.)	40% (Approx.)
Overhaul of turbine	Simple	Complicated
Padding repair of runner	Possible	Impossible
Speed rise/pressure rise at full load trip	1.1 1.5/1.05 1.1	1.25 1.3/1.3 1.4
Cost of turbine-generator unit	1.2	1.0

Note: 1) The effective head of the Francis turbine is increased approximately 2 m by its draft tube.

2) Cavitation will be severe if a Francis turbine is operated at a partial load of less than 40%, and production of vibration and reduction of efficiency are liable to occur.

As shown in the above table, the Francis turbine is of slightly higher rated output and the price of the main equipment is approximately 20% cheaper. However, with regard to other items, the Pelton turbine is superior. Particularly, in Peru, there is generally a high content of silica sand in river water. Since silica sand is hard and shapes are angular, it can cause severe abrasion of runners and guide vanes of turbine. For example, at Machu-pichu Power Station (output 40 MW), abrasion of runners and guide vanes of the Francis turbines is severe and overhauling is done once every year. Runners and guide vanes are very often repaired. Other than this example, there have been many cases of abrasion of Francis turbines by sediment. In the electric power circles of

Peru, Pelton turbines have been favored over Francis turbines so far as the design conditions of hydraulic equipment have permitted. Judging by the geological conditions of the Rio San Juan drainage basin it is imagined there is a high content of silica sand. In the water quality test carried out in December 1982, 28 ppm of silica sand was detected in sample water of low turbidity. The silica sand will increase in the high water stages. The Pelton turbine is selected for the Project because of the above reasons.

Outline of Turbine

Type	:	Horizontal-shaft, 1-runner, 2-nozzle Pelton turbine
Effective head	:	242 m
Maximum power discharge	:	2.2 cu.m/sec
Rated output	:	4,450 kW
Rated revolution speed	:	450 rpm
Specific speed	:	22.2 m-kW
Inlet valve	:	Butterfly valve
Control system	:	Electric-hydraulic

3.6.3 Generator

The generator directly coupled with the hydraulic turbine is a horizontal-shaft, alternating-current, 3-phase, synchronous generator, with the cooling system an air-cooled type, a water cooler not to be especially provided.

The rated power factor was selected at 0.82 taking into account the power factor of Huanzala Mine and the rated power factor of the existing diesel plants.

The exciting system is to be a brushless type aiming for a maintenance-free condition. As for the automatic voltage regulator, a transistorized one will be used.

Outline of Generator

Type	:	Horizontal-shaft, AC, 3-phase, synchronous generator
Rated output	:	5,200 kVA
Rated voltage	:	6.6 kV
Rated power factor	:	0.82
Rated frequency	:	60 Hz
Rated revolution speed	:	450 rpm
Exciting system	:	Brushless (AC exciter with semi-conductor-type rectifier)

3.6.4 Main Transformer

The main transformer is an outdoor, 3-phase, oil-immersed, self-cooled type with a primary delta connection, secondary star connection, and resistance-grounded neutral system, the primary-side matching the generator voltage, and the secondary side the transmission voltage. It is thought that a Peruvian-made product will be adequate for this class of transformer.

Outline of Transformer

Type	:	Outdoor, 3-phase, oil-immersed, self-cooled
Rated capacity	:	5,200 kVA
Rated voltage	:	6.6/33 5% kV
Rated frequency	:	60 Hz

3.6.5 Switchgear

For both the generator side, 6.6 kV, and the transmission line side, 33 kV, vacuum circuit breakers is adopted aiming for a maintenance-free condition. The 6.6-kV side is to comprise indoor-type, metal-clad switchgear, while the 33-kV side is a conventional type installed outdoors. The one-line diagram of the power station is shown in Fig. 3-10-11. For the equipment layout Fig. 3-10-9 should be referred to.

3.6.6 Control System

The control system of the power station is a one-man control type, and the main control procedures is possible to undertake at the control room. Since the output control systems below will be respectively adopted in the high- and low-water seasons for this power station, it is expected that there will be extremely little "start stop" controlling of main equipment.

(1) High-water season: Governor-free load, frequency control operation

(2) Low-water season (runoff less than 2.2 cu.m/sec): Level governor operation by water level governor provided at head tank

As described above, considerations are given in design of the control system so that an operator would be almost unnecessary during normal operation.

3.6.7 INSULATION DESIGN

The powerhouse equipment, transmission line, and mine interconnection and transforming facilities to be provided in connection with this hydro-electric power station will all be in an area of elevations from 3,850 to 4,000 m. Consequently, it will be necessary for designing to be done taking into consideration reduction in air insulation.

Altitude correction factors do not exist in Japanese Standards, but the factors in U.S. standards are specified as shown in the table below.

American National Standard Definitions and Requirements for Altitude Correction Factors

Altitude Correction Factor to be applied to:

Altitude Feet	Altitude Meters	Rated Withstand Voltage Col 1	Current Rating Col 2	Ambient Temperature Col 3
3300	1000	1.00	1.00	1.00
4000	1200	0.98	0.995	0.992
5000	1500	0.95	0.99	0.980
6000	1800	0.92	0.985	0.968
7000	2100	0.89	0.98	0.956
8000	2400	0.86	0.97	0.944
9000	2700	0.83	0.965	0.932
10000	3000	0.80	0.96	0.920
12000	3600	0.75	0.95	0.896
14000	4200	0.70	0.935	0.872
16000	4800	0.65	0.925	0.848
18000	5400	0.61	0.91	0.824
20000	6000	0.56	0.90	0.800

For maximum ambient of 40°C for nonenclosed switches and 40°C outside the enclosure for enclosed switches

For operation at continuous current rating

All electrical facilities installed in the highlands of South America are designed in accordance with the above standards. It was accordingly decided to apply the above standards for the Project and an altitude correction factor of 0.70 was adopted.

3.7 Transmission Line

3.7.1 Topographical Conditions of Transmission Route

The length of the transmission line from Huanzala Hydro-electric Power Station (EL. 3,546 m) to Huanzala Mine (EL. 4,000 m) will be approximately 10 km. The transmission line route goes through the basin area of Huallanca for a distance of approximately 2 km from the power station, passes the west edge of the town of Huallanca, through a eucalyptus forest, and running along the right bank of the Rio Torres, reaches the mine. The vegetation in this area, except for eucalyptus trees, is grass with nothing else noticeable. The greater part of the eucalyptus forest under the transmission line is the property of Santa Luisa.

The transmission line route was selected along the road at the right bank of the Rio Torres for convenience of approach at the stages of construction and future maintenance. The route is shown in Fig. 3-10-13.

3.7.2 Meteorological Conditions

The climate of this region may be divided into dry seasons (May - August, October - November), a rainy season (December - April), and a minor rainy season in September, the annual precipitation being 900 to 1,100 mm. The temperature has an annual mean of 15°C with the minimum being -2 to -3°C for a typical highland climate of diurnal temperature variation being greater than seasonal variation.

The isokeraunic level (IKL) is seen to be about 60 days and it is necessary for consideration to be given to lightning protection design. The maximum wind speed is unknown since there is no observation station in the vicinity. But there will be no problem if the value of 25 km/sec given in the design standards of the Republic of Peru is applied.

3.7.3 Design of Transmission Line

(a) Selection of Voltage

With the length of the transmission line as 10 km and the power transmitted as 4,200 kW, it will suffice for the transmission voltage to be 22 kV. However, a voltage of 22 kV is never used in the Republic of Peru. Therefore, 33 kV matching Peruvian standards is adopted.

(b) Number of Circuits

For the Project, a proposal for 33 kV and two circuits was first contemplated. But as a result of studies it is deemed possible for transmission between the power station and Huanzala Mine to be carried out with ample stability even with a single circuit. Therefore, single circuit was adopted in order to reduce initial investment.

(c) Conductor

Either steel-reinforced aluminum stand (ACSR) or hard-drawn copper conductor (HDCC) is conceivable as the conductor to be used, but ACSR is to be used in view of economy.

Regarding conductor size, it will be possible for transmission to be done with transmission loss of not more than 5% even with AWG 2/0 (67.42 sq.mm). But with AWG 2/0 the steel core is single and deficient in reliability. This size is not generally used for trunk lines. Therefore, ACSR 120 sq.mm with a stranded steel core is used for the conductor.

(d) Insulator

Insulators are the 250-mm suspension type and the number in a string is 4 insulators as a result of adjustment using the previously-mentioned altitude correction factor.

(e) Overhead Ground Wire

As mentioned in connection with meteorological conditions, the IKL in this region is about 60 days so that one overhead ground wire is to be provided. The overhead ground wire used is to be galvanized steel strand (GSC) of 38 sq.mm, and the shielding angle is to be not more than 30 deg.

(f) Supports

Assuming the average span of this transmission line to be 100 m, steel towers, steel poles, concrete poles, and wood poles are conceivable to be used as supports. Concrete poles are to be adopted for the following reasons:

- Steel Tower, Steel Pole

  Since there is no prominent manufacturer in Peru, Electro-Peru and other electric utilities are relying on imports. Although it appears some fabrication is being done in small factories, galvanizing is not possible and the products lack reliability.

- Concrete Pole

  There are three companies manufacturing according to Peruvian standards who have abundant experience, while a complete assortment of arms and other accessories are also available.

- Wood Pole

  The supply of coniferous trees such as cedar and pine in Peru is small, and the only species of tree from which straight trunks of 15 to 16 m can be obtained is the eucalyptus. However, eucalyptus is prone to cracking while drying. Furthermore, there is no creosoting plant available. Therefore, wood poles used by electric utilities of Peru are being imported from Canada.

For the reasons given above, concrete poles which can be produced in Peru and which are standard products of high reliability from a technical standpoint are to be adopted. Standard pole assemblies are shown in Figs. 3-10-14 and 3-10-15.

3.8 Interconnecting Substation at Mine Site

As shown in Fig. 3-10-10, an outdoor substation (see Fig. 3-10-13) is provided at the east side (downstream side) of the existing diesel power plants in order to interconnect the existing Huanzala Mine system with Huanzala Hydro-electric Power Station and the transmission line to be constructed in this Project.

One 3-phase transformer of 5,200 kVA required for interconnection stepping down from the transmission voltage of 33 kV to the diesel power plant bus voltage of 2.2 kV is to be provided at this substation. Since both the primary and secondary sides of this transformer will have star connections, a tertiary winding (stabilizing winding) will be required.

Vacuum circuit breakers are to be used at both the 33-kV and 2.2-kV sides in aiming for maintenance-free operation. The 2.2-kV side breaker is to be an indoor, metal-clad type, while the 33-kV side breaker is to be a conventional type installed outdoors. The one-line diagram and equipment layout are shown in Fig. 3-10-12.

Outline of Transformer

Type	:	Outdoor, 3-phase, oil-immersed, self-cooled, with tertiary winding
Rated capacity	:	5,200 kVA
Rated voltage	:	33 5%/2.2 kV
Rated frequency	:	60 Hz

3.9 Interconnection with the Existing Distribution Systems of Electro Peru

There are, as previously stated, two power systems operated by Electro Peru. The one is Huallanca power system which has been supplied by the existing Huallanca power station (156 kW), and the other is by the existing portable diesel power plants (125 kVA) in La Union. In order to connect the both systems with the Huanzala power system to be realized by the Project, outgoing equipment of ElectroPeru is designed to be installed at the end of 6.6 kV-Bus as shown on Fig. 3-10-12 "One-line Diagram of Huanzala Hydro-electric Power Station".

It is understood that the connection works after the vacuum circuit breaker (VCB) of the said outgoing should be constructed by Electro Peru which is regarded as the supplier of the electricity for public use in Huallanca and La Union. Since the electrical equipment of the existing facilities at Huallanca and La Union are not such mechanism that the combination operation can be made with the Huanzala Hydro-electric Power Station of the Project, it will depend on the judgement of Electro Peru whether the said two will be completely abandoned or re-used after improving governor and various equipment for the connection purpose.

Table 7-1-1 Project Feature

1. General

River	;	Rio San Juan
Catchment area	;	153.7 km^2

2. Power Generation

Intake water level	;	3802.5 m
Head tank water level	;	3796.5 m
Center of turbine	;	3547.3 m
Normal head	;	249.2 m
Effective head	;	242.0 m
Maximum discharge	;	2.2 cu.m/sec
Output	;	4200.0 kW
Annual energy production	;	32187 10^3 kWh

3. Intake Dam

Type	;	Concrete gravity
Crest elevation	;	3802.5 m
Overflow length	;	15.0 m
Height	;	3.5 m

4. Sedimentation Basin

Width	;	3.5 m
Length	;	40.0 m
Height	;	1.7 3.5 m

5. Head Race

(1) Canal

Type	;	Trapezoidal stone pitching (Type I), concrete rectangular (Type II) or concrete culvert (Type III)
Length	;	3000 m
Width height	;	Type I : 1.2 1.8 (1:0.3) m Type II : 1.2 1.8 m Type III: 1.2 1.7 m
Slope	;	1:1000

(2) Tunnel

Type	;	Lined (Type II) or unlined (Type I) top-round, bottom-rectangular type
Length	;	1650 m
Width height	;	Type I : 2.0 2.3 m Type II : 1.6 2.1 m
Slope	;	1:1000

6. Head Tank

Type	;	Cylindrical type
Diameter	;	9.0 m
Height	;	5.5 m

7. Penstock

Type	;	All welded steel pipe, exposed type
Length	;	664.0 m
Diameter	;	1.10 m 0.55 m

8. Powerhouse

Type	;	Surface type
Length width height	;	23.0 m 10.0 m 8.3 m

10. Transmission Line

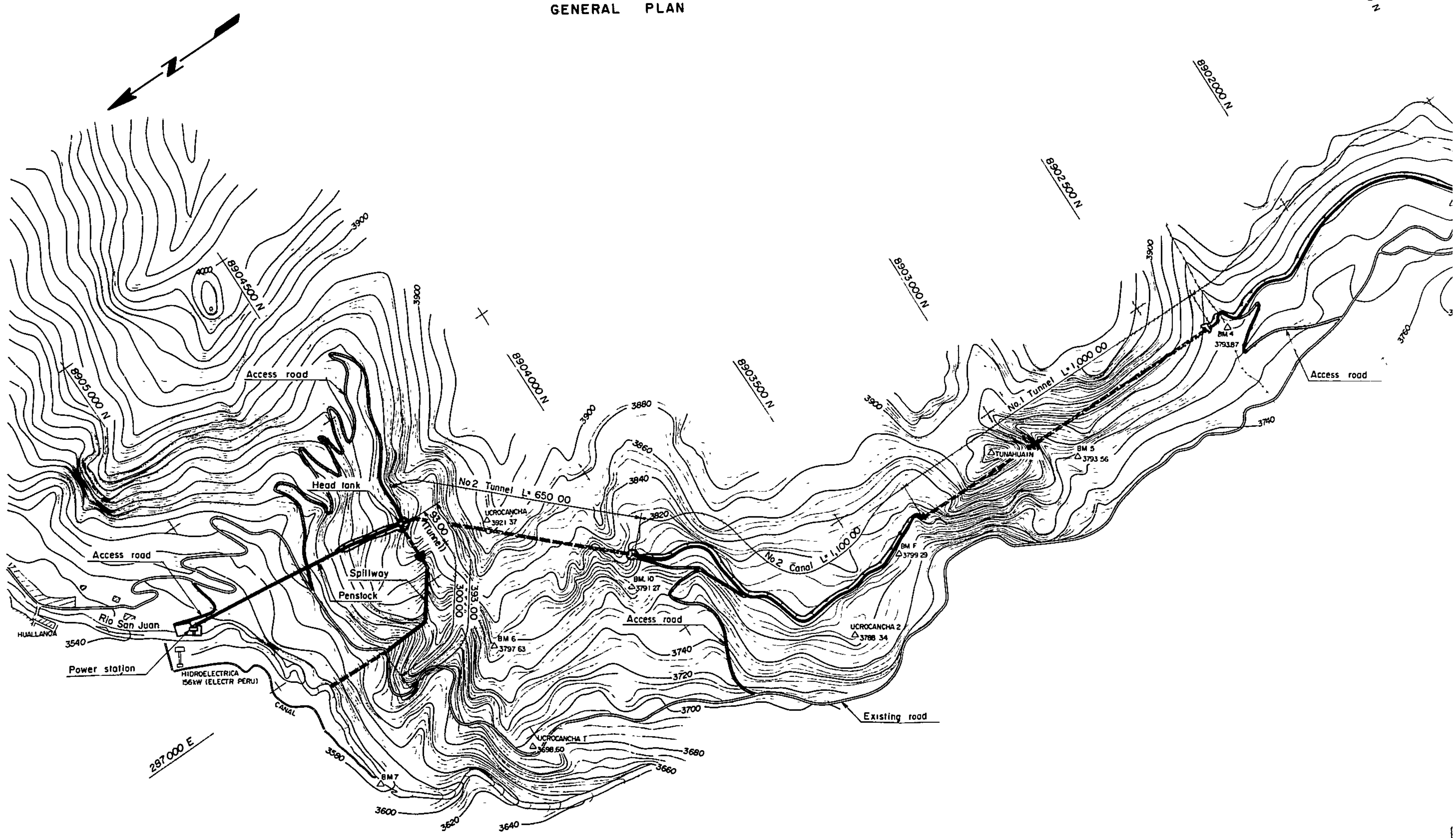
Number of Circuit	;	1
Conductors	;	A.C.S.R. 120 mm^2
Insulators	;	250 mm suspension type, 4 for 1 string
Ground Wire	;	38 mm^2 GSC, 1 line
Support	;	Concrete poles
Voltage	;	33 kV
Length	;	10 km

11. Huanzala Mining Side Substation

Transformer

Type	;	Outdoor, 3-phase, oil immersed transformer
Number of Unit	;	1
Capacity	;	5,200 kVA
Voltage	;	33 5%/2.2 kV

GENERAL PLAN



GENERAL PLAN

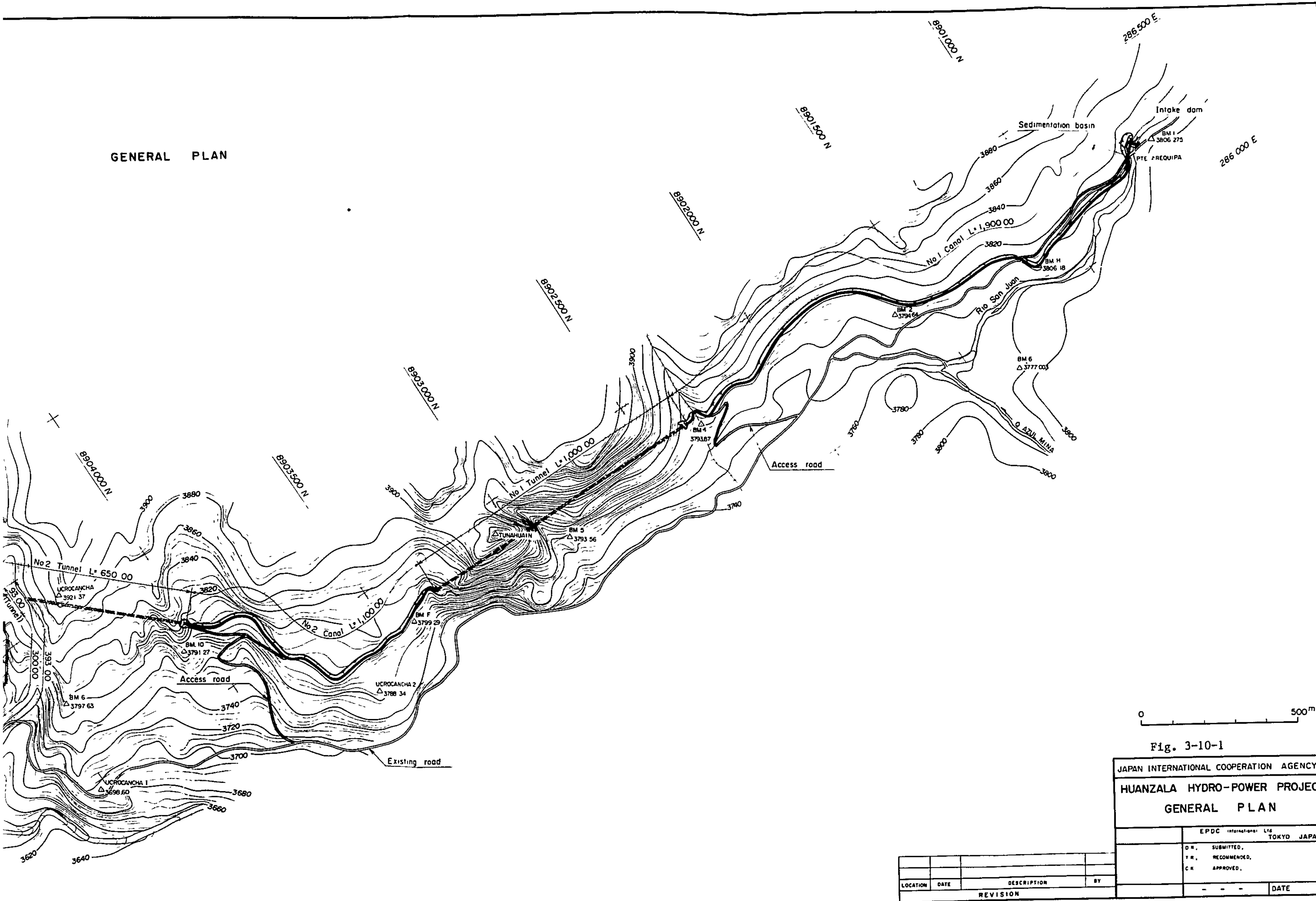


Fig. 3-10-1

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
GENERAL PLAN

EPDC International Ltd TOKYO JAPAN

DR.	SUBMITTED,
TR.	RECOMMENDED,
CK	APPROVED,
- - -	DATE

LOCATION	DATE	DESCRIPTION	BY
REVISION			

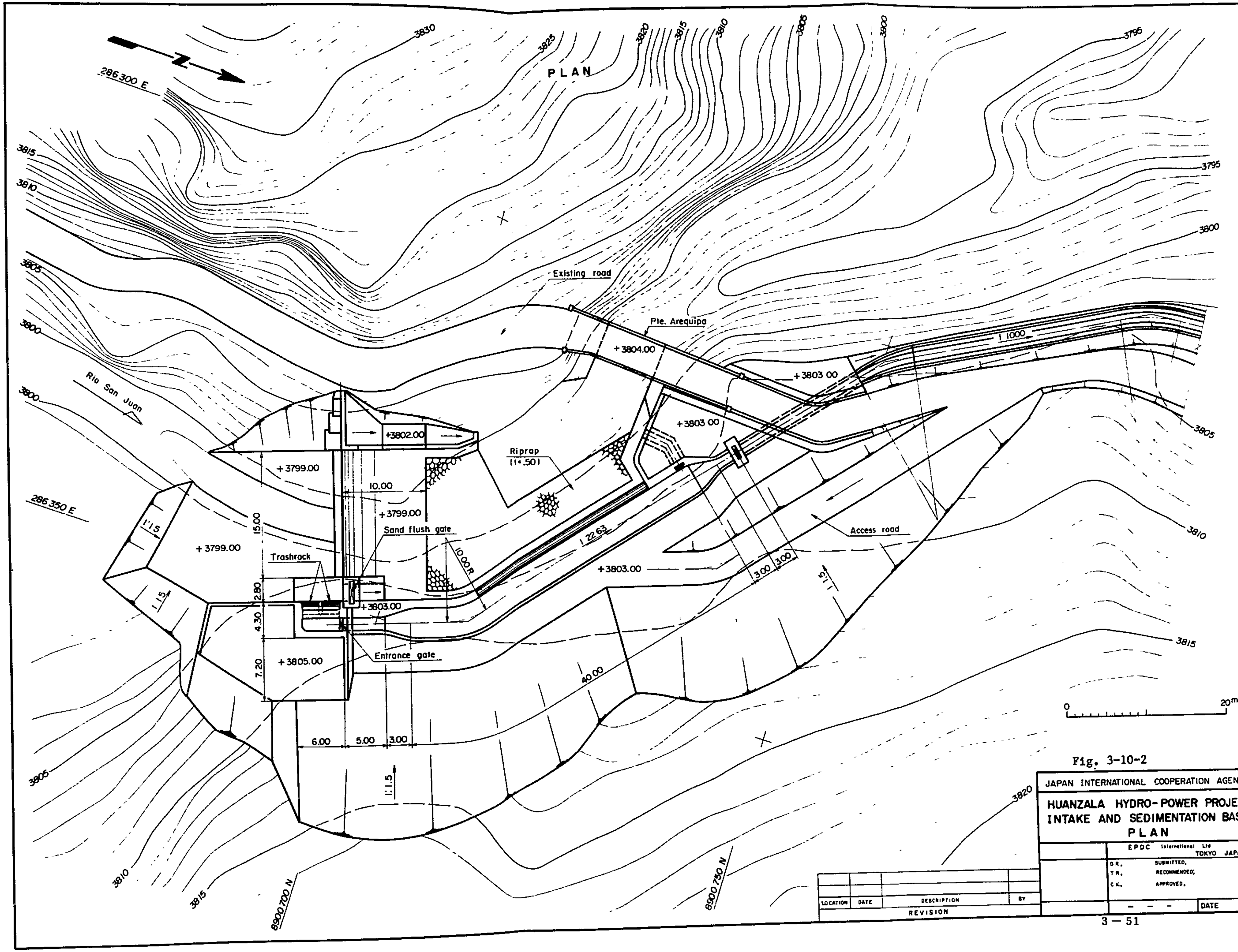


Fig. 3-10-2

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
INTAKE AND SEDIMENTATION BASIN
PLAN

EPDC International Ltd TOKYO JAPAN

LOCATION	DATE	DESCRIPTION	BY
		REVISION	

DR. SUBMITTED,
TR. RECOMMENDED;
CK. APPROVED,
DATE

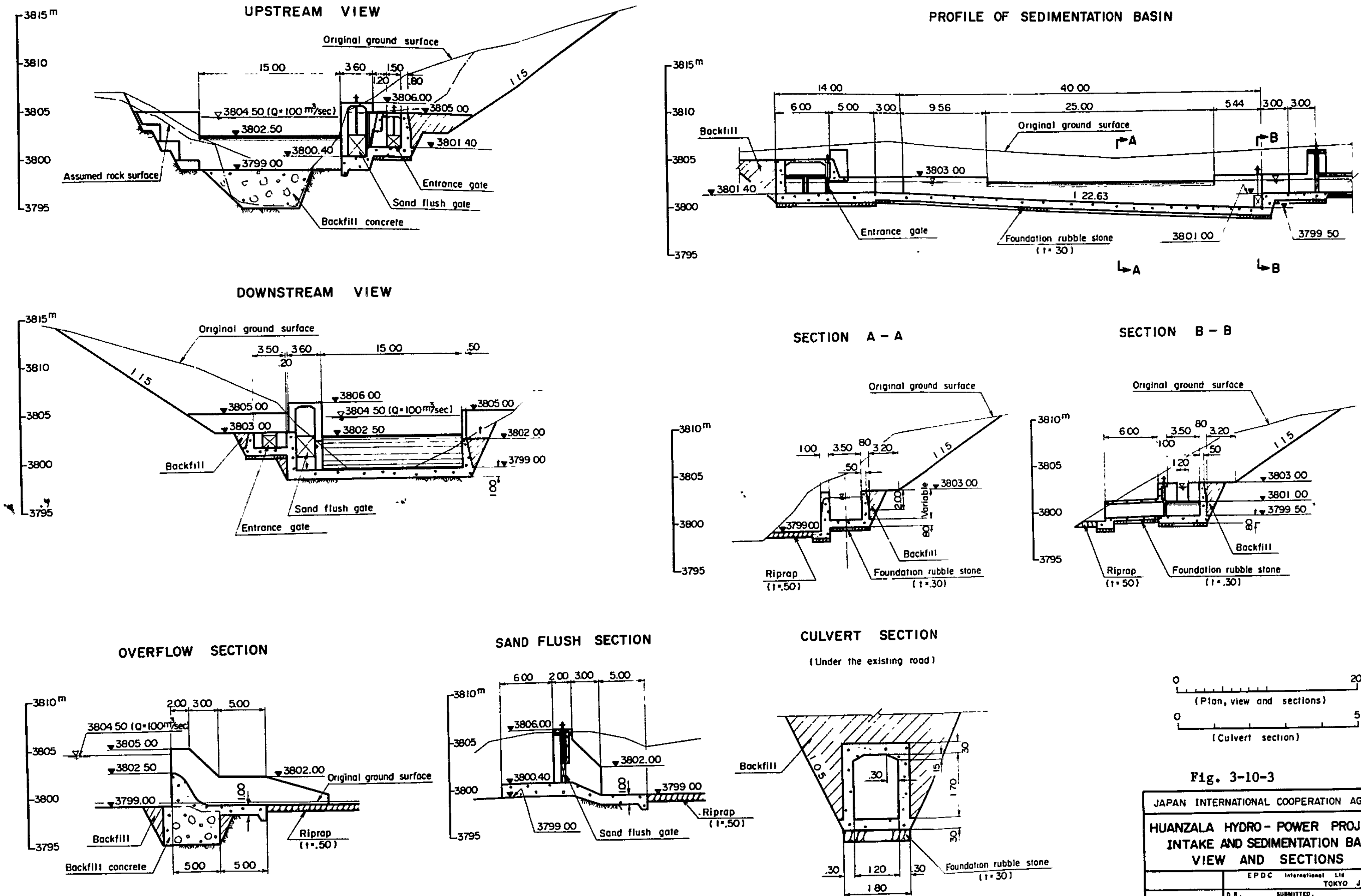


Fig. 3-10-3

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
INTAKE AND SEDIMENTATION BASIN
VIEW AND SECTIONS

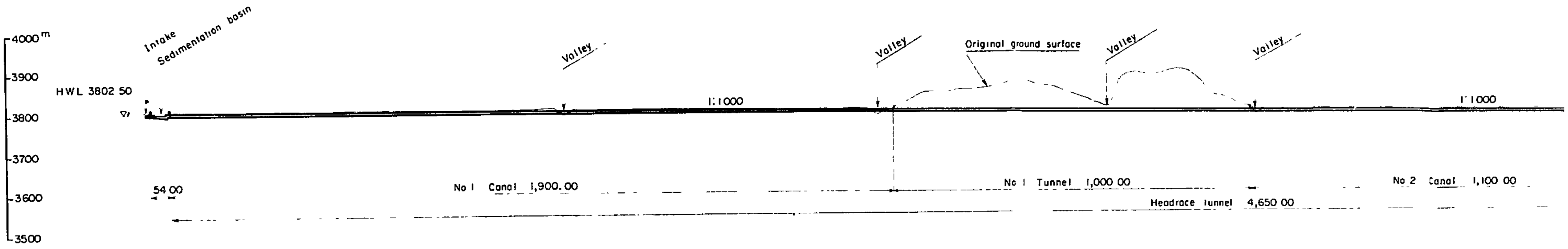
EPDC International Ltd TOKYO JAPAN

DR., SUBMITTED,
TR., RECOMMENDED,
CK., APPROVED,

DATE

LOCATION	DATE	DESCRIPTION	BY
		REVISION	

PROFILE

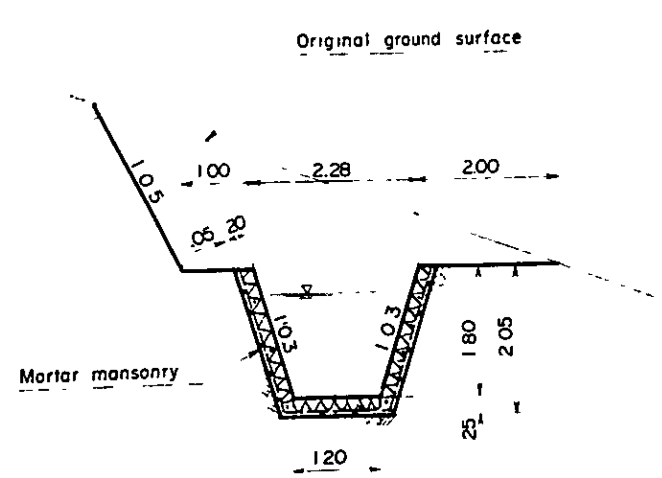


TYPICAL SECTIONS

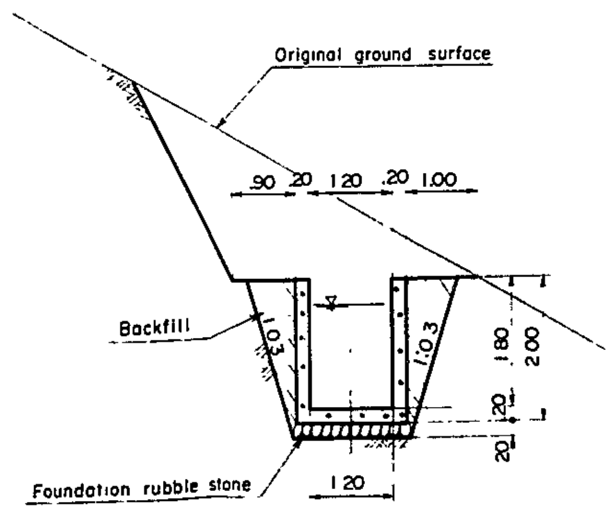
SECTION OF CANALS

SE

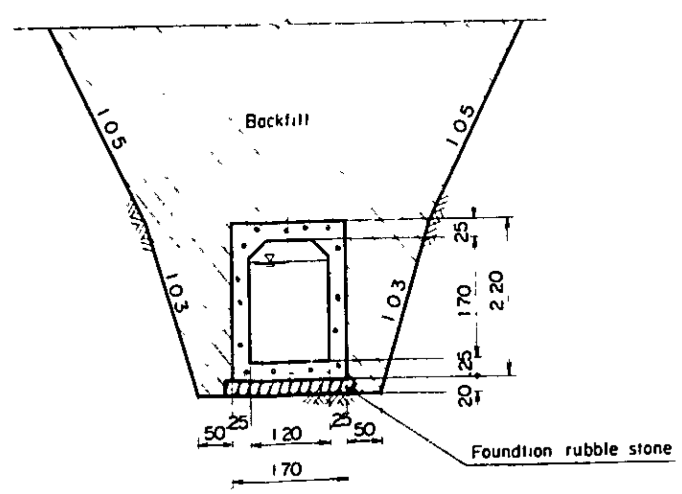
TYPE - I



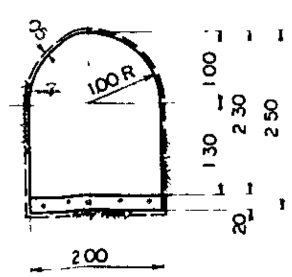
TYPE - II



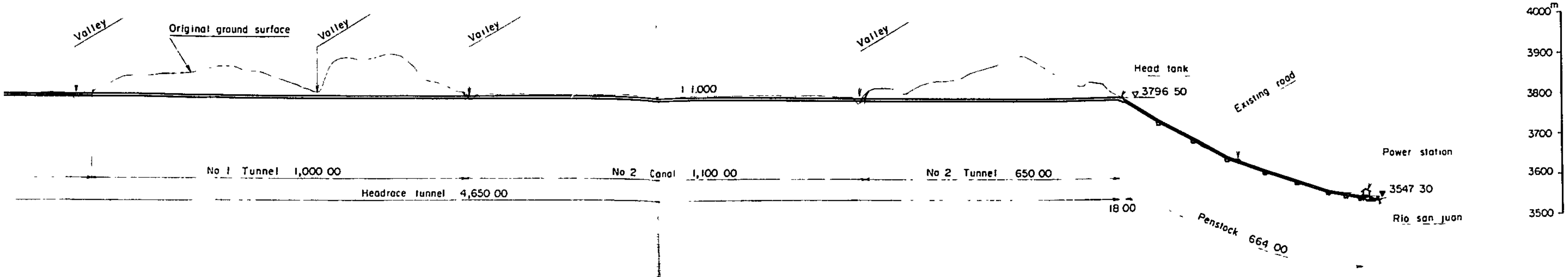
TYPE - III



TYPE - I



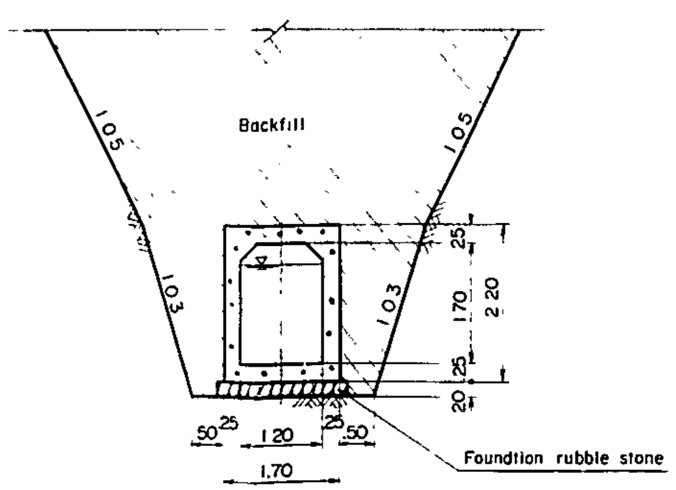
PROFILE



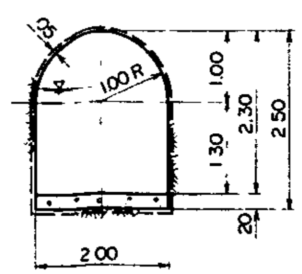
ICAL SECTIONS

SECTION OF TUNNEL

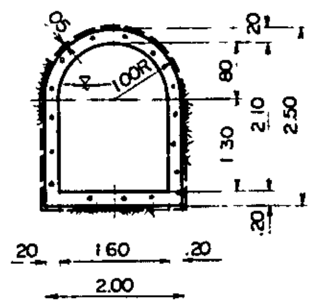
TYPE - III



TYPE - I



TYPE - II



0 500m
(Profile)
0 5 m
(Typical sections)

Fig. 3-10-4

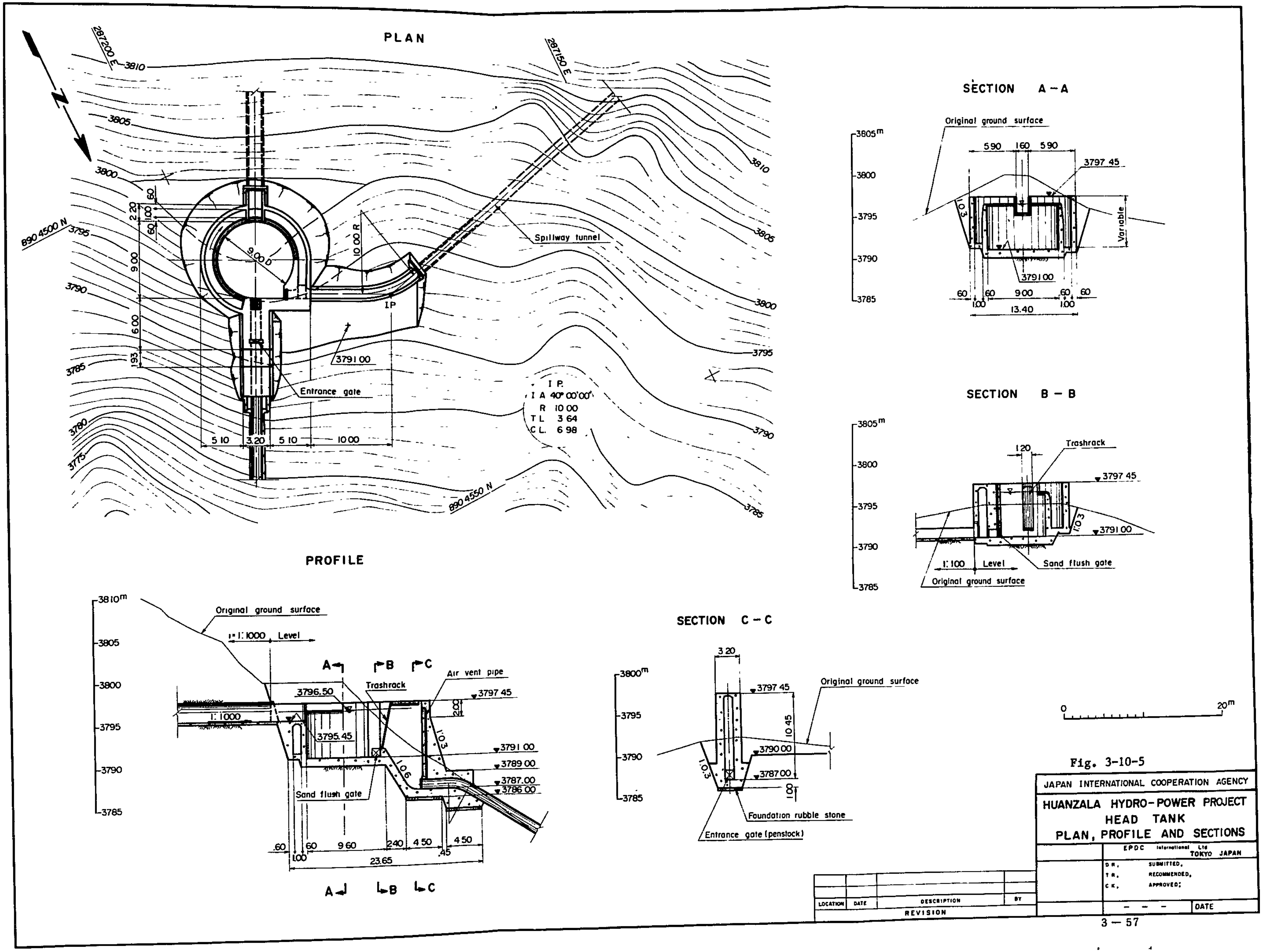
JAPAN INTERNATIONAL COOPERATION AGENCY

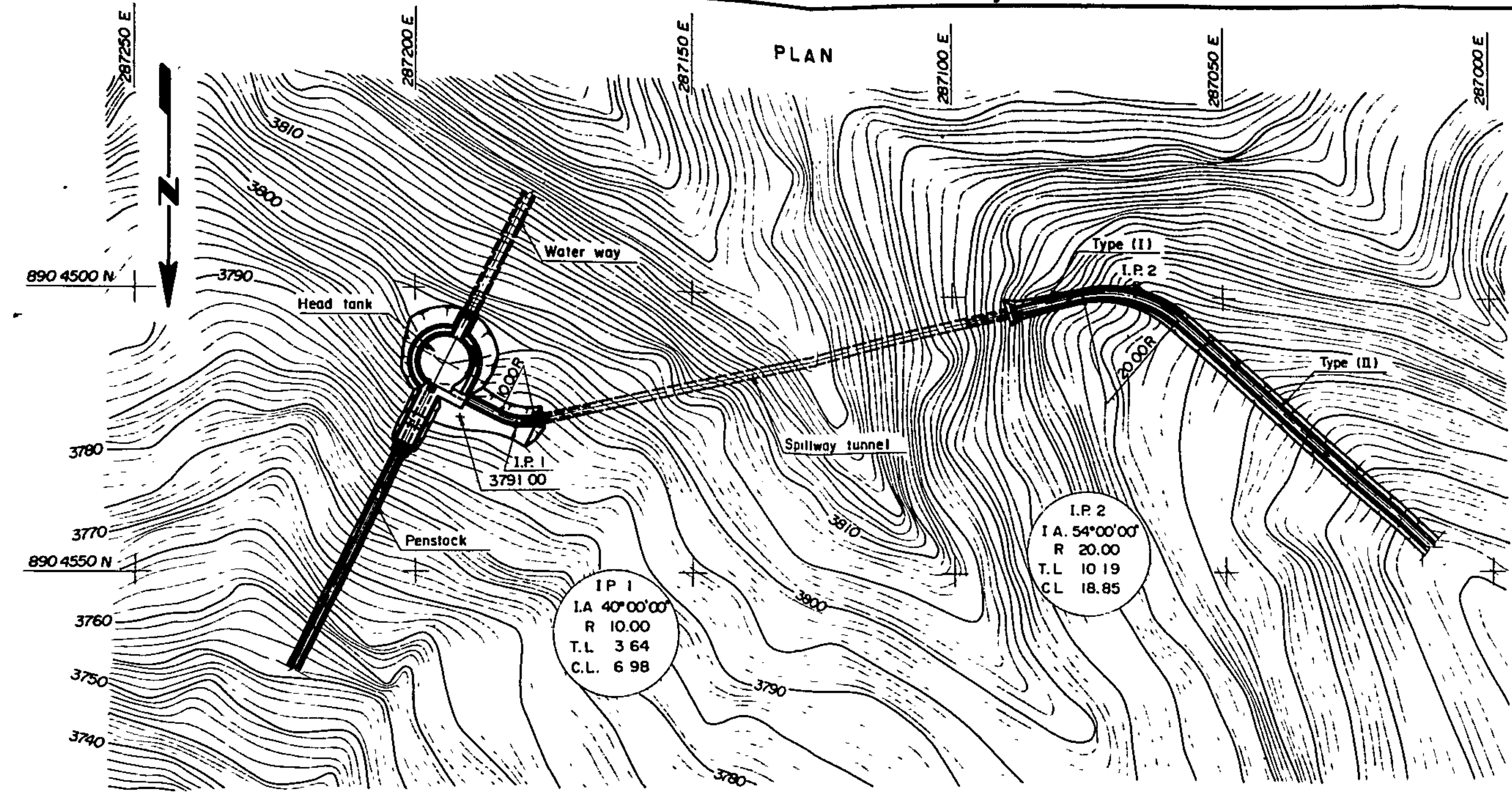
HUANZALA HYDRO-POWER PROJECT
WATERWAY
PROFILE, TYPICAL SECTIONS

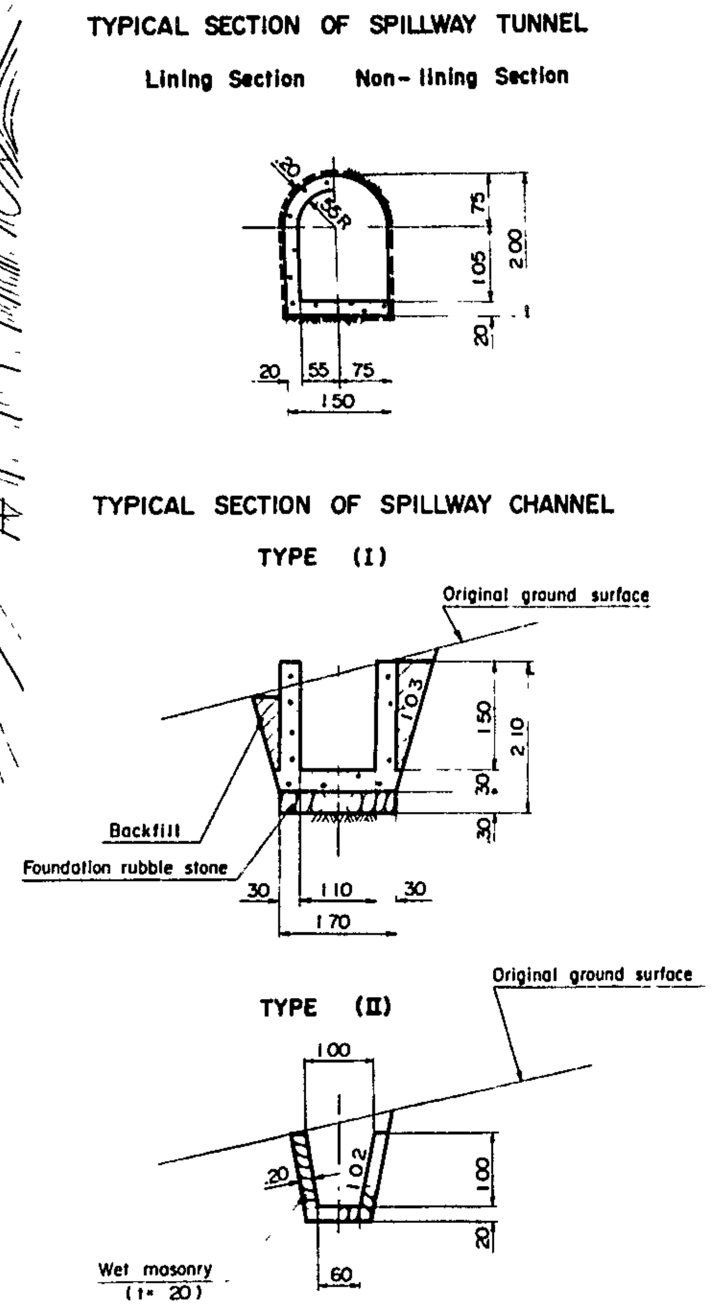
EPDC International Ltd TOKYO JAPAN

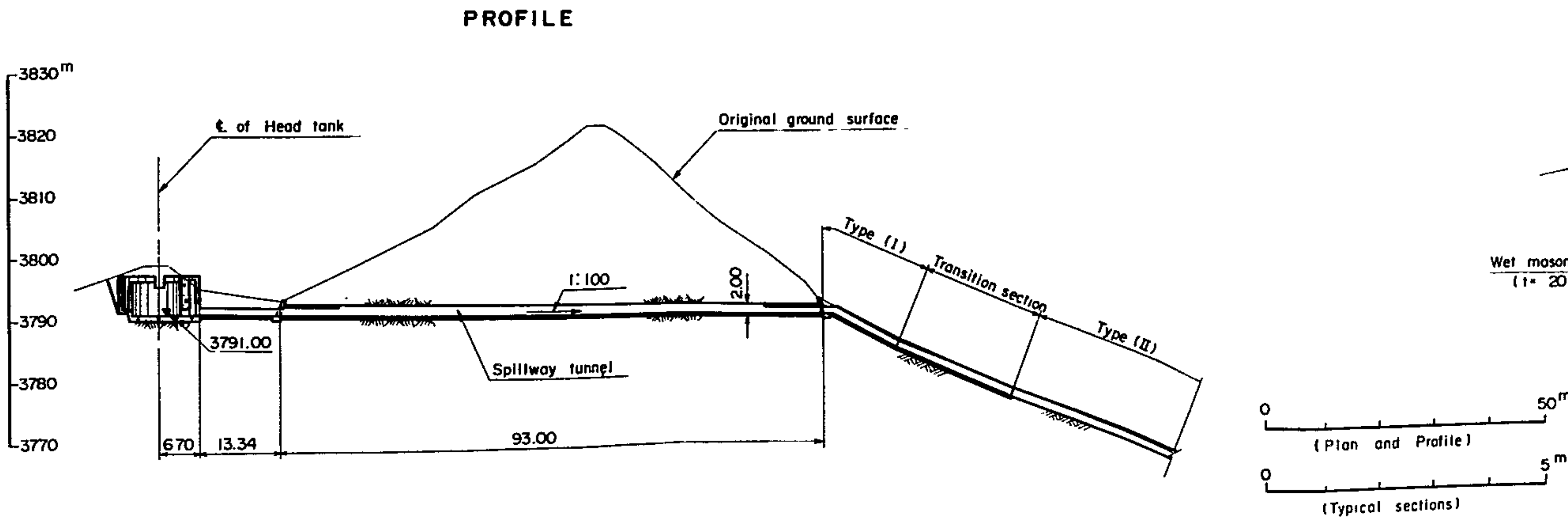
DR.	SUBMITTED,		
TR.	RECOMMENDED,		
CR.	APPROVED,		
	– – –	DATE	

LOCATION	DATE	DESCRIPTION	BY
REVISION			


PLAN
Spillway tunnel
Entrance gate
IP
3791.00
I.P.
I.A 40°00'00"
R 10.00
T.L. 3.64
C.L. 6.98
SECTION A – A
Original ground surface
Variable
SECTION B – B
Trashrack
Sand flush gate
Original ground surface
PROFILE
Original ground surface
Trashrack
Air vent pipe
Sand flush gate
SECTION C – C
Original ground surface
Foundation rubble stone
Entrance gate (penstock)
Fig. 3-10-5
JAPAN INTERNATIONAL COOPERATION AGENCY
HUANZALA HYDRO-POWER PROJECT
HEAD TANK
PLAN, PROFILE AND SECTIONS
EPDC International Ltd TOKYO JAPAN
DR., SUBMITTED,
TR., RECOMMENDED,
CK., APPROVED;
DATE
LOCATION
DESCRIPTION
BY
REVISION







Note: 1 As for spillway tunnel, lining or non-lining would depend on the geological condition.
2 Spillway channel (Type II) would be provided from the end of transition section to the intersection of the existing road.

Fig. 3-10-6

LOCATION	DATE	DESCRIPTION	BY
REVISION			

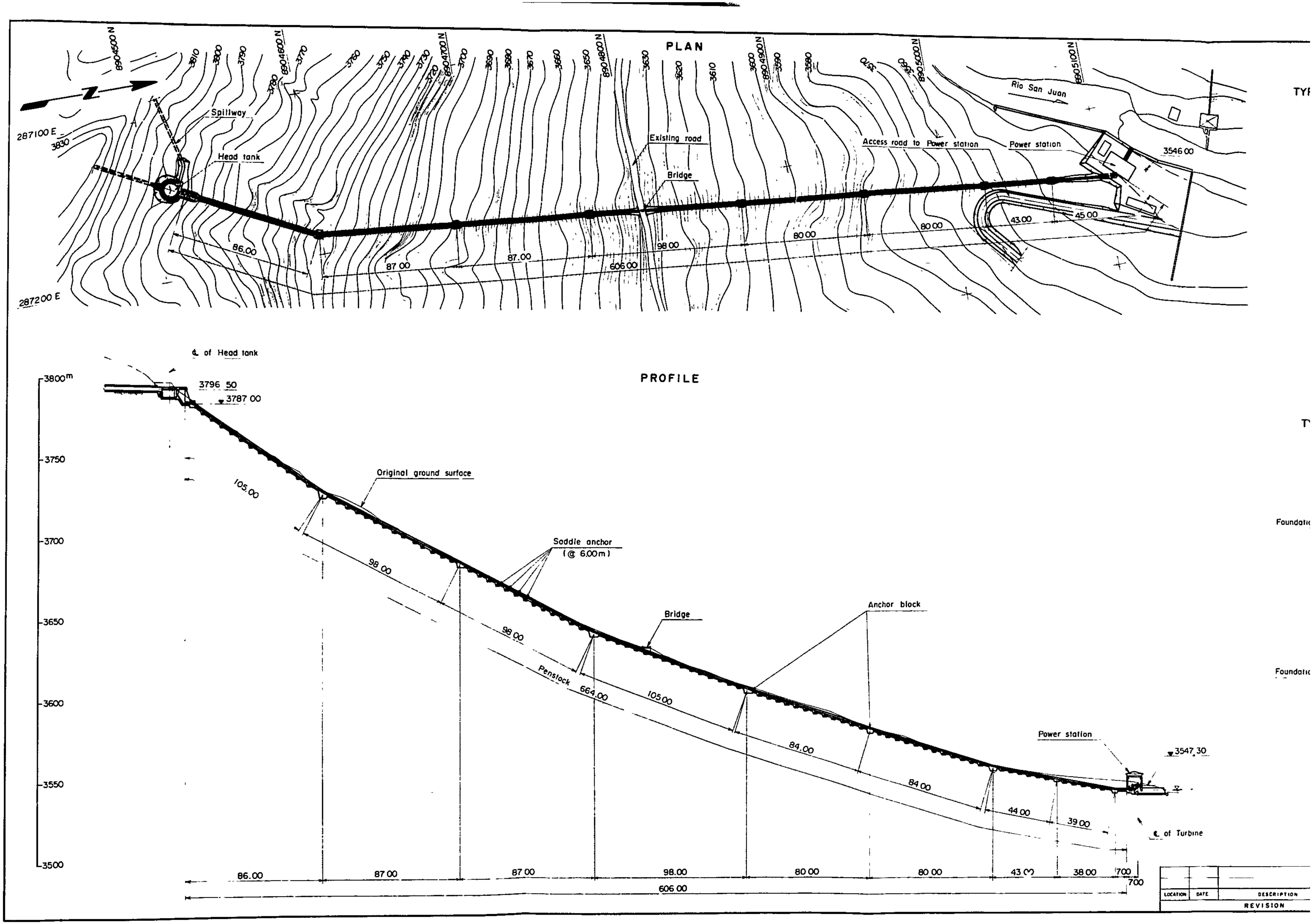
JAPAN INTERNATIONAL COOPERATION AGENCY

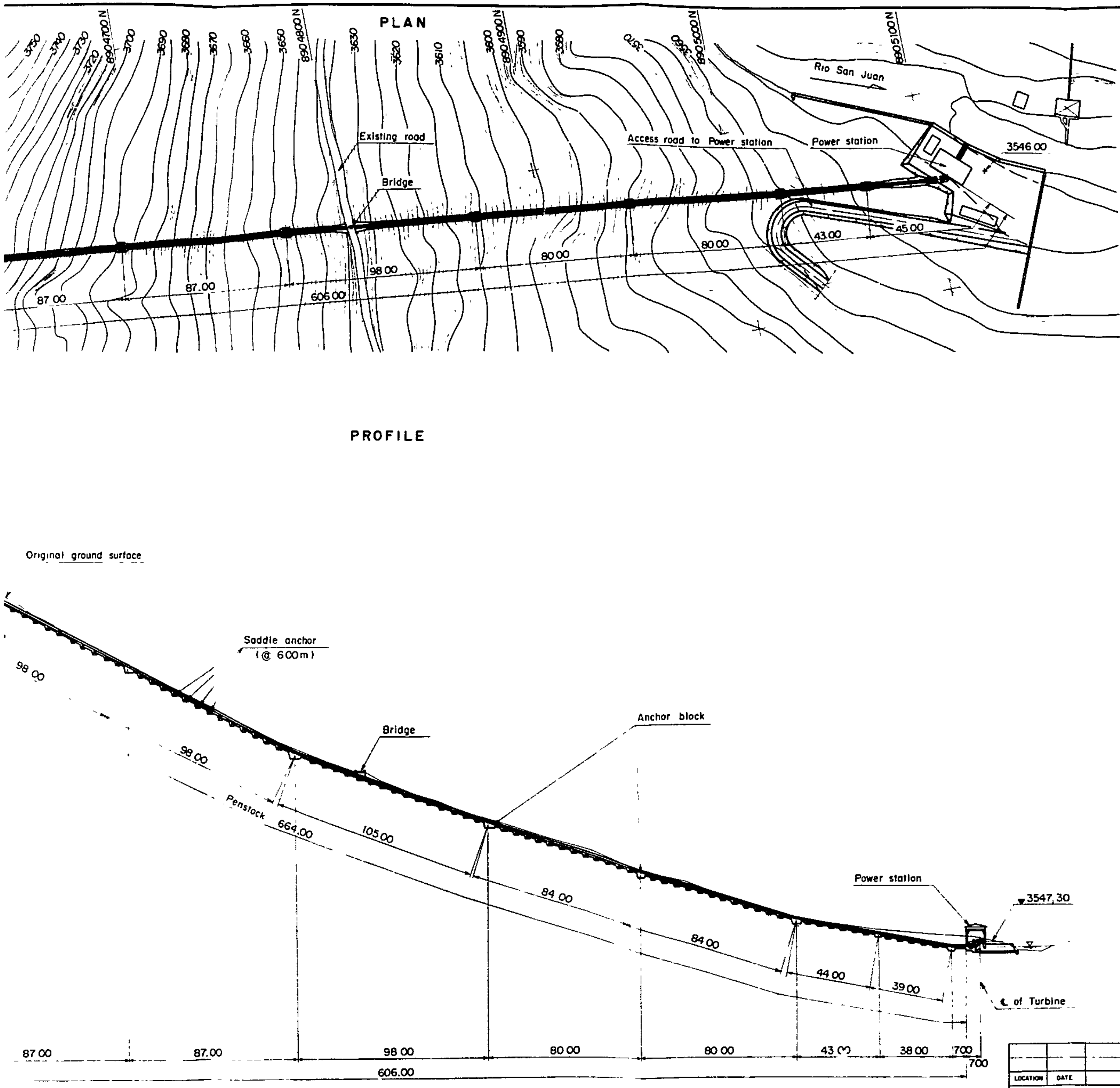
HUANZALA HYDRO-POWER PROJECT
SPILLWAY
PLAN, PROFILE AND SECTIONS

EPDC International Ltd TOKYO JAPAN

DR. SUBMITTED,
TR. RECOMMENDED,
CK. APPROVED,

DATE


PLAN
Spillway
Head tank
Existing road
Bridge
Access road to Power station
Power station
Rio San Juan
287100 E
287200 E
8904500 N
8904600 N
8904700 N
8904800 N
8904900 N
8905000 N
8905100 N
3546.00
86.00
87.00
87.00
98.00
80.00
80.00
43.00
45.00
606.00
PROFILE
℄ of Head tank
3796.50
3787.00
Original ground surface
Saddle anchor
(@ 6.00m)
Bridge
Anchor block
Penstock 664.00
Power station
3547.30
℄ of Turbine
105.00
98.00
98.00
105.00
84.00
84.00
44.00
39.00
3800m
3750
3700
3650
3600
3550
3500
86.00
87.00
87.00
98.00
80.00
80.00
43.00
38.00
7.00
606.00
7.00
LOCATION
DATE
DESCRIPTION
REVISION
Foundation
Foundation



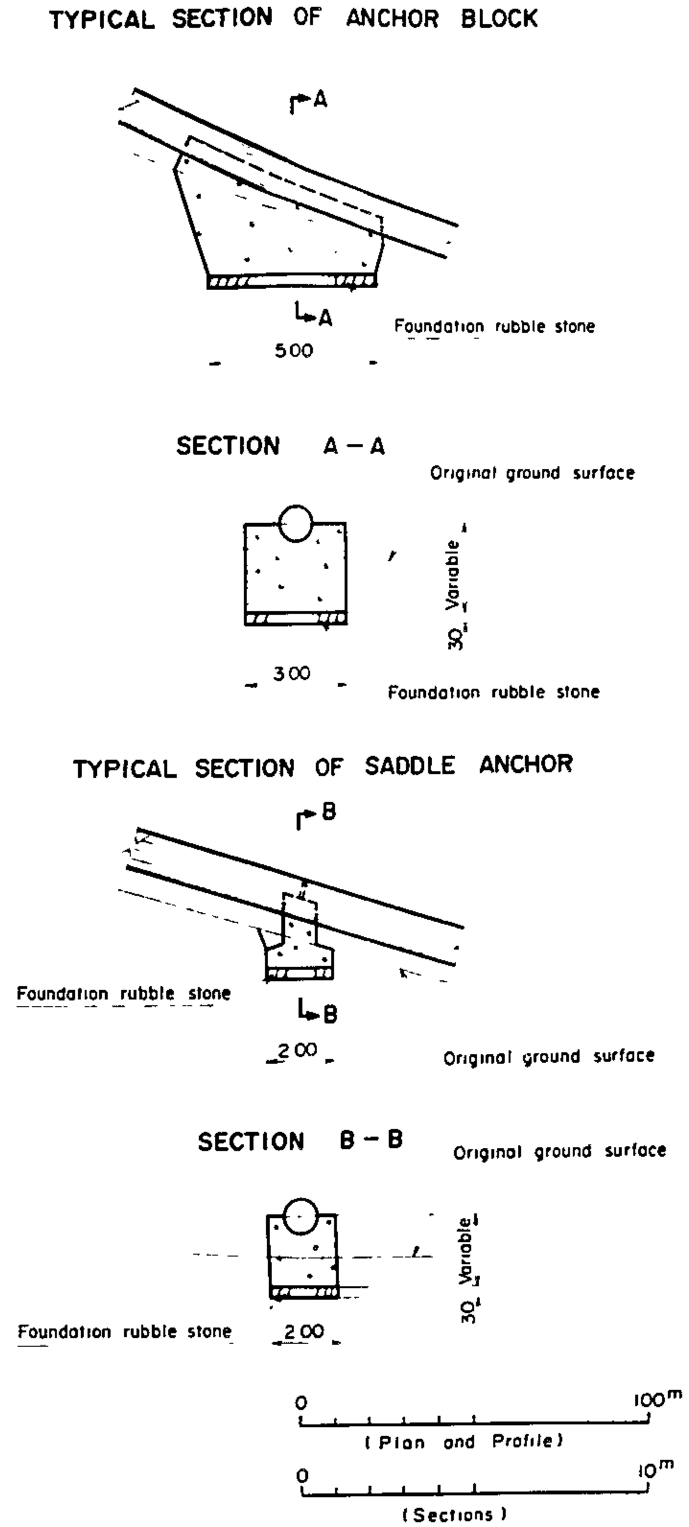


Fig. 3-10-7

JAPAN INTERNATIONAL COOPERATION AGENCY

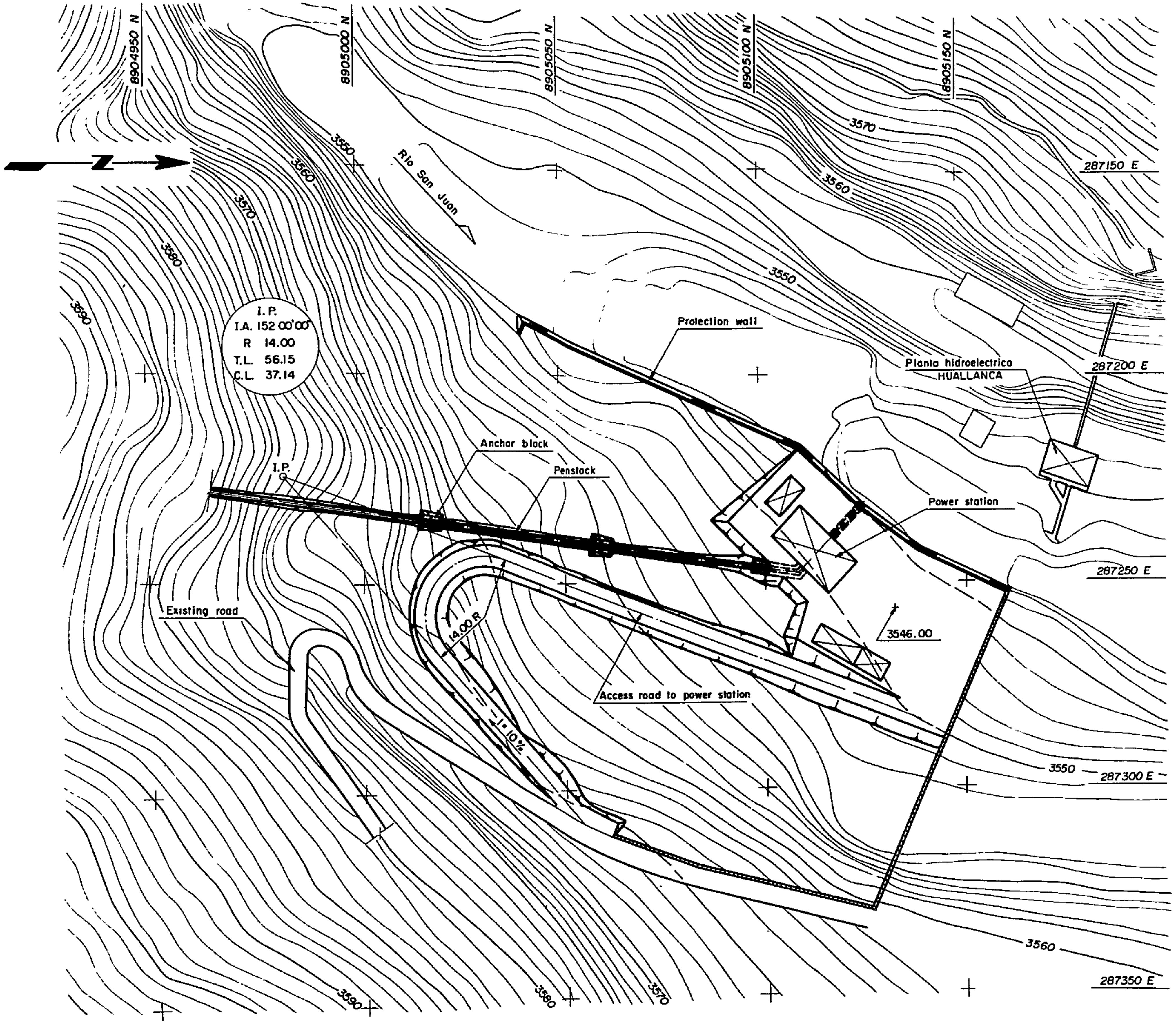
HUANZALA HYDRO-POWER PROJECT
PENSTOCK
PLAN AND PROFILE

EPDC International Ltd TOKYO JAPAN

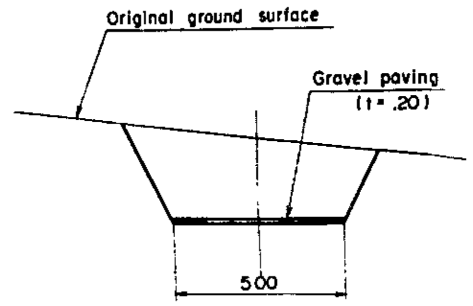
LOCATION	DATE	DESCRIPTION	BY
REVISION			

DR. SUBMITTED.
TR. RECOMMENDED
CK. APPROVED.
DATE

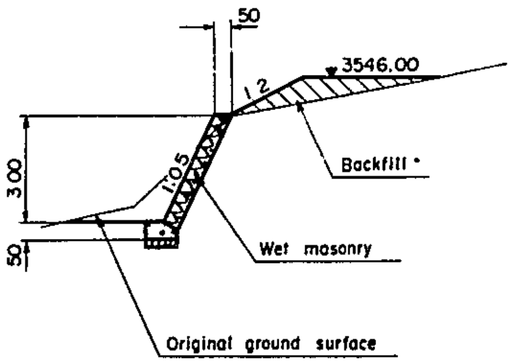
GENERAL PLAN



TYPICAL SECTION OF ACCESS ROAD



TYPICAL SECTION OF PROTECTION WALL



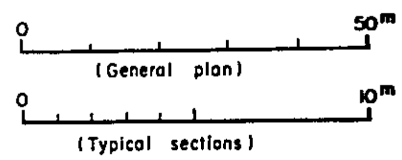
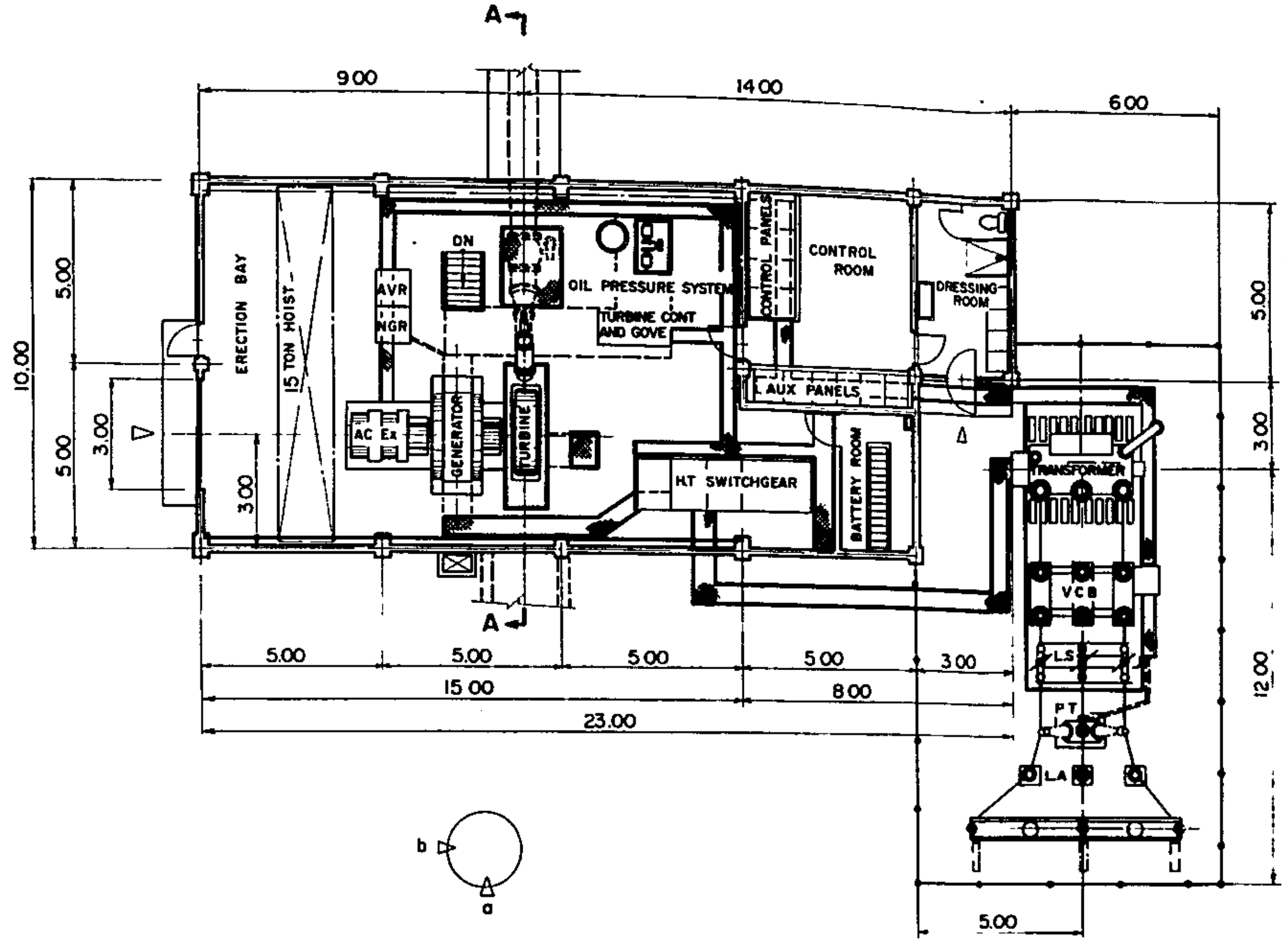


Fig. 3-10-8

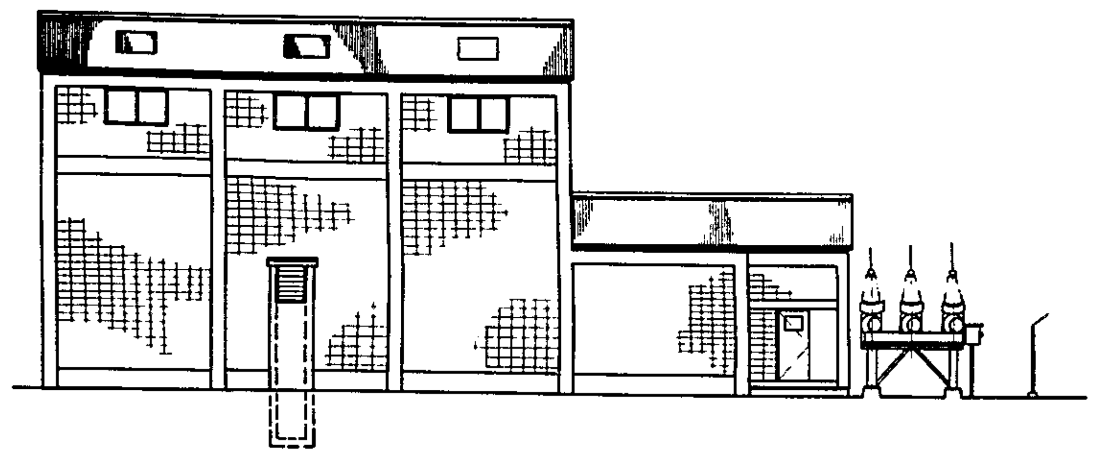
JAPAN INTERNATIONAL COOPERATION AGENCY			
HUANZALA HYDRO-POWER PROJECT POWER STATION GENERAL PLAN			
	EPDC International Ltd TOKYO JAPAN		
	DR.	SUBMITTED;	
	TR.	RECOMMENDED;	
	CK.	APPROVED.	
	— — —	DATE	

LOCATION	DATE	DESCRIPTION	BY
REVISION			

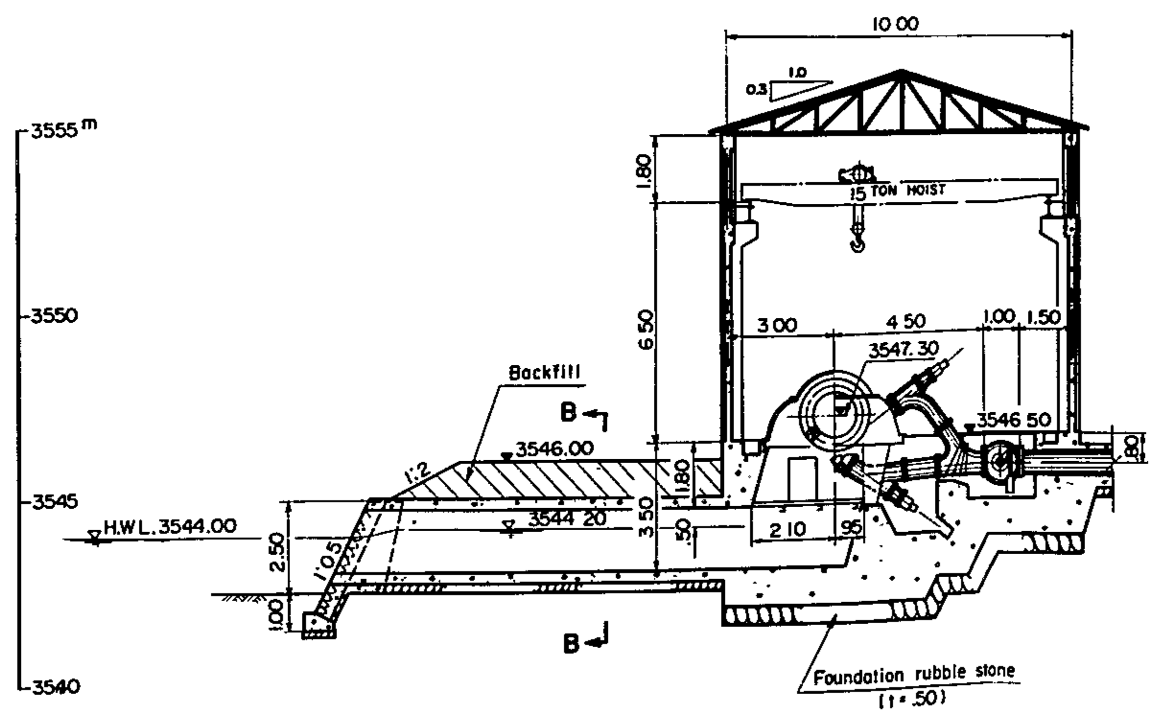
PLAN



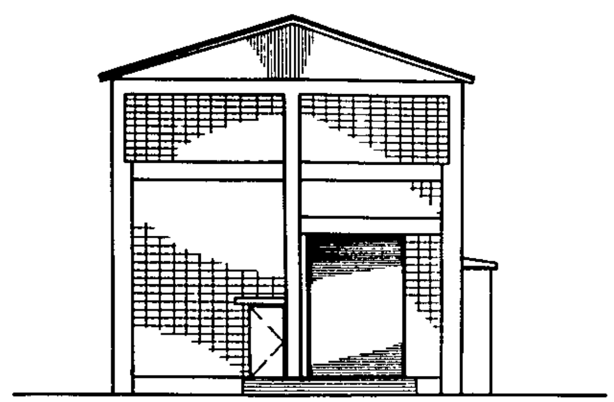
ELEVATION a



SECTION A – A



ELEVATION b



SECTION B – B

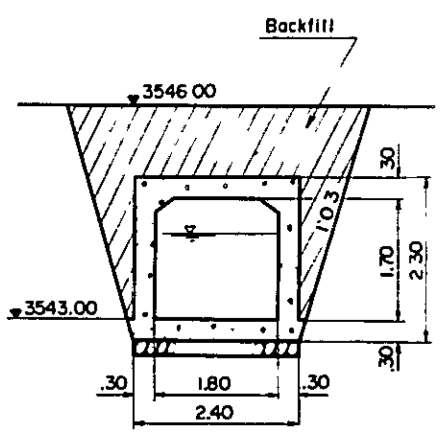


Fig. 3-10-9

0 10 m
(Plan, Section A-A, Elevation a and b)

0 5 m
(Section B - B)

LOCATION	DATE	DESCRIPTION	BY
		REVISION	

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO-POWER PROJECT
POWER STATION
PLAN, ELEVATIONS AND SECTIONS

EPDC International Ltd TOKYO JAPAN

DR., SUBMITTED,
TR., RECOMMENDED,
CK., APPROVED,

– – – DATE

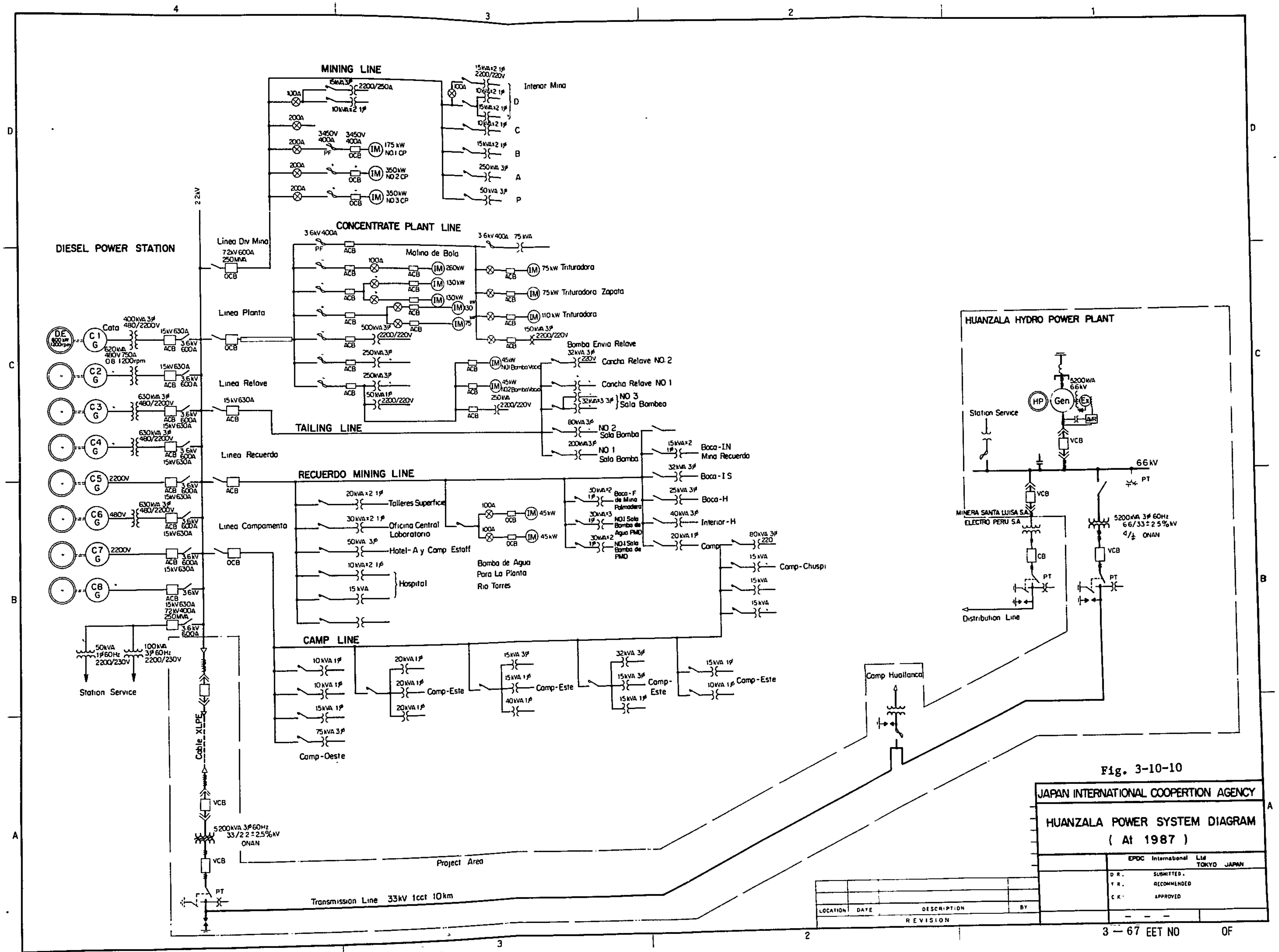


Fig. 3-10-10

JAPAN INTERNATIONAL COOPERTION AGENCY

HUANZALA POWER SYSTEM DIAGRAM
(At 1987)

EPDC International Ltd TOKYO JAPAN

LOCATION	DATE	DESCRIPTION	BY
REVISION			

DR. SUBMITTED.
TR. RECOMMENDED
CK. APPROVED

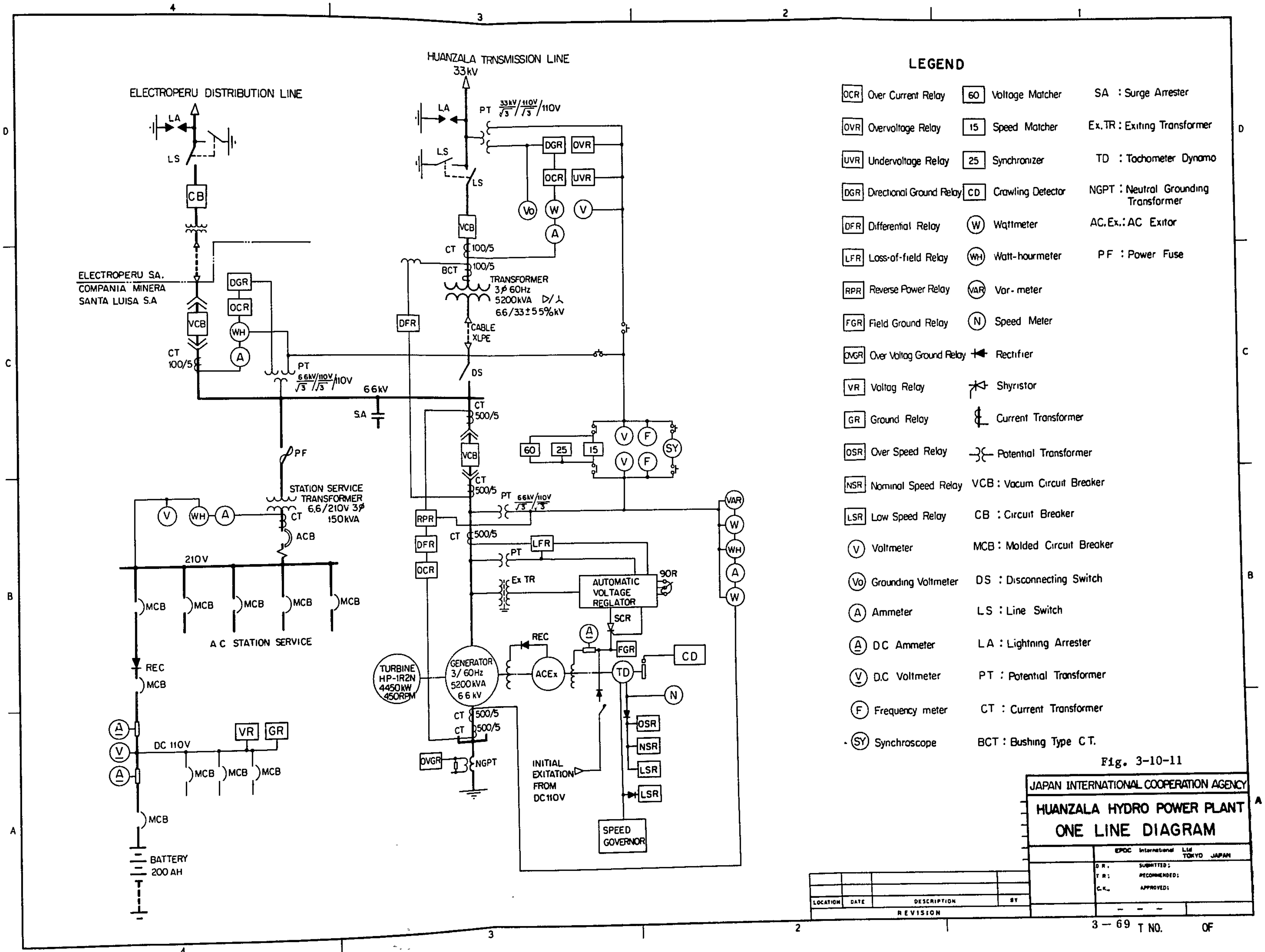

HUANZALA TRNSMISSION LINE
33kV
ELECTROPERU DISTRIBUTION LINE
LA
LS
CB
VCB
DGR
OCR
WH
CT
100/5
ELECTROPERU SA.
COMPANIA MINERA
SANTA LUISA S.A
PT
6.6kV/√3 / 110V/√3 / 110V
6.6kV
SA
PF
STATION SERVICE
TRANSFORMER
6.6/210V 3∅
150kVA
ACB
210V
MCB
A C STATION SERVICE
REC
DC 110V
VR
GR
BATTERY
200 AH
PT 33kV/√3 / 110V/√3 / 110V
DGR
OVR
OCR
UVR
Vo
W
V
A
CT 100/5
BCT 100/5
TRANSFORMER
3∅ 60Hz
5200kVA
6.6/33±5.5%kV
CABLE
XLPE
DS
CT 500/5
DFR
60
25
15
SY
PT 6.6kV/√3 / 110V/√3
RPR
LFR
Ex TR
AUTOMATIC
VOLTAGE
REGLATOR
90R
SCR
FGR
CD
ACEx
TD
N
TURBINE
HP-1R2N
4450kW
450RPM
GENERATOR
3/ 60Hz
5200kVA
6.6kV
CT 500/5
OVGR
NGPT
INITIAL
EXITATION
FROM
DC110V
OSR
NSR
LSR
SPEED
GOVERNOR
VAR
LEGEND
OCR Over Current Relay
OVR Overvoltage Relay
UVR Undervoltage Relay
DGR Directional Ground Relay
DFR Differential Relay
LFR Loss-of-field Relay
RPR Reverse Power Relay
FGR Field Ground Relay
OVGR Over Voltag Ground Relay
VR Voltag Relay
GR Ground Relay
OSR Over Speed Relay
NSR Nominal Speed Relay
LSR Low Speed Relay
V Voltmeter
Vo Grounding Voltmeter
A Ammeter
A DC Ammeter
V D.C Voltmeter
F Frequency meter
SY Synchroscope
60 Voltage Matcher
15 Speed Matcher
25 Synchronizer
CD Crawling Detector
W Wattmeter
WH Watt-hourmeter
VAR Var-meter
N Speed Meter
Rectifier
Shyristor
Current Transformer
Potential Transformer
VCB : Vacum Circuit Breaker
CB : Circuit Breaker
MCB : Molded Circuit Breaker
DS : Disconnecting Switch
LS : Line Switch
LA : Lightning Arrester
PT : Potential Transformer
CT : Current Transformer
BCT : Bushing Type C.T.
SA : Surge Arrester
Ex.TR : Exiting Transformer
TD : Tachometer Dynamo
NGPT : Neutral Grounding Transformer
AC.Ex. : AC Exitor
PF : Power Fuse
JAPAN INTERNATIONAL COOPERATION AGENCY
HUANZALA HYDRO POWER PLANT
ONE LINE DIAGRAM
EPDC International Ltd TOKYO JAPAN
DR.
TR.
CK.
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
T NO.
OF

Fig. 3-10-11

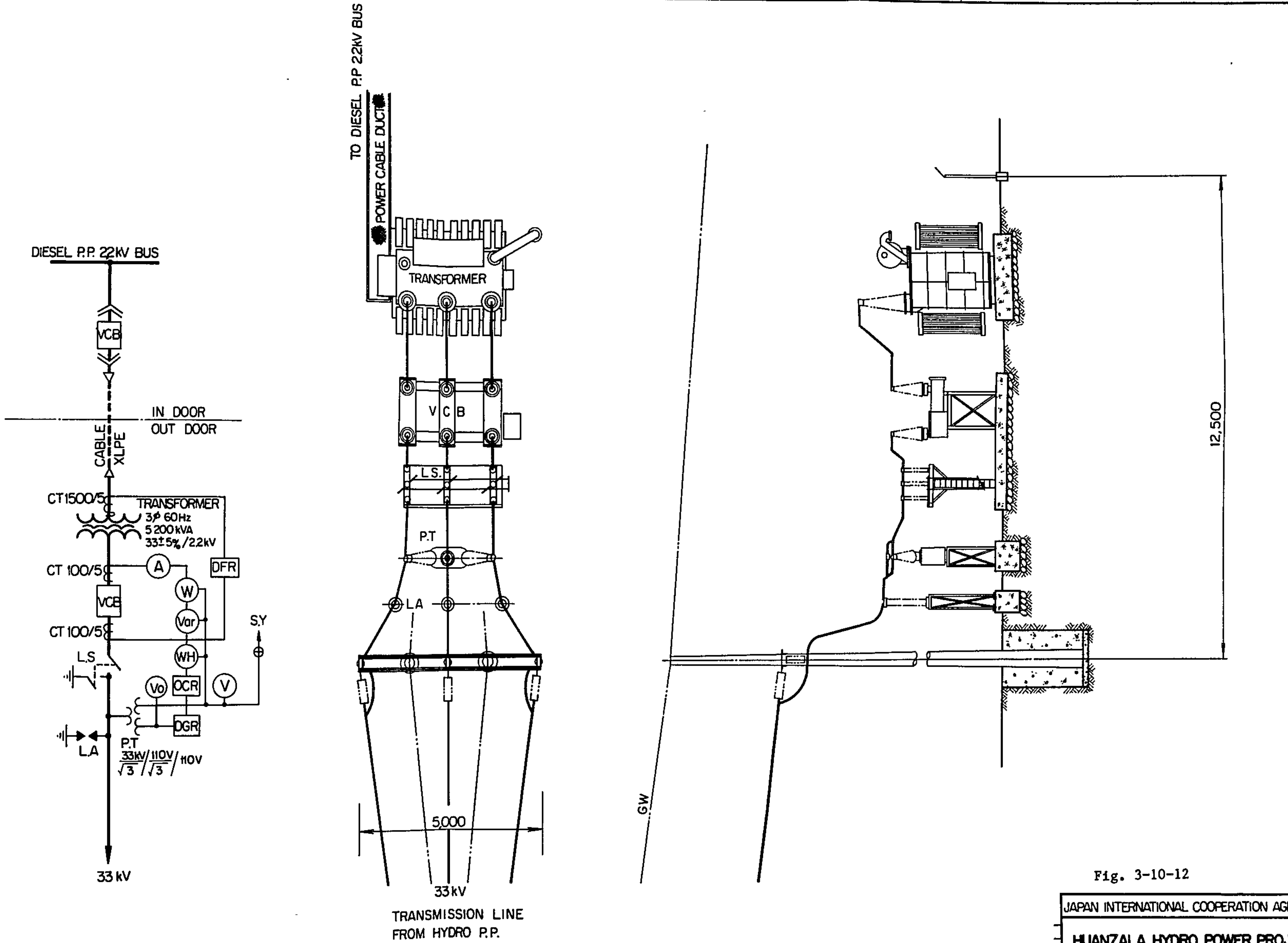


Fig. 3-10-12

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDRO POWER PROJECT
HUANZALA MINING SIDE SUBSTATION

EPDC International Ltd. TOKYO JAPAN

LOCATION	DATE	DESCRIPTION	BY
REVISION			

D.R.: SUBMITTED:
T.R.: RECOMMENDED:
C.K.: APPROVED:

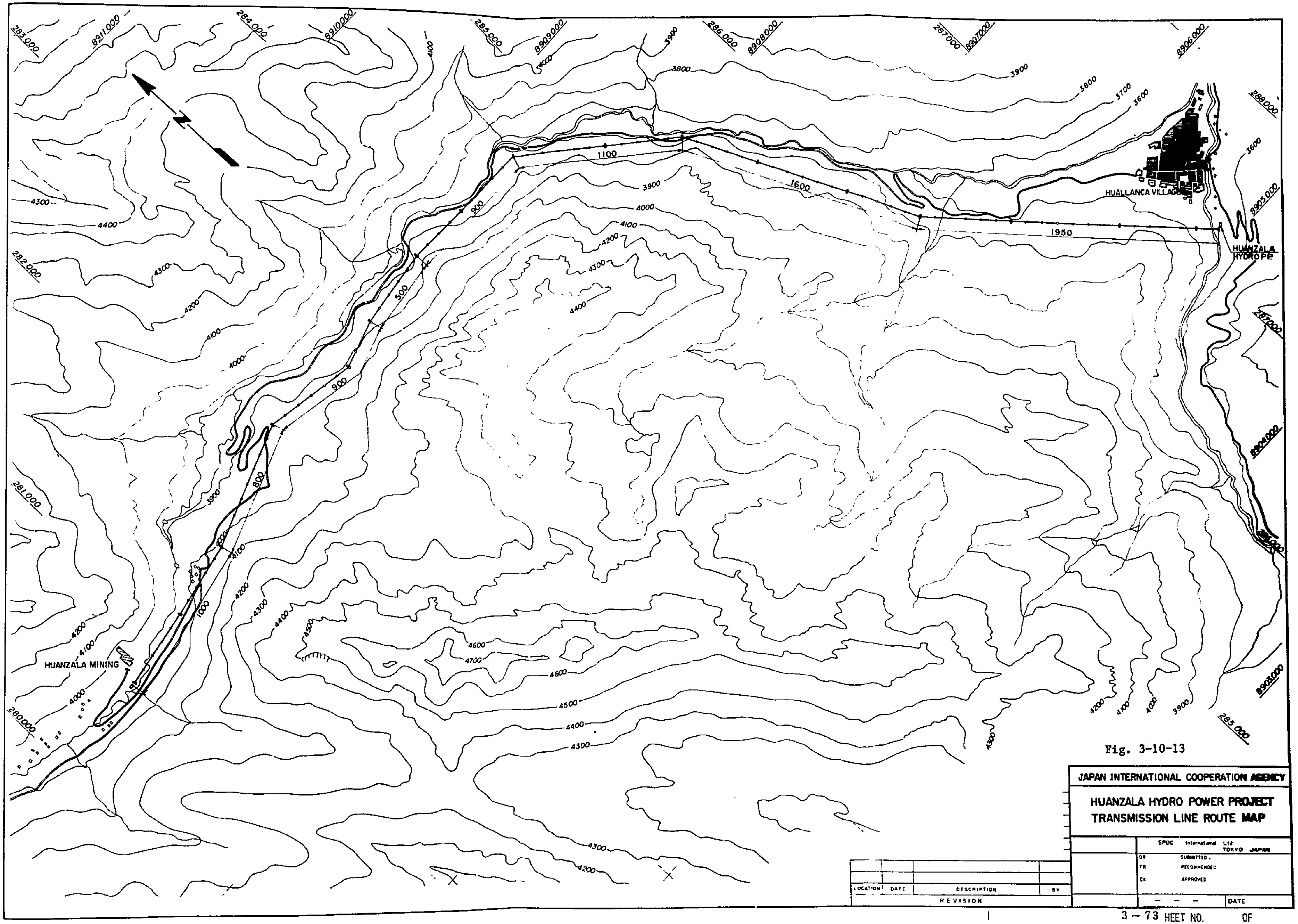

N
HUALLANCA VILLAGE
HUANZALA HYDRO P.P.
HUANZALA MINING
1100
1600
1950
900
500
800
1000
JAPAN INTERNATIONAL COOPERATION AGENCY
HUANZALA HYDRO POWER PROJECT
TRANSMISSION LINE ROUTE MAP
EPDC International Ltd TOKYO JAPAN
SUBMITTED
RECOMMENDED
APPROVED
DATE
LOCATION
DATE
DESCRIPTION
BY
REVISION

Fig. 3-10-13

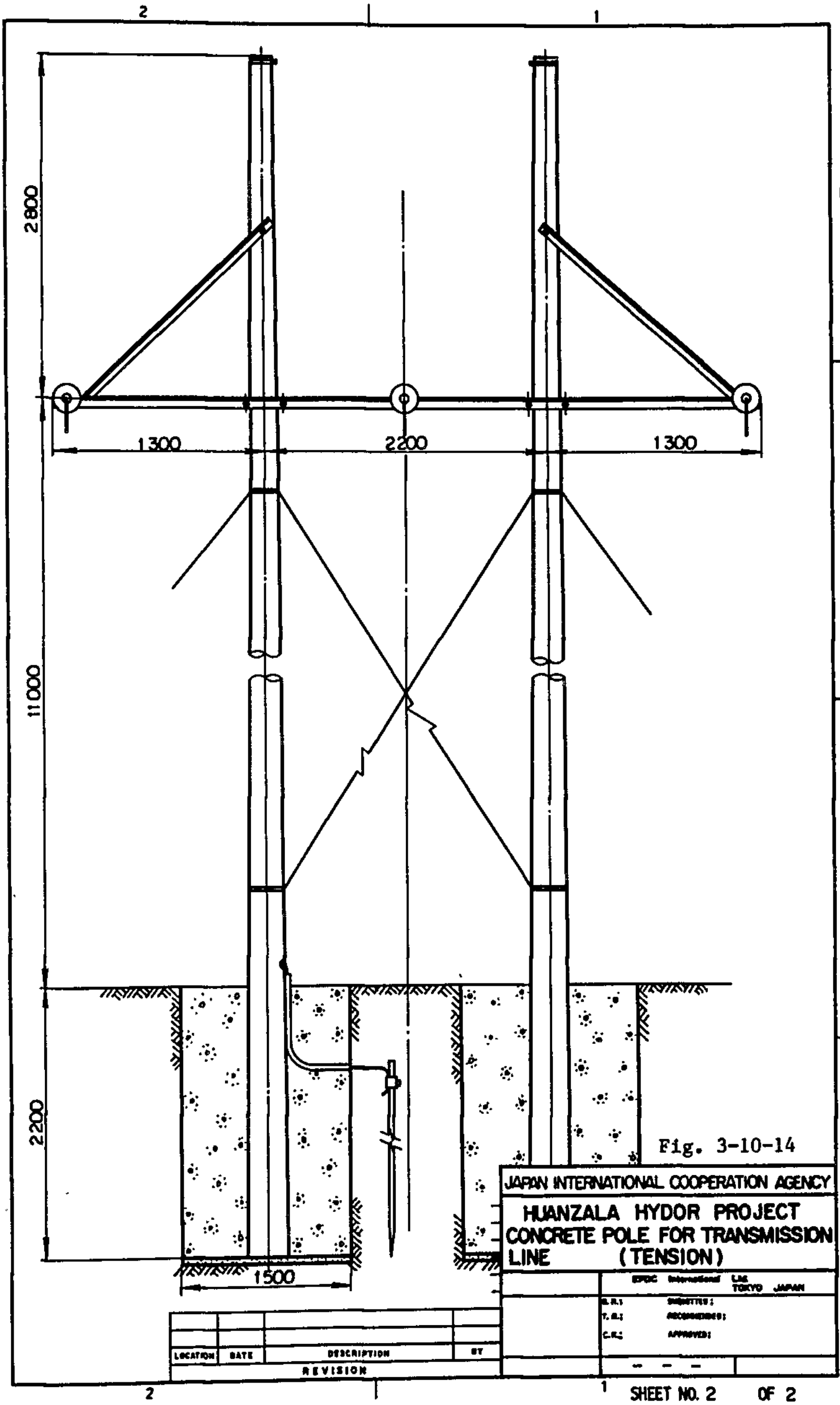

2800
1300
2200
1300
11000
2200
1500
JAPAN INTERNATIONAL COOPERATION AGENCY
HUANZALA HYDOR PROJECT
CONCRETE POLE FOR TRANSMISSION LINE (TENSION)
LOCATION
DATE
DESCRIPTION
BY
REVISION
SHEET NO. 2 OF 2

Fig. 3-10-14

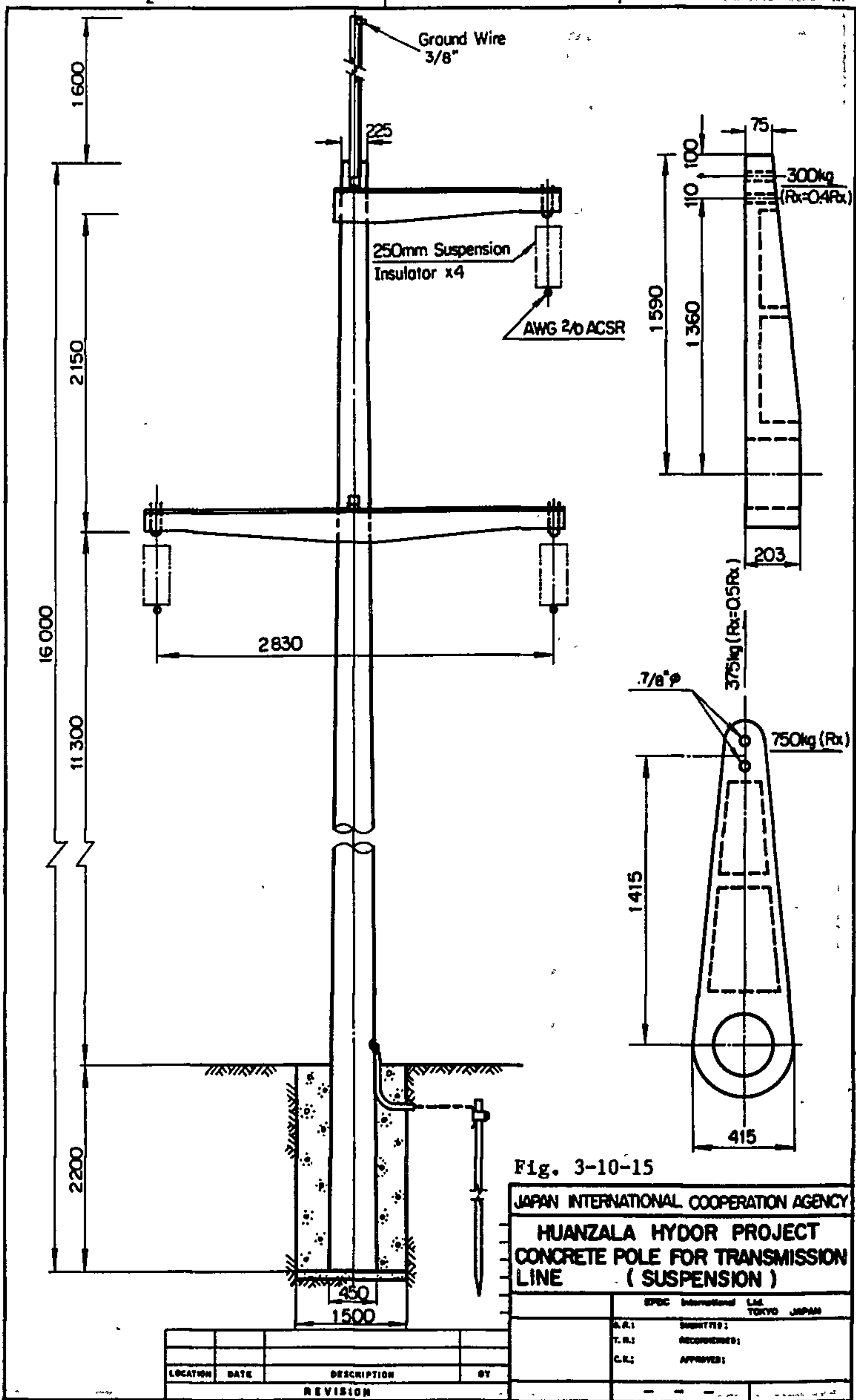


Fig. 3-10-15

JAPAN INTERNATIONAL COOPERATION AGENCY

HUANZALA HYDOR PROJECT
CONCRETE POLE FOR TRANSMISSION LINE (SUSPENSION)

EPDC International Ltd. TOKYO JAPAN

SHEET NO. 1 OF 2