gravel will be thrown and gabions laid on the upstream concrete slab under the water up to EL.285.0.

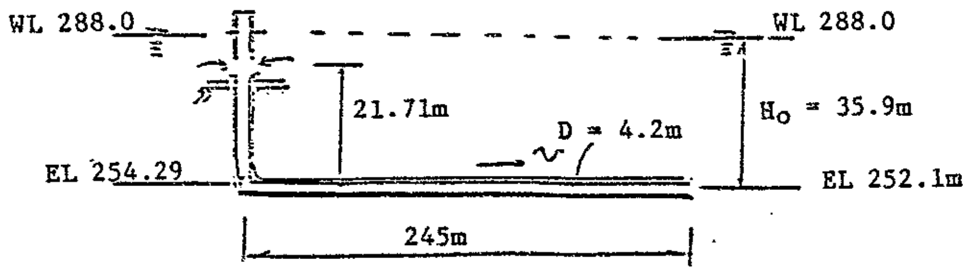
5. Spillway Facilities

5.1. Layout of Service Spillway

The service spillway of the Caliraya Dam comprises an intake tower constructed about 100 m offshore to the north-east of the Dam with a connecting horizontal tunnel approx. 245 m in length.

The intake tower has a cylinder gate of 4.2 m in diameter and 4.0 m in height, located around 10 m below the high water level. Beneath the cylinder gate is a vertical shaft of approx. 10 m with reinforced concrete lining. The vertical shaft and the horizontal tunnel are connected by an elbowshaped rivetted steel liner with 12.5 m curvature radius. The cross section of the horizontal tunnel is round, with 4.2 m internal diameter. (See Fig. V-10)

The spillway was constructed more than 40 years ago. Signs of extreme old age are evident on the gate, lining concrete and steel liner. Detailed drawings of the intake tower, gate and steel liner were not available for this field investigation. Since the gate opening was already submerged at the time of field investigation, detailed measurement was not possible. It is presumed from visual inspection of the intake tower and each part of structure that the gate comprises six openings each 1.2 m wide and 3.5 m high arranged in a circular formation. With the gate fully opened at high water level of the reservoir, the maximum possible discharge through the gate would exceed the discharge capacity of the tunnel and, therefore, the discharge capacity of the spillway would be governed by the discharge capacity of the tunnel. Our calculations indicate that the discharge capacity of the spillway in its present condition is about 250 m^3/sec. with the gate fully opened and the water level at EL.288.0.



$$H_o = \frac{v^2}{2g}\left(1 + 0.15 + \frac{124.5\ n^2}{D^{4/3}} L\right)$$

$$v = \sqrt{2gH_o/(1.15 + 0.963)} = 18.25 \text{ m/s}$$

$$Q = Av = 252.7 \text{ m}^3/\text{s}$$

$$L = 266.7 \text{ m}, \quad D = 4.2 \text{ m}, \quad n = 0.014$$

5.2. Water Leakage at the Service Spillway

On October 28, 1985, a member of the JICA Study Team entered the vertical shaft to inspect water leakage. Leakages were found in the following four (4) sections.

- Cylinder gate
- Concrete lining (vertical shaft)
- Curved steel liner
- Concrete lining (horizontal tunnel)

a) Leakage from the cylinder gate

No leakage was evident from the gate leaf. However, leakage was occurring through one spot in the upper gate seal and through three spots in the lower gate seal. The water spouting through these leaks was 2-3 cm in diameter.

b) Leakage from concrete lining (vertical shaft)

The reinforced concrete lining between the lower end of the cylinder gate and the steel liner is approx. 9.3 m long and 1.0 m thick. This concrete lining is seriously damaged at many points and has a number of cavities.

The cavities are grouped in 3 cross sections, each group distributed around the circumference of the lining. The largest, about 120 cm long, about 70 cm wide and 60 cm maximum depth, is located in the mid-portion of the lining with reinforcing bars exposed. The water leakage from this cavity is relatively small in connection with the size of the cavity. Water merely seeps through the concrete in droplets.

A more serious leakage occurs about 1.0 m above the steel liner. This damaged portion is of an irregular shape, about 70 cm wide and 20 cm high. Water gushes from the whole damaged area with sufficient power to strike against the opposite concrete lining.

The surfaces of the reinforced concrete lining including the cavities themselves, are contaminated with dark brown clayey materials 3 to 5 mm thick.

In the damaged portions, coarse aggregate is exposed representing a honeycomb structure. As mentioned above, the damage is aligned around the circumference of the lining. These facts indicate that the damage is not the result of corrosion due to water leakage but has developed as a result of insufficient compaction of the cold joints when the concrete lining was poured.

c) Leakage from the steel liner

The curved steel liner connects the vertical shaft and the horizontal tunnel. It was fabricated from riveted steel plates, about 90 cm in width and about 120 cm in length.

At least one quarter of the total number of rivets had become detached from the liner plates and water was gushing out of the vacant holes. These empty rivet holes spread over the entire surface but particularly at a point about 3.5 m below the top of the liner. The water spouts from these holes are of similar diameter and velocity, from which we can assume that the voids allowing free passage of water must lie between the steel liner and the backfill concrete, and the continuous cracks and voids in the backfill concrete and surrounding ground must enable easy passage of pressurised water from the reservoir.

d) Leakage from the horizontal tunnel

At the time of the field investigation, the horizontal tunnel was full of water. This prevented our access to the tunnel and direct inspection of leakage areas. By analogy with the leakages in the vertical shaft concrete and steel lining portions, and in consideration of the fact that the horizontal tunnel was constructed at the same time as the vertical shaft, it is considered likely that the lining concrete in the horizontal tunnel is damaged and affected by leakage. It is recommended that an earlier inspection of the actual conditions be made and appropriate remedial measures, taken.

According to NAPOCOR's observation report, the overall leakage volume at the spillway is estimated to be approx. 200 lit./sec.

5.3. Emergency Spillway (Saddle Spillway)

In addition to the service spillway, an emergency spillway is provided to avoid dam overflow due to any unexpected increase in water level. This emergency spillway, located about 1.6 km east-northeast of the main dam, comprises an open excavated channel, about 160 m wide and EL.289.2 - 289.3 crest elevation, constructed by means of excavation of a topographic saddle. This spillway is in the form of a broad-crested weir with a very gentle slope. The crest is paved with concrete 7 m in width, and is used as a public access road between Caliraya dam and Lumot Dam. Except this pavement, the emergency spillway comprises only excavated areas, without slope protection, covered with natural shrubbery and weeds. Downstream of the spillway crest is now private land, occupied by the gardens and recreation grounds of a hotel. In the event of overflow via this emergency spillway, water will flow across the garden and recreation ground to the Lumot River located about 3 km downstream. The overflow route is badly arranged, and unsuitable for a spillway. Therefore, practical steps should be taken to avoid overflow via this emergency spillway. The discharge capacity of this emergency spillway can be roughly calculated by the following formula.

$$Q = 273\ H^{3/2}$$

where,

Q : Overflow discharge (m^3/sec.)

H : Overflow water depth (m)

5.4. Restoration of Spillway and Increase of Discharge Capacity

The service and emergency spillways installed at the Caliraya Dam have the problems as mentioned above.

Only one tunnel spillway is provided for the Caliraya Dam as a service spillway, especially the concrete lining, which is damaged by age and it has the possibility of tunnel collapse if it is left unrepaired.

In case of a tunnel collapse, the function of the service spillway installed at the Caliraya Dam Will be stopped and it will become impossible to control the increase in water level at a time of flood because the overflow crest of the emergency spillway is 1.3 m higher than H.W.L. and it is of free overflow type.

The discharge capacity of this service spillway is about 250 m^3/sec at H.W.L. 288 m and is restricted by its tunnel section. Therefore, it is impossible to expect the increase of its discharge capacity to cope with any unexpected increase in water level.

At a flood time (20% increase of 200 years flood), the water level in the Caliraya reservoir with the present spillway facilities will be over EL.290.5, which leaves only a 1.5 m freeboard against the non-over flow dam portion elevation 292.0. Since there is a possible danger of dam collapse due to wave overtopping the non-overflow section of the dam at a time of wind velocity more than 25 m/sec., restoration of the existing service spillway and installation of a new spillway for increase of discharge capacity are emergency works.

FIG. V-1 Landslide at the East dyke

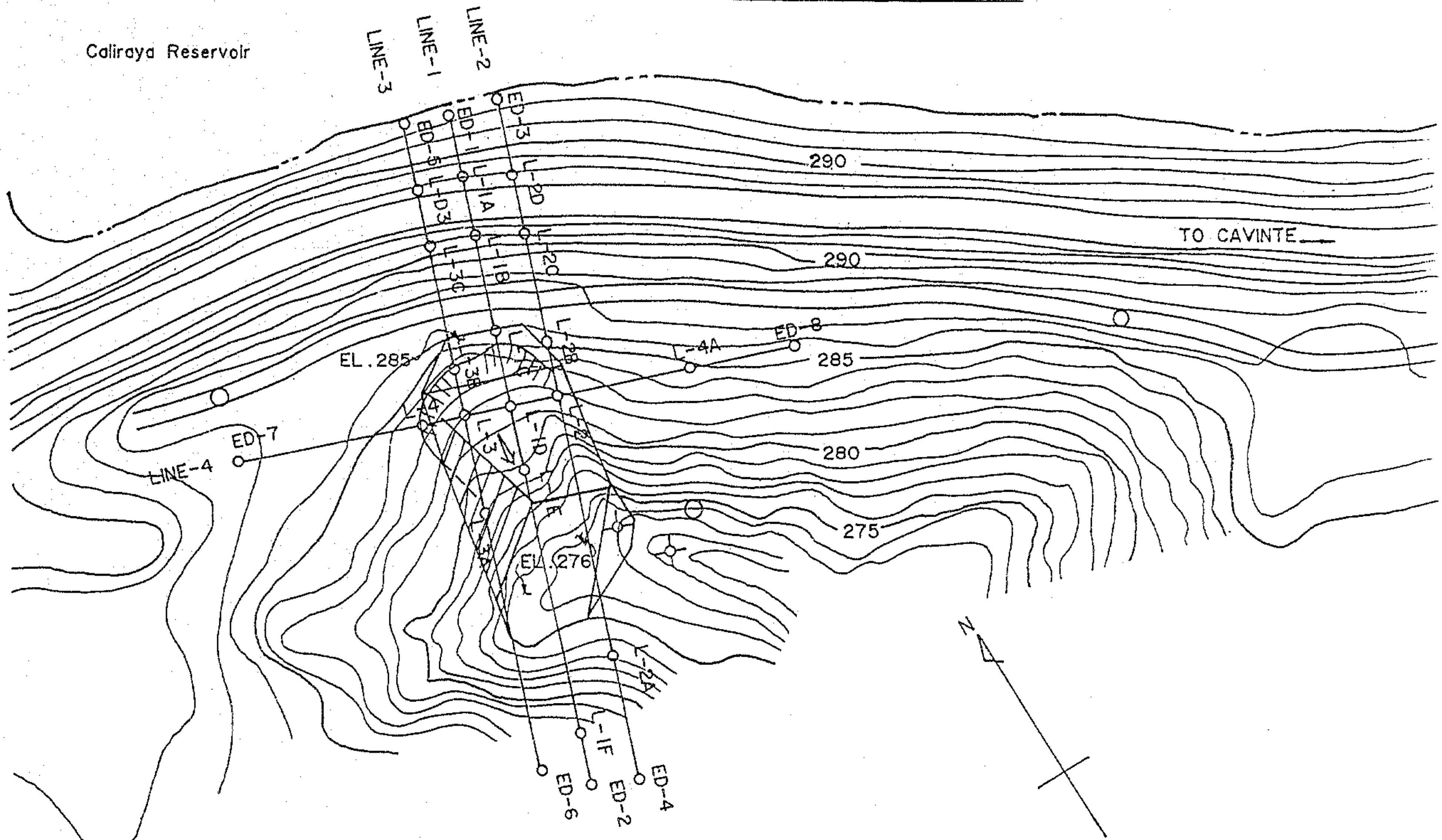
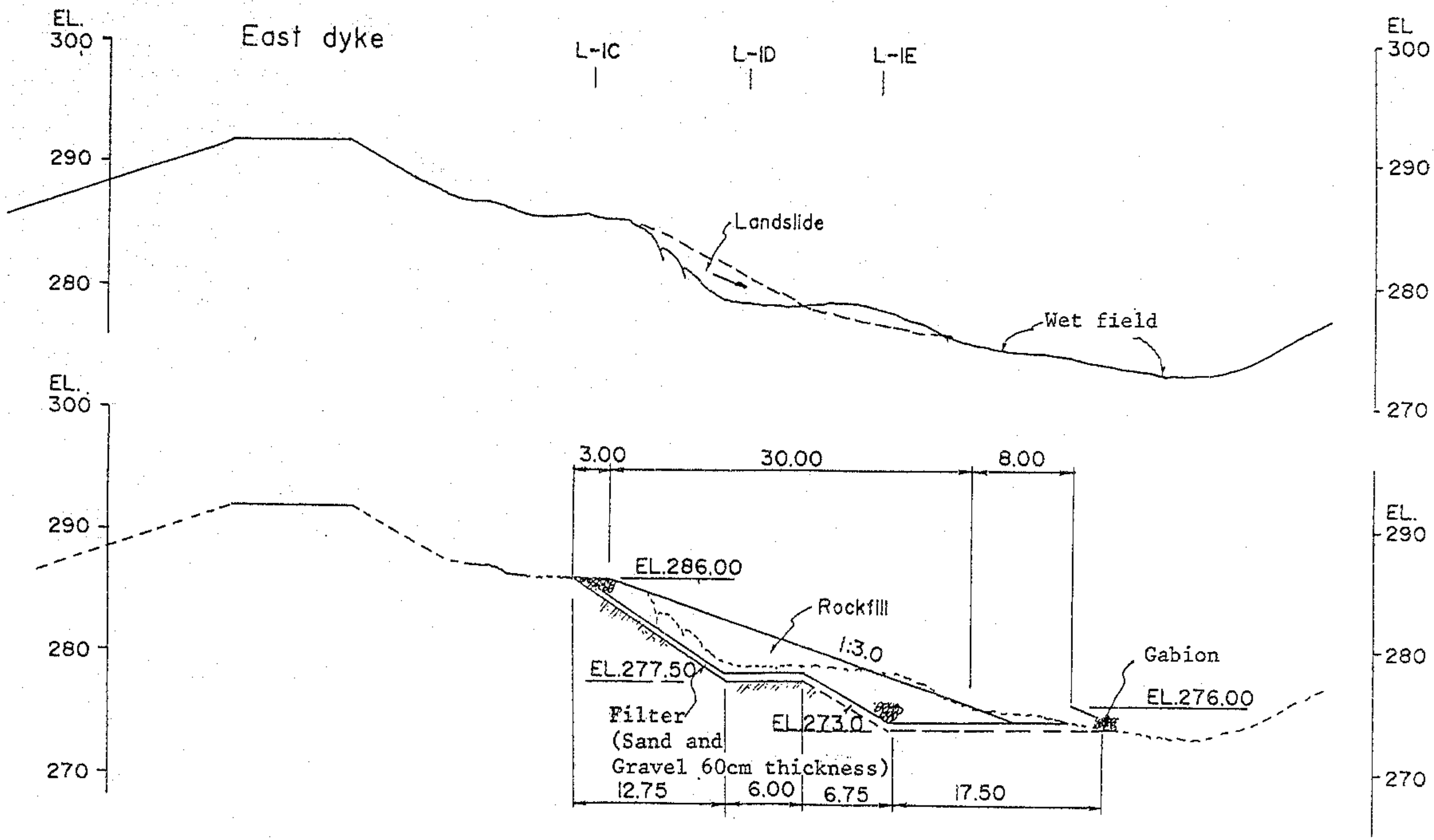


FIG. V-2 Landslide at the East Dyke



Rehabilitation Plan

FIG. V-3 Ground-Water Level of Caliraya Dam

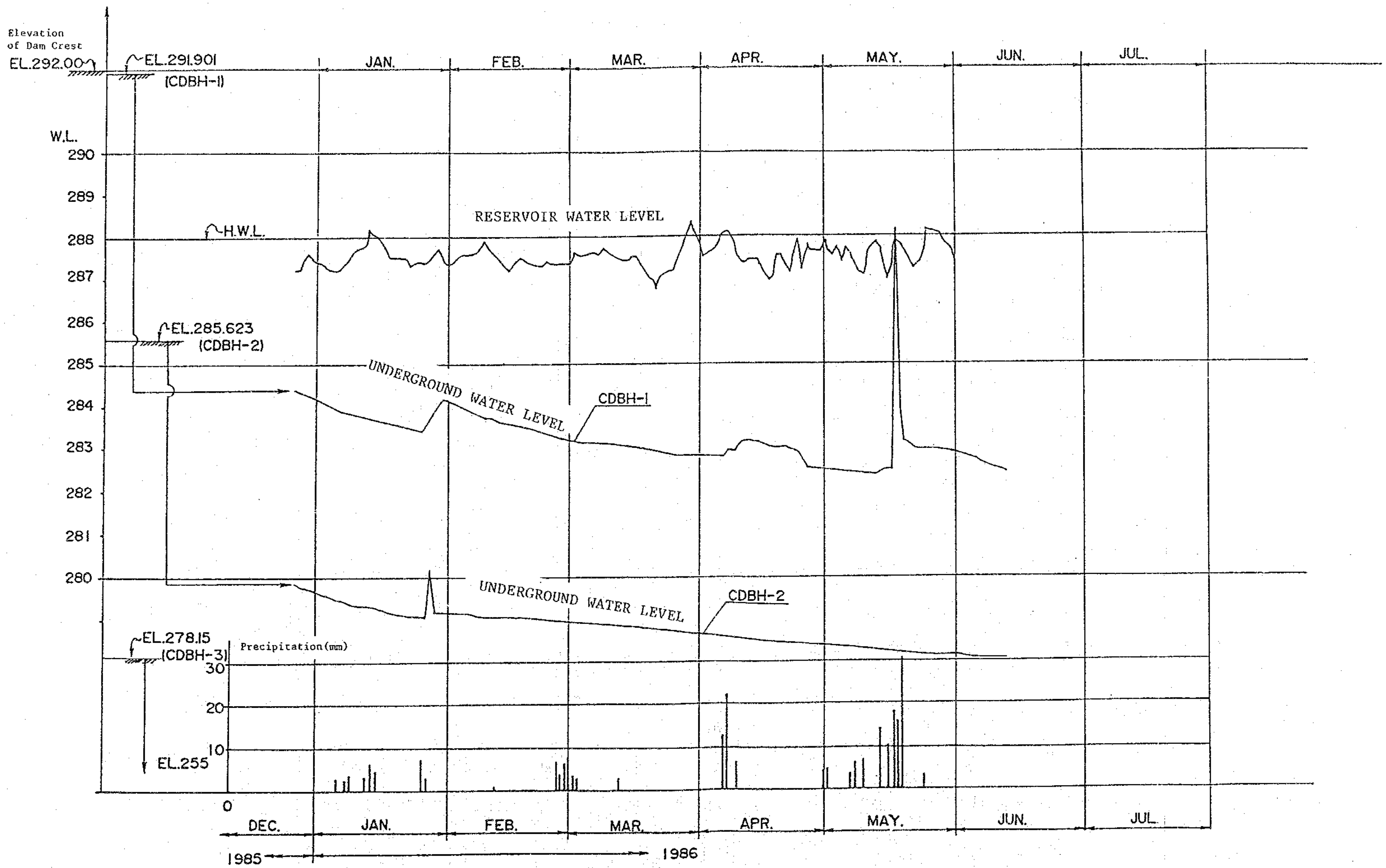


FIG. V-4 Horizontal Displacement

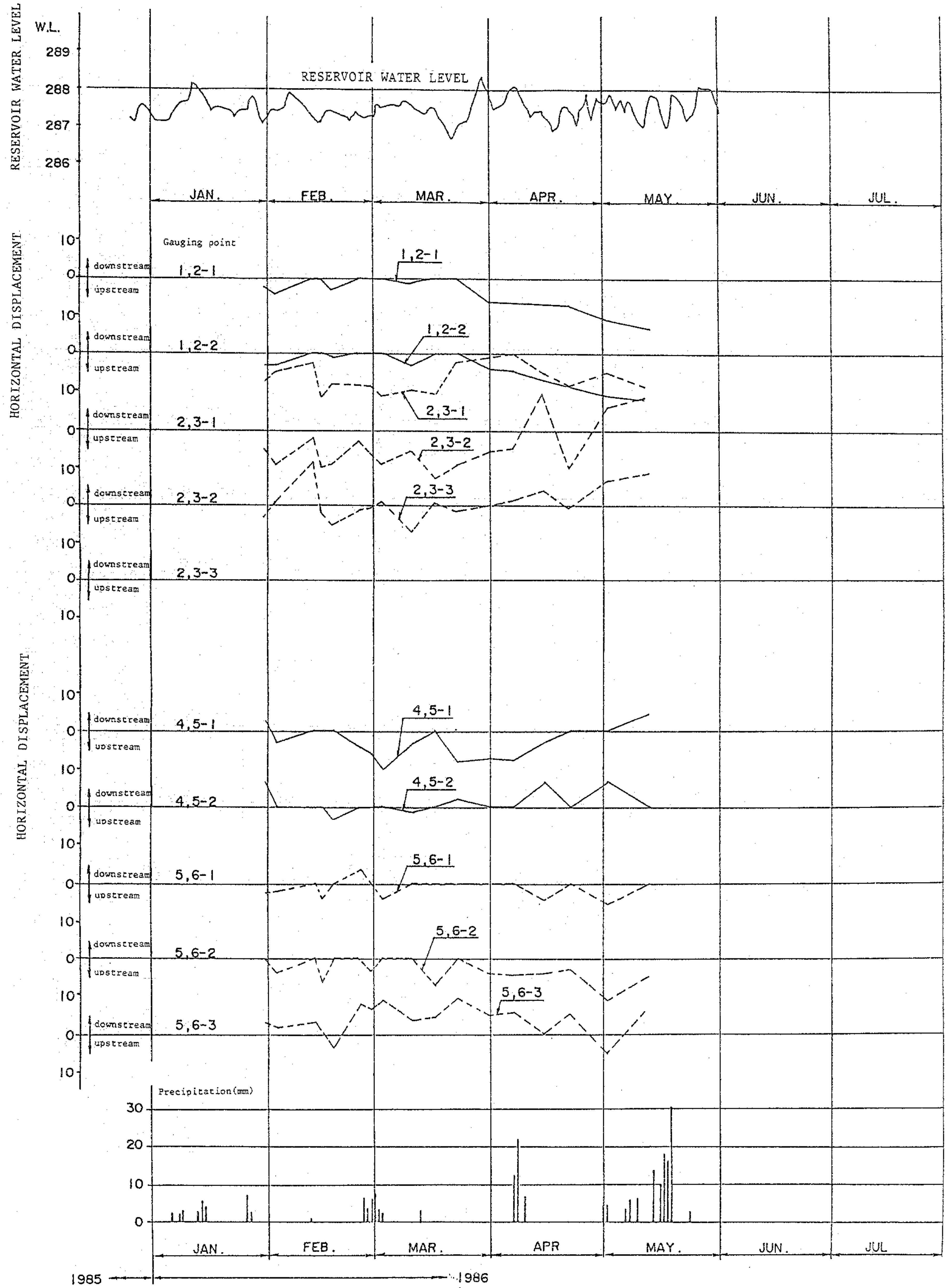


FIG. V-5 Vertical Displacement

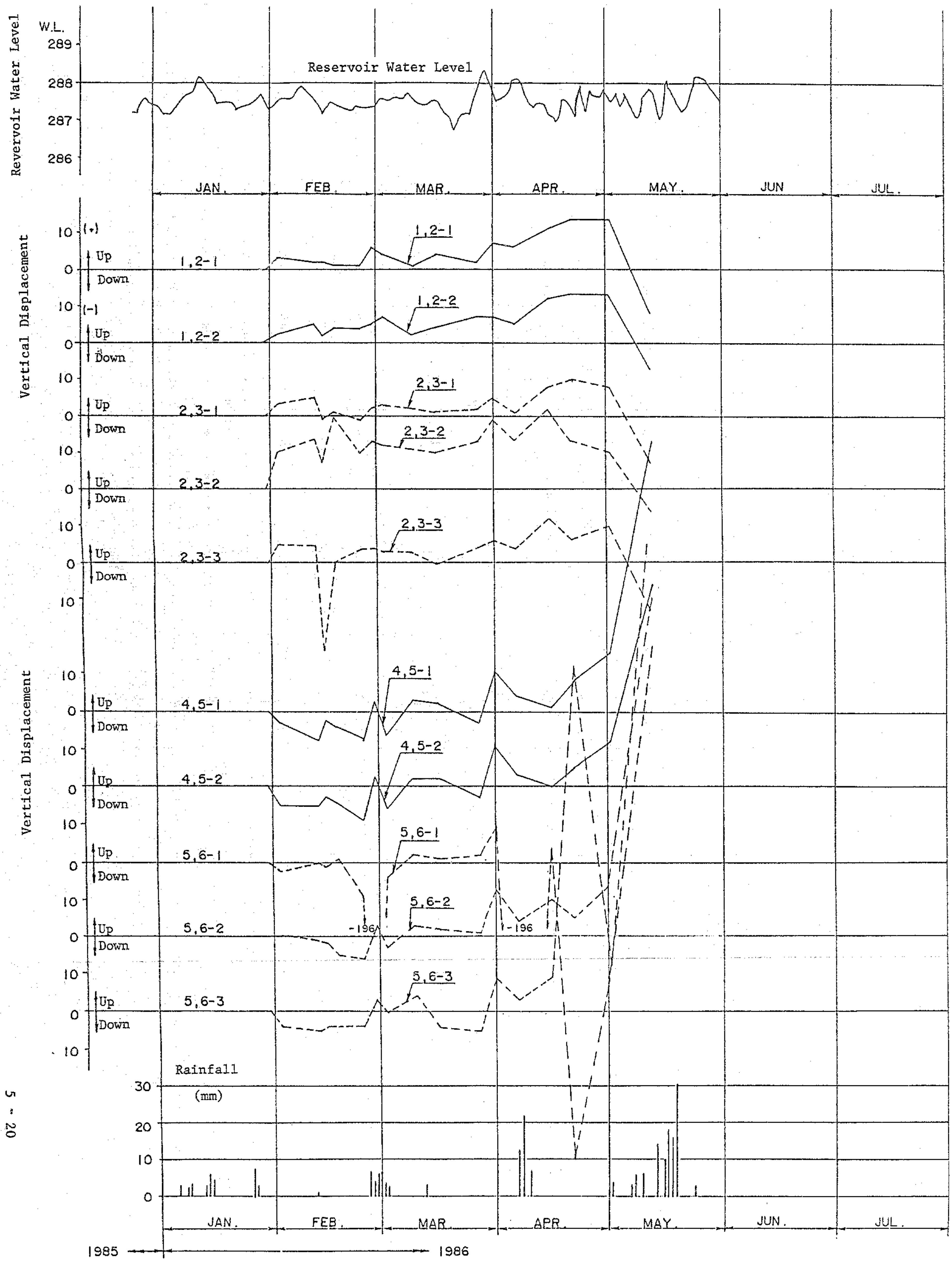


FIG. V-6 Leakage at Weir

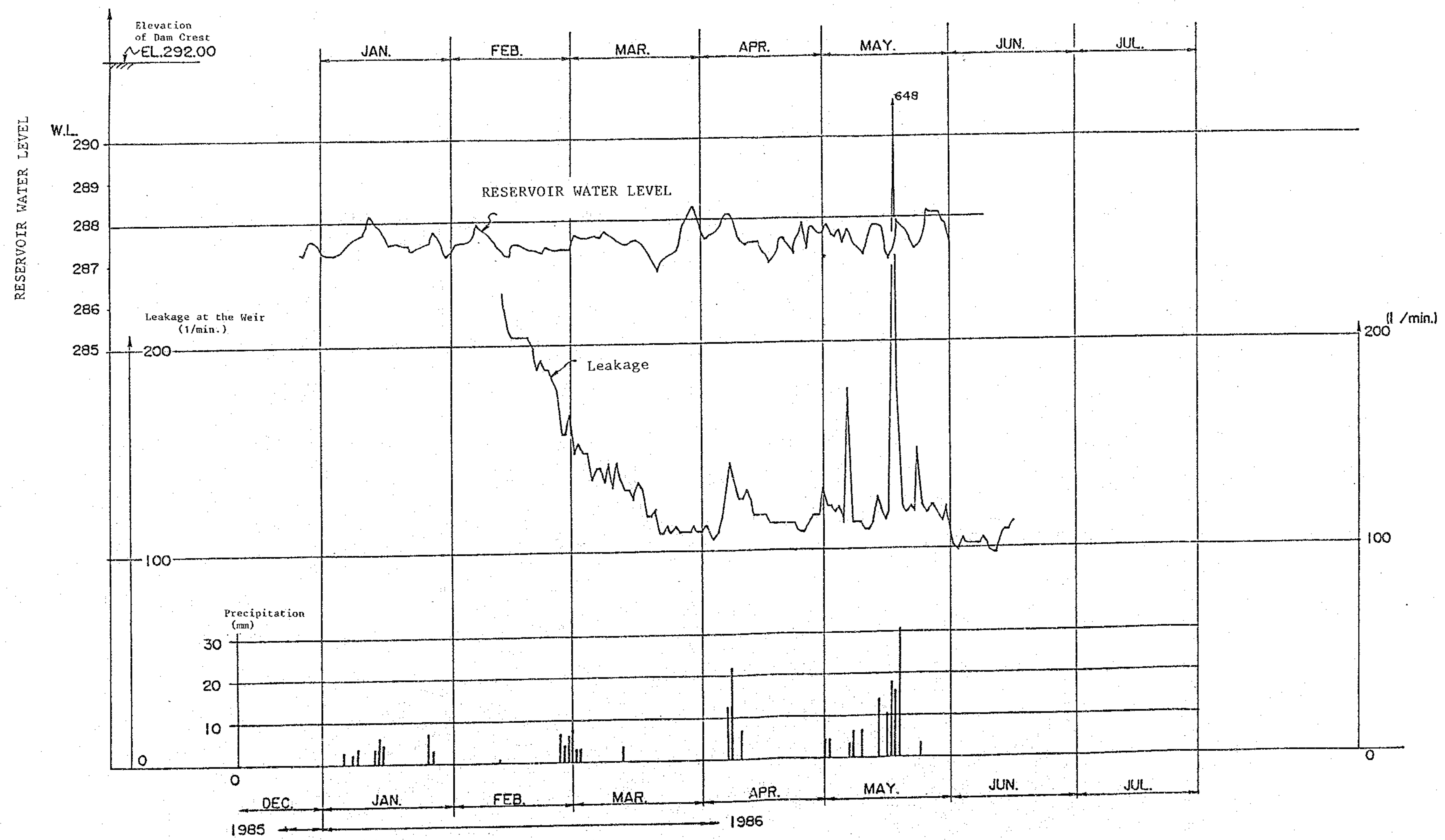


FIG. V-7 Leakage at Weir

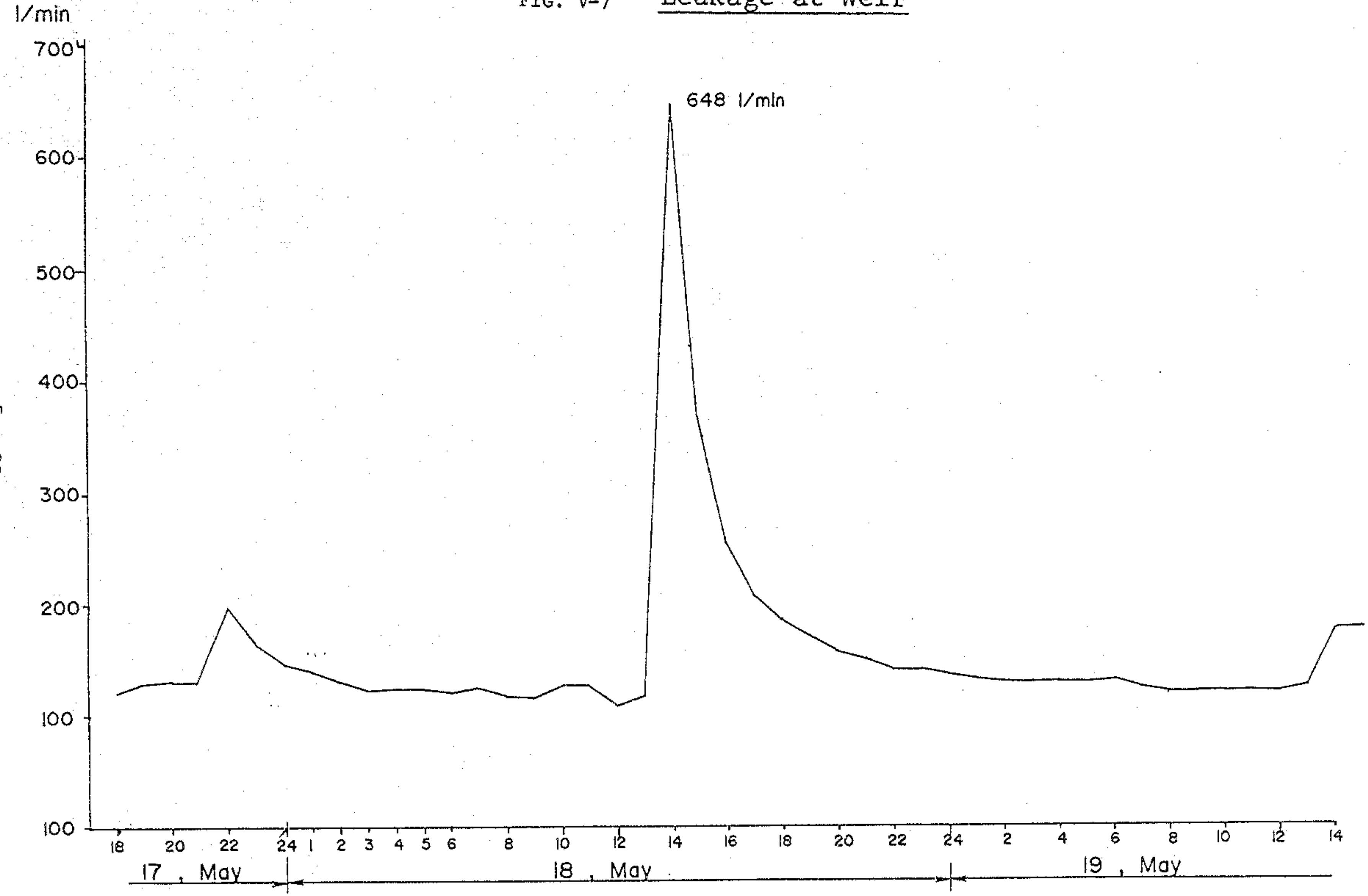
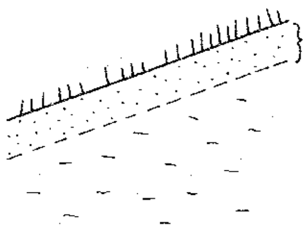


FIG. V-8 Explanation of Erosion Process at Downstream Face of Caliraya Dam

1. Normal Slope

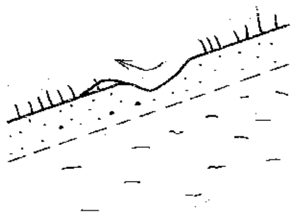


Turfed area is used as a grazing ground for domestic animals.

Surface ground: Not clear (10 to 20 cm)

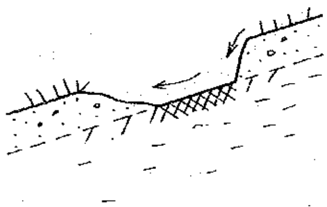
Dam embankment: Composed of weathered tuff breccia, with good grain distribution but easy to crack by drying.

2.



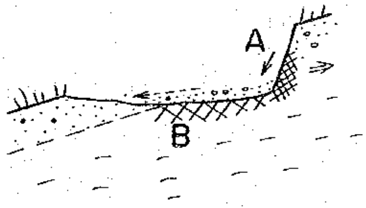
Face of slope is damaged by animal footmarks. This damage develops through rainwater erosion. Thus, the surface becomes exposed.

3.



Cracks occur on the exposed surface of the embankment of which material is already easily detachable and broken into small pieces. These pieces are washed out by rainwater.

4.



"A" surface is always exposed. (Since the broken pieces are washed out by rainwater).

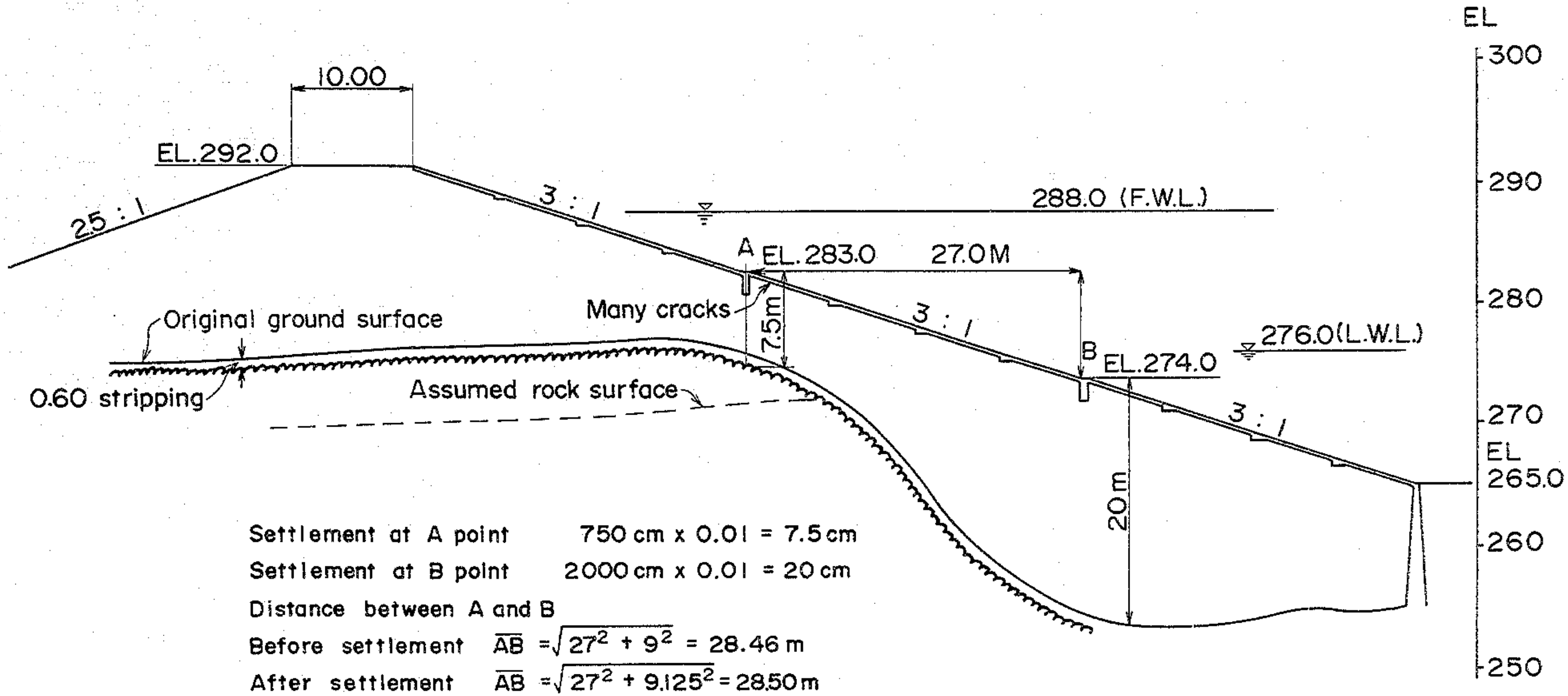
Some of the washed pieces are settled on "B" surface until the next rainfall. Because of this, the generation of cracking is mitigated on "B" surface.

As a result, "A" surface is moved towards upstream.

Fig. V-9 Location of Cracks on the Concrete Face Slab

(Section STA. 0 + 280)

S : 1/500



Settlement at A point 750 cm x 0.01 = 7.5 cm

Settlement at B point 2000 cm x 0.01 = 20 cm

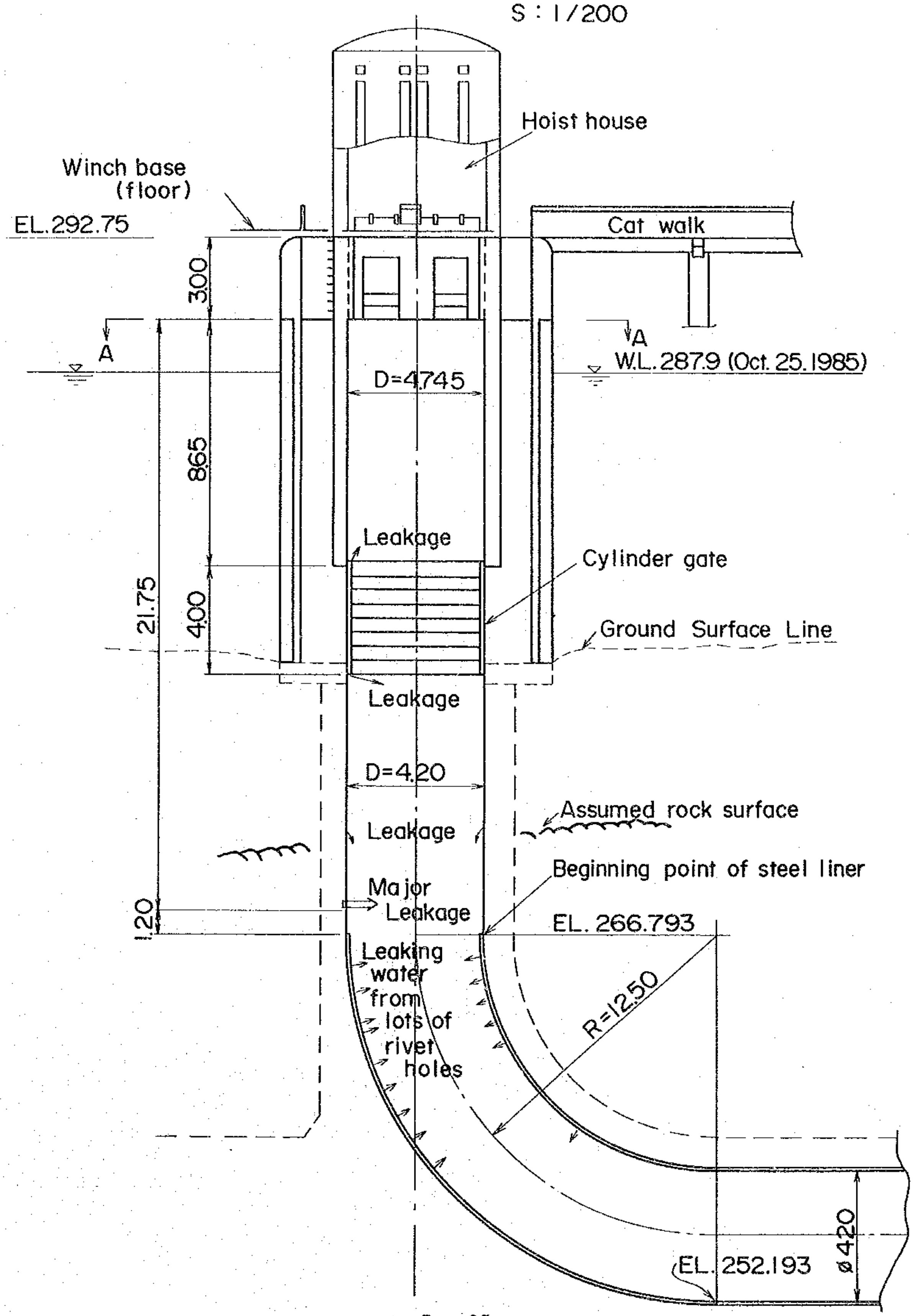
Distance between A and B

Before settlement $\overline{AB} = \sqrt{27^2 + 9^2} = 28.46$ m

After settlement $\overline{AB} = \sqrt{27^2 + 9.125^2} = 28.50$ m

Elongation $\Delta\overline{AB} = 28.50 - 28.46 = 0.04$ m

Fig. V-10 Service Spillway (Caliraya Dam)



VI. FLOOD OF THE CALIRAYA RESERVOIR

VI. Flood of the Caliraya Reservoir

The catchment areas of the Caliraya and Lumot Dams are relatively small, only 92 km^2 and 37 km^2 respectively, but the total reservoir capacity at HWL is large - approximately 115.6 x $10^6 m^3$ (Caliraya area of 76.8 x $10^6 m^3$ and Lumot area of 38.8 x $10^6 m^3$) which corresponds to about 40% of the total annual run off.

A waterway tunnel, of circular section 2.0 m in diameter and 1,850 m in length, connects the Caliraya Reservoir with the Lumot Reservoir. HWL is EL 288 at the Caliraya Reservoir and EL 290 at the Lumot Reservoir. The inflow into the Lumot Reservoir is transmitted to the Caliraya Reservoir through a connecting waterway, but because of the small capacity of the connecting waterway, the most of flood flow would be released through the Lumot reservoir spillway ("morning glory" type).

The average annual rainfall in this area is 3,380 mm. The rainy season is October to December, the dry season is January to May, and July to September is an average rainfall period. (See Figs. VI-1 & VI-2).

Since there is no stream-gauging station along the Caliraya river, the rate of inflow into the reservoir was calculated from rainfall data recorded by the rainfall gauge installed at the Caliraya Dam spillway watchman's cottage.

The available rainfall records are:

- Daily rainfall records from 1950 to October 1985.
- Hourly rainfall records from October 28, 1984 to April 5, 1985 and from May 12 to October 16, 1985.

The Caliraya Reservoir is located on the Caliraya Plateau, and its circumference is surrounded by hills of gentle slope at an elevation of about 400 m. The reservoir presents a typical dendritic drainage pattern basin, its water edge entering in a complicated form into hills of relatively small height. A distance from the boundary line of the basin to the reservoir is as short as less than 7 km even in the longest span, and an average slope of the river bed until it reaches the reservoir end is about 1/60. This topographic feature carries a rainfall in the basin to the reservoir within about one (1) hour.

Since the travelling hour of peak discharge caused by a rainfall into the reservoir is as short as about one (1) hour, hourly distribution of the rainfall is required to establish flood pattern. Table VI-1 shows maximum precipitation in one day and maximum precipitation in successive two days in each year for the period of 35 years from 1950 to 1985. The probable rainfalls for different return periods which were computed with the values shown on Table VI-1 are shown on Table VI-3. This table shows three (3) results of probable rainfalls computed by the Gumbel-Chow Method, the Moment Method and L.N.D. Further analysis was made, for sake of safety, with the probable rainfalls estimated by the Gumbel-Chow Method which show the largest values.

The hourly rainfall rt can be obtained from the daily rainfall (R24) by the following formula;

$$rt = R24 \left(\frac{t}{24}\right)^{k} \ldots\ldots\ldots\ldots\ldots \text{(VI-1)},$$

where

rt = precipitation in "t" time

R24 = precipitation in 24 hrs.

k = constant

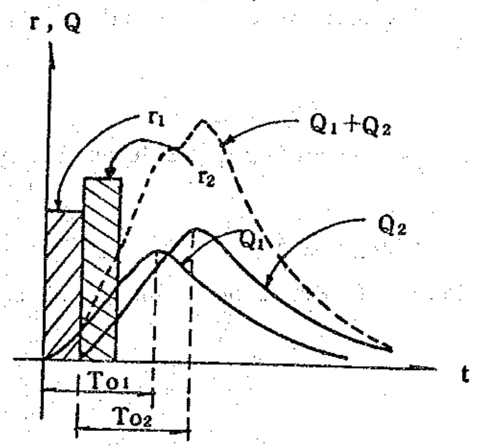
The constant k can be counted back by the formula (VI-1), if the daily precipitation and hourly precipitation are given. Table VI-2 shows the constant k which was obtained from the largest 10 daily rainfalls and the hourly rainfalls during the period when both of them are known. Generally, it is known that as the daily rainfall increases, the constant k becomes smaller and it comes near to a certain value under a large daily rainfall. Dr. Mononobe proposes 1/3 as this value. For the following analysis, k=1/3 was used.

The daily rainfall pattern which is required for estimate of flood discharge into the Caliraya Reservoir was worked out by distributing the hourly rainfall which equalizes the gross precipitation and daily rainfall, on both sides of the peak hourly rainfall which was obtained from the daily rainfall on Table VI-3 by the formula (VI-1). The hourly rainfall distribution chart shown on the upper part of Fig.VI-4 was obtained in this method.

A hourly rainfall at a time will carry a discharge Q at a location in the reach of travelling hour To. This Q can be expressed by the following run-off distribution function:

$$Q = \frac{r}{3.6} A.f.\alpha^2.te^{-\alpha t} \quad \ldots\ldots\ldots \quad \text{(VI-2)}$$

where Q = discharge (m^3/sec)
r = hourly rainfall (mm/hr)
A = catchment area (km^2)
f = coefficient of run off
α = 1/To
To = time of flow
(flood travelling hour) = 1 hr
t = elapsed time



The inflow peak flood into the Caliraya Reservoir obtained based on the hourly rainfall distribution chart and the run-off distribution function of the formula (VI-2) is summarized below.

Inflow Peak Flood into the Caliraya Reservoir

Return Period (Year)	Peak Discharge (m^3/sec)
1000	2632
200	2173
100	1968
50	1770
10	1300

Changes in the reservoir water level in case of the flood inflow patterns obtained by the above method is as indicated on the Figures VI-3 and VI-4. Similar computations were carried out for various cases and the flood water level in the reservoir obtained by those computations is as shown on the Table VI-4.

According to the standards in Japan, the probable flood volume to be considered for the Caliraya dam which is a fill-type dam is 120 per cent of a 200 year return flood, and the spillway should be equipped with a spilling capacity so that the reservoir water level may be controlled safely enough against this flood inflow.

The height of the non-overflow section of a dam should be more than the values indicated below (according to the design standard of dams in Japan).

A) In case the dam is equipped with spillway gates,

i) $Hn + hw + he + 0.5$

$Hn + 2.0$ where $hw + he < 1.5$ m

ii) $Hs + hw + \frac{he}{2} + 0.5$

$Hs + 2.0$ where $hw + \frac{he}{2} < 1.5$

iii) $Hd + hw + 0.5$

$Hd + 1$ where $hw < 0.5$

(3) In case the dam is not equipped with spillway gates,

i) $Hn + hw + he$

$Hn + 2$ where $hw + he < 2$

ii) $Hs + hw + \frac{he}{2}$

$Hs + 2$ where $hw + \frac{he}{2} < 2$

iii) $Hd + hw$

$Hd + 1$ where $hw < 1$

The above are applicable to a concrete dam and the abovementioned design standard specifies that the applicable values for a fill-type dam should be more than the values obtained by the above formula added further by 1.0 m.

In the above formula, Hn, Hs and Hd m represent a normal HWL, a surcharge water level and a design flood water level respectively, and hw and he mean a height (m) of waves from the reservoir water surface due to winds and a height of waves from the reservoir water surface due to earthquakes respectively.

In case of the Caliraya dam, the normal HWL (Hn) is EL.288.0 and the design flood level is EL.290.0, however, the surcharge water level is not made clear.

The height of waves from the reservoir water surface due to winds (hw) can be obtained in combination of the S.M.B. method with the Saville method. In case of the Caliraya dam, hw becomes 1.8 m using an average velocity of 30 m/s, a distance to the opposite bank of 2 km and a grade of dam upstream face of 1:3.0.

The height of waves from the reservoir water surface due to earthquakes (he) is given by

$$he = \frac{1}{2}\frac{k.t}{\pi}\sqrt{gHo}$$

where k (seismic coefficient) is 0.15, T (period of seismic wave) is 1.0 second and the reservoir water depth 28 m in front of dam is used, he becomes 0.4 m.

Therefore, for the normal HWL Hn,

$$hw + he + 0.5 + 1.0 = 3.7 \text{ m}$$

$$288.0 + 3.7 = 291.7 \text{ m} \quad 292.0 \text{ m} \quad \text{(height of dam crest)}$$

the standard in Japan is satisfied, but for the design flood water level,

$$Hd + hw + 0.5 + 1.0 = 293.3\text{m} \quad 292.0\text{m}$$

the standard in Japan is not satisfied.

Therefore, it is strongly desired in the Caliraya dam to increment spillway facilities so that the raise of reservoir water level can be restrained below Hd = 290.0 m. The new spillway proposed in this Report is considered to enable the reservoir water level at the time of flood to become less than 290.0 m and the new spillway equipped with two gates of 9.0 m in width and 5.0 m in height each is the minimum requirement in this respect. (See Tables VI-5 and VI-6).

Till the new spillway is realized, it is necessary to lower the reservoir water level preliminarily in case much rainfalls are anticipated by attack of typhoons, etc.

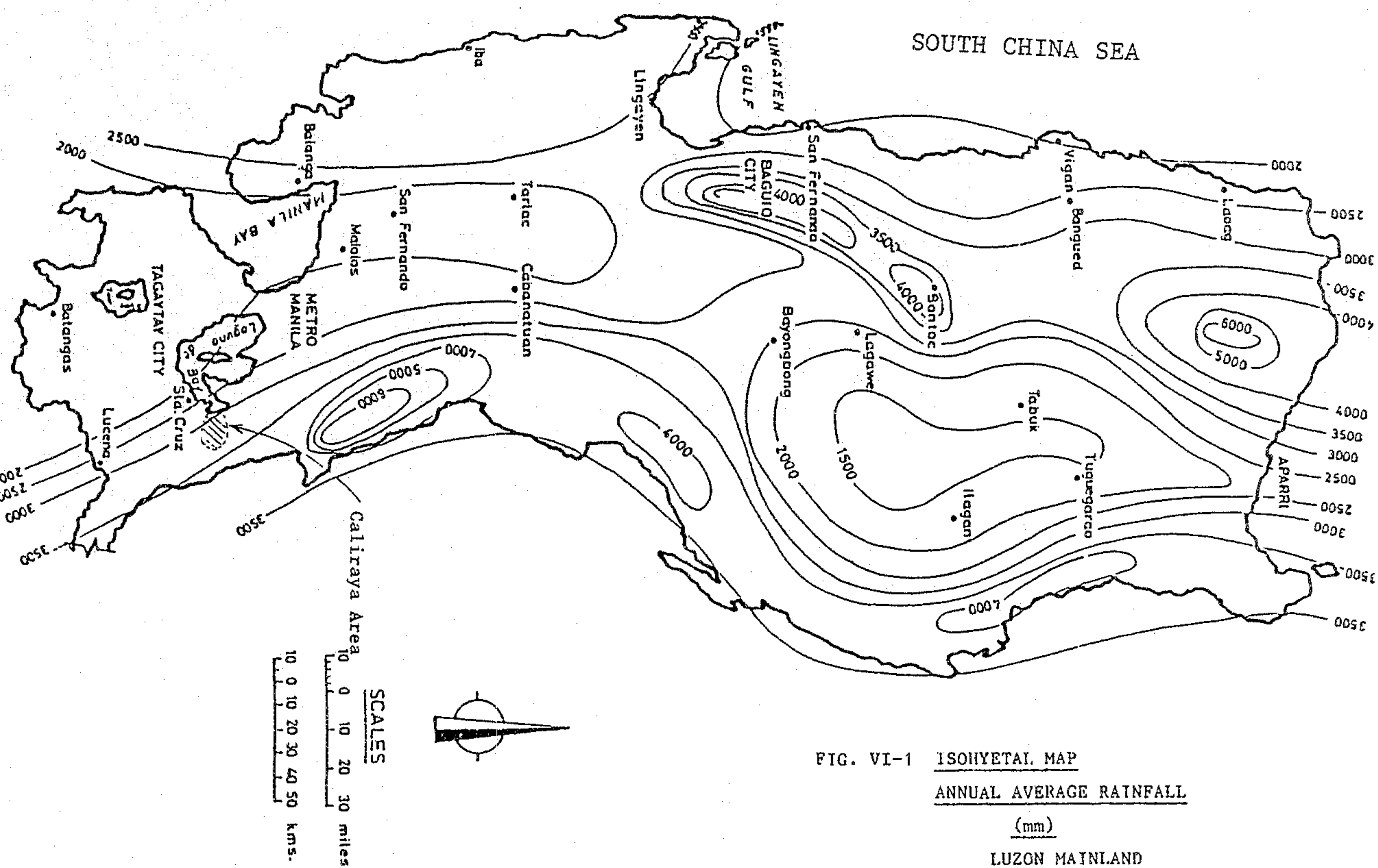
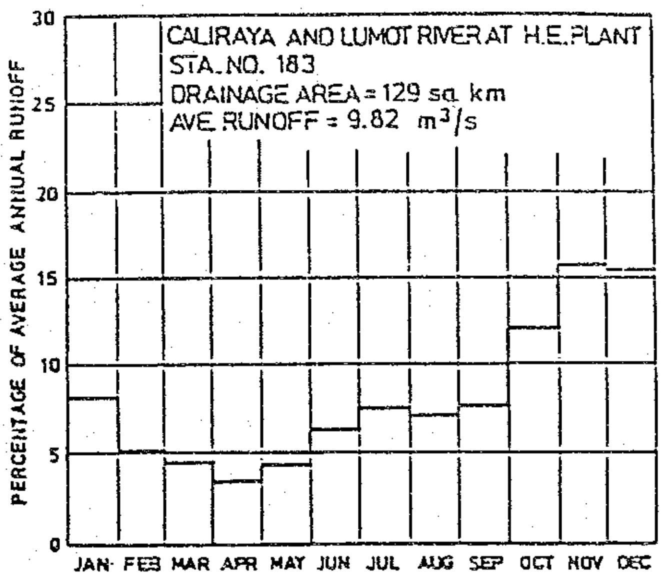


FIG. VI-1 ISOHYETAL MAP
ANNUAL AVERAGE RAINFALL
(mm)
LUZON MAINLAND

FIG. VI-2 SEASONAL DISTRIBUTION OF RAINFALL AND RUNOFF

(CALIRAYA)



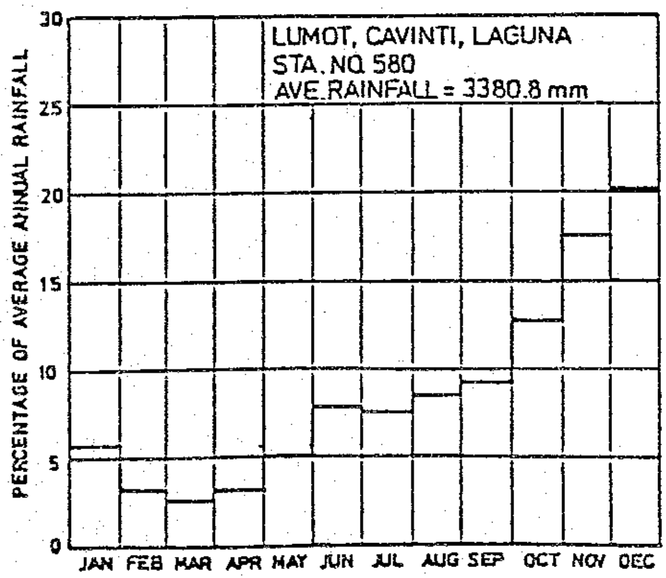


FIG. VI-3 Fluctuation of Reservoir Water Level

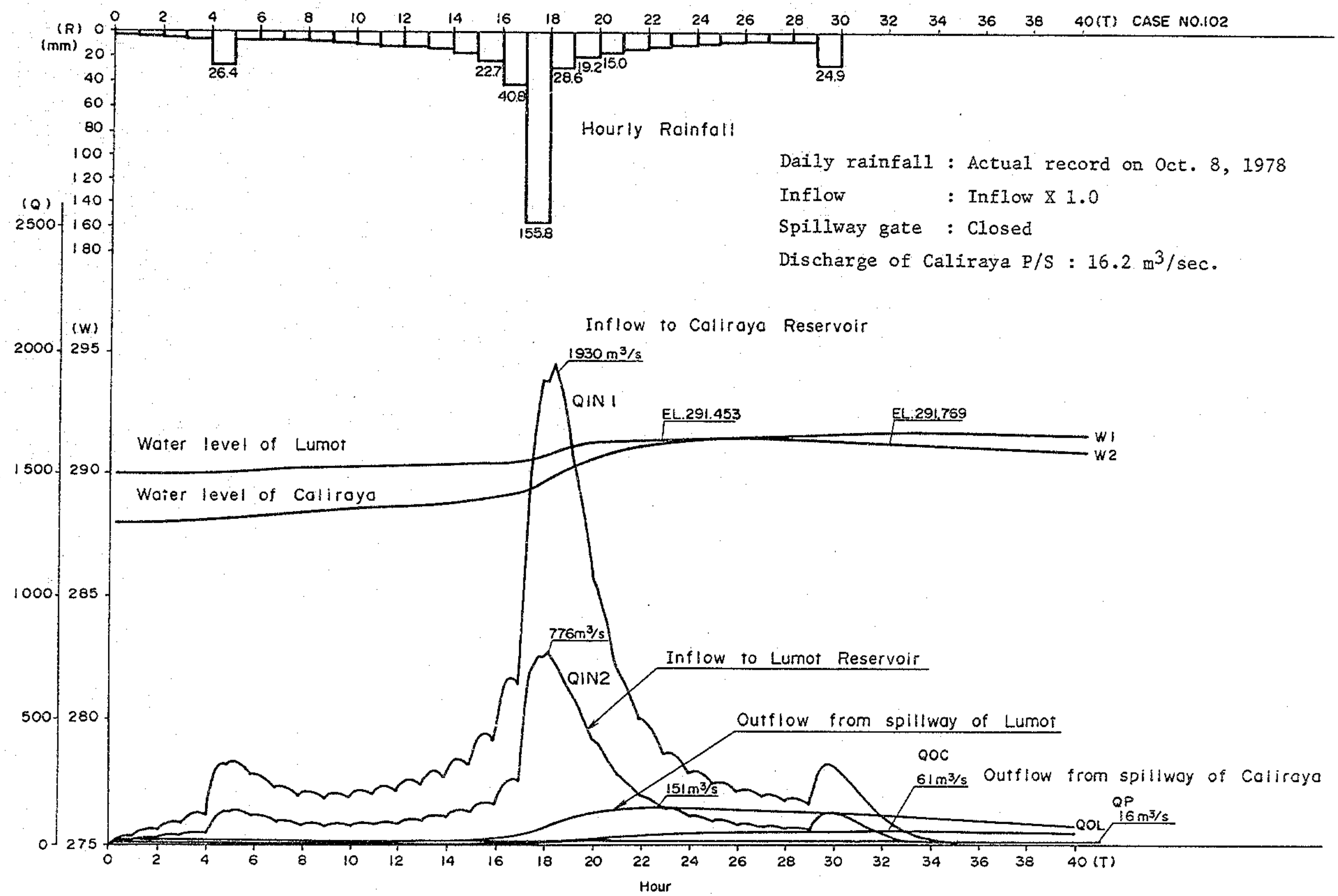


FIG. VI-4 Fluctuation of Reservoir Water Level

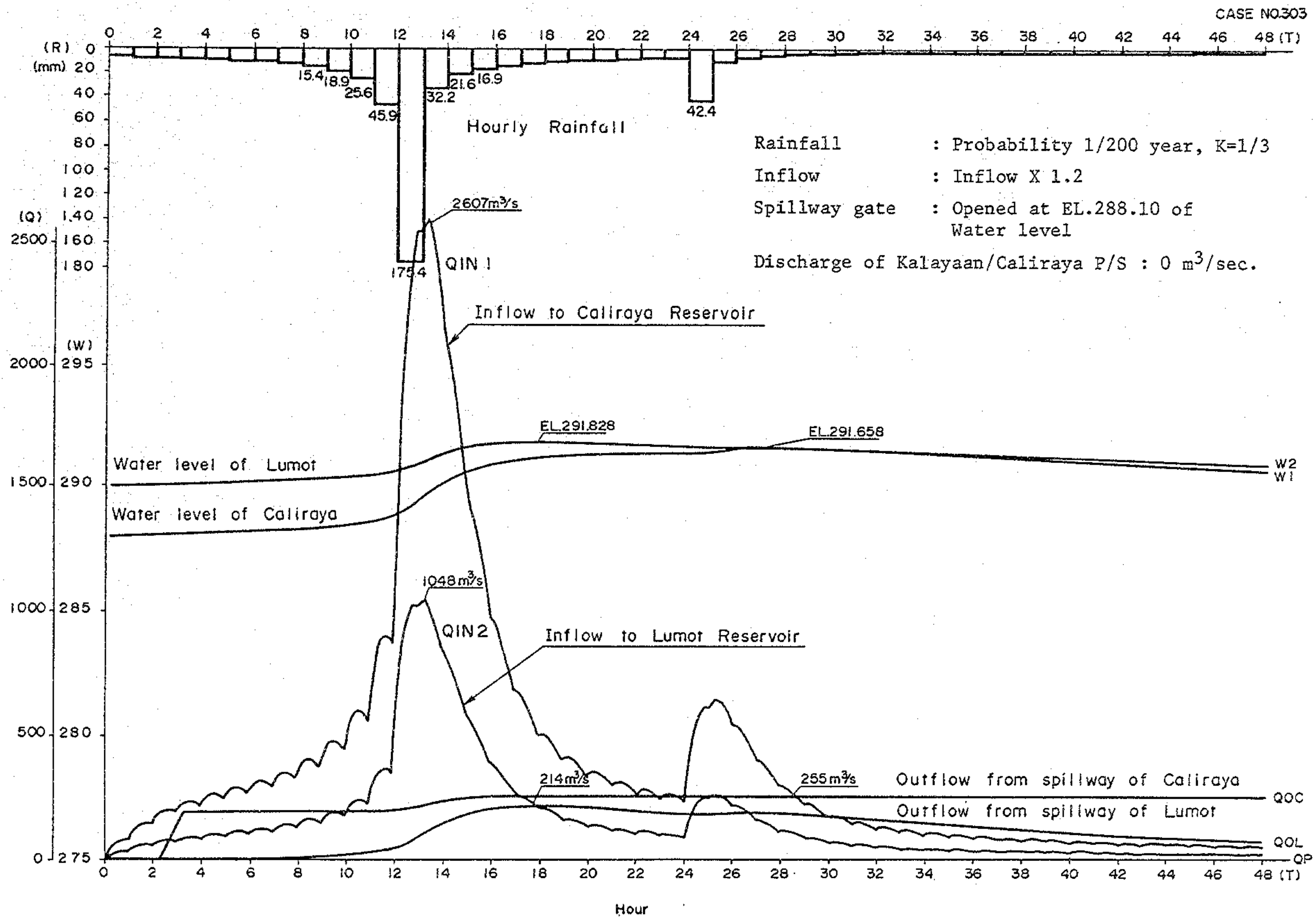


TABLE VI-1 Maximum Average Annual Rainfall

(Caliraya rainfall records)

Year	Max. Precipitation in one day	Month of occurrence	Max. Precipitation in successive two days
1950	216.4 mm	Dec.	253.5 mm
51	190.5	Nov.	201.2
52	156.0	Aug.	187.0
53	167.9	Nov.	248.2
54	101.4	Dec.	179.4
55	231.7	Nov.	270.3
56	113.8	Dec.	144.3
57	186.4	Oct.	278.9
58	201.9	Oct.	237.7
59	151.9	Dec.	209.3
1960	295.3	Jun.	335.9
61	242.3	Nov.	484.6
62	150.8	Sep.	265.6
63	136.6	Aug.	149.4
64	457.2	Jun.	473.5
65	130.8	Dec.	197.9
66	342.7	Dec.	377.3
67	185.9	Jan.	243.1
68	98.0	Jul.	133.1
69	137.9	Jul.	156.7
1970	213.6	Oct.	161.2
71	152.4	Dec.	268.0
72	145.6	Jun.	212.7
73	129.0	Nov.	115.8
74	135.1	Dec.	175.7
75	245.1	Nov.	247.6
76	224.1	May.	448.2
77	215.1	Nov.	244.3
78	452.1	Oct.	528.6
79	139.2	May.	167.4
1980	188.4	Nov.	296.6
81	115.3	Nov.	134.4
82	143.6	Sep.	269.8
83			
84	129.3	Oct.	250.0
85			

TABLE VI-2 Relation between Daily Rainfall and Peak Hourly Rainfall

(Caliraya rainfall records)

Date of occurrence	Daily Rainfall (R_{24}) mm	Peak Hourly Rainfall (r_p) mm	$K = \frac{\log r_p - \log R_{24}}{\log \left(\frac{1}{24} \right)}$
26 ~ 27 Nov. '84	113.0	23.0	0.50
21 ~ 22 Jun. '85	89.5	13.0	0.61
22 ~ 23 Jun. '85	85.5	18.5	0.48
27 ~ 28 Jun. '85	188.0	29.5	0.58
4 ~ 5 Jul. '85	117.5	27.5	0.46
5 ~ 6 Jul. '85	104.5	25.0	0.45
2 ~ 3 Sep. '85	101.0	16.0	0.58
9 ~ 10 Oct. '85	106.5	24.5	0.46
10 ~ 11 Oct. '85	101.5	43.5	0.27
12 ~ 13 Oct. '85	73.0	11.5	0.58

$$r_t = R_{24} \left(\frac{t}{24} \right)^k$$

r_t = Rainfall Within t hours

K = coefficient of Rainfall Intensity

TABLE VI-3 Probable Rainfall

Presumed year	Presumed daily rainfall (mm/day)			Presumed 2 consecutive days rainfall (mm/2 days)		
	Gumbel - chow	Moment	L.N.D	Gumbel - chow	Moment	L.N.D
1000	615.6	582.9	569.9	761.3	752.5	736.1
200	508.2	478.9	470.1	631.9	620.2	608.9
100	461.8	435.5	428.2	576.1	564.7	555.4
50	415.3	392.5	386.7	520.1	509.8	502.4
10	305.2	292.4	289.7	387.5	381.4	378.0
2	179.7	179.4	179.4	236.3	235.7	235.7

L.N.D. Logarithmic Normal Distribution Method

TABLE VI-4 SUMMARY OF CALCULATION OF CALIRAYA RESERVOIR WATER LEVEL
(Without Use of Emergency Spillway)

Case No.	Calculation Conditions								Calculation Results						
	Initial Water Level		Quantity used by Powerhouse		Spillway Gate		Hydrological Presumed Year	Magnification of Inflow	Max. Inflow		Max. Water Level		Max. Discharge		Flow of Connecting Waterway
	Caliraya	Lumot	Cagayan	Caliraya	Existing	Newly Established			Caliraya	Lumot	Caliraya	Lumot	Caliraya	Lumot	
	EL m	EL m	m^3/s	m^3/s				times	m^3/s	m^3/s	EL m	EL m	m^3/s	m^3/s	m^3/s
100	288.0	290.0	120	0	closed	none	200 years K = 0.45	1.2	2027	816	292.081	291.737	72	199	4.4
200	"	"	0	0	"	"	"	"	"	"	293.123	291.74	111	200	4.4
300	"	"	0	0	opened	"	"	"	"	"	291.527	291.736	255	199	4.4
400	"	"	120	0	"	"	"	"	"	"	290.616	291.733	250	198	4.5
500	"	"	0	0	-	h = 4 B = 18	"	"	"	"	290.291	291.734	524	198	4.4
actual 101	287.95	289.91	0	12.2	closed	none	9 Oct.'78 K = 0.45	1.0	1505	606	291.765	291.371	61	140	4.3
actual 102	"	"	0	"	"	"	9 Oct.'78 K = 1/3	1.0	1931	777	291.769	291.453	62	152	4.4
103	288.0	290.0	120	0	"	"	200 K = 1/3	1.2	2608	1049	292.178	291.829	76	215	
203	"	"	0	0	"	"	"	"	"	"	293.191	291.833	114	216	
303	"	"	0	0	opened	"	"	"	"	"	291.658	291.828	255	215	
313	"	"	0	0	"	"	100 K = 1/3	1.0	1968	792	290.441	291.409	244	135	4.5
403	"	"	120	0	"	"	200 K = 1/3	1.2	2608	1049	290.791	291.826	253	215	
503	"	"	0	0	-	h = 4 B = 18	"	1.2	"	"	290.505	291.826	550	214	4.6
304	"	"	0	0	opened	none	200 K = 1/3	1.0	2173	874	290.799	291.545	252	167	
404	"	"	120	16.2	"	"	"	1.0	"	"	290.056	291.544	230	167	

TABLE VI-5 Combination of Spillways and Maximum Water Level of Reservoir

		200 yr return flood	(200 yr return flood) x 1.2
	Peak inflow	2,173 m^3/s	2,608 m^3/s
Case 1	Service Spillway	EL 290.799 m	EL 291.658 m
Case 2	Service Spillway + Emergency Spillway	290.182 m	290.547 m
Case 3	Service Spillway + New Installed Spillway	289.400 m	289.774 m
Case 4	Service Spillway + New Installed Spillway + Emergency Spillway	289.400 m	289.740 m

Note: Existing spillway facility is Case 2.

TABLE VI-6 Discharge Capacity of Spillways

(Unit m^3/s)

Water Level of Reservoir	Service Spillway	Emergency Spillway	New Installed Spillway	Total
288.0	252.74	0	295.82	548.56
288.5	254.49	0	359.72	614.21
289.0	256.24	0	428.65	684.89
289.5	257.97	24.42	494.77	777.16
290.0	259.68	159.89	563.75	983.33
290.5	261.39	358.87	635.66	1255.92

Note: New Installed Spillway with 2 gates of width 9.0 m and height 5.0 m at 284.0 m of overflow crest level.

VII. CALIRAYA DAM STABILITY ANALYSIS

VII. Caliraya Dam Stability Analysis

Fig.VII-1 shows the largest cross section of the Caliraya Dam.

CDB-1, CDB-2 and CDB-3 in Fig.VII-1 are the boreholes used at present for measurement of the underground water level of the Dam. The phreatic line of the Dam assumed from the underground water level measured at the boreholes is also shown in Fig.VII-1.

The characteristic value of the embankment materials on undisturbed test pieces obtained from the test pit are given at Table III-2.

Figs. VII-2 & 3 show the results of stability analysis on the dam faces using the characteristic value of the embankment materials obtained by the soil tests and the section shown in Fig. VII-1. In case of computing the stability of the slope for fill-type dam, the safety factor for slip circle shall be generally more than 1.1 + α, taking into account of seismic force (according to Japan Dam Design Standard). In this calculation, α means an allowance of the safety factor to be settled by dam type, importance of dam, etc. Generally, α is regarded as 0.1. Therefore, the safety factor shall be more than 1.2 at a time of earthquake.

The continuous lines in Fig.VII-2 show sliding safety factor of the downstream slope of the dam at HWL of the reservoir, and the dotted lines show sliding safety factor when rapid drawdown occurs from HWL to LWL at the reservoir. Fig.VII-3 shows the safety factor at the time of an earthquake (k=0.15).

Fig. VII-4 assumes that the ground water level reaches to the surface of the downstream slope of the dam at an elevation lower than HWL. Even in this case, the stability of the downstream slope of the dam can be maintained at over 1.5, and slightly below 1.0 in the event of seismic condition of k=0.15.

These calculations assume that the embankment materials possess cohesion (C) and internal friction angle (ϕ) as shown in Table III-2.

As the sampling areas of soil test materials were limited, it cannot be said that these values would apply over the whole dam body. Fig. VII-5 shows the calculation results in case of the characteristic values being varied.

The lines in the Fig.VII-5 show the combination of C and ϕ values corresponding to the safety factor of just 1.0 and the characteristics over these lines give the sliding safety factor of more than 1.0.

Line (1) indicates the downstream slope of the dam, normal condition (without earthquake) and the water level being the observed value.

Line (2) indicates the downstream slope of the dam, seismic condition being k=0.15 and the ground water level being the observed value.

Line (3) indicates the upstream slope of the dam, normal condition and the reservoir water level with a rapid drawdown (EL.288 $\rightarrow$ EL.276).

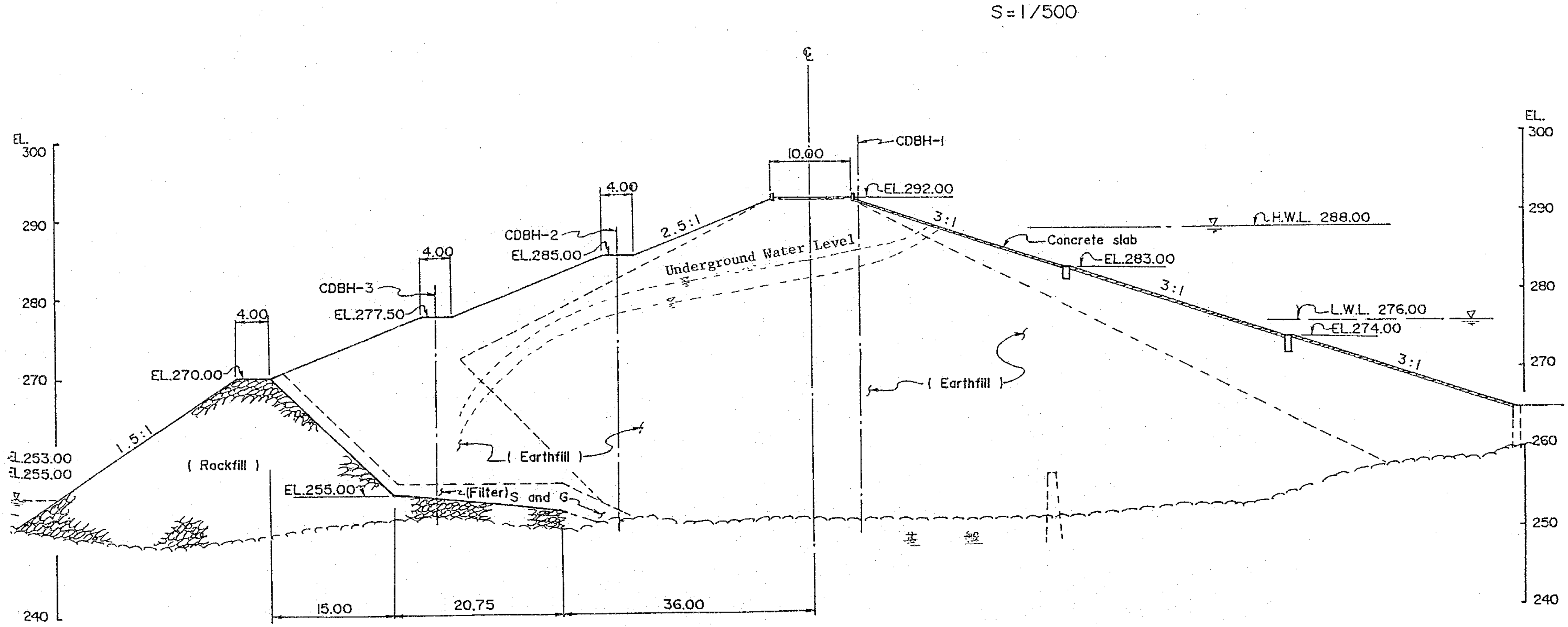
Line (4) indicates the upstream slope of the dam, seismic condition being k=0.15 and the reservoir water level with a rapid drawdown (EL.288 $\rightarrow$ EL.276).

Line (5) indicates the downstream slope of the dam, normal condition and on the assumption that the ground water level reaches to the surface of downstream slope of lower than EL.288.

Characteristics of embankment obtained from the triaxial compression test at this time and previous time (soil test on the samples taken from the test pie excavated by NAPOCOR during the period from 1984 to 1985) are also shown in Fig.VII-5 by 'O' marks and 'X' marks respectively.

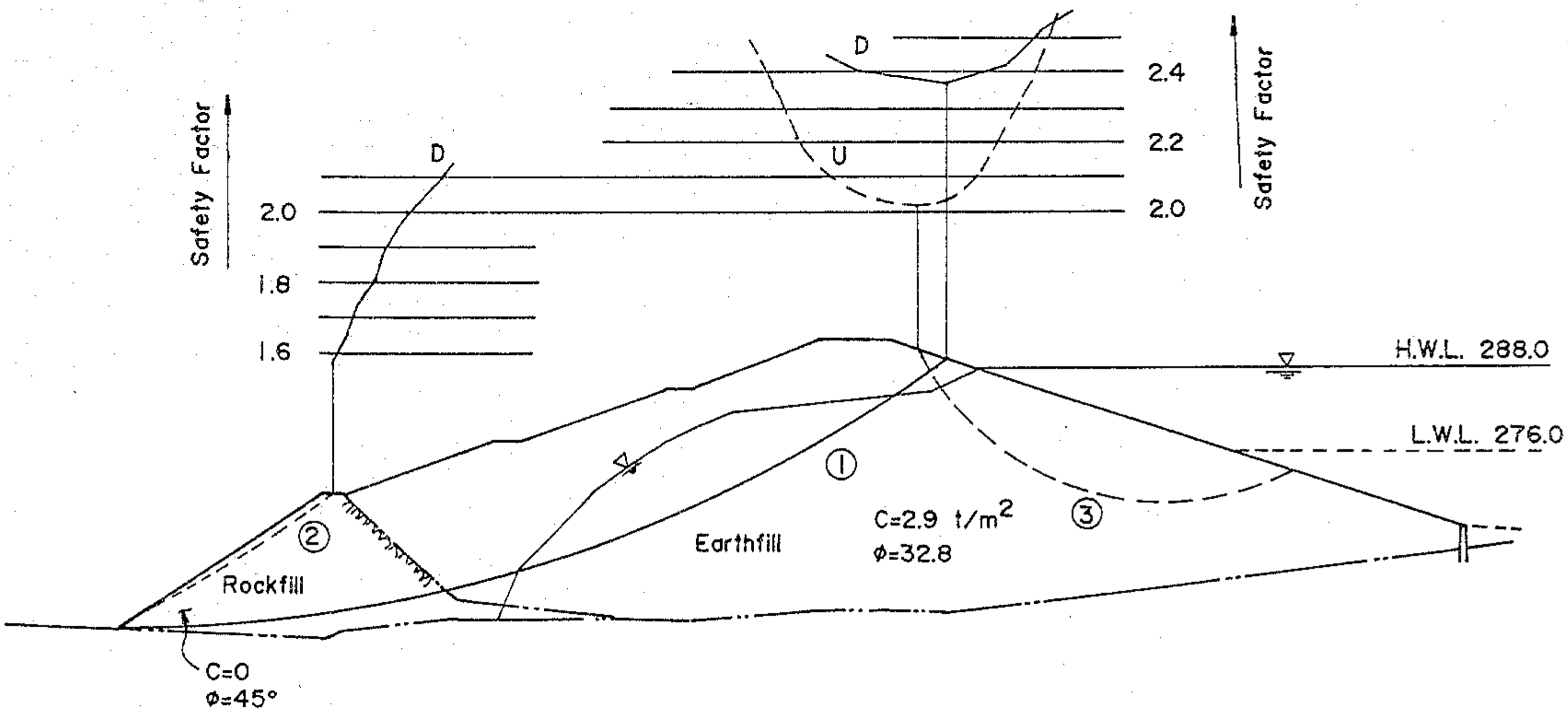
Referring to Fig.VII-5, it is considered that at present no significant landslide is likely to occur on either the upstream slope or the downstream slope of the Caliraya Dam, whether under normal conditions or even in case of a rapid drawdown of the reservoir water level from H.W.L. to L.W.L.

Fig. VII-1 Caliraya Dam - Largest cross-sectional view



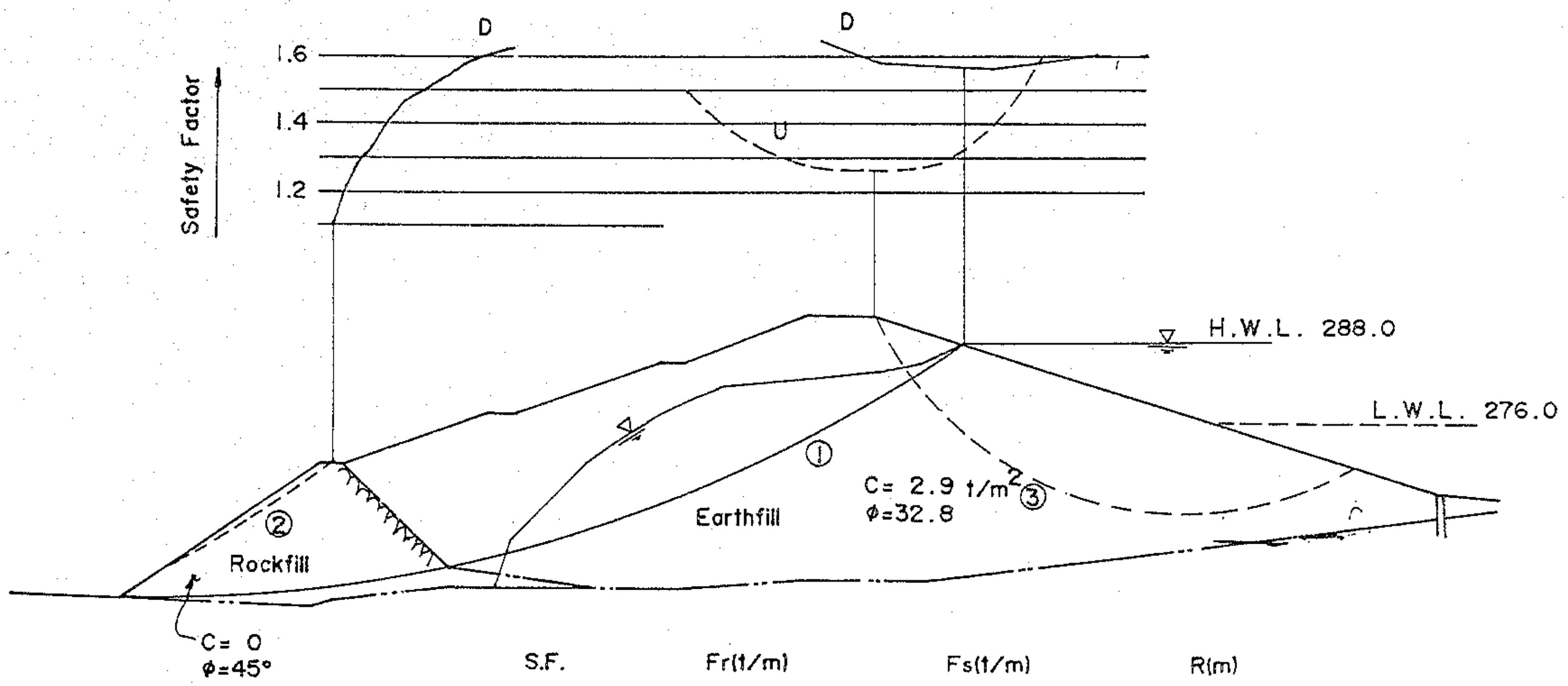
SECTION STA.0 + 384

FIG. VII-2 CALIRAYA DAM STABILITY (Normal)
(Section STA. 0 + 384)



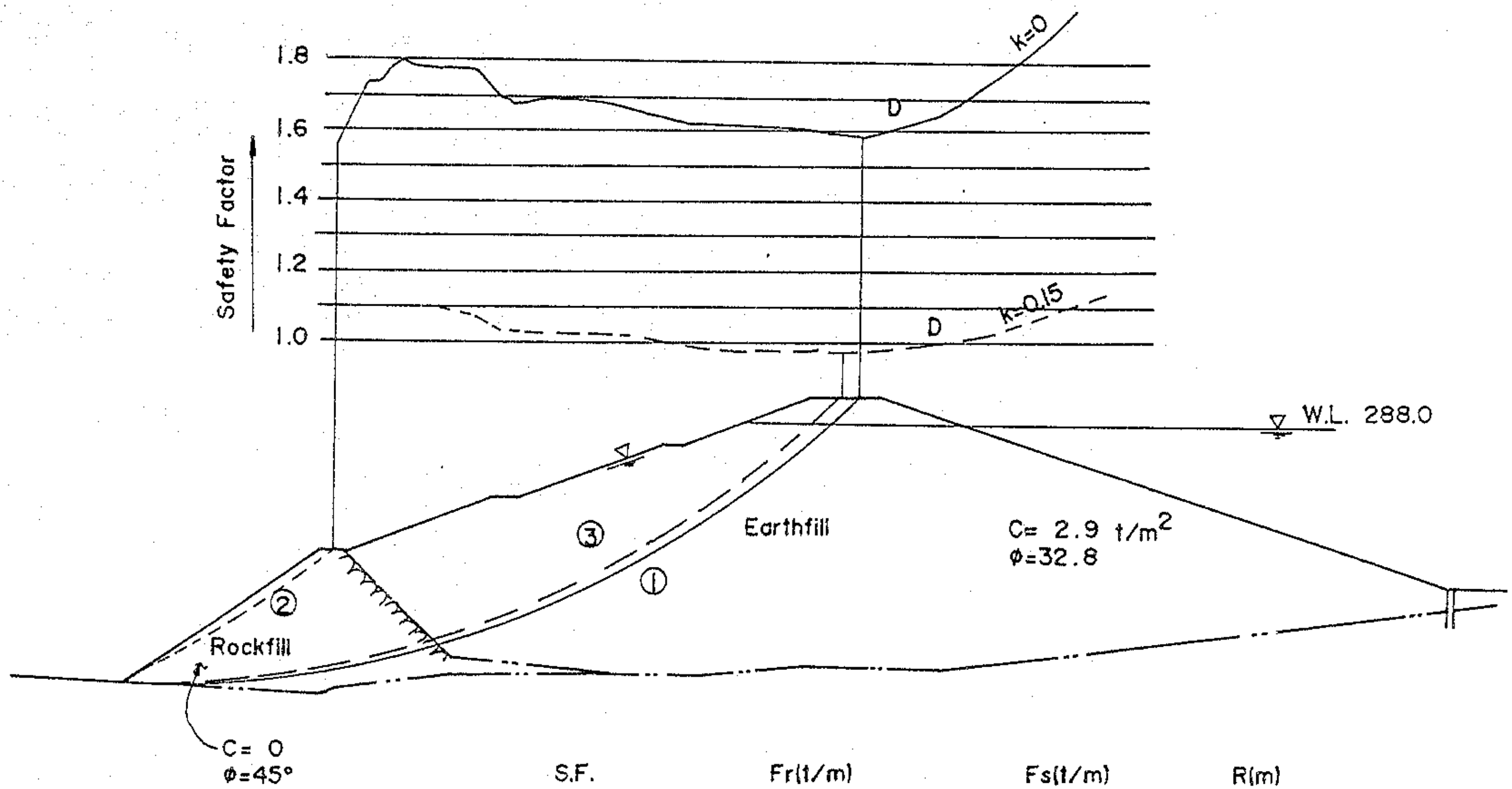
	S.F	Fr(t/m)	Fs(t/m)	R(m)
①	2.366	2,193.6	127.1	202.7
②	1.561	68.9	44.1	175.5
③	2.018	464.7	230.3	40.1 (WL.288.0 → 276.0, a sudden drop)

FIG. VII-3 CALIRAYA DAM STABILITY (Seismic Condition k = 0.15)
(Section STA. 0 + 384)



	S.F.	Fr(t/m)	Fs(t/m)	R(m)	
①	1.558	2,456.9	1,577.5	214.9	
②	1.145	62.3	54.4	175.5	
③	1.261	672.2	533.2	51.9	(W.L. 288.0 → 276.0, a sudden drop)

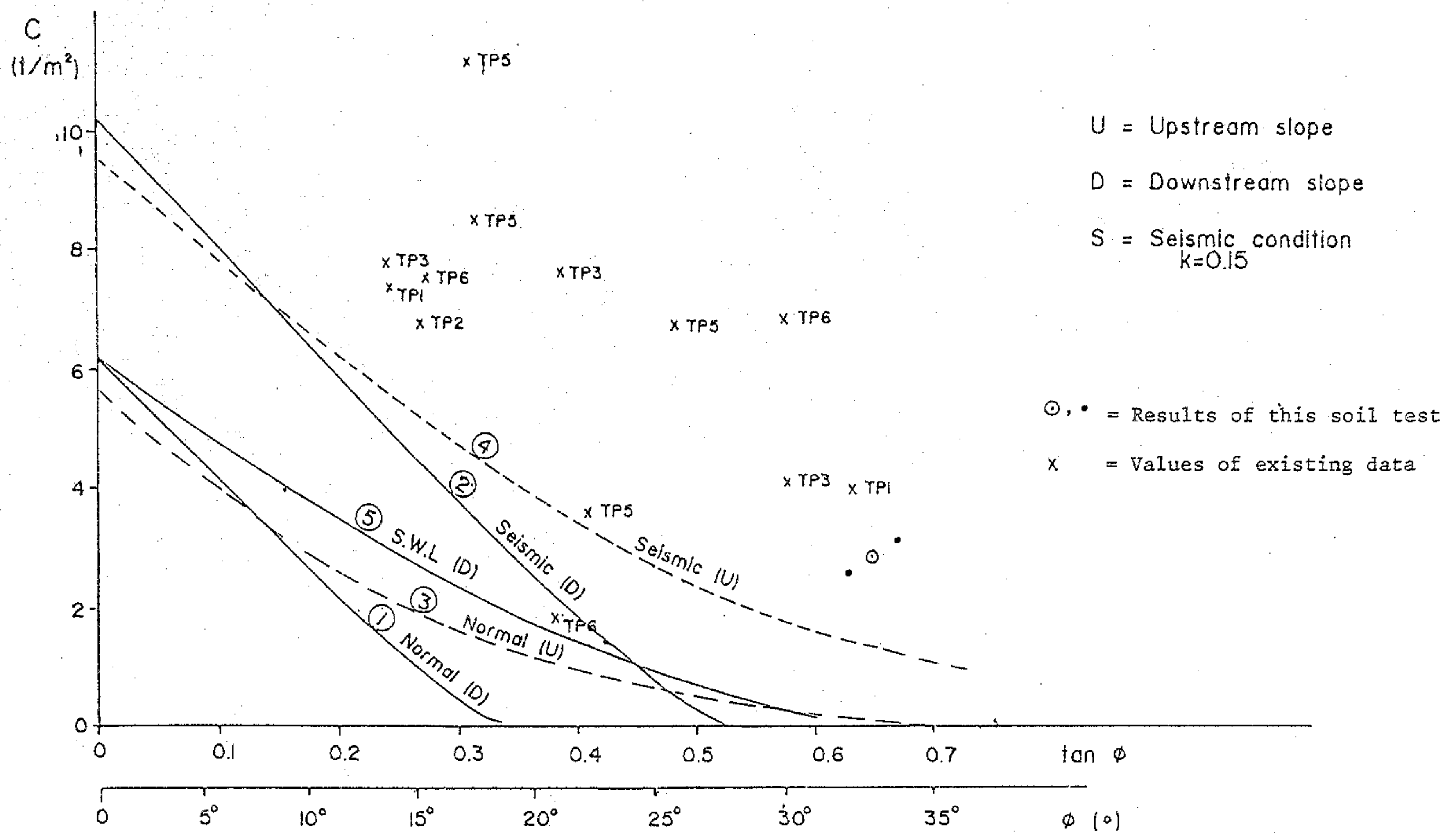
FIG. VII-4 CALIRAYA DAM STABILITY
(Section STA. 0 + 384)



	S.F.	Fr(t/m)	Fs(t/m)	R(m)	
①	1.592	1,461.3	917.9	141.9	(Q=0)
②	1.561	68.9	44.1	175.5	(k=0)
③	0.964	1,189.1	1,233.3	140.0	(k=0.15)

FIG. VII-5 : Range of C & ϕ Valves for 1.0 of Sliding Stability

(Section STA 0+384)



VIII. REHABILITATION PLAN AND REHABILITATION COSTS

1. Downstream Face of the Dam

2. Upstream Face of the Dam

3. Service Spillway

4. Sliding at East Dyke

5. Costs Schedule and Outlined Time Schedule of the Caliraya Dam Repair Works

6. Investigation Costs, Engineering Cost and NAPOCOR Administration Cost for Repair Works

VIII. Rehabilitation Plan and Rehabilitation Costs

1. Downstream Face of the Dam

As reported above, widespread damage caused by rainfall erosion stripping grass and top soil was found on the downstream face of the dam. Without remedial measures it is probable that the damage will rapidly deteriorate. It is therefore recommended that a programme of remedial work is established without delay.

The remedial work proposed would include installation of drainage, and repairing the damaged areas. The damage is spread over a wide area. A systematic remedial scheme is therefore proposed rather than a continuation of the present piecemeal repair work.

For a permanent rehabilitation plan, it is proposed to cover the downstream slope of the dam with the rock material, having first placed filter materials, as follows.

1) Slightly stripping by bulldozer.

2) Installation of perforated hume concrete drainage pipes (ϕ300 - ϕ600) at the existing middle berm (EL.280.00 & EL.277.50).

3) Placing sand and gravel aggregates (to a depth of 1 meter)

4) Cover the aggregate materials with rock materials (to a depth of 1.5 m) topped with rock riprap (300 mm - 600 mm).

For these repairs, about 38,600 m^3 of rock materials will be required. Although it is difficult to collect those rock materials near the Caliraya dam because they are not found existing on the surface of the earth, they could be collected from the basaltic zone existing along the Laguna lake shore road which is located about 3 km northeast of the Kalayaan pumped storage power station.

If this plan of permanent repairs involves difficulties in collecting rock materials and consumes time for its implementation, emergency repairs can also be done, without leaving the downstream face of the dam as it is, by using soil materials which are made available around the dam. In this case, however, similar damages to the present ones are feared to reoccur and it would be required to repeat constant monitoring and repair works to prevent such reoccurrence.

The costs of such emergency repairs Plan A and permanent repairs Plan B are approximately estimated as follows:

Plan A	(Emergency repairs)	US$254,000.-
Plan B	(Permanent repairs)	US$1,560,000.-

The scheme of the permanent repairs plan is shown on Fig. VIII-1.

2. Upstream Face of the Dam

As reported above, the cracks in the concrete slab of the upstream face are considered to be mainly caused by differential settlement of the embankment fill materials. The embankment having been completed some 40 years ago, it is not considered likely that further settlement will occur or that further cracking will develop. However, the underlying materials will continue to be gradually washed out through the existing cracks and joints, and voids beneath the slab will thus increase.

Without repair, these voids will eventually cause instability of the entire upstream face of the dam, so remedial work should be undertaken at the earliest opportunity.

This remedial work should be directed towards stilling water disturbance due to waves in the cracks and voids of the concrete slab.

The remedial work should be done at a time when the reservoir water level is lowered for instance, at the time of repair of cylinder gate for the service spillway. The cracked areas should be drilled out and the underlying voided areas filled with sand and the larger openings should be backfilled with mortar and broken joints and cracks covered with shotcrete and wire mesh. (the pipes through which the sand was poured shall be remained as a role of drainage).

As for the rehabilitation of submerged areas of the upstream slope of the dam, gravels and sands should be dumped in order to cover the upstream slope of the dam. (See Fig.VIII-2/1).

The estimated rehabilitation costs are US$377,000.-.

The reservoir water level at which the Kalayaan Pumped Storage Power Plant can intake the water is higher than EL.286.0. At a lower reservoir water level, its waterway sucks air at the intake inlet and the plant can not be operated. In practice, it is very difficult to lower the reservoir water level for repair of the dam upstream surface under the present condition that the Kalayaan Plant is operated almost everyday.

If the dam upstream face concrete slabs are repaired without lowering of the reservoir water level, sand and gravel shall be thrown on the concrete slab up to EL.285.0 under the water where many cracks take place. The sand and gravel layer under the water near the reservoir water surface may be washed out by water disturbance caused by waves, therefore it is required to be protected with gabions. The cost of repairs in this method is estimated at US$527,000. The conception of this repair is shown in Fig.VIII-2/2.

3. Service Spillway

3.1. Repair of Existing Service Spillway

a) Gate

The sealing parts of the cylinder gate must be replaced. Since corrosion of the gate leaf is widespread, it is necessary to renew the entire gate. The estimated renewal cost is US$90,000.-.

b) Vertical shaft

Damage to the lining concrete is serious and a considerable amount of leakage is seen. Grouting with mortar or cement milk in the rock surrounding the lining concrete is essential. However, the large cavity in the lining concrete from which a large quantity of water is escaping, should be temporarily blocked, and then grouted. The proposed temporary leakage sealing works are shown in Fig.VIII-3.

c) Steel liner

As reported above, many rivets have become detached from the curved steel liner connecting to the bottom of the vertical shaft and water is gushing from the vacant rivet holes. Grouting in the rocks surrounding the backfill concrete should be carried out, the vacant holes having first been plugged with short tapered steel rods.

d) Horizontal tunnel

As reported above, the leakage condition of the horizontal tunnel is not clear at present. For rehabilitation, chiefly, grouting should be carried out in the surrounding ground.

Grouting shall be carried out from inside of the tunnel at section of 3 m intervals. In the vertical shaft, 6 holes of grouting (to a depth of 4 m) at a section should be carried out, and in the horizontal tunnel, 3 holes of grouting (to a depth of 6 m) at a section should be carried over a length of 100 m from upstream side to near the Dam core area.

The estimated rehabilitation cost on the grouting in the spillway is US$187,500.-.

3.2. Construction of New Spillway

The discharge capacity of the existing service spillway is approximately 250 m^3/sec. with the reservoir water level at H.W.L. On the other hand, the peak flood inflow to the Caliraya reservoir will be about 2,173 m^3/sec. in the event of a 200 years return period flood. The design spillway discharge of the dam is estimated at 2,608 m^3/sec. which is 120% of the 200 years return flood. If no discharge for generation is released to Kalayaan and

Caliraya Power Stations, in other words, the discharge be released from the existing spillway only, the maximum water level of the reservoir will reach EL.290.5 m. In this case, a freeboard to the dam crest will remain only 150 cm. Taking account of wave action caused by strong wind (1.8m high wave under a wind of 30 m/sec.), there will be a clear possibility of over-topping, which may result in the collapse of the dam.

The existing service spillway is classified as tunnel spillway, and the discharge capacity of such tunnel spillway is limited by its diameter. It is, of course, practically impossible to increase the discharge capacity. Moreover, its defective condition causes many problems in operation. For these reasons, the existing service spillway should be repaired, and the construction of a new spillway is inevitable.

Figs.VIII-4 and VIII-5 show an example of the new spillway construction which will be jointly used with the existing spillway after repaired. The estimated cost of the new spillway construction is US$5,200,000.-.

4. <u>Sliding at East Dyke</u>

Judging from the fact that the down end of slide material is saturated with water, there will be water seepage at the bottom of the slide area. After the loosen material is removed and the ground surface is trimmed, the slide area should be backfilled with pervious materials such as rock fragment, gravel and sand. In this case, impervious soil material is not suitable as backfill material.

Fig. V-9 shows the suggested rehabilitation plan and the estimated cost is US$96,400.-.

5. Costs Schedule and Outlined Time Schedule of the Caliraya Dam Repair Works

Shown on Tables VIII-1 and VIII-2 are the cost schedule of the Caliraya Dam, Repair Works, and Table VIII-3 shows the priority of work classified by priorities I, II, and III and an outlined time schedule of each element of the Repair Works.

6. Investigation Costs, Engineering Cost and NAPOCOR Administration Cost for Repair Works

Prior to the implementation of the repair work, core drilling work should be carried out to investigate raw material distribution at the quarry site and geology of foundation rock under the new spillway route for the quantity of 40m x 5 holes at the quarry site and 30 m x 10 holes along the spillway route. And topographic survey of the quarry site and longitudinal and cross-sectional survey of the new spillway route should also be carried out. In addition, the engineering cost for employment of experienced consultants and NAPOCOR administration cost will be required for detailed design, preparation of tender documents, tenders evaluation and construction supervision of the repair work. The estimated cost of these works is shown in Table VIII-1.

Fig. VIII-1 Remedial Works for Downstream Slope of Dam

S : 1/500

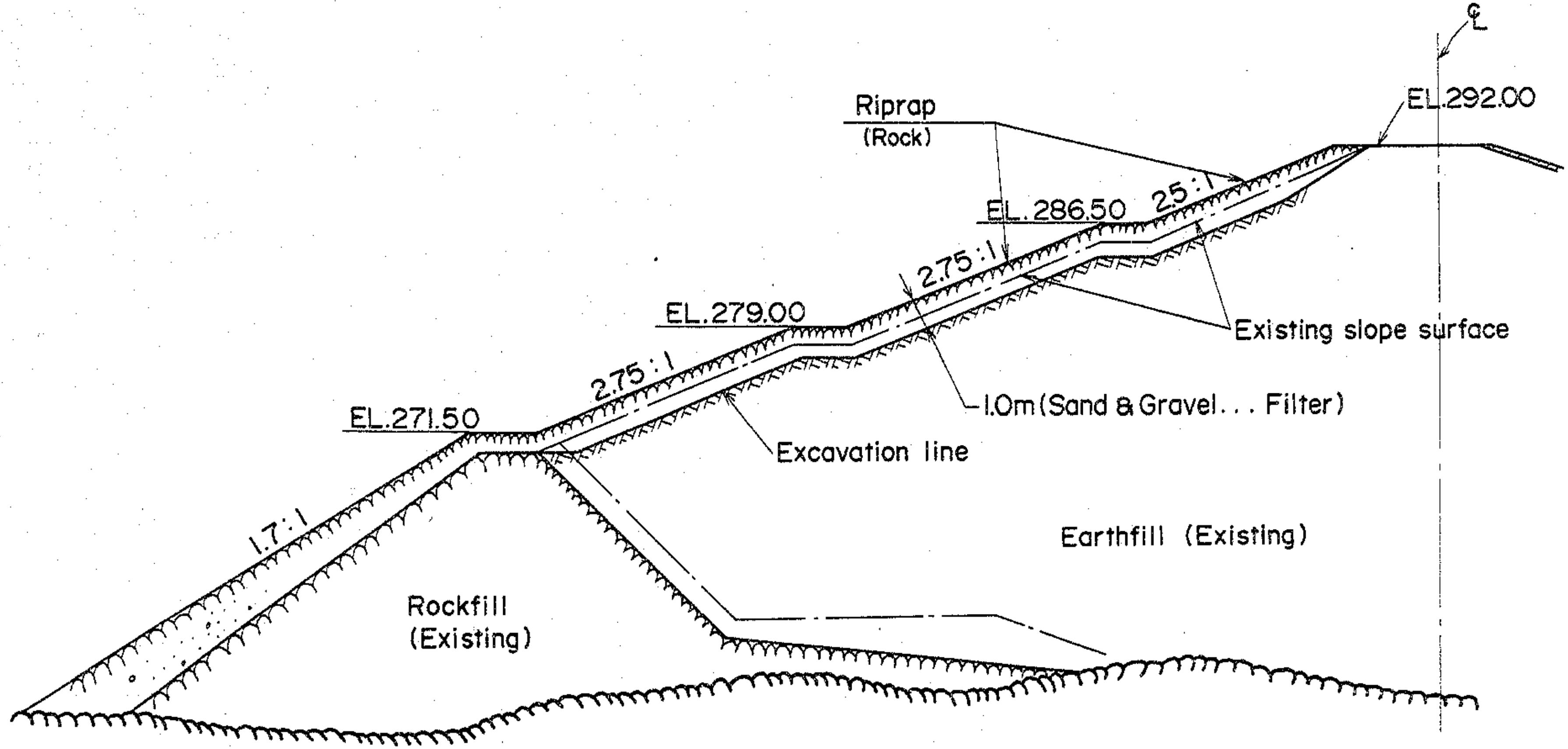


Fig. VIII-2/1 Remedial Works for Downstream Slope of Dam

S : 1 / 500

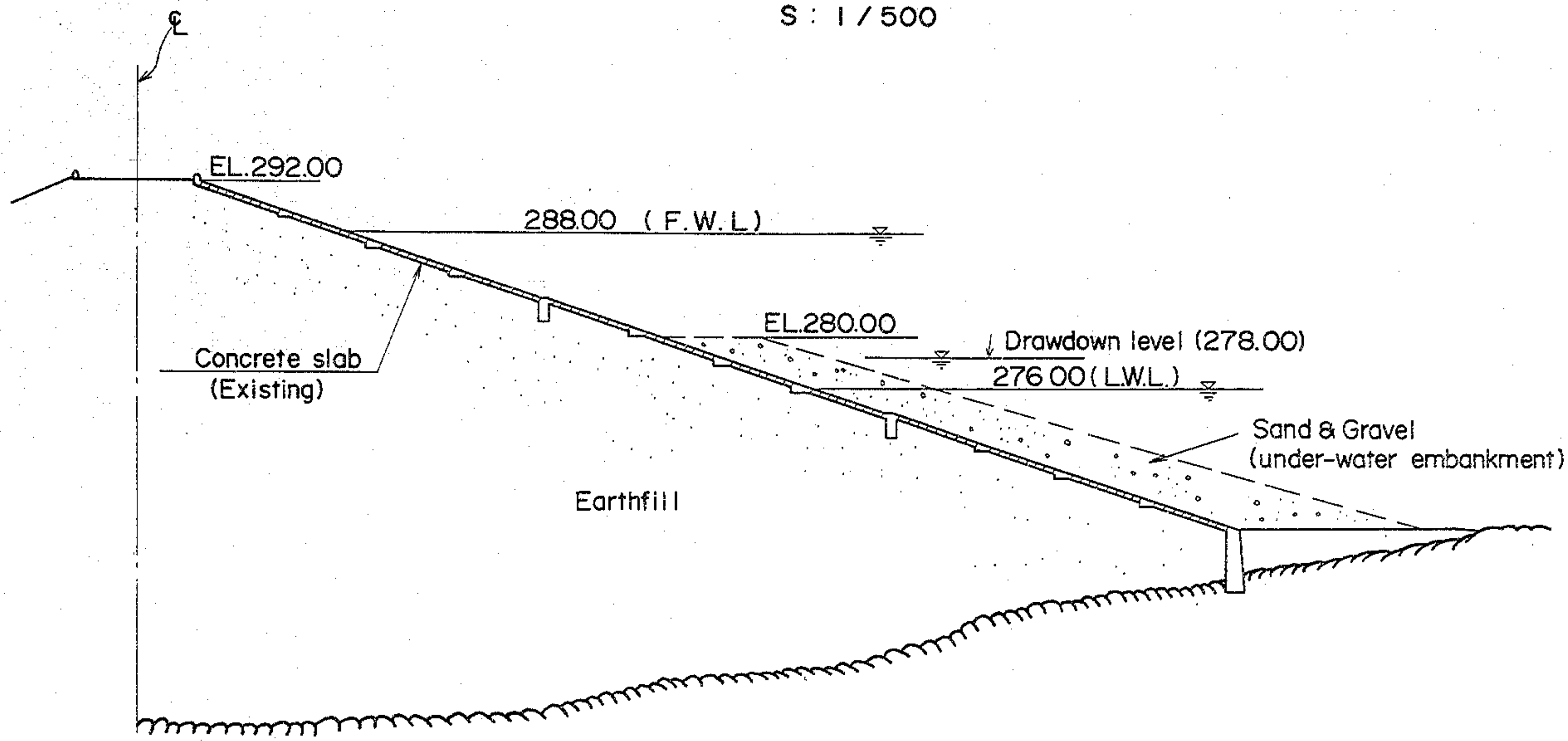


Fig. VIII-2/2 Remedial Works for Upstream Slope of Dam

(Without lowering of reservoir water level)

S : 1/500

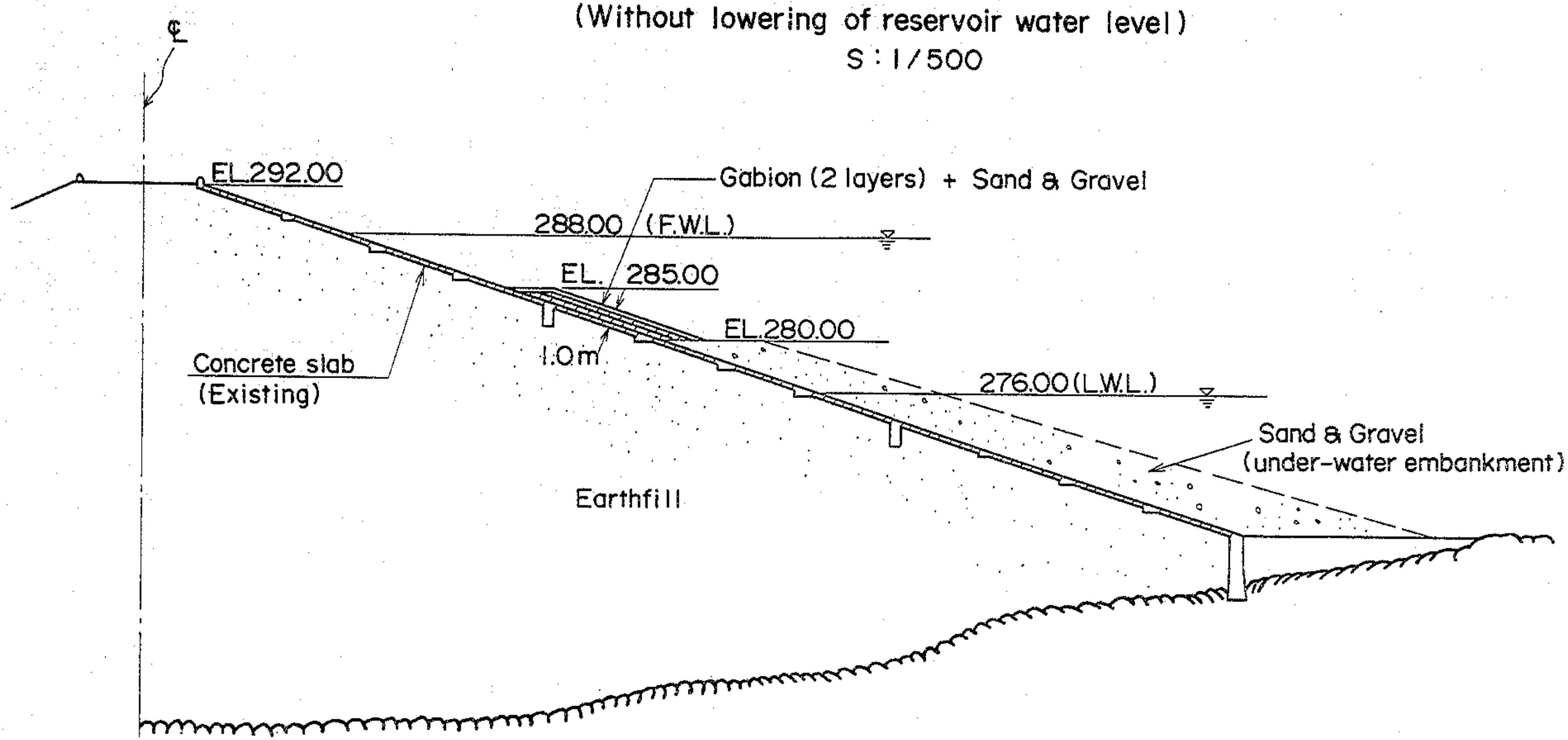
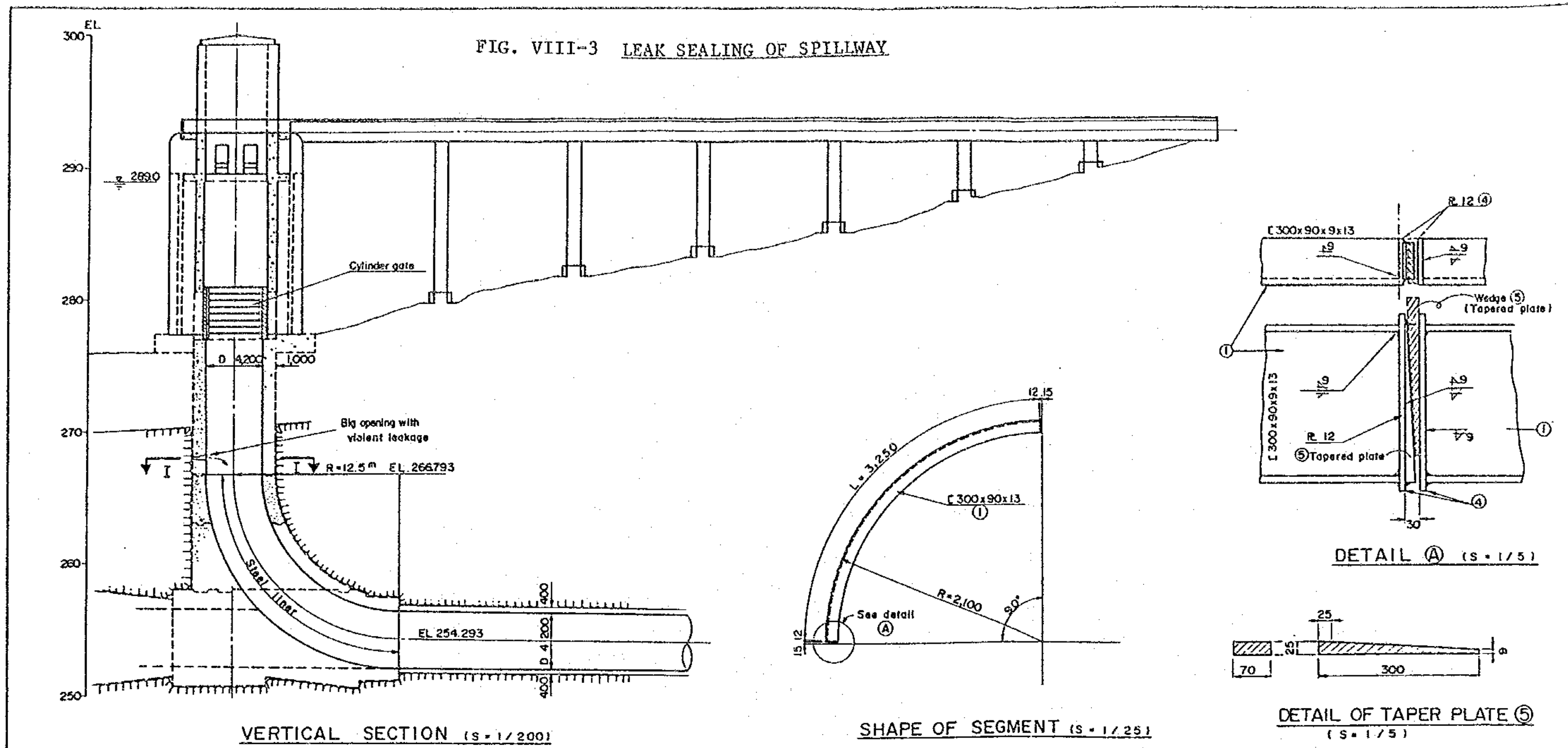
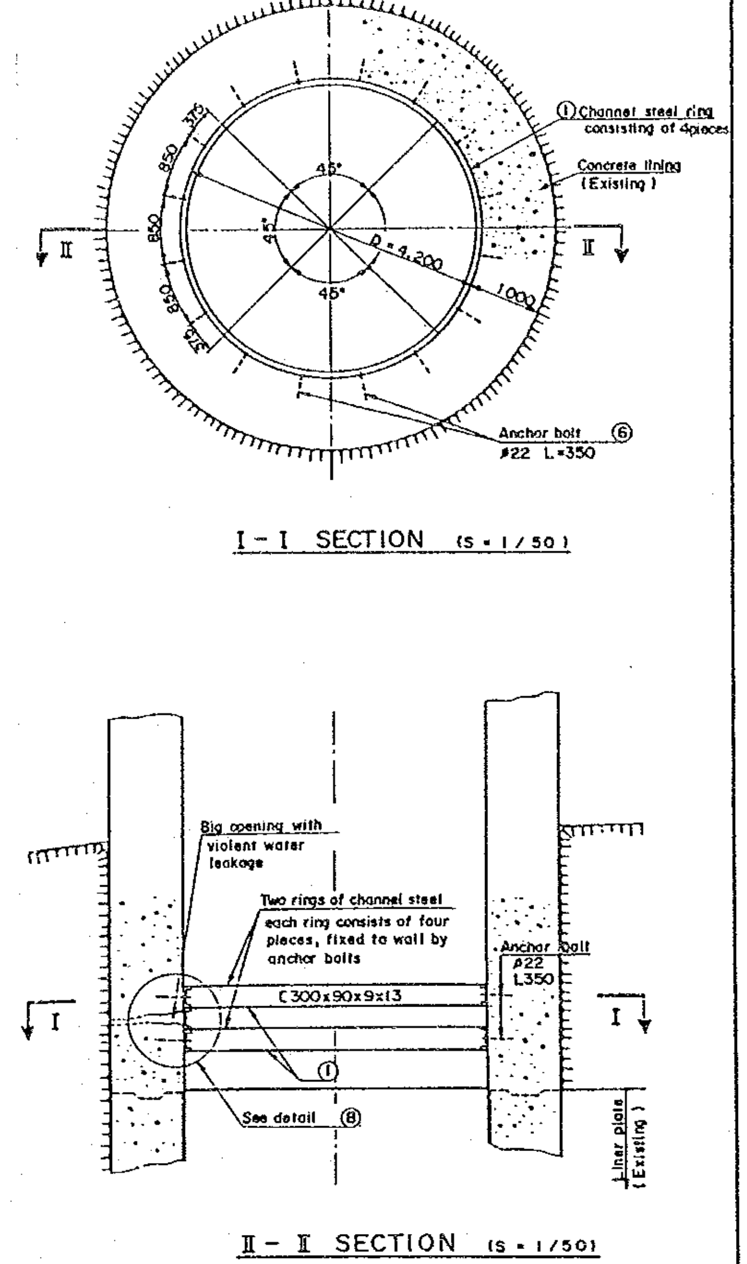
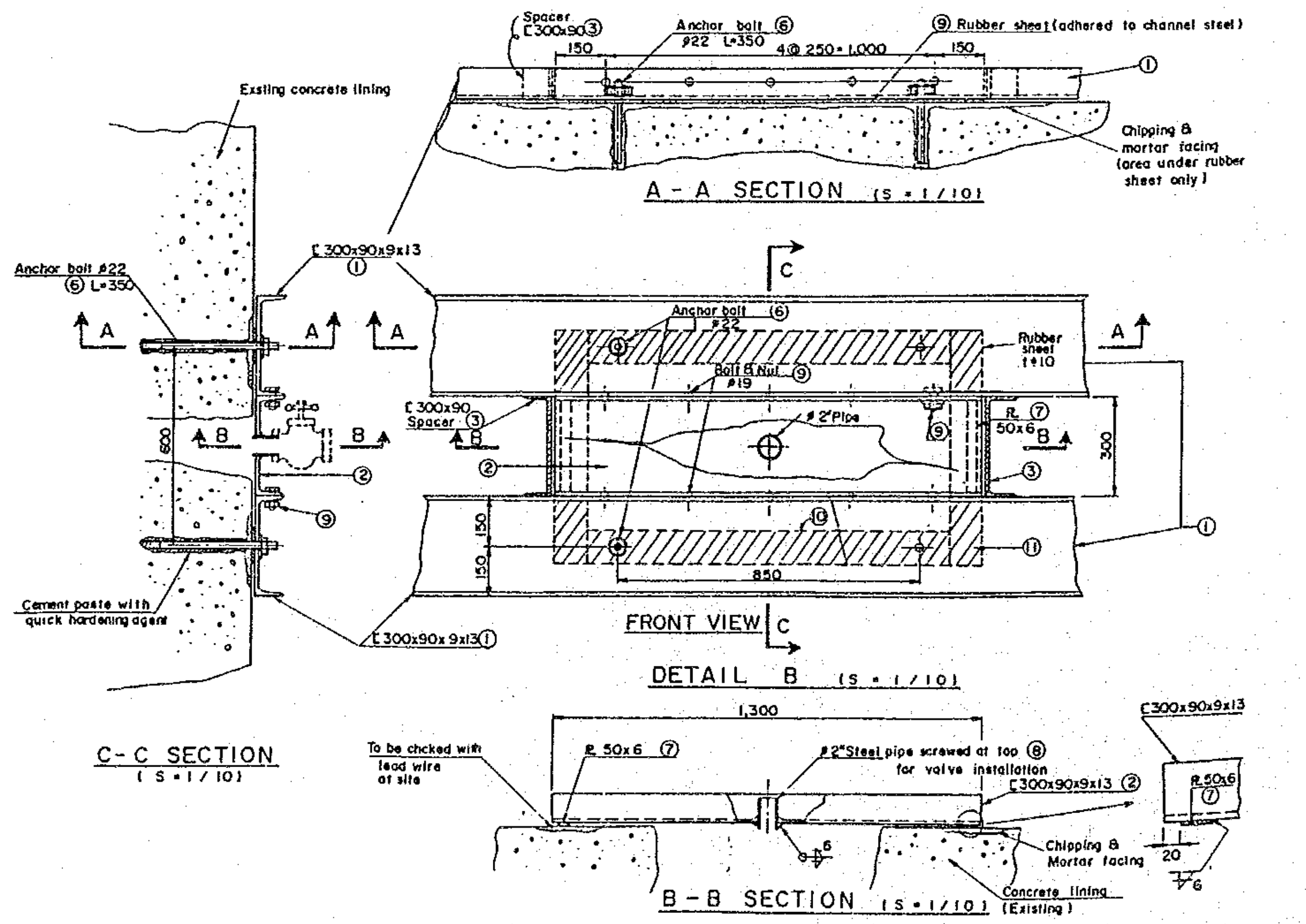


FIG. VIII-3 LEAK SEALING OF SPILLWAY







NOTE

I) Should no bending machine be available, Segments for steel rings can be made with steel plate properly shaped and welded.

II) Prior to installation of steel ring at its location, concrete surfaces under the ring shall be cleaned and smoothened with cement mortar.

III) Prior to installation of steel ring at its location, all cracks of significant size shall be plugged with cement paste after chiseling along the cracks.

Iv) Prior to installatin of steel ring, big cavities other than that shown in this drawing shall be back filled with cement mortar with quick hardening agent.
Those cavities which bear leaking water shall be provided with drainage of steel pipe(s) screwed at its one end for connection of grouting hose.
Surfaces of cavities and adjacent concrete shall be cleaned by wire. brush

v) After completion of steel ring installation and water leakage being blocked by the method indicated in this drawing. The opening shall be grouted with cement mortar.

vI) Leak holes at the existing steel liner plates shall be plugged with tapered iron rods by tapping. Diameter of iron rods will be chosen in accordance with the measurement of diameters of actual holes.

vII) After completion of procedures shown in this drawing and note items II) to vI), the surrounding rocks along the vertical and horizontal tunnel. Shall be grouted.(2nd step)

CALIRAYA REHABILITATION PROJECT	
SPILLWAY LEAK SEALING (Ist step) PLANS, SECTIONS and DETAILS	
JAN. 1986	CHEP-R-J-I
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

Fig. VIII-4

Caliraya Dam - Plan of New Spillway

S = 1/1,000

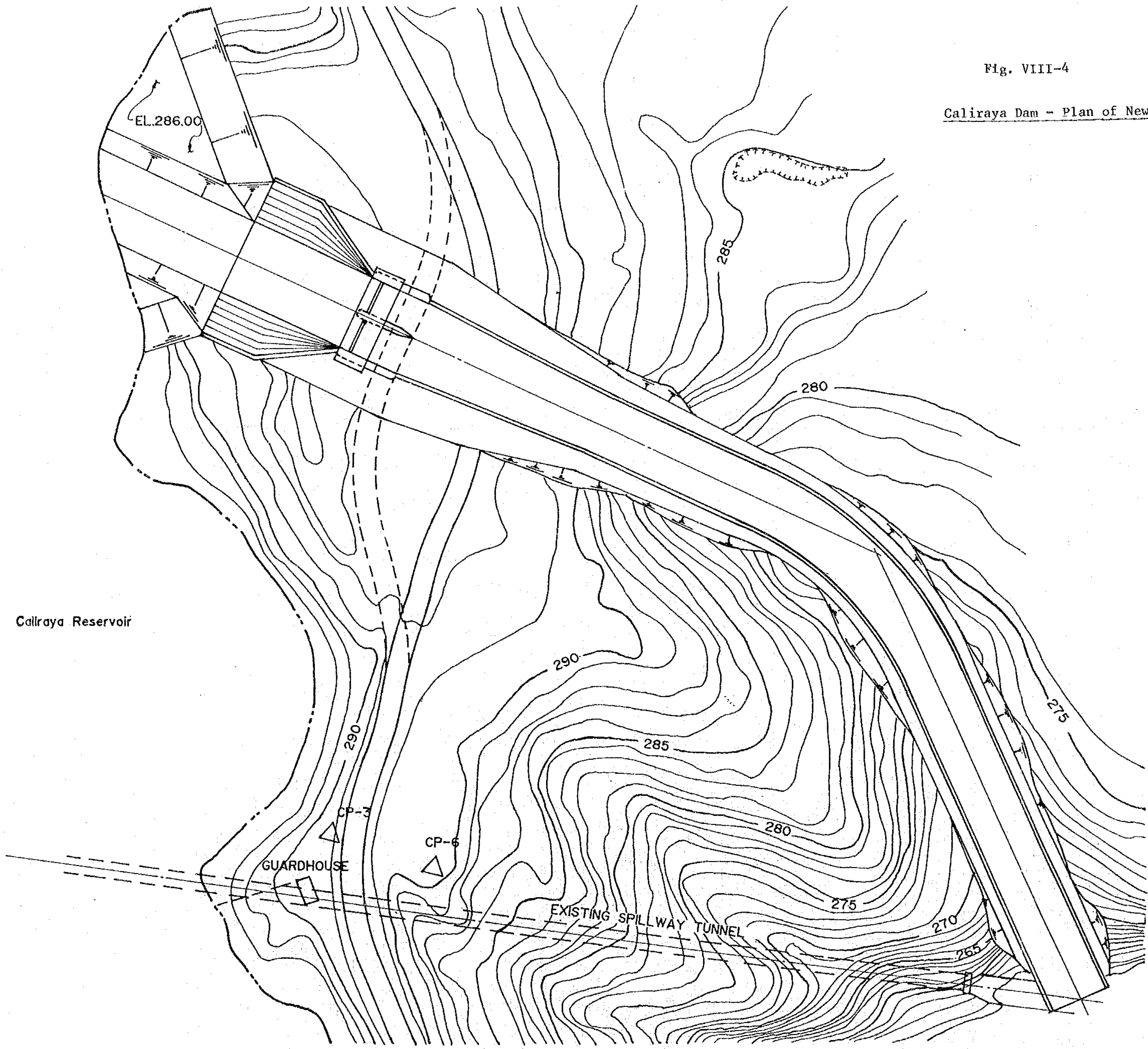


Fig. VIII-5 Cross Section and Profile of Spillway

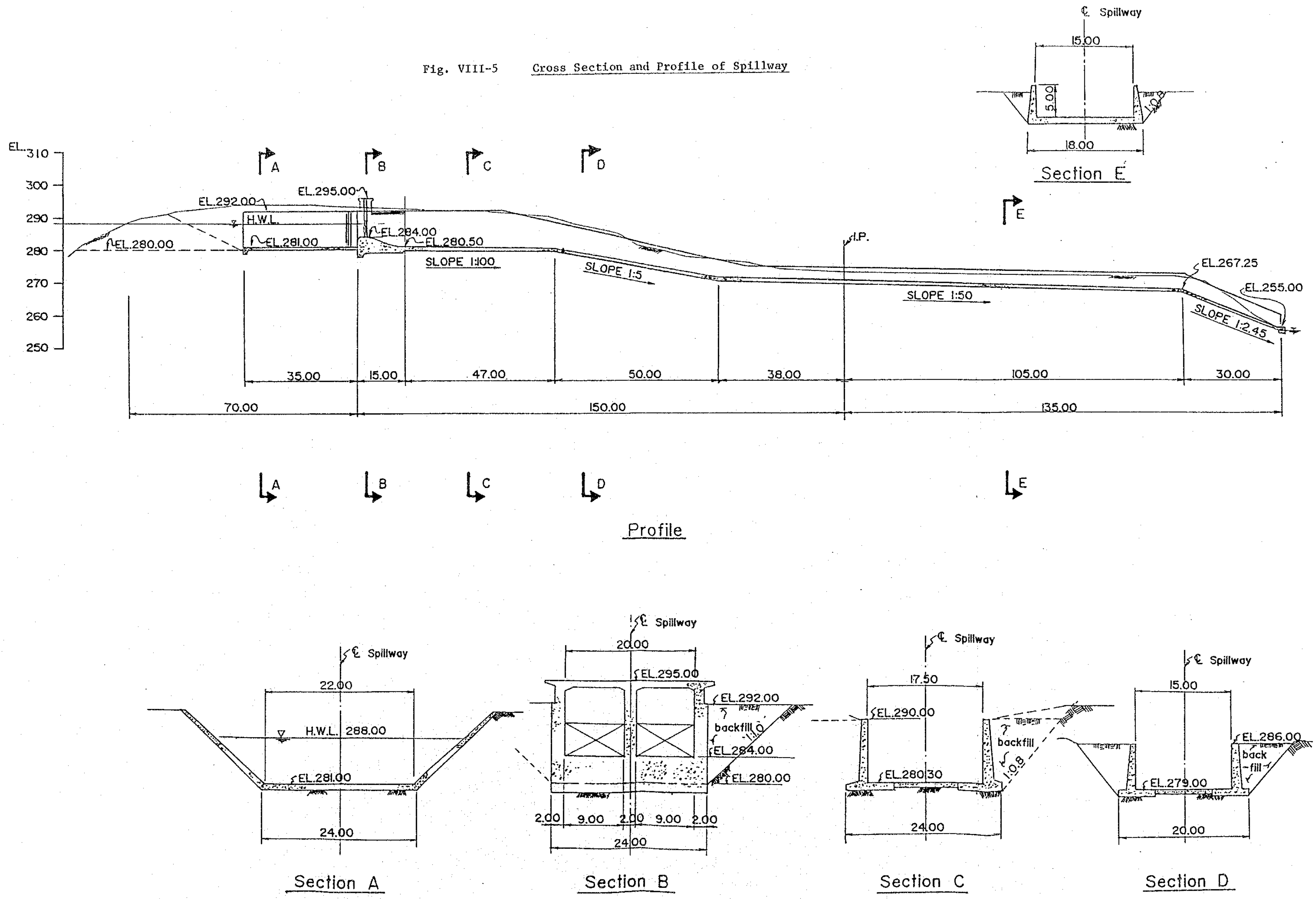


TABLE VIII-1 Cost Schedule of the Caliraya Dam Repair Works

Item	Foreign Currency Portion (US$)	Local Currency Portion (US$)	Total (US$)	Remarks
1. Construction Cost				
Repair of Existing Spillway	20,000	167,500	187,500	
Replacement of Cylinder Gate	80,000	10,000	90,000	
New Spillway Construction	2,538,000	2,662,000	5,200,000	
Repair of Downstream Slope of Dam				Plan A: Repair with earth material
Plan A	(50,000)	(204,000)	(254,000)	Plan B: Repair with rock material
Plan B	702,000	858,000	1,560,000	
Repair of Upstream Slope of Dam	56,000	321,000	377,000	
Repair of Landslides at East Dyke	0	96,400	96,400	
Sub-total	(2,744,000) 3,396,000	(3,460,900) 4,114,900	(6,204,900) 7,510,900	
2. Field Investigation Cost				
Core Drilling	0	60,000	60,000	
Topographic Survey	0	30,000	30,000	
Sub-total	0	90,000	90,000	
3. Land Acquisition Cost	0	200,000	200,000	Quarry site (about 100,000 m^2)
4. Engineering Cost	826,000	0	826,000	
5. NAPOCOR Administration	0	165,000	165,000	
6. Contingency	(274,400) 339,600	346,090 411,490	(620,490) 751,090	
7. Total	(3,844,400) 4,561,600	(4,261,990) 4,981,390	(8,106,390) 9,542,990	

Figures inside ( ) show the costs in case only the Plan A is carried out.

Table VIII - 2 Details of Construction Cost for Caliraya Rehabilitation Project

(As of 1986)

(Unit: US Dollar)

1. Summary (US$ equivalent)

A. Repair of Existing Spillway

20,000.- (F/C) + 167,500.- (L/C) = 187,500.-

B. Replacement of Cylinder Gate

80,000.- (F/C) + 10,000.- (L/C) = 90,000.-

C. New Spillway Construction

2,538,000.- (F/C) + 2,662,000.- (L/C) = 5,200,000.-

D. Repair of Downstream Slope of Dam

D1. (A) : repair with earth material

50,000.- (F/C) + 204,000.- (L/C) = 254,000.-

D2. (B) : repair with rock material

702,000.- (F/C) + 858,000.- (L/C) = 1,560,000.-

E. Repair of Upstream Slope of Dam

56,000.- (F/C) + 321,000.- (L/C) = 377,000.-

F. Repair of Landslides at East Dyke

0.- (F/C) + 96,400.- (L/C) = 96,400.-

2. Details of Costs

2.1. Repair of Existing Spillway

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation Works	1	LS	48,620	48,620
2.)	Drilling	768	m	100	76,800
3.)	Set-up	142	hole	80	11,360
4.)	Grouting	768	m	40	30,720
5.)	Sub total				167,500
6.)	Protection work for grouting	1	LS	20,000	20,000
7.)	Total (3+4)				187,500

2.2. Replacement of Cylinder Gate

	Description	Quantity	Unit	Unit Price	Amount
1.)	Cylinder Gate	15	ton	6,000	90,000
2.)	Total				90,000

2.3. New Spillway Construction

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation works (Quarrying and Crushing & Batching Plants)	1	LS	1,260,060	1,260,060
2.)	Excavation (Earth and rock)	83,000	m^3	9	747,000
3.)	Backfill (Earth)	15,700	m^3	5	78,500
4.)	Concrete	16,150	m^3	140	2,261,000
5.)	Reinforcement	648	ton	780	505,440
6.)	Sub total				4,852,000
7.)	Gate	58	ton	6,000	348,000
8.)	Total				5,200,000

2.4. Repair of Downstream Slope of Dam

2.4.1. Repair with Earth Material (A)

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation works (Borrow area and haul road)	1	LS	65,460	65,460
2.)	Removal of Existing Surface	21,705	m^2	4/3	28,940
3.)	Drain work	1	LS	51,100	51,100
4.)	Earth Backfill	10,850	m^3	10	108,500
5.)	Total				254,000

2.4.2. Repair with Rock Material (B)

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation works (Quarrying and haul road)	1	LS	401,705	401,705
2.)	Removal of existing surface	21,705	m^2	4	86,820
3.)	Filter Material embankment	21,705	m^3	15	325,575
4.)	Rockfill embankment	38,600	m^3	18	694,800
5.)	Drain pipes (ϕ300 mm, 550 m)	550	m	30	16,500
6.)	Drain pipes (ϕ450 - 600 mm, 775 m)	775	m	40	31,000
7.)	Drainage Junction etc.	18	number	200	3,600
8.)	Total				1,560,000

3. Repair of Upstream Slope of Dam

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation works (Haul road and barge)	1	LS	97,500	97,500
2.)	Shotcrete (8 cm thick)	1,050	m^2	15	15,750
3.)	Miscellaneous (Drain, etc.)	1	LS	5,000	5,000
4.)	Disposal in water	25,875	m^3	10	258,750
6.)	Total				377,000

4. Repair of Landslides at East Dyke

	Description	Quantity	Unit	Unit Price	Amount
1.)	Preparation works (Access and haul roads)	1	LS	24,960	24,960
2.)	Excavation	1,840	m^3	6	11,040
3.)	Filter	560	m^3	15	8,400
4.)	Rockfill	2,800	m^3	18	50,400
5.)	Gabion	40	m^3	40	1,600
6.)	Total				96,400

TABLE VIII-3 Construction Schedule of Rehabilitation Works of the Caliraya Dam

Priority	Item No.	Work Item	1986 (9 10 11 12)	1987 (1 2 3 4 5 6 7 8 9 10 11 12)	1988 (1 2 3 4 5 6 7 8 9 10 11 12)	1989 (1 2 3 4 5 6 7 8 9 10 11 12)
I	A	Repair of Existing Spillway				Reservoir water level to be lowered
		1. Additional investigation				
		2. Detailed Design				
		3. Tendering/Contracting				
		4. Construction				
	C	New Spillway Construction				
		1. Additional investigation				
		2. Detailed Design				
		3. Tendering/Contracting			Order to proceed for gates	Gate installation
		4. Construction				
	F	Repair of Landslides at East Dyke				Removal of coffer dam
		1. Additional investigation				
		2. Detailed Design				
		3. Tendering/Contracting				
		4. Construction				
II	D	Repair of Downstream Slope of Dam				
		1. Additional investigation				
		2. Detailed Design				
		3. Tendering/Contracting				
		4. Construction				
III	B	Replacement of Cylinder Gate				
		1. Additional investigation				
		2. Detailed Design			Order to proceed for gate	Gate installation
		3. Tendering/Contracting				
		4. Construction				
	E	Repair of Upstream Slope of Dam				
		1. Additional investigation				
		2. Detailed Design				Sand and gravel to be thrown down into water
		3. Tendering/Contracting				
		4. Construction				Above water works

IX. ECONOMIC EVALUATION OF REHABILITATION WORK

IX. Economic Evaluation of Rehabilitation Work

In case where the Caliraya Dam continues to be left unrepaired, erosion will be advancing at the downstream face of the dam, and 10 years hence, the safety factor against land sliding will decrease to less than 1.2 at the time of earthquake of K=0.15, which means that there could be no alternatives but reconstruction of the dam.

Total cost and lead time for reconstruction of the dam are estimated to be US$26,000,000.- and 3 years, respectively. During this period for reconstruction, both Kalayaan and Caliraya power stations can not be placed into service. On the other hand, total cost and lead time for rehabilitation as recommended herein are estimated to be US$9,542,990.- and some 3 years, respectively. Besides, there is no need to empty the dam reservoir during the period for rehabilitation work. Only thing to do is to lower the water level in the reservoir for about 3 months. Though this will keep the Kalayaan Pumped Storage Power Station from being in service for about 3 months, Caliraya Power Station may continue to be put in service even in this period. (Refer to Table VIII-3).

1. Method of Economic Evaluation

The economic justification of the proposed rehabilitation plan can be made by comparing with the reconstruction plan in which no measures will be given until the dam will become useless. The comparison of economics of both plans was made on the sum of construction cost and cost evaluation of losses due to suspension of power production during the work, which made it possible to obtain I.R.R. of the sum of cost and generation losses for the rehabilitation plan (COST) against the sum of cost and generation losses for the reconstruction plan (BENEFIT). Generation losses were evaluated on both aspects of KW (capacity) and KWH (energy generation), assuming that they may be made up by construction and operation of a typical coal-fired power station.

A question may arise as to whether such generation losses may not be made up by the existing reserve margin of the system.

The reserve margin of a power system is, in general, to cover deficiencies of supply capability arisen out of forced outages of power sources which are unpredictable in the planning stage of power development program, or of a decrease in output of hydro power stations due to extremely low level of water availability, or to meet unexpected rise in power requirements.

If the reserve margin should make up for any generation losses, power development program must be worked out so as to include such magnitude of losses over such an extended period of time as caused by suspension of power sources like Caliraya and Kalayaan power stations, in addition to the normally required reserve margin to make up for the unpredictable deficiencies in supply capability.

If it should be the case, generation losses way in no may be made up by the existing reserve margin, and should be covered by an increase in supply capability with addition of some type of power sources. This is the reason for making the assumption to construct a typical coal-fired power station in determining the economics of the proposed rehabilitation plan as mentioned above.

Conditions made for the economic analysis are as cited below:

1.1. Cost for Reconstruction Plan

(1) Construction Cost

Direct construction cost	:	US$21,000,000.-
Site investigation	:	US$ 300,000.-

Land acquisition	:	US$ 200,000.-
Engineering	:	US$ 2,100,000.-
NAPOCOR administration	:	US$ 300,000.-
Contingency	:	US$ 2,100,000.-
Total	:	US$26,000,000.-
		(As of July, 1986)

Total cost of US$ 26,000,000 was supposed to be disbursed in equal amount each year over three years from 1997 to 1999.

(2) Cost Evaluation of Generation Losses

During the period for reconstruction of the dam from 1997 to 1999, both Caliraya and Kalayaan power stations cannot be placed into service. The generation losses due to non-operation of these power stations should be evaluated on both aspects of KW and KWH (hereinafter referred to as KW losses and KWH losses, respectively).

KW losses were estimated on the assumption that they may be made up by construction of a typical coal-fired power station with a capacity of 400 MW, and KWH losses were estimated on the assumption that the amount of electricity to be generated by using water inflow to the Caliraya reservoir (175 GWH a year) would be replaced by generation of the coal-fired power station. Values used for the estimation are as follows:

Construction cost of a typical coal-fired power station (400 MW)	:	US$1,030.-/kW
Fuel:		
Oil (9,700 Kcal/ℓ)	:	US$15.-/barrel
Coal (6,500 Kcal/kg)	:	US$30.-/ton
Thermal efficiency	:	38%

1.2. Cost for Rehabilitation Plan

(1) Construction Cost : US$9,542,990.-

The breakdown of the cost is as shown in Table VIII-1. The cost was supposed to be disbursed in equal amount each year over three years from 1987 to 1989.

(2) Cost Evaluation of Generation Losses

As shown in Table VIII-3, Construction Schedule of Rehabilitation Work, it becomes necessary to lower the water level in the reservoir for three months in 1989. During this period, Kalayaan Pumped Storage Power Station cannot be put into operation, but Caliraya Power Station can continue to be in service. With no ineffective inflow, no KWH losses were supposed to be produced at Caliraya Power Station. But, KW losses to be produced at the 300-MW Kalayaan Pumped Storage Power Station over three months were evaluated.

Described below are the results of economic evaluation of the reconstruction plan and the rehabilitation plan on the basis of the aforementioned assumptions.

2. Cost Evaluation of Reconstruction Plan

2.1. Present Worth Value of Construction Cost

The present worth value (P_1) of the construction cost (Co), assuming that the construction be started (m) years hence and last (k) years, and the cost be disbursed in equal amount each year over the construction period, may be expressed as:

$$P_1 = \frac{Co}{k} \left[\frac{1}{(1+i)^m} + \frac{1}{(1+i)^{m+1}} + \ldots\ldots + \frac{1}{(1+i)^{m+k-1}} \right] \ldots\ldots (1)$$

2.2. KW and KWH Losses

Mentioned below is the methodology for evaluation of any generation losses which occurred at certain hydro power sources in a specific year:

Evaluation was made on the assumption that generation losses be made up by construction and operation of a typical coal-fired power station. KW and KWH made up by the coal-fired power station should be the net values after subtraction of station losses, maintenance and planned outages and forced outages so as to correspond to the real power requirements.

Factors used in the cost evaluation were as follows:

x_c : Cost per net capacity of a typical coal-fired power station .... US$/KW

y_c : Cost per net energy generation of the typical coal-fired power station at a load factor of ℓ_c..... US$/KWH

ℓ_c : Annual load factor of the typical coal-fired power station

X_L : Net capacity of a hydro power station to be taken out of service (the object of evaluation of generation losses) .... KW

Y_L : Net energy generation of the hydro power station to be taken out of service (the object of evaluation of generation losses) .... KWH

ℓ_L : Annual load factor of the hydro power station to be taken out of service (the object of evaluation of generation losses)

T : Annual period hours

Hence, Y_L may be expresses as:

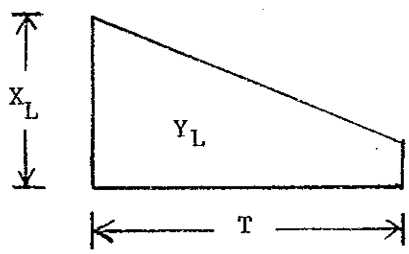
$$Y_L = X_L \cdot T \cdot \ell_L$$

y_o : Cost per net evergy generation of oil-fired power station(s) to be saved by operation of the coal-fired power station .... US$/KWH

V_L : Monetary value of X_L and Y_L .... US$

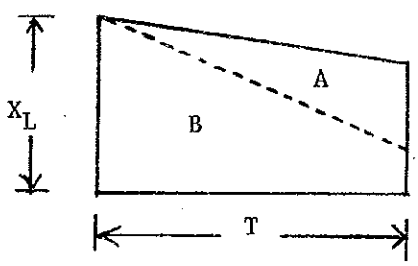
V_L can be obtained in the following manner:

(1) X_L and Y_L, the object of evaluation, may be as shown in the following figure:



$$Y_L = X_L \cdot T \cdot \ell_L$$

(2) If X_L is made up by a typical coal-fired power station, then the figure should be as shown below:



$$(A + B) = X_L \cdot T \cdot \ell_c$$

$$B = Y_L = X_L \cdot T \cdot \ell_L$$

Hence, $A = X_L \cdot T \cdot \ell_c - Y_L$

This illustrates that the coal-fired power station to make up for generation losses may produce the amount of electricity covering portions of A and B, as it should be the most advanced plant with the highest thermal efficiency in the system, and, accordingly, would be placed into service for longer hours with a higher load factor of ℓ_c.

However, Y_L, the object of evaluation of generation losses, being equal to the portion of (B), it was necessary to adjust the cost evaluation of X_L and Y_L in the following manner:

First, X_L and the portion of (A+B) were evaluated on the cost per net KW and net KWH of the coal-fired power station, then the resultant cost of (A+B) was deducted by the amount of the portion (A) multiplied by the cost per net KWH (y_o) of the oil-fired power station(s) to be saved by operation of the coal-fired power station. y_o may be the highest in the fuel cost per net KWH among all thermal power stations in the system.

(3) V_L can be obtained by the following equation.

$$V_L = X_L \cdot x_c + (X_L \cdot T \cdot \ell_C) \cdot y_c - (X_L \cdot T \cdot \ell_C - Y_L) \cdot y_o$$

$$= X_L \cdot x_c - X_L \cdot T \cdot \ell_C \,(y_o - y_c) + Y_L \cdot y_o$$

$$= X_L \,[\, x_c - T \cdot \ell_C (y_o - y_c) \,] + Y_L \cdot y_o$$

where, if $x_c - T \cdot \ell_C (y_o - y_c) \equiv x_e$, and $y_o \equiv y_e$, then,

$$V_L = X_L \cdot x_e + Y_L \cdot y_e$$

Values of x_e and y_e can be obtained in the following manner:

$$x_c = \frac{\text{Construction Cost} \times \text{Annual Levelized Cost Ratio}}{\text{Installed Capacity} \times (1-\text{Station Loss}) \times \left(1-\frac{\text{Maintenance}}{\text{Outage Ratio}}\right) \times \text{Availablity Factor under Operation}}$$

$$= \frac{412 \times 10^6 \text{ US\$} \times 0.1708}{400 \text{ MW} \times 10^3 \times (1-0.073) \times (1-\frac{50}{365}) \times 0.95} = \text{US\$231/KW}$$

$$y_c = \frac{860 \times \text{Coal Price/ton} \times (1 + \text{Other Viriable Cost Ratio})}{\text{Thermal Efficiency} \times \text{Heating Value} \times (1 - \text{Moisture Content}) \times (1-\text{Station Loss})}$$

$$= \frac{860 \times \text{US\$30/ton} \times 10^{-3} \times 1.01}{0.38 \times 6500 \text{ Kcal/kg} \times (1-0.07) \times (1-0.073)} = \text{US\$0.0122/KWH}$$

$$y_o = \frac{860 \times \text{Oil Price/BL} \div 159 \times (1 + \text{Other Variable Cost Ratio})}{\text{Thermal Efficiency} \times \text{Heating Value} \times (1-\text{Station Loss})}$$

$$= \frac{860 \times \text{US\$15/BL} \div 159 \times 1.01}{0.38 \times 9700 \text{ Kcal}/\ell \times (1-0.05)} = \text{US\$0.0234/KWH}$$

If $\ell_c = 0.98$, then

$$x_e = x_c - T \cdot \ell_c (y_o - y_c) = 231 - 8760 \times 0.98 \times (0.0234 - 0.0122)$$
$$= \text{US\$135/KW}$$

$$y_e = \text{US\$0.0234/KWH}$$

Since the annual energy generation by both of Kalayaan Pumped Storage Power Station and Caliraya Hydro Power Station (only those corresponding to water inflow to the Caliraya reservoir) is estimated at 175 GWH, the value V_L can be obtained by the following equation:

$$V_L = 336 \times 10^3 \text{ kW} \times 135 \text{ US\$/kW} + 175 \times 10^6 \text{ kWH} \times 0.0234$$
$$= \text{US\$}45.36 \times 10^6 + \text{US\$}4.10 \times 10^6$$
$$= \text{US\$}49.46 \times 10^6$$

Now that the cost evaluation of annual generation losses during construction work was made available, the present worth value (P_2) of generation losses (V_L) over three-year period of construction can be expressed as:

$$P_2 = V_L \left[\frac{1}{(1+i)^m} + \frac{1}{(1+i)^{m+1}} + \cdots\cdots + \frac{1}{(1+i)^{m+k-1}}\right] \cdots\cdots (2)$$

where m = 11 and k = 3.

2.3. Present Worth Value of Sum of Construction Cost and Cost Evaluation of Generation Losses

The present worth value (P) of the sum of construction cost and cost evaluation of generation losses, or the sum of Eq.(1) and Eq.(2), can be shown as:

$$P = P_1 + P_2 = \left(\frac{26}{3} + 49.46\right) \left[\frac{1}{(1+i)^{11}} + \frac{1}{(1+i)^{12}} + \frac{1}{(1+i)^{13}}\right] \times 10^6$$

$$= \text{US\$}58.13 \left[\frac{1}{(1+i)^{11}} + \frac{1}{(1+i)^{12}} + \frac{1}{(1+i)^{13}}\right] \times 10^6$$

3. Cost Evaluation of Rehabilitation Plan

3.1. Present Worth Value of Construction Cost

The present worth value (P'_1) of construction cost for the rehabilitation plan can be expressed as:

$$P'_1 = \frac{P_o}{3} \left[\frac{1}{(1+i)} + \frac{1}{(1+i)^2} + \frac{1}{(1+i)^3} \right] \quad \cdots\cdots\cdots\cdots\cdots \quad (3)$$

Where P_o = US\$9.543 x 10^6

3.2. KW Losses

The KW losses for the rehabilitation plan are only for Kalayaan Pumped Storage Power Station, which would be taken out of service three years hence for a period of three months. The cost evaluation of KW losses (V'_L) and the present worth value (P'_2) in this case can be expressed as:

$$V'_L = 300 \times 10^3 \text{ kW} \times \text{US\$135/kW} \times \frac{3\text{m}}{12\text{m}} = \text{US\$10.125} \times 10^6$$

$$P'_2 = V'_L \frac{1}{(1+i)^3} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4)$$

3.3. Present Worth Value of Sum of Construction Cost and Cost Evaluation of Generation Losses

The present worth value (P') of the sum of construction cost and cost evaluation of generation losses, or the sum of Eq.(3) and Eq.(4), can be shown as:

$$P' = P'_1 + P'_2$$

$$= \frac{\text{US\$9.543}}{3} \left[\frac{1}{(1+i)} + \frac{1}{(1+i)^2} + \frac{1}{(1+i)^3} \right] \times 10^6 + \text{US\$10.125} \times \frac{1}{(1+i)^3} \times 10^6$$

The I.R.R. of the total cost for rehabilitation plan (P') against the total cost for reconstruction plan (P) amounts to some 26%, and the pertinent conclusion from this study is that the rehabilitation plan proves very high in the effect of capital investment.

X. ESTABLISHMENT OF MAINTENANCE CONTROL SYSTEM

X. Establishment of Maintenance Control System

Good maintenance will reduce the possibility of unforeseen accidents, improve the level of operation and reliability of the power supply and prolong the lifespan of the plants.

It is highly recommended that the equipment and the plants under NAPOCOR's control should be maintained by establishment of a maintenance control system.

Table X-1 shows the actual values for the past five (5) years of Installed Capacity and Gross Energy Generation operated by NAPOCOR. The plant factor obtained from this table is generally low.

The hydro power plants are chiefly reservoir-type large power stations, and as they seem to operate mainly at peak load, the plant factors of oil and coal are remarkably low, although the plant factor of this group is a little low. Several conditions may have contributed to this and of these the decline in generating capability and accidents resulting from inferior maintenance are the likeliest reasons. To meet the increasing demand, considerable effort has been directed to construction of new power plants; maintenance of newly constructed plants has been of secondary importance, and this tendency seems to prevail.

As the growth in power demand has now become static, or even decreasing, this presents a suitable opportunity to review maintenance procedures at existing plants, because they have enough reserve in installed capacity of generation.

Continuous maintenance will serve to increase the plant factor up to 60's%. Raising the ratio of operation of existing plants by 10% would equal the affect of construction of a new 700 MW power plant.

Turning to Caliraya Dam, this belongs to Souther Luzon Regional Center under control of NAPOCOR's Operation Department.

Although an equipment maintenance crew is assigned to the Kalayaan and Caliraya Power Plants, no specialist civil engineer is assigned. One civil engineer is based at Southern Luzon Regional Center to look after the civil installations for each power plant, but, working above he can only act in a liaison role. Depending on the type of problem, the Head Quarters may send assistance, but only after the problem has occurred. Therefore, at present no responsible body exists to carry out maintenance and monitoring of the civil structures. If the present situation does not permit the assignment of a civil engineer to each main plant, it is highly advisable that a permanent civil section and monitoring team comprising several civil engineers be established at the Regional Center to carry out systematic checks of the plants and dams under their control. In this way, if sufficient personnel monitoring equipment and vehicles are provided, early discovery of problems and early preventive and remedial measures should be achieved. It is also advisable to establish at Head Quarters a section responsible for Civil Engineering Maintenance in order to avoid inconsistencies in decision making, and to deal with the problems by integrated consideration and decision.

Fig. X-1 shows the proposed addition of the aforementioned Civil Engineering Monitoring Section and Civil Maintenance Section to the present NAPOCOR's organizational setup. It is considered pertinent to establish similar organizational units at the regional centers other than Southern Luzon Regional Center which have important hydro power sources under their administrative control.

A monitoring team engaged in the daily inspection and patrol should act on a pre-arranged work schedule, which is the point of successful maintenance inspection.

Table X-2 shows a standard frequency in observation for each monitoring item which relates to the dam behavior. The Caliraya Dam, completed a considerable time ago is assumed to have become stable. However, there has been no relevant long-term continuous data was available prior to establishment of monitoring facilities for this study. Therefore, it is recommended that the observation be continued in the same frequency as the first stage shown on Table, at least for a round of seasons (1 year from the beginning), and thereafter to determine the scope of monitoring in the next stage by evaluating the data obtained until then.

Table X-3 shows places and items on which attention should be paid at the time of the inspection and patrol.

Fig. X-I Proposed Organization Chart of NAPOCOR

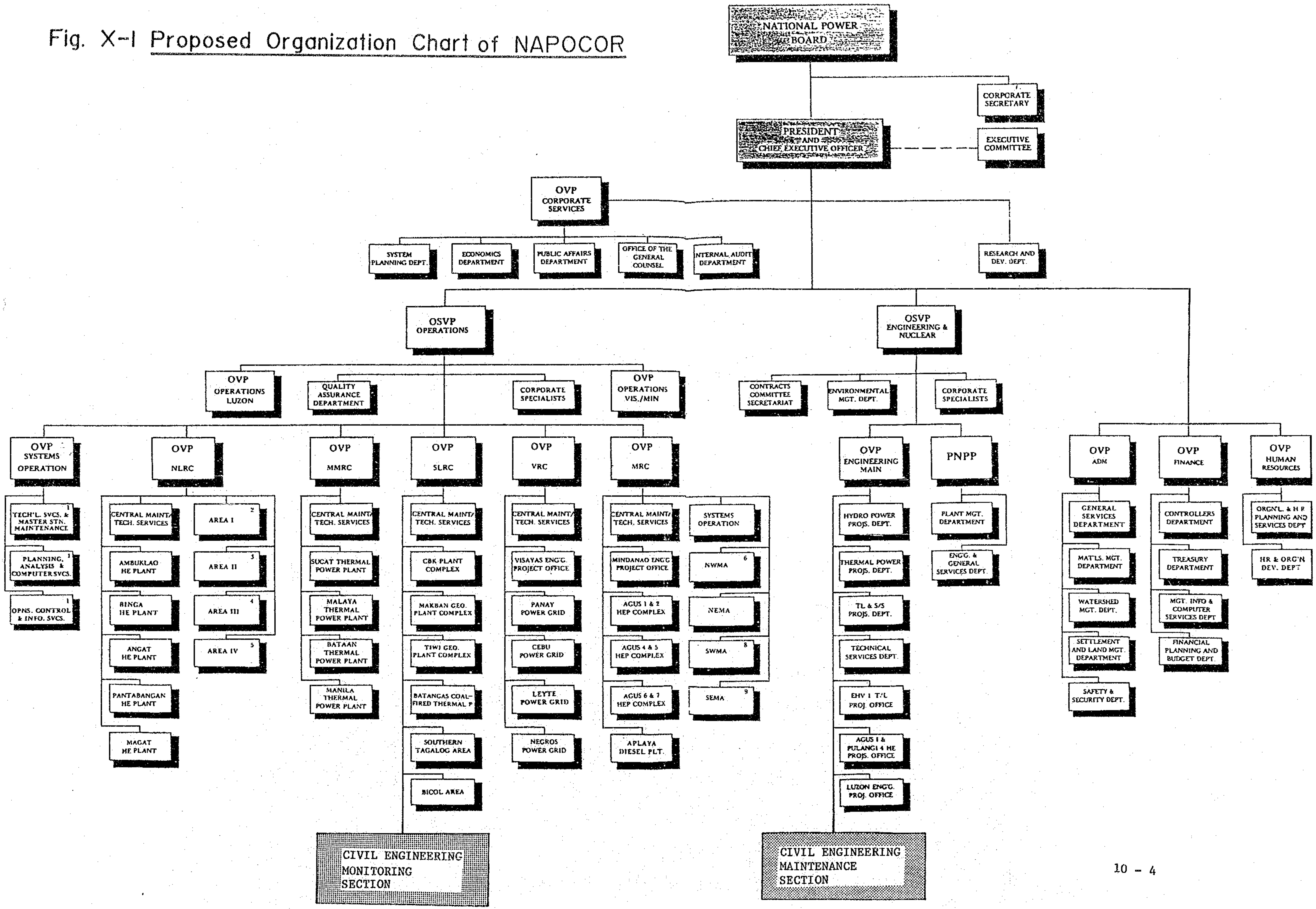


Table X-1 : Installed Capacity and Energy Generation

	Plant Type	Year	1980	1981	1982	1983	1984	1985
Installed Capacity (MW)	TOTAL	PHILIPPINES	3,821	4,016	4,460	5,001	5,196	5,550
		LUZON	3,226	3,281	3,636	3,906	4,101	4,101
	Oil-based	PHILIPPINES	2,435	2,525	2,584	2,603	2,298	2,362
		LUZON	2,230	2,230	2,230	2,230	1,925	1,925
	Hydro	PHILIPPINES	940	940	1,267	1,564	1,654	1,944
		LUZON	556	556	856	1,126	1,216	1,216
	Geo-thermal	PHILIPPINES	446	501	559	784	894	894
		LUZON	440	495	550	550	660	660
	Coal	PHILIPPINES	-	50	50	50	350	350
		LUZON	-	-	-	-	300	300
Gross Energy Generation (GWH)	TOTAL	PHILIPPINES	15,086	15,988	17,413	18,682	18,666	18,757
		LUZON	13,115	13,666	14,398	15,294	14,655	14,449
	Oil-based	PHILIPPINES	9,507	9,494	10,016	11,514	8,536	6,713
		LUZON	9,173	8,894	9,011	10,145	7,787	5,825
	Hydro	PHILIPPINES	3,502	3,724	3,751	2,964	5,167	5,514
		LUZON	1,873	2,033	1,832	1,274	2,519	2,869
	Geo-thermal	PHILIPPINES	2,077	2,770	3,586	4,093	4,540	4,945
		LUZON	2,069	2,739	3,555	3,875	4,125	4,284
	Coal	PHILIPPINES	-	-	60	111	423	1,585
		LUZON	-	-	-	-	224	1,471
Plant Factor (%)	TOTAL		45.1 (46.4)	45.4 (47.5)	44.6 (45.2)	42.6 (44.7)	41.0 (40.8)	38.6 (40.2)
	Oil-based		44.6 (47.0)	42.9 (45.5)	44.2 (46.1)	50.5 (51.9)	42.4 (46.2)	32.4 (34.5)
	Hydro		42.5 (38.5)	45.2 (41.7)	33.8 (24.4)	21.6 (12.9)	35.7 (23.6)	32.4 (26.9)
	Geothermal		53.2 (53.7)	63.1 (63.2)	73.2 (73.8)	59.6 (80.4)	58.0 (71.3)	63.1 (74.1)
	Coal				13.7 (-)	25.3 (-)	13.8 (8.5)	51.7 (56.0)

Figures in parenthesis are for LUZON grids.

Table X-2 : Standard List of Measurement Items and Frequency

stages	dam type & height	items	leakage	deformation	seepage line
first stage	surgace impervious wall type	-	everyday	once a week	
	zone type	-	everyday	once a week	
	uniform type	-	everyday	once a week	once a week
second stage	surface impervious wall type	-	once a week	once a month	
	zone type	-	once a week	once a month	
	uniform type	-	once a week	once a month	once a month
third stage	surface impervious wall type	less than 70m more than 70m	once a month once a month	(once three months) (once three months)	
	zone type	less than 70m more than 70m	once a month once a month	(once three months) (once three months)	
	uniform type	-	once a month	(once three months)	(once three months)

First Stage : Period of initial impounding
Second Stage : Successive period of first stage, until behaviour comes to steady value (usually 3 years after first stage)
Third Stage : After second stage

Table X-3 : Inspection Items & Inspection Particulars

points	items	particulars
dam levee crown	crack	developed or not (direction, width, depth etc.)
	settlement	disorder like depression
	deformation	disorder like curve or bend
	security fence	any defection
slope	protection work	damage to riprap, impervious wall
	erosion	situation of erosion and damage to slope face
	settlement	depression, opening, crack
	deformation	disorder like swell
	vegetation	vegetation situation
	water spring	seepage, saturation point
surrounding bed rock	slope	slide, collapse
	crack joint layer	any disorder
	water spring	new spring, disorder of existing springs (amount, water pressure, muddiness)
	snow avalanche	situation
measurement locations	instruments	operational situation
	record	normal or abnormal