5.6.2. Results of Field Investigation

(1) Reservoir Area

There are many slide portions along the reservoir bank, especially in the section 2.5 km upstream from the dam site and from the confluence of the main Agno and its tributary Bokod River to a some 3 km upstream point.

These slides produce a large amount of rock debris discharging into the reservoir but most of them are done in flood time. Among the tributaries flowing into the reservoir such as, Balubac River, Bantey Creek and Labey Creek are discharging much sediments. That means they have many slides in the water-shed.

(2) Spillway Left Abutment

The mountain mass of spillway left abutment is small and skinny, having a steep slope of 35-40° on the southern (downstream) slope. The upper and middle portions are composed of much cracked metamorphic rocks and the lower portion of hard and less-cracked diorite. The metamorphic rocks are in general much weathered up to the depth of 5.

Since an open crack developed in the flood time of 1976, it is still observed in the test pit digged on the road side of spillway left bank, the three drainage adits were carefully observed, keeping in mind whether there are such cracks in the adits or not.

The main fractured planes are, (1) N20 - 30°W/74°N - 90°, (2) N25 - 35°E/82°N - 75°S, (3) N50°W/85°N - 60°S.

The combination of these fractured planes seems to be concerned with the open crack developed in the flood time of 1976. But it is very significant that all these planes are high-angled and no low angled slip planes are there.

Judging from these facts the surface rock mass of the upper portion of spillway left abutment seems to be not slided but slightly toppled in a range of at least 23 m in horizontal as illustrated on Fig.-5.8 and 5.9.

Since the rocks of the spillway left abutment are much weathered and closely fractured, and no particular slide plane is defined, it is better to appreciate the average physical properties of rocks by considering the in-situ rock test data conducted in the similar rock type, when the slope stability is calculated.

The rocks from portal to 5 m are estimated as 'D-CL' class (see Table-5.3) which have a shearing strength of 4-5 kgf/cm^2 and an internal friction angle of around 40°. On the other hand, the rocks of much deeper portions are estimated as 'CM' class which have a shearing strength of more than 10 kgf/cm^2 and an internal friction angle of around 45°.

The values above-mentioned are only assumed by experience. Considering the important situation of the spillway structure and some particular geological differences between this site and others, it is recommendable to carry out in-situ rock shearing tests here to determine the actual rock strength.

(3) Quarry site

There is a good rock exposure just at the opposite side of river to the spillway flip-backet. The rocks are composed of massive diorite and seems to be suitable for lip-rap materials for rock-fill dam.

Taking into account the convenient position and the adequate size of mountain, this place might be deserved for further geological investigation.

(4) Remodel intake tower

The portion of remodel intake tower is composed of metamorphic & diorite. The former constitutes the upper portion of the tower and the latter constitutes the lower portion of the tower .

In these rocks many high-dip fractures are developed. Therefore, drilling investigations should be executed for the selection of best portion for the intake tower.

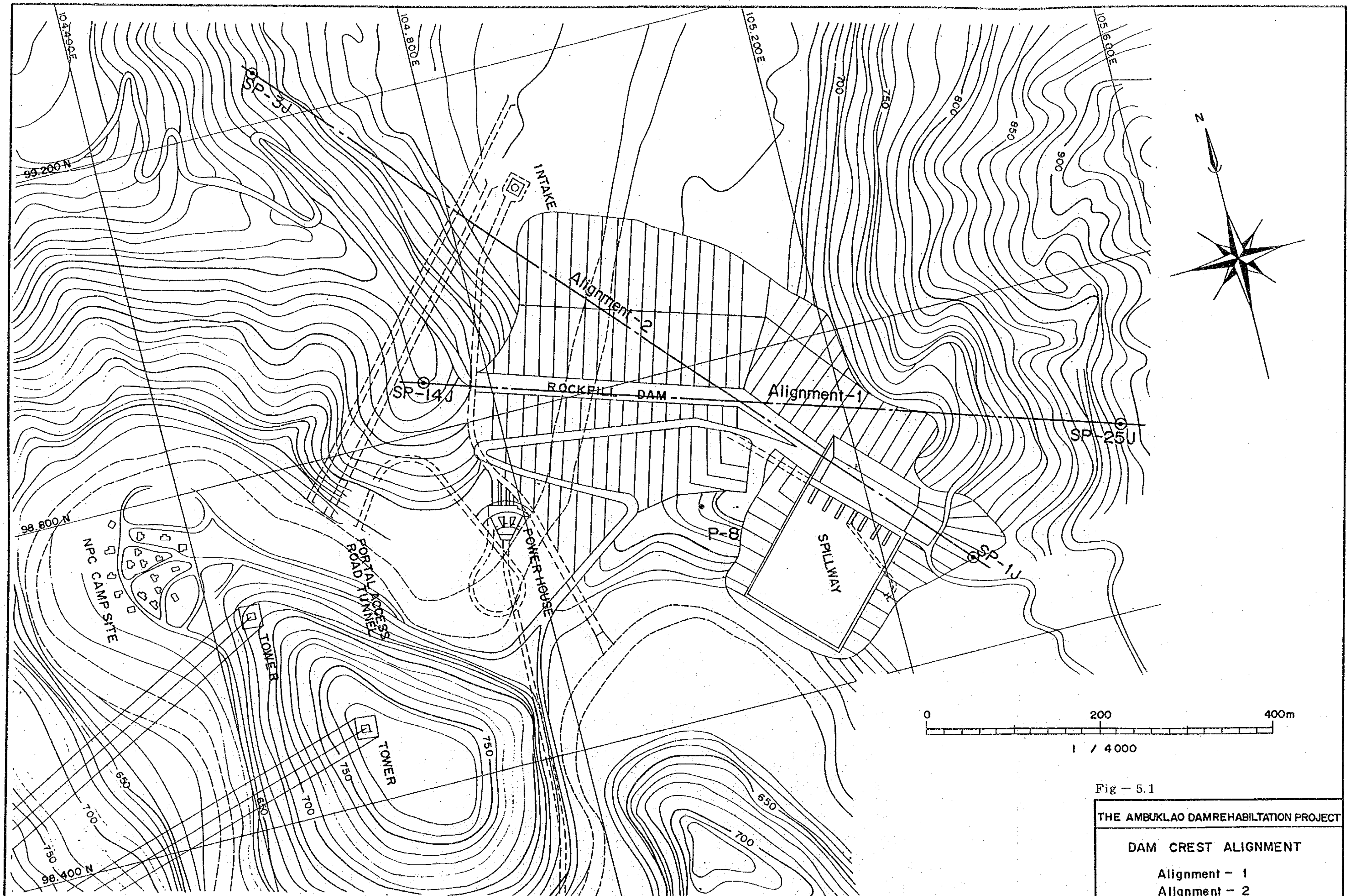
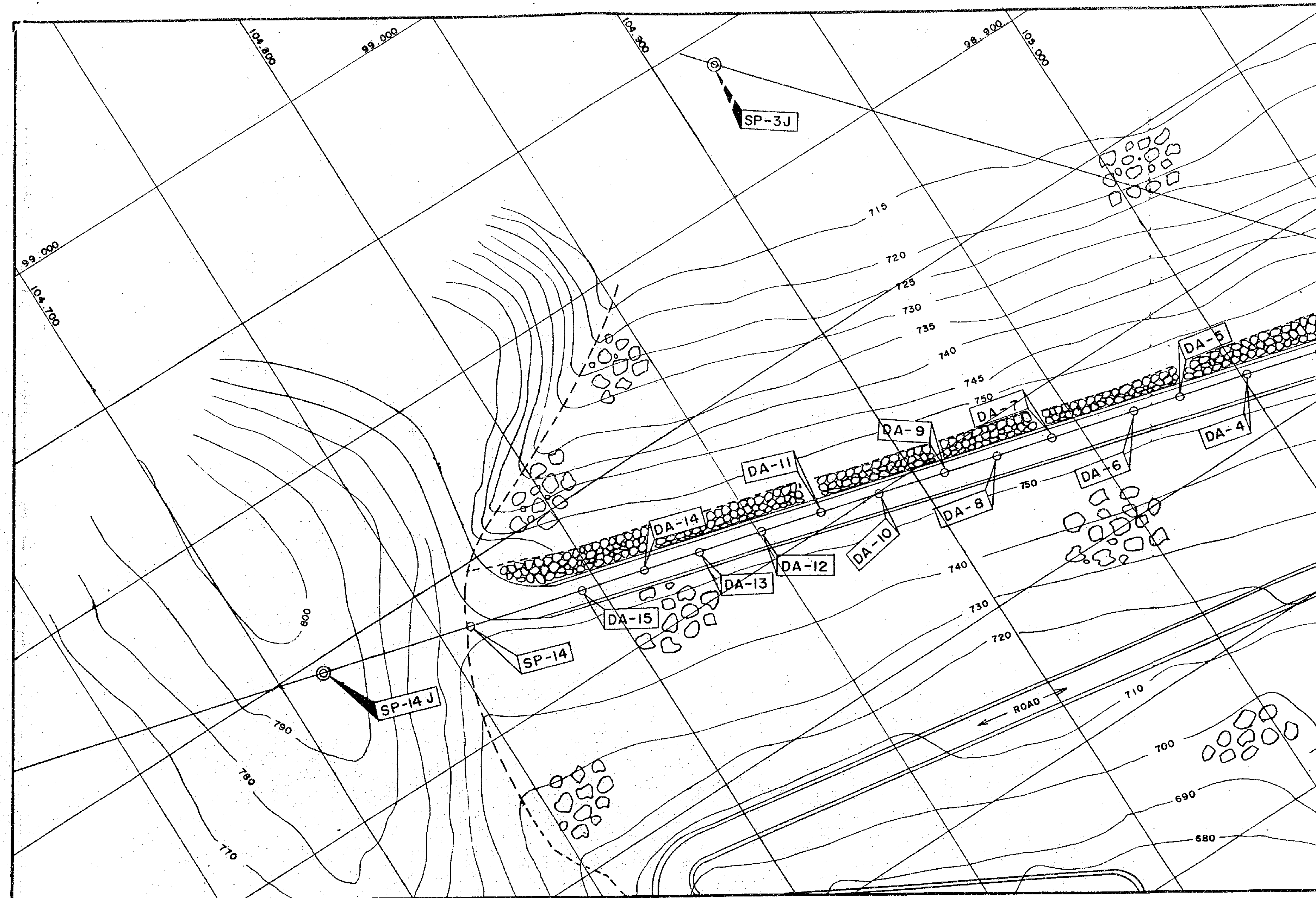
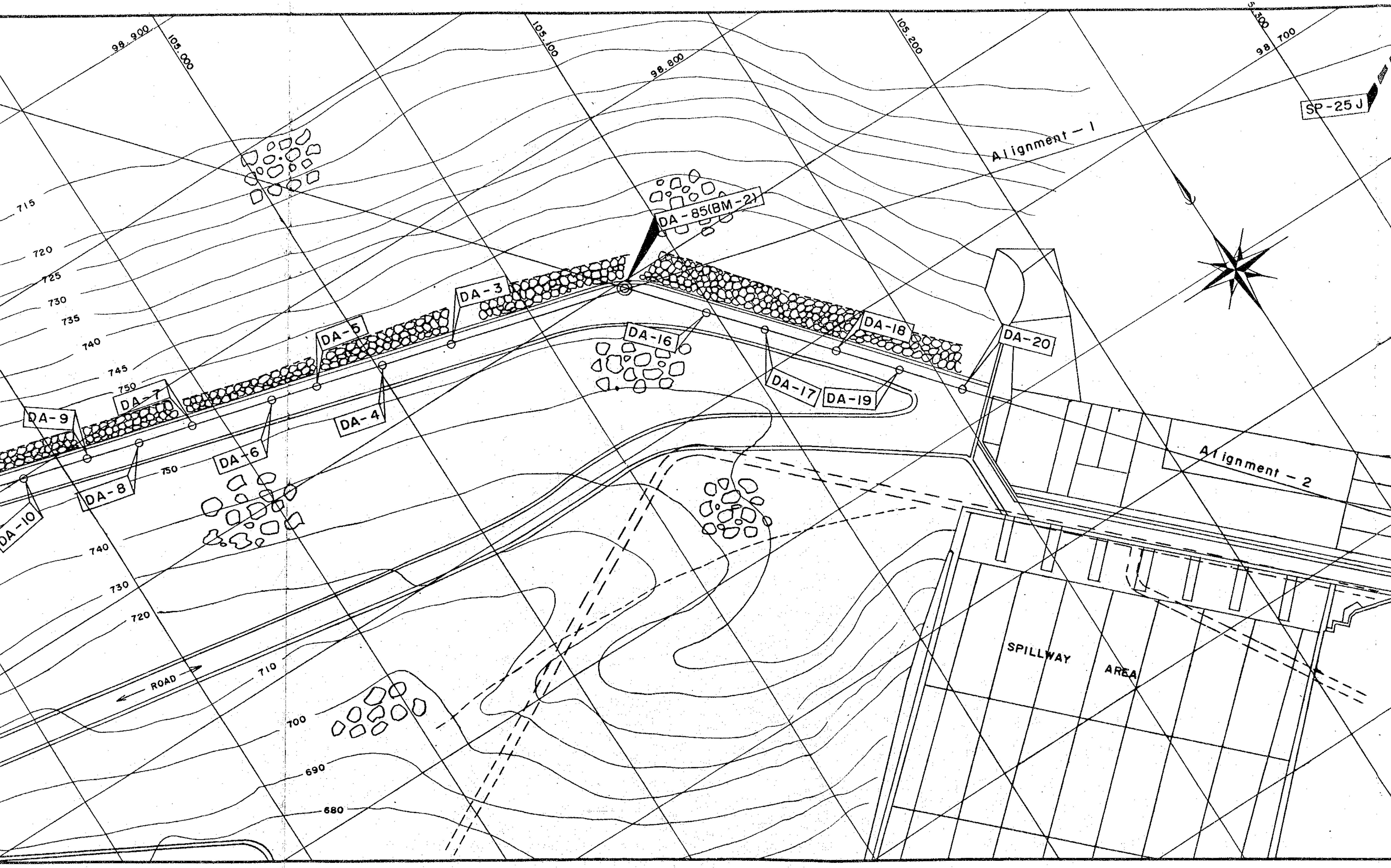


Fig − 5.1

THE AMBUKLAO DAMREHABILTATION PROJECT	
DAM CREST ALIGNMENT Alignment − 1 Alignment − 2	
SCALE 1:4000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	


99.000
104.800
104.900
98.900
105.000
104.700
SP-3J
715
720
725
730
735
740
745
750
DA-5
DA-4
DA-7
DA-9
DA-6
DA-11
DA-8
750
DA-14
DA-10
DA-12
DA-13
740
DA-15
730
SP-14
720
800
SP-14 J
ROAD
710
790
700
780
690
680
770


98.900
105.000
105.100
98.800
105.200
5.300
98.700
SP-25 J
Alignment - 1
DA-85(BM-2)
DA-3
DA-5
DA-4
DA-6
DA-7
DA-8
DA-9
DA-10
DA-16
DA-17
DA-18
DA-19
DA-20
Alignment - 2
SPILLWAY AREA
ROAD
715
720
725
730
735
740
745
750
710
700
690
680

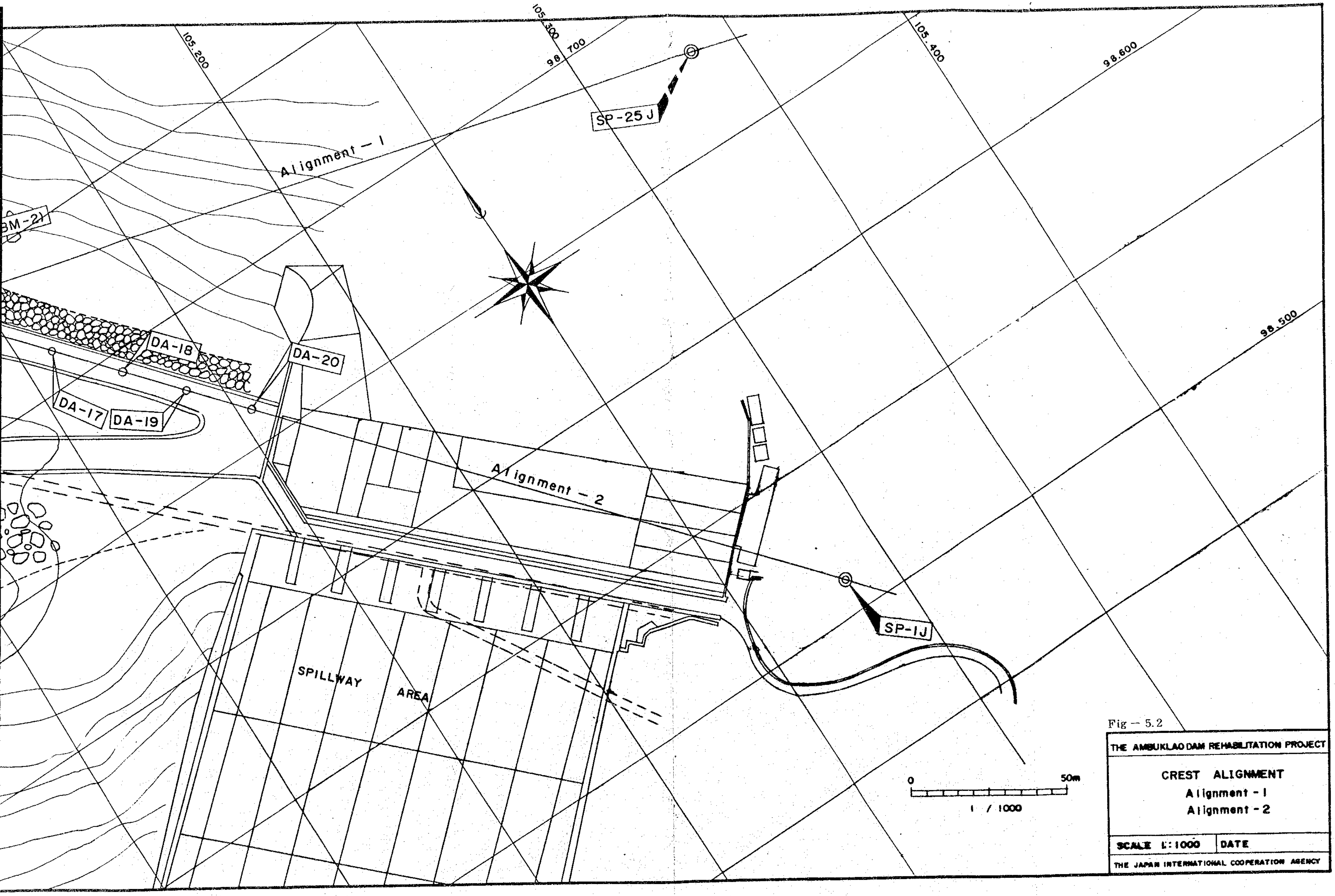


Fig — 5.2

THE AMBUKLAO DAM REHABILITATION PROJECT

CREST ALIGNMENT
Alignment - 1
Alignment - 2

SCALE 1:1000 | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

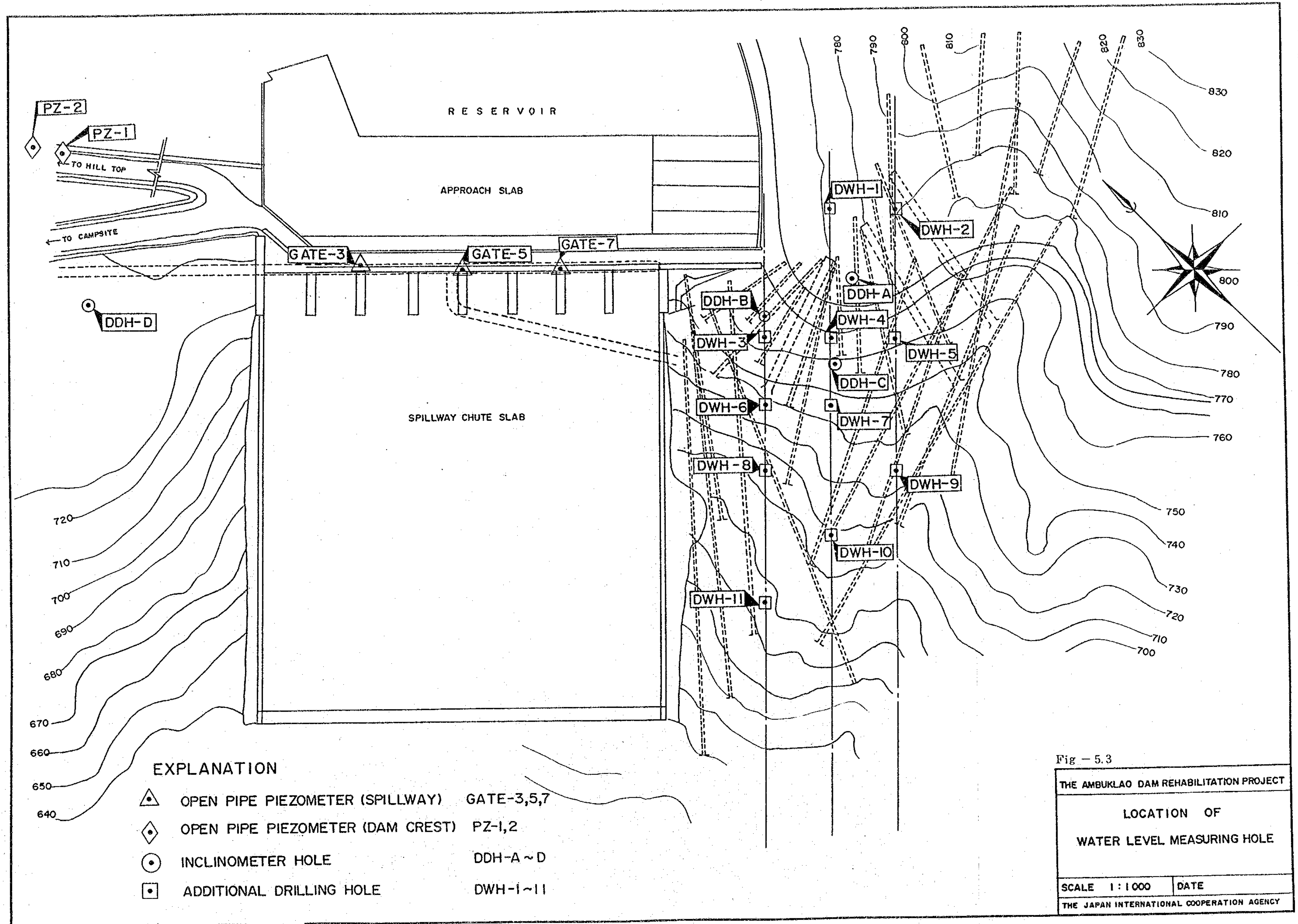


Fig − 5.3

THE AMBUKLAO DAM REHABILITATION PROJECT	
LOCATION OF WATER LEVEL MEASURING HOLE	
SCALE 1:1000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

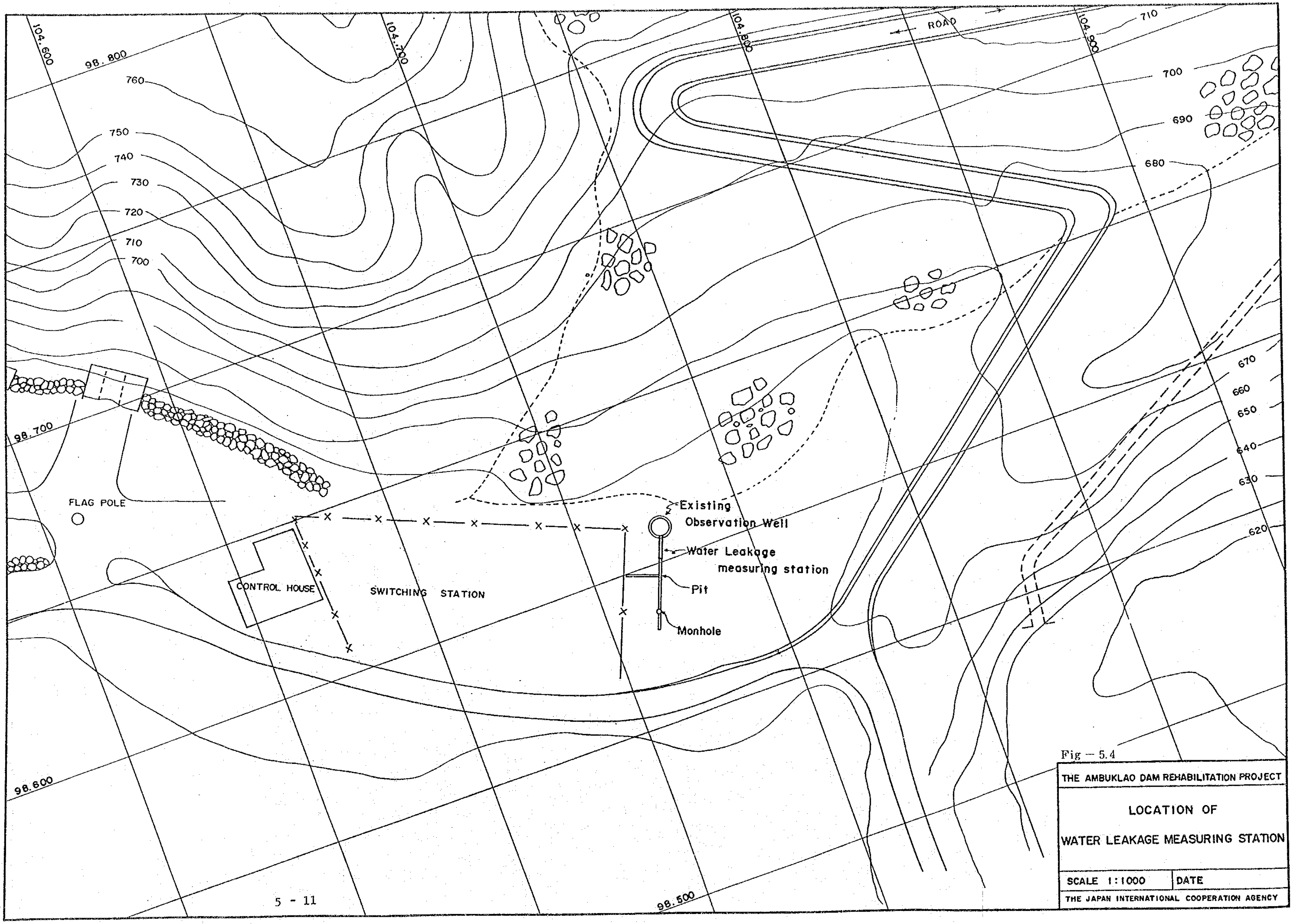


Fig — 5.4

THE AMBUKLAO DAM REHABILITATION PROJECT	
LOCATION OF WATER LEAKAGE MEASURING STATION	
SCALE 1:1000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

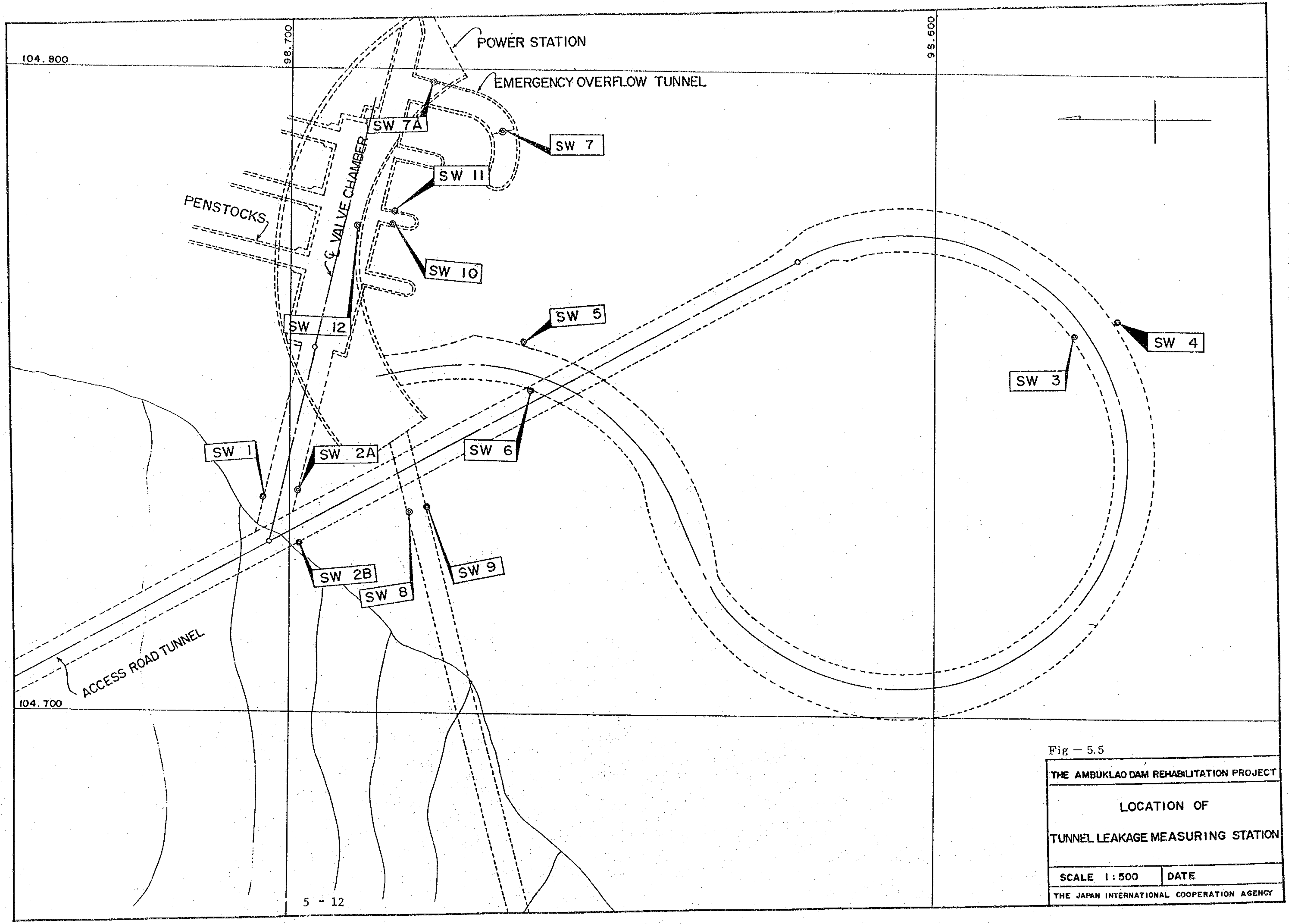


Fig − 5.5

THE AMBUKLAO DAM REHABILITATION PROJECT	
LOCATION OF TUNNEL LEAKAGE MEASURING STATION	
SCALE 1 : 500	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

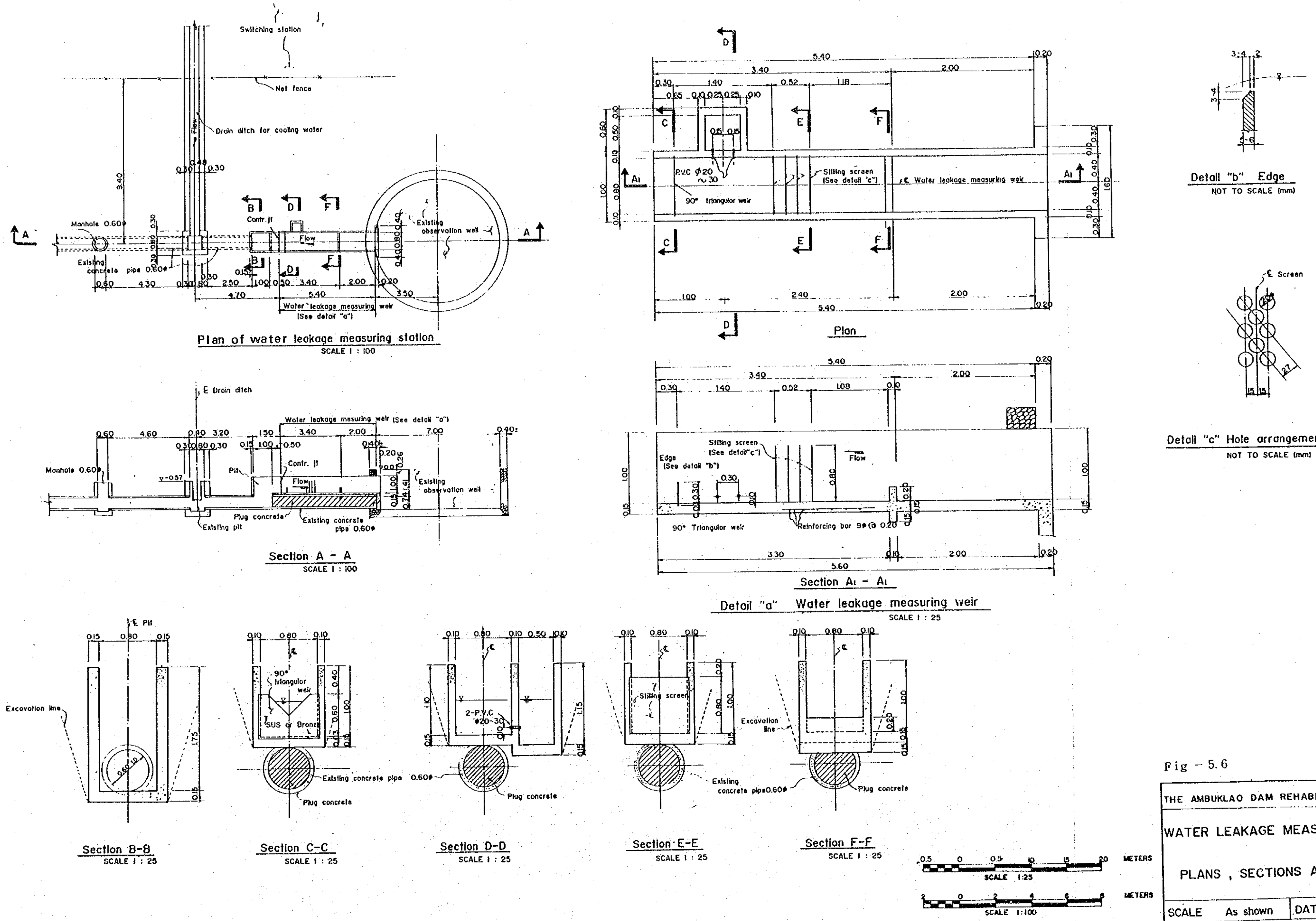


Fig - 5.6

THE AMBUKLAO DAM REHABILITATION PROJECT	
WATER LEAKAGE MEASURING STATION	
PLANS , SECTIONS AND DETAILS	
SCALE As shown	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

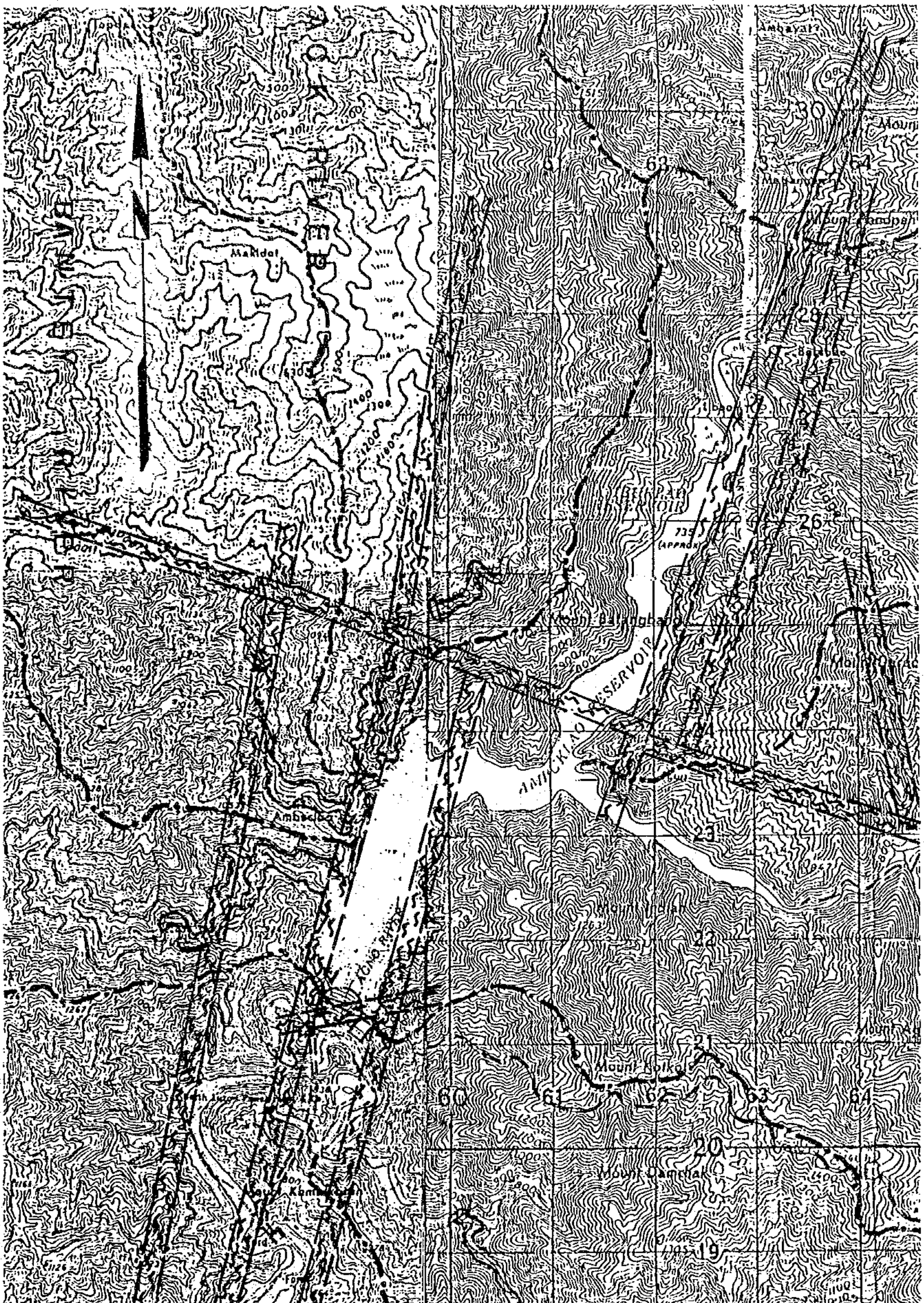


Fig - 5.7 Fault or Fractured Zone
Scale = 1 : 50,000

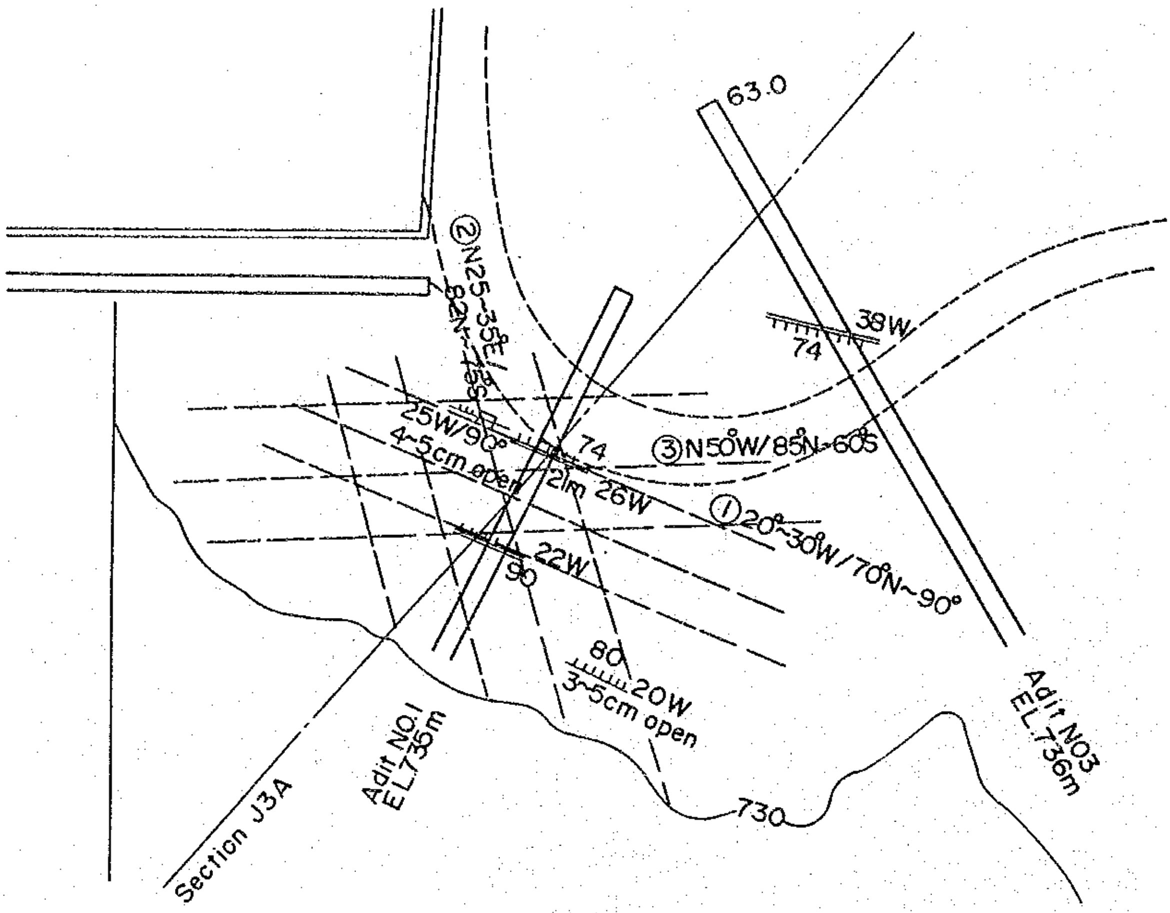


Fig-5.8 Horizontal Slice at EL.730m
of Spillway Left Abutment

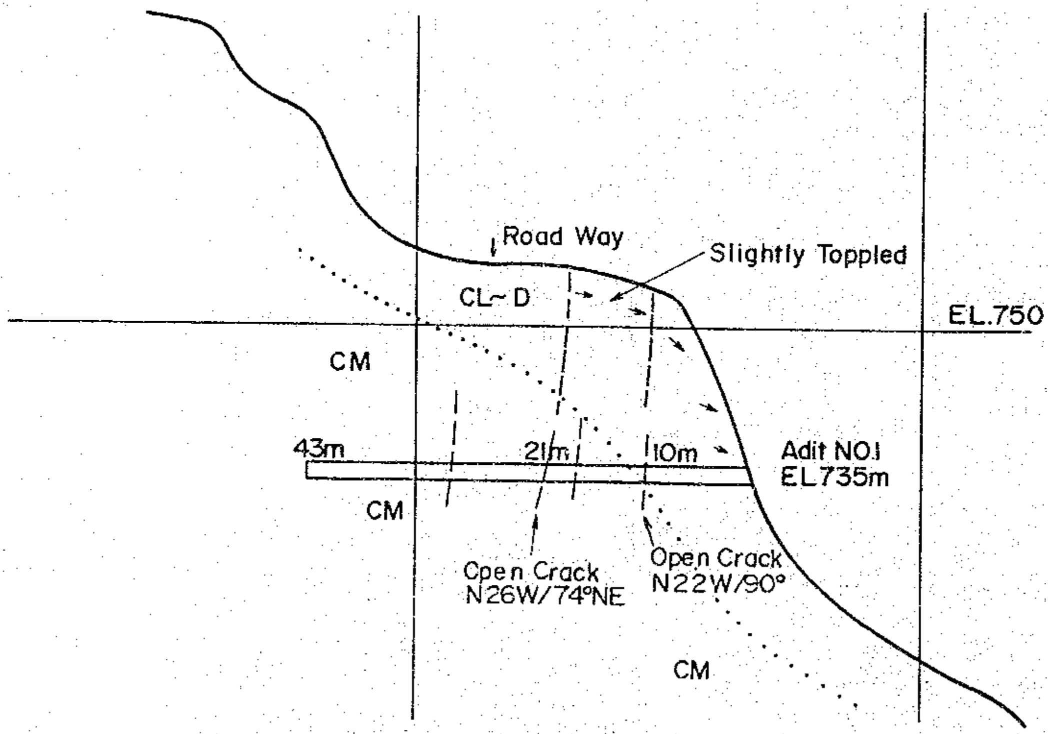


Fig-5.9 Geological Section along
Section J3A

Table – 5.1 Details of Measurement Weirs

Station No.	Location	Total Width B (m)	Depth D (m)	Width of Weir D (m)	Type of Weir *
SW – 1	ACCESS TUNNEL	0.19	0.12	0.14	TYPE – I
SW – 2A	ACCESS TUNNEL	0.175	0.11	0.14	TYPE – I
SW – 2B	ACCESS TUNNEL STA. 0+187.0	0.22	0.12	–	TYPE – II
SW – 3	ACCESS TUNNEL STA. 0+324.0	0.25	0.09	–	TYPE – III
SW – 4	ACCESS TUNNEL STA. 0+325.0	0.23	0.07	–	TYPE – III
SW – 5	ACCESS TUNNEL STA. 0+485.0	0.28	0.11	–	TYPE – II
SW – 6	ACCESS TUNNEL STA. 0+482.0	0.265	0.17	–	TYPE – II
SW – 7	SURGE CHAMBER	0.48	0.205	–	TYPE – IV
SW – 7A	PENSTOCK ADIT	0.43	0.10	–	TYPE – II
SW – 8	CABLE TUNNEL	0.14	–	–	Mensurement of flow velocity
SW – 9	CABLE TUNNEL	0.31	–	–	– ditto –
SW – 10	POWERHOUSE	0.21	–	–	– ditto –
SW – 11	POWERHOUSE	0.19	–	–	– ditto –
SW – 12	POWERHOUSE	0.31	–	–	– ditto –

* TYPE – I ………… Rectangular weir
TYPE – II ………… Right angled triangular weir
TYPE – III ………… 60° angled triangular weir
TYPE – IV ………… Suppressed rectangular

Table — 5.2 Schedule of Additional Field Works for 1987

Work Item			January	February	March	April	May	June	July	August	September	October	November	December
			10 20	10 20	10 20	10 20	10 20	10 20	10 20	10 20	10 20	10 20	10 20	10 20
Drilling	DWH - 1	Tentative												
		Actual												
	DWH - 2	Tentative												
		Actual												
	DWH - 3	Tentative												
		Actual												
	DWH - 4	Tentative												
		Actual												
	DWH - 5	Tentative												
		Actual												
	DWH - 6	Tentative												
		Actual												
	DWH - 7	Tentative												
		Actual												
	DWH - 8	Tentative												
		Actual												
	DWH - 9	Tentative												
		Actual												
	DWH - 10	Tentative												
		Actual												
	DWH - 11	Tentative												
		Actual												
	DWR - 1 (Reservoir)	Tentative												
		Actual												
	DWR - 2 (Reservoir)	Tentative												
		Actual												
Leakage Measuring Station	Tunnel	Tentative												
		Actual												
	Dam	Tentative												
		Actual												
Water Level Gage	Bokod	Tentative												
		Actual												
	Kabayan	Tentative												
		Actual												
Installation of fixed survey point		Tentative												
		Actual												
Location and levelling of drill holes		Tentative												
		Actual												
Cross sectional survey		Tentative												
		Actual												
Detailed topographic survey		Tentative												
		Actual												

Legend:
- ●→ -- beginning of monitoring
- A --- constructing trail access way
- B --- mobilization of drilling, water pumps, Laying water pipe line
- C --- drilling and water pressure test
- D --- demobilization

Table — 5.2

Table-5.3. **Examples of Quality Classifications of Rock in Dam Foundations**

Classification	Characteristics
A	Rock-forming minerals(1) are fresh and not weathered or altered. Joints and cracks are very closely adhered with no weathering along their planes. A clear sound is emitted when hammered.
B	Rock-forming minerals are weathered slightly or partially altered, the rock being hard. Joints and cracks are closely adhered. A clear sound is emitted when hammered.
C_H	Rock-forming minerals are weathered but the rock is fairly hard. The bond between rock blocks is slightly reduced and each block is apt to be exfoliated along joints and cracks by strong hammering. Joints and cracks sometimes contain clay and other material which may be coloured by limonite. A slightly dull sound is emitted when hammered.
C_M	Rock-forming minerals are weathered and the rock is slightly soft. Exfoliation of the rock occurs along joints and cracks by normal hammering. Joints and cracks sometimes contain clay and other material. A somewhat dull sound is emitted when hammered.
C_L	Rock-forming minerals are weathered and the rock is soft. Exfoliation of the rock occurs along joints and cracks by light hammering. Joints and cracks contain clay. A dull sound is emitted when hammered.
D	Rock-forming minerals are weathered, and rock is very soft. There is virtually no bond between rock blocks, and collapse occurs at the slightest hammering. Joints and cracks contain clay. A very dull sound is emitted when hammered.

(1) : Except quartz

Rock class and its physical properties

Rock class	Shear strength	Elastic constant
C_H	$\tau=18.0\text{kg/cm}^2+\sigma\tan 45°$	$E=30{,}000\text{kg/cm}^2$
C_L	$\tau=$ 7.5 " $+\sigma\tan 40°$	$E=13{,}000$ "
D	$\tau=$ 4.0 " $+\sigma\tan 35°$	$E=$ 6,500 "

6. Measurement Manual

6. Measurement Manual

6.1. Seepage Measurement

6.1.1. Seepage through the Dam and its Foundations

The location of seepage measurement stations for the dam and the appurtenant structures are indicated in Figs.-5.4 and 5.6, respectively. The measurement of seepage is carried out by measuring the depth of overflow through a triangular weir with a right (90 degree) angle installed at the downstream end of the water flow measuring pit, as indicated in Fig.-5.6. An Automatic Water Level Recorder which is to be provided by JICA will be used for this measurement.

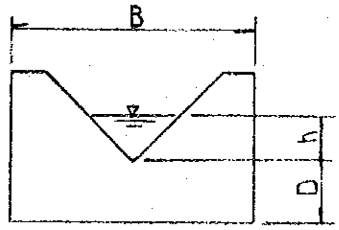
The operation method of the automatic water level recorder is described in detail in the operation and instruction manual provided with the instrument.

The seepage can be calculated by the use of formula (6-1) below. However, it will be easier to make use of the H-Q curve indicated in Fig.-6.1.

The formula for determining the flow over a triangular weir with a 90° angle is as follows:

$$Q = C\,h^{5/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6\text{-}1)$$

$$C = 81.2 + 0.24/h + (8.4 + 12/\sqrt{D}) \times (h/B - 0.09)^2$$



where, Q = Discharge (m^3/min)

h = Overflow Depth (m)

C = Coefficient of overflowing

B = Width (m)

D = Height from bottom of weir to Point A (m)

Records measured by the instrument are automatically printed hourly. These data are compiled on the data sheet.

The amount of seepage is read on the instrument once a day at the predetermined time (for example, 9 AM).

The reservoir water level and precipitation records measured separately should be filled on the same data sheets, and such data should be graphically presented by the method indicated in Fig.-6.2.

The automatic recording paper should be replaced bi-monthly at which time a test printout of the program should be made.

6.1.2. Seepage around the Powerhouse

Seepage in the tunnels situated around the underground powerhouse are measured at the site (see Fig.-5.5). In this case, the depth of the flow over the weir is measured from SW-1 to SW-7A and the seepage discharge is calculated by the use of the formula given below. Water flow is calculated by measuring the velocity at five locations, SW-8 to SW-12, and multiplying the velocity with the cross-sectional area of flow obtained from the depth and width of the waterway.

(1) SW-1 and SW-2A (in case of rectangular weir)

$$Q = Cbh^{3/2} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (6\text{-}2)$$

$$C = 107.10 + 0.177/h + 14.22\ h/D - 25.68\sqrt{(B-b)h/B\cdot D} + 2.04\sqrt{B/D}$$

where, Q = Seepage Discharge (m^3/min)
b = Width of sink (m)
C = Coefficient of discharge
B = Total Width (m)
D = Height to the weir (m)

(2) SW-2B, SW-5 and SW-6 (triangular weir with a right angle)

Formula (6-1) is used.

(3) SW-3 and SW-4

$Q = Ch^{5/2}$ .... (6-3)

$C = 1.363 (34.38 + 0.1098/h)$

The symbols are the same as those of formula (6-2).

(4) SW-7 (in case of suppressed rectangular weir)

$Q = CBh^{3/2}$ .... (6-4)

$C = 107.10 + (0.177/h + 14.22 h/D)$

For symbols see formula (6-2).

The discharge of seepage calculated by the above formulas should be graphically presented so that the time passage could be understood clearly. The reservoir water levels and precipitation records should also be presented on the same graph.

6.2. Dam Deformation Measurement

6.2.1. Horizontal Deformation

Horizontal deformations are calculated by line-of-sight the survey along Alignment 1 and Alignment 2 (See Fig.-5.1 and 5.2) using a theodolite provided by JICA. For Alignment 1, the fixed control point is SP-3J and the reference point is SP-1J. For Alignment 2, the fixed control point is SP-25J and the reference point is SP-14J. For both Alignments, the movable target point (DA-3, etc.), to measure deformation, is pointed at by the theodolite installed at the fixed control point. The slope distance, the vertical angle and the horizontal angle, are measured. Furthermore, the horizontal deformation is calculated by formula (6-5), using the above measured values, as follows:

$$\delta x = L \cdot \sin \theta v - \sin \theta h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6\text{-}5)$$

where, δx = Horizontal Deformation (mm)

L = Distance between the fixed control point and the target point (mm)

θv = Zenith Angle, degrees

θh = Horizontal Angle, degrees

6.2.2. Vertical Deformation

Vertical deformations of the movable points are calculated on the bases of the datum point P-8 (coordinates, lat. 16°27'40.46"N and long. 120°44'35.11"E, EL 742.464 m) by leveling survey from the level of each movable point. The location of the datum point P-8 is indicated in Fig.-5.1.

6.3. Groundwater Level Measurement

The water levels of 11 boreholes, four inclinometer holes and two piezometer holes, located on the dam crest and as indicated in Fig.-5.3, are measured respectively. Measurement of the groundwater level is carried out by measuring the depth from the top of the casing pipe to the water surface using ground water level gauge provided by JICA. The measured data should be recorded as shown in Table-6.1 and periodically presented graphically as shown in Fig.-6.4.

6.4. Measurement Intervals

The measurement intervals should be set up taking into consideration the fact that 30 years have passed since the construction of the Ambuklao Dam.

On the other hand, considering that the groundwater level gauging holes have been newly installed at the left abutment of the spillway and, further, the groundwater level is very important for the stability of the abutments, slopes, relatively frequent measurement intervals should be set up until the movements of groundwater levels in a yearly cycle are properly understood.

Table 6-2 shows the summary of the interval of measurements.

Table – 6.1 Data Sheet of Ground Water Level

(Spillway left abutment)

HOLE No.	DATE					
DWH-1 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-2 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-3 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-4 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-5 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-6 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-7 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-8 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-9 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-10 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DWH-11 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					

(Dam crest and Inclinometer hole)

HOLE No.	DATE					
PZ-1 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
PZ-2 Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DDH-A Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DDH-B Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DDH-C Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					
DDH-D Casing(Top) EL.	Reading(m) (Depth)					
	Water Level (EL.)					

Table – 6.2 Interval of Measurements

Measuring Item	Interval of Measurements	Discriptions
Precipitation	Once a Day	
Reservoir Water Level	Once a Day	
Leakage Dam	Once a Week	
Leakage Power Station	Once a Day	to be reduced in future
Water Level Openpipe Piezometer	Once a Month	to be reduced water level rising up in future
Water Level O.P. Dam Crest	Once a Week	
Water Level Inclinometer Hole	Once a Week	
Water Level Spillway Left Abutment	Once a Week	to be reduced in future
Dam Displacement Alignment – 1 Alignment – 2	 Once a Month Once a Month	to be reduced in future
Inclinometer	Once a Week	

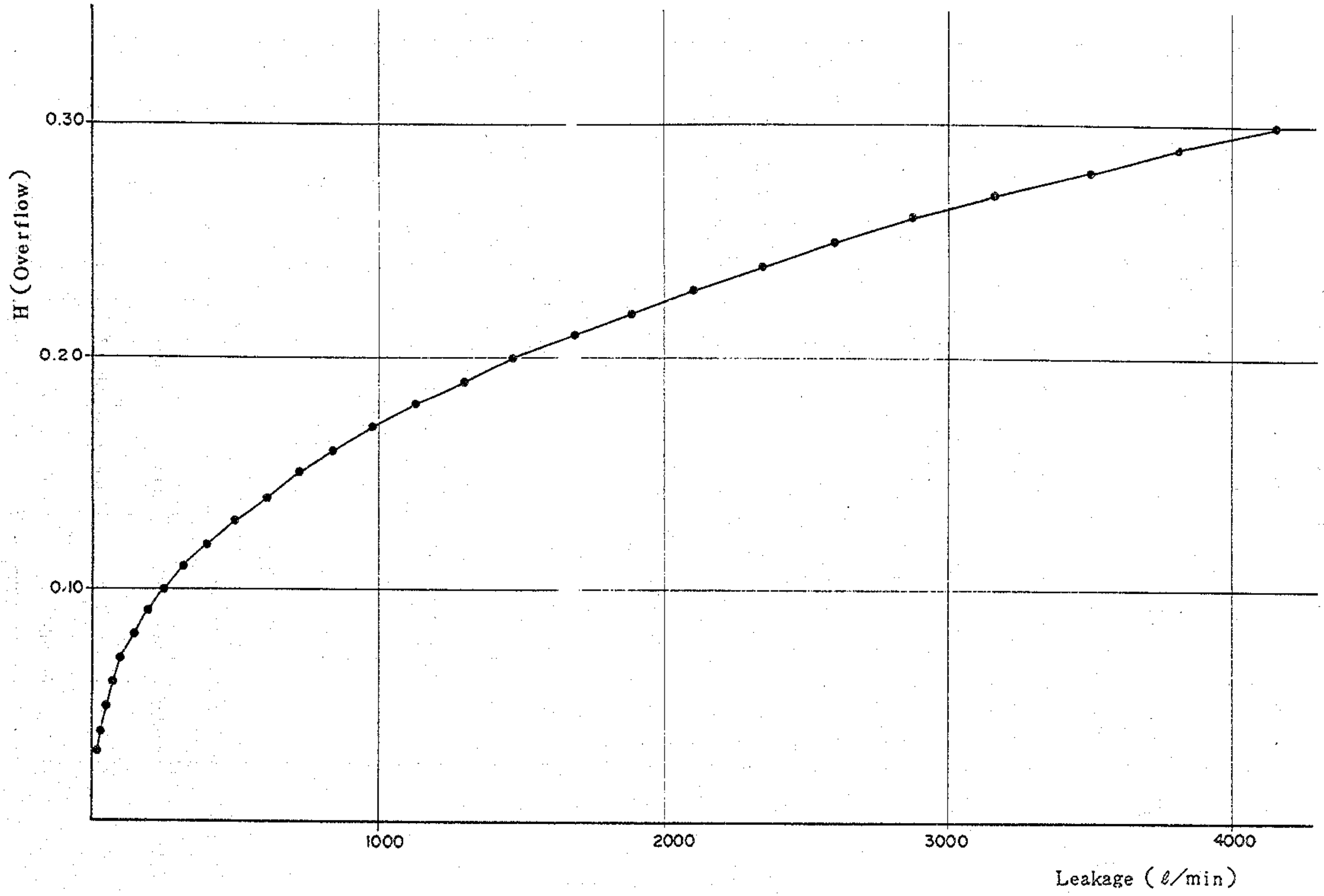


Fig — 6.1   H — Q Curve

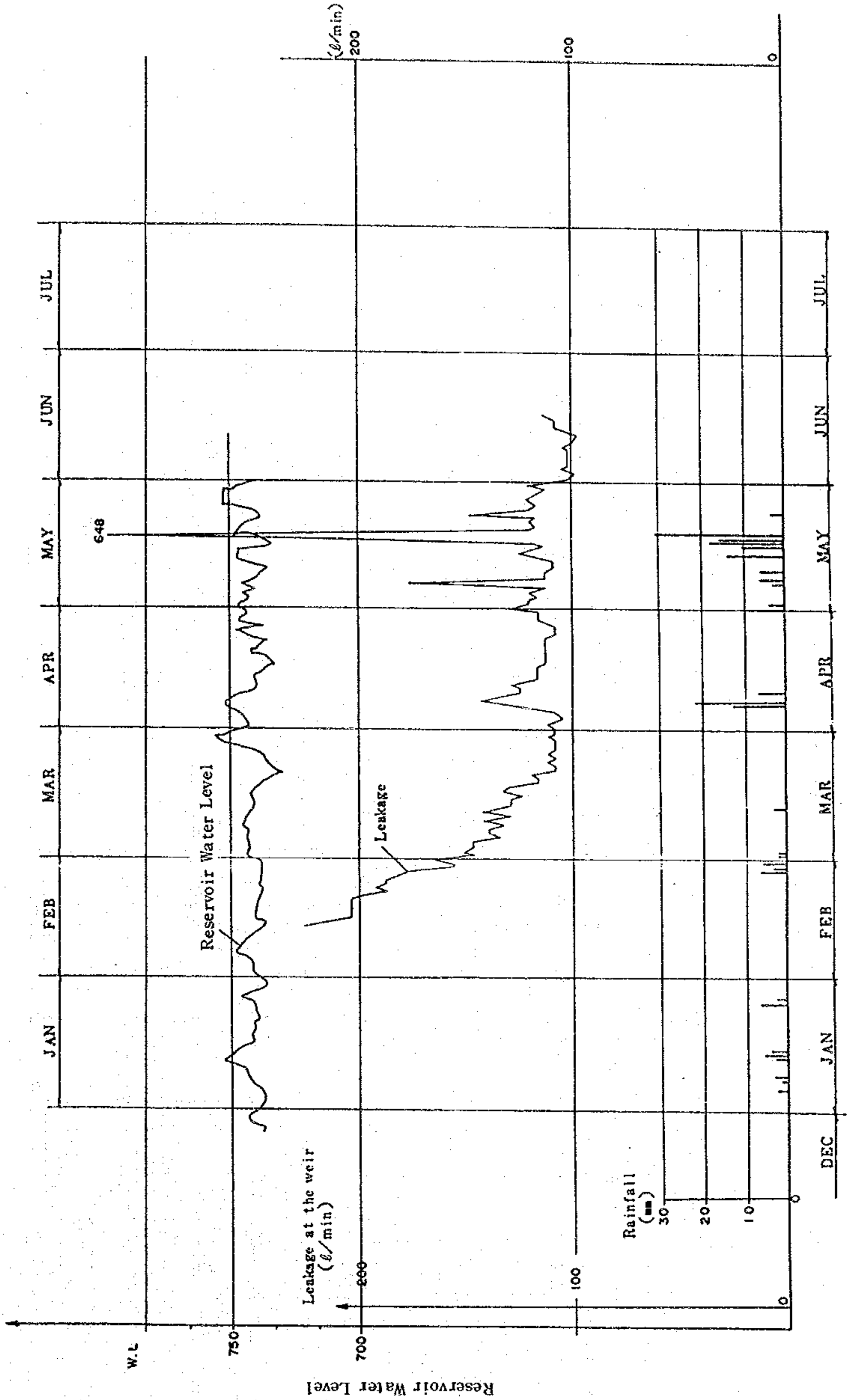


Fig - 6.2 EXAMPLE (Water Leakage)

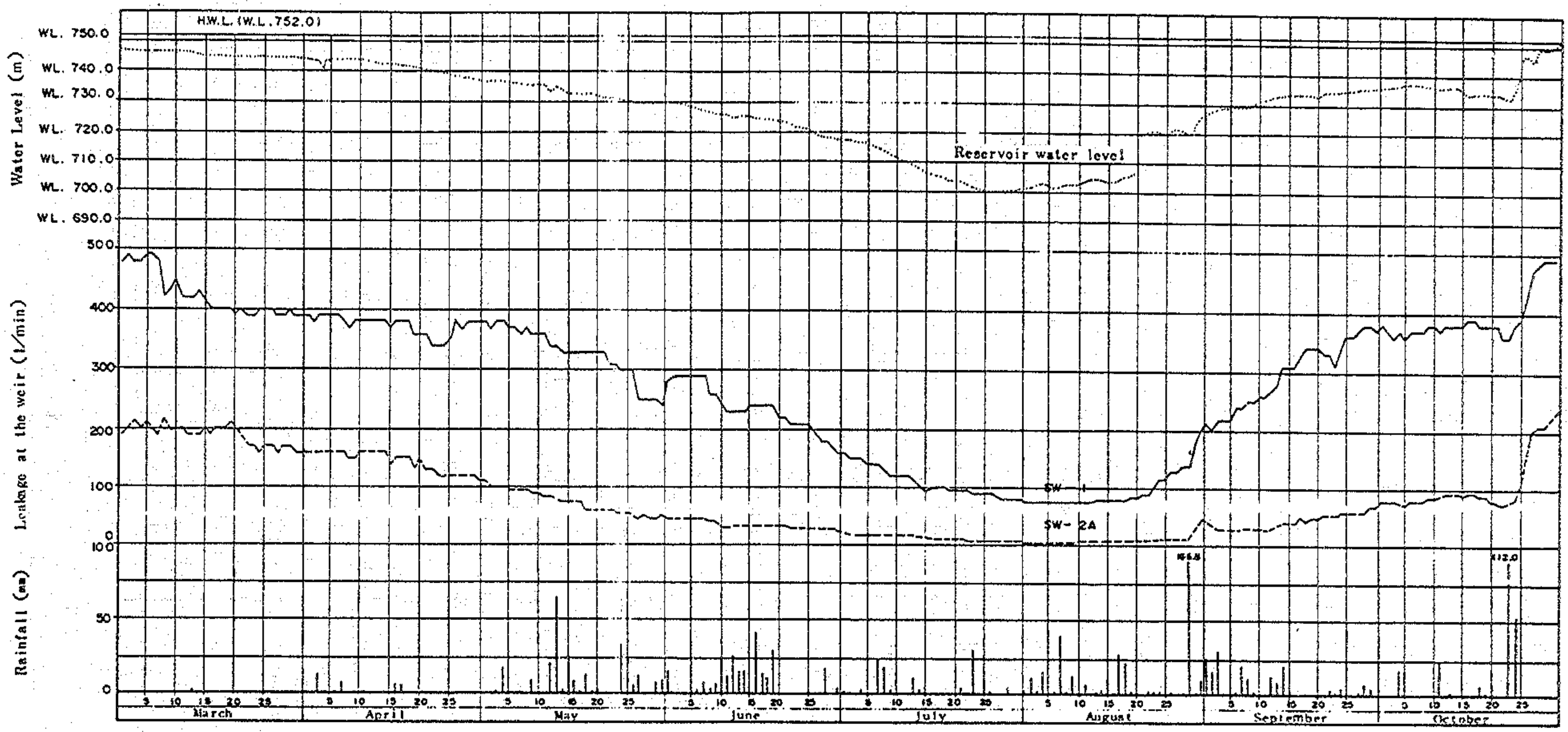


Fig − 6.3 GRAPHIC EXAMPLE (Dam Leakage)

Ground — Water Level of Ambuklao Dam

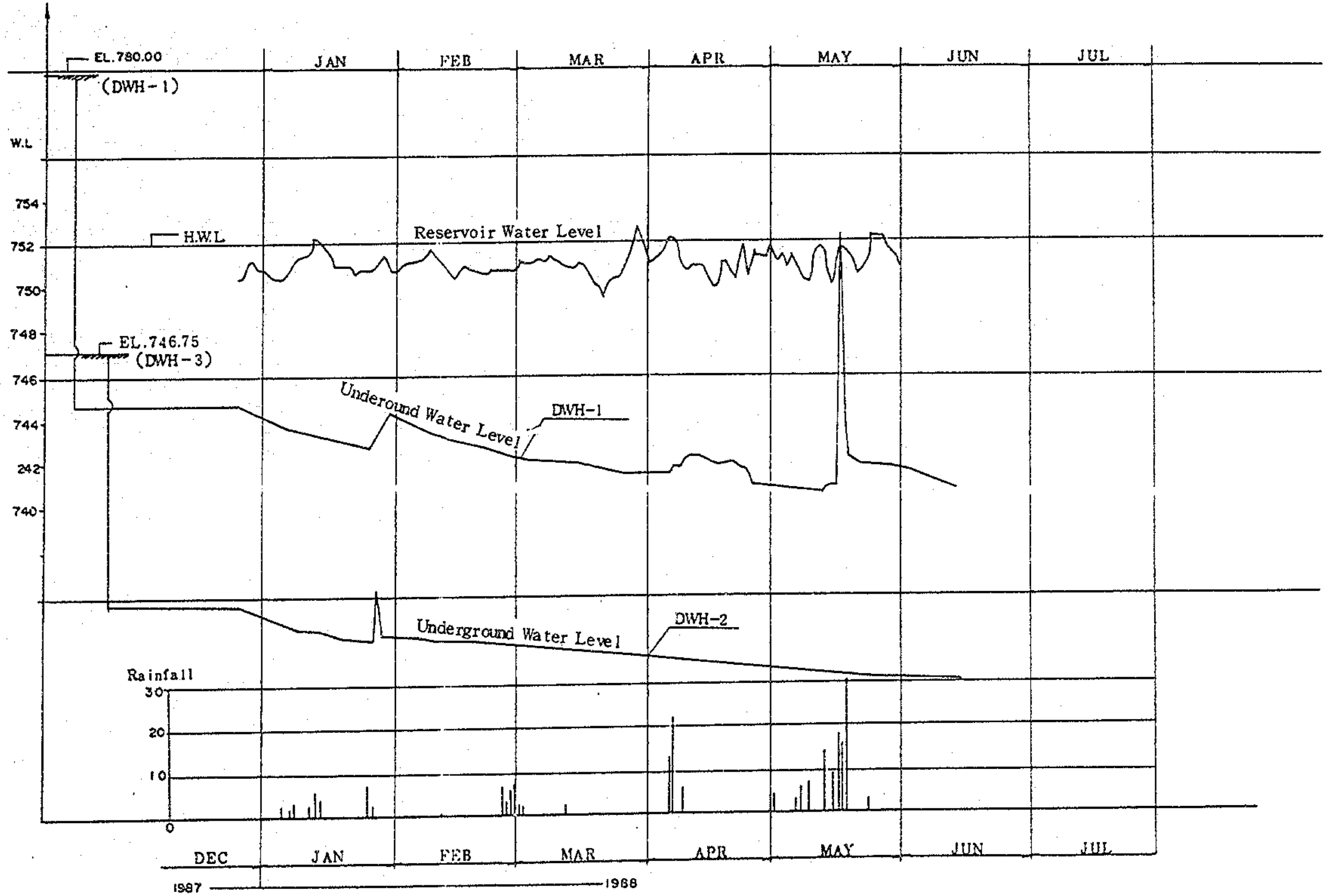


Fig — 6.4 EXAMPLE (Ground Water Level)

7. Monitoring

7. Monitoring

The monitoring work carried out at the Ambuklao Dam during the course of these studies was as follows:

7.1. Discharge and Quality of Seepage

The discharge of seepage was measured by a water seepage measurement device installed at the edge of the downstream face of the dam and water flow measurement devices installed at the auxiliary structures around the underground powerhouse. Figs.-5.4 and 5.5 show the distribution and the location of these devices. The measurements performed during the period of this study, were as described below:

Seepage measurement for the dam

Continuous monitoring from March 1, 1987, and recording of measurement results until November 12, 1987.

Seepage measurement around the powerhouse

Continuous monitoring from March 1, 1987, and recording of measurement results until November 12, 1987.

The investigation of the water quality was not carried out as it was not considered to be necessary.

7.2. Deformation

(1) Deformation of the Dam

The deformation measurements for the dam were carried out along two alignments, Alignment 1 and Alignment 2 located along the dam crest.

Fig.-5.1 shows the location of these alignments.

The deformation measurements for the dam were carried out continuously since March 17, 1987, and the results obtained until November 6, 1987, were recorded.

(2) Inclinometer

The measurement by inclinometer was carried out using the existing borehole at both the dam and the spillway left abutments drilled in 1979. Its location is indicated in Fig.-5.3.

The measurement was carried out continuously since March 13, 1987. The results of the measurement until August 10, 1987, were recorded.

7.3. Borehole on the Left Abutment of the Spillway and Water Levels in the Dam Embankment

The measurements of the borehole on the left abutment of the spillway and the water levels in the dam embankment were carried out as indicated in Fig.5-3.

Water levels in the dam embankment

Continuous monitoring and recording of measurement results from February 5, 1987 until August 25, 1987, were carried out.

Borehole on the left abutment of the spillway

Continuous recording was conducted since completion of the hole.

Recording of measurements for DWH-1, 2, 4, 5 and DDH-A, B, C was conducted till November 9, 1987. Recording of the measurements for the hole in the dam was performed till August 25, 1987.

7.4. Reservoir Water Level

The reservoir water levels were measured at the Ambuklao Powerstation hourly and recorded at the time of reading.

7.5. Inflow and Outflow

The inflows and outflows were recorded at the Ambuklao Powerstation and these data can be obtained as necessary.

7.6. Rainfall

Rainfall was measured daily at the Ambuklao Powerstation and recorded at the time of monitoring.

7.7. Record of Earthquakes

No useful records on earthquakes could be obtained from sources around the area of the Ambuklao Dam. However, the data of Bongel and Kanoong Stations located downstream of the Agno River were available and they were obtained.

8. Stability Analysis of Structures

8. Stability Analysis of Structures

8.1. Dam Stability

8.1.1. General

The principal design and construction drawings of the dam were not available and assumed to have been lost as 30 years have passed since its construction. There were no detail data on the design assumptions, loading conditions and the design parameters used for the design of the dam.

The paper "Ambuklao Rockfill Dam, Design and Construction" by E. Montford Fucik and Robert F. Edbrooke, ASCE Paper No. 3057, 1967, is the only article on this project known to have been published.

Generally, for stability of dams, it is quite adequate to study the (1) structural and (2) hydraulic stability of the dam.

With regard to structural stability, two stability conditions have to be investigated. They are (a) stability against sliding of the upstream and downstream slopes of the dam, and (b) sliding of the dam body in the downstream direction due to reservoir water pressure.

In addition to the above, studies are required to determine the subsidence of the embankment materials due to consolidation, deformation of the embankment due to water pressure, and the effects on the overall stability of the dam of the irregular subsidence at the boundaries between the zones of the dam. These are problems which relate to the deformations of the dam. However, as 30 years have passed since the construction of the dam, these deformation studies are not considered to be relevant, and, thus, not necessary for the present stage.

The following paragraphs describe the structural and hydraulic stability studies conducted for the Ambuklao Dam as part of the Rehabilitation Project.

8.1.2. Stability Analysis

The method for stability analysis of the dam was selected not only on the basis of the above ASCE Report, but also taking into consideration the state of the art of the design and construction of rockfill dams in the USA in the 1950's at the time when the Ambuklao Dam was designed.

According to the above ASCE Report, the stability analysis of the Ambuklao Dam was conducted by the so-called "Wedge Theory". This method examines the stability against sliding of the downstream part of the dam including core zone against water pressure.

In the "Wedge Theory", a simple formula is used and, as a result, the safety factor is calculated using formulae (8-1) and (8-2).

$$S.F. = \left(\frac{L}{H}\right) \cdot \frac{\gamma t}{\gamma w + K \cdot \gamma t (L/H)} \tan \phi \quad \text{.......... (8-1)}^{*1}$$

$$= \left(\frac{L}{H}\right) \tan \phi \quad \text{..................... (8-2)}^{*1}$$

---

*1 - "Study on Stability Analysis, Design and Construction of Rockfill Dams" by Dr. Tsuguo HARADA, March, 1977
- Interim Report on the Study for Ambuklao Dam Rehabilitation Project, Japan International Cooperation Agency, June 1987, Page 2-2

where,

- S.F. = Safety factor against sliding
- H = Static water head (m)
- γt = Average unit weight by volume of embankment materials (t/m^3)
- γw = Unit weight by volume of water (t/m^3)
- K = Design coefficient of earthquake
- L = Horizontal distance from the upstream end of core to the toe of the downstream face (m)
- ϕ = Internal friction angle of embankment materials (°)

As condition of L/H 2 is satisfied by both formulae (8-1)and (8-2), the safety factor against sliding in the downstream direction due to water pressure when the downstream portion of the dam is constructed of rockfill, is normally very high as compared with the safety factor obtained for the case of sliding of the slopes.

Therefore, it is judged that the stability study of a rockfill dam, if only "Wedge Theory" is applied, to be insufficient. In view of the above, the stability analysis against sliding of the upstream and downstream slopes of a rockfill dam becomes a very essential and significant requirement.

For the Ambuklao Dam, there are no references on the performance of the stability analysis of the upstream and downstream slopes of dam. If stability analysis against sliding of the slopes were carried out, the extremely steep slopes of 1:1.75 of the upstream and downstream faces of the dam would not have been adopted.

The design methods for rockfill dams practiced in the USA in the 1950's were somewhat different from those used presently. Twenty to thirty years ago, as shown in the examples of Fig.-8.1(a) Miboro Dam (completed in 1960) and Fig.-8.1(b) Kisenyama Dam (completed in 1970), trial and error method was used in order to determine the minimum safety factor against sliding so as to be fixed the beginning point of sliding at the dam crest shoulder. The above method, as required to check the stability of the upstream and downstream slopes of the Ambuklao dam, was also applied during the present studies.

The safety factor against sliding based on this method was computed using the following formula:

$$S.F. = \frac{\Sigma\{C \cdot \ell \quad (N - U - Ne) \tan \phi}{\Sigma (T + Te)} \quad \cdots\cdots\cdots\cdots\cdots \quad (8\text{-}3)$$

where,

SF = Safety factor

N = Normal component of load acting on slip circle of each slice (t/m)

T = Tangential component of load acting on slip circle of each slice (t/m)

U = Pore water pressure of acting on slip circle of each slice (t/m)

Ne = Normal component of earthquake load acting on slip circle of each slice (t/m)

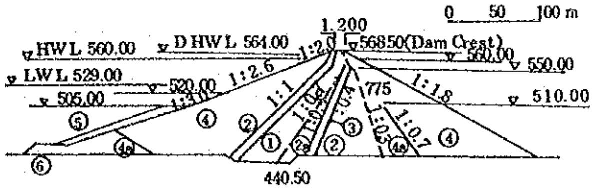
Te = Tangential component of earthquake load acting on slip circle of each slice (t/m)

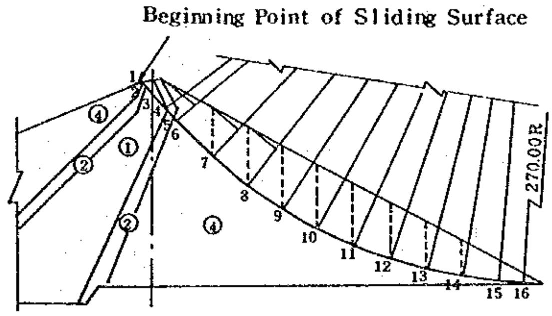
ϕ = Internal friction angle of material (°C)

C = Cohension of material (t/m^2)

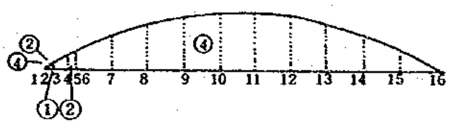
ℓ = Arc length of slip circle of each slice (m)

Typical Section of Dam

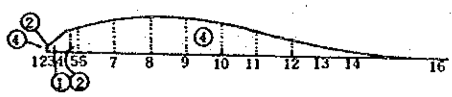




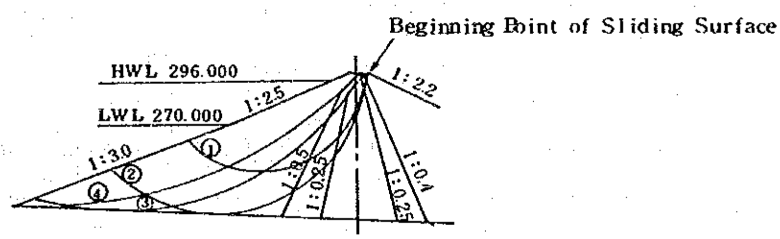
Vertical Forces



Tangential Forces



(a) Miboro Dam (Height 131m, Cmpleted in 1960)



Safety Factors

Circle No.	At H. W. L		At Water Level Drawdown	
	K = 0	K=0.12	K = 0	K=0.12
①	3.73	2.13	2.70	1.80
②	4.69	2.58	3.35	2.24
③	2.73	1.61	2.49	1.54
④	2.69	1.47	2.43	1.46

(b) Kisenyama Dam (Height 95m, Completed in 1970)

Fig — 8.1 Example of Dam Slope Analysis

8.1.3. Design Conditions and Material's Properties

Table-8.1 shows the shearing strength of the embankment materials quoted from the above ASCE Report. Fig.-8.2 shows zoning of the dam.

Table-8.1

Characteristics of Embankment Materials

Zone	Material Characteristics		Unit Weight
	ϕ	tan ϕ	
Zone A (core material)	15°	0.268	Not indicated
Zone B (filter material)	35°	0.700	
Zone C	35°	0.700	
Zone D (rock material)	40°	0.839	
River gravels	30°	0.577	

The test condition conducted the values given in Table-8.1 are not clarified. It is reasonable to assume, then, that these values are the test values in the practical conditions of consolidation and drainage. Furthermore, the soil tests were carried out only for the material of Zone A (core zone) during the rehabilitation work after the earthquake of 1985.

The results of the soil tests mentioned above are shown in Table-8.2.

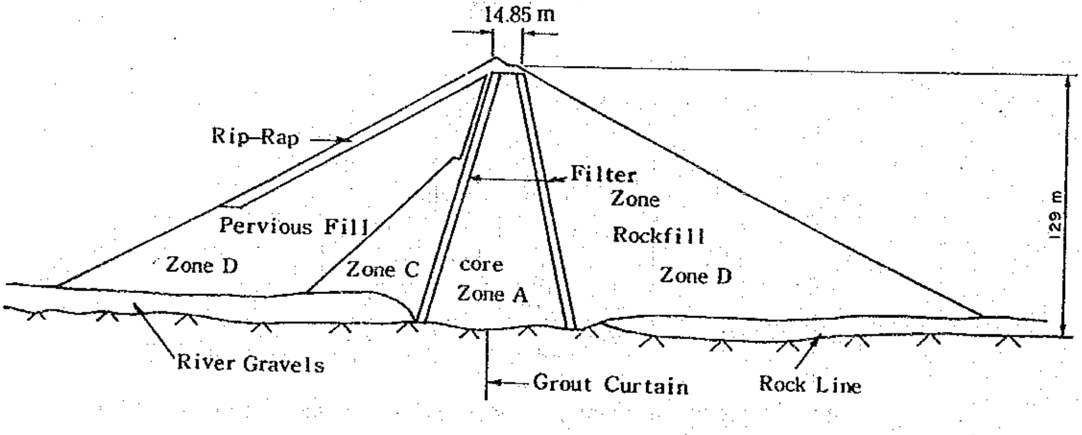


Fig-8.2 Zoning of the Dam

Table-8.2

Shear Strength of Zone A Material Tested in August 1985

(1) CU Test

Ccu : 86 =Kpa, 33Kpa, 71Kpa

ϕcu : 26°, 26°, 23°

(2) CD Test

Ccd : 66Kpa, 12Kpa, 33Kpa

ϕcd : 28°, 31°, 28°

(3) Average values converted to kg/cm^2

Ccu = 0.645 kg/cm^2

Ccd = 0.377 kg/cm^2

ϕcd = 29°

Table-8.3 shows the unit weight by volume of the embankment materials of which each value was selected based on the results of the above soil tests and the other results of the field investigations. The fact that Zones B and D consisted of diorite was also taken into consideration. From experience it is known that, for calculation of the safety factor against sliding, the effect of the unit weight of the materials is very small. Thus, although the analysis was based on the unit weights of the materials shown in Table-8.3, the results of the calculations are considered to be quite reliable.

Table-8.3

Unit Weight of Embankment Materials

Zone	γd *1 (g/cm^3)	γt *2 (g/cm^3)	γsat *3 (g/cm^3)
Zone A (core material)	1.83	2.20	2.28
Zone B (filter material)	1.90	1.95	2.18
Zone D (rock material)	1.80	1.85	2.11

Notes: *1 dry condition

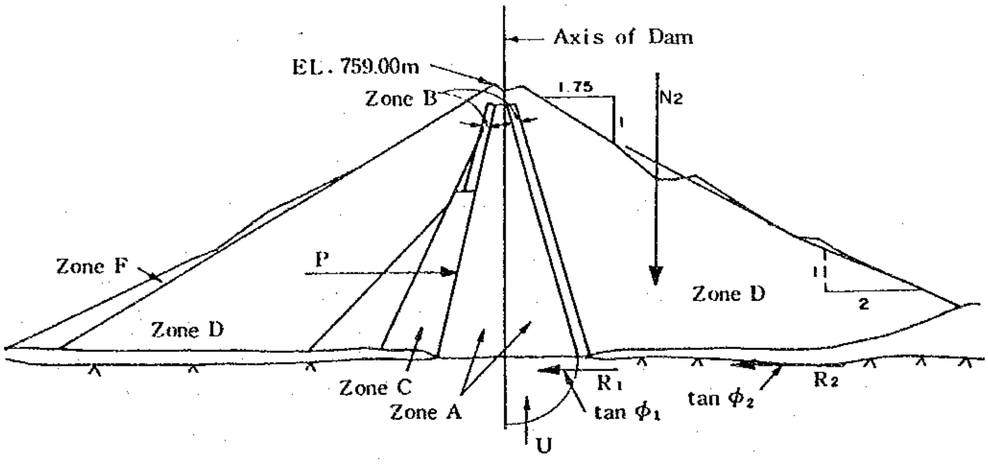
*2 wet condition

*3 saturated condition

Cohesion is normally not taken into account in making stability analysis. This is particularly true for rockfill materials used in rockfill zones. On the other hand, cohesion of core materials (clays) is often considered.

As is well known, design of dams is greatly affected by earthquake condition. The earthquake proof design should be properly considered as the Ambuklao Dam is located in an active earthquake area. In the ASCE Report, the safety factor (S.F.) against sliding of 1.21, in case of earthquake, was reported as the minimum allowable.

However, there is no description in the above Report of the magnitude of the earthquake coefficient used in the calculations and the basis for its determination. By making back calculations using formula (8-4) as applied to the dam geometry shown in Fig.-8.3, an earthquake coefficient of K = 0.15 was determined. As the uplift pressure was not considered in stability calculations of the dam for the case of maximum water level in the reservoir with earthquake, the earthquake coefficient of 0.15 is considered to be quite appropriate.



Fig— 8.3 Sliding Wedge Analysis

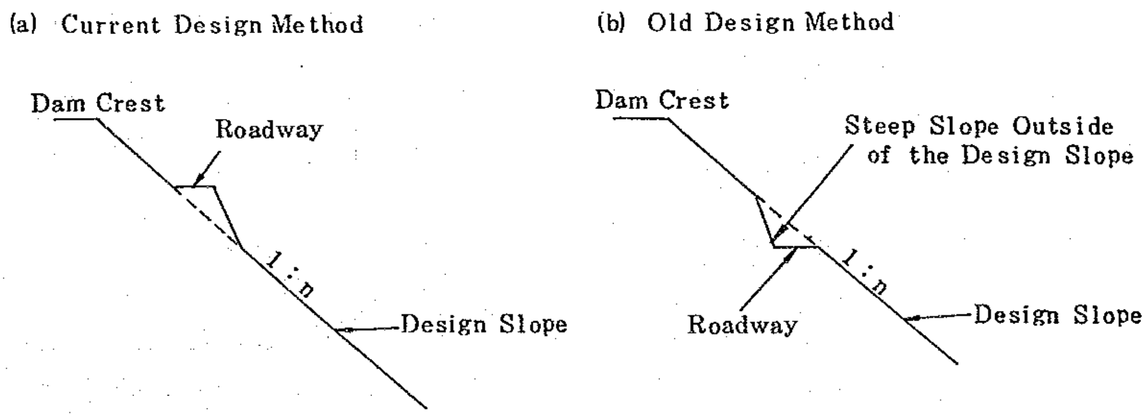


Fig — 8.4 Design Concept of Roadway on the Dam Slope

$$S.F. = \frac{(N_1 - U)\tan\phi_1 + N_2 \tan\phi_2}{P + (N_1 + N_2)K} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8-4)$$

where,

S.F. = Safety factor against sliding
(Minimum allowable in case of earthquake is equal to 1.21)

K = Design coefficient of earthquake

The steep slope caused by the construction of the access road on the downstream face of the dam is considered not to have any relation with the design slope of the dam. (See Fig.-8.3 and Fig.-8.4(b))

The design and construction concept for the access road on the downstream face of the dam, used in the original design, is not applicable any more. This concept results in undercutting the design slope and reduction of the safety factor as originally computed. The proper method which is currently in practice, is to place the road on the outside the theoretical line of the dam required by design. For illustration, see Fig.-8.4 (a).

8.1.4. Stability Analysis of the Upstream Face of the Dam

(1) Basic Considerations

Stability of the upstream face of rockfill dams is analyzed for various operating conditions corresponding to the various stages of construction and operation, such as end of construction stage or some intermediate stage, full reservoir level or steady seepage, and a drawdown stage from full or partial pool. Earthquake effects may be included for the case of a partial or full pool or the end of construction condition.

In case of the Ambuklao Dam, however, it is quite adequate to study the stability of the slope for the case when the reservoir is at maximum water level with the earthquake effects included. This is the case of a maximum loading which will produce minimum safety factor. The other loading cases mentioned above are not relevant, as this is an old dam which has been in service for over 30 years and has been subjected to the above loadings for long time without any adverse effects. For reference purposes only, the loading condition consisting of a low reservoir water and earthquake has been also studied and the results are presented herein.

In the present analysis, the upper end of the sliding surface was assumed to be located on the shoulder of the downstream face. In case of an optional sliding, the surface sliding for which the safety factor was expected to be minimum was selected, and, then, the analysis for each case was carried out.

The physical properties of the embankment materials shown in Table-8.1 were used. The physical properties of the materials for Zone A determined on the basis of soil tests as described in Paragraph 8.1.3, were also used as applicable to these materials.

Section S6 with the steeper average upstream slope was selected for analyzing the cross section of the dam for stability. For analyzing surface sliding, however, section S16 was selected as a section with steepest partial slope (1 vertical to 1.32 horizontal). The flow net of the core zone, shown in Fig.-8.5, drawn on the basis of a

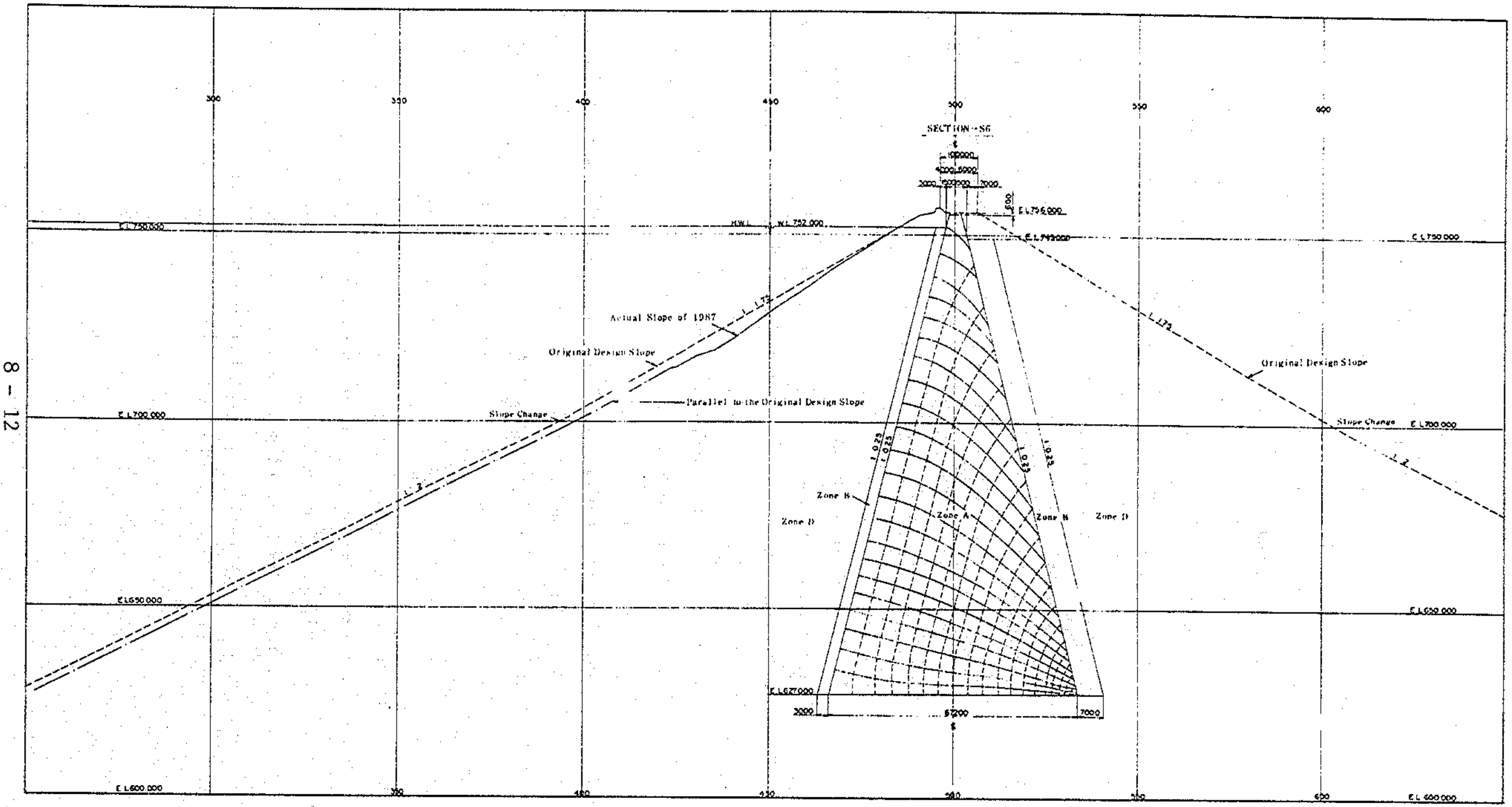


Fig — 8.5 Dam Profile and Flow-Net Applied for Stability Analysis trial and error principles, was used as required for the analyses.

Total of ten cases were analyzed including the surface sliding. They are all presented in Table-8.4. They were calculated for Section S16. For the first six cases, the reservoir water was assumed at maximum level, Elevation 752.0 meters, and the earthquake effect with a coefficient of K = 0.15.

The cases analyzed differ from each other mainly in physical properties of the materials as assumed for the purposes of the analysis. Case 7 was analyzed for the minimum reservoir level, Elevation 694.0 meters. Cases 8, 9 and 10 comprise surface slidings assuming a semi-infinite slope.

Table-8.4

Ambuklao Dam

Upstream Slope

Data on Analyzed Cases

Case Number	Shear Strength of Materials			Coefficient of Seismicity	Remarks
	Core	Filter	Rockfill		
	Cd = 0	Cd = 0	Cd = 0		
1	∅d = 15°	∅d = 35°	∅d = 40°	0	Original Design Condition in 1987
2	Ccu = 0 ∅cu = 15°	- ditto -	- ditto -	0.15	Core zone ∅ cu
3	Ccd=0.377 ∅cd = 29°	- ditto -	- ditto -	0	Soil test results (CD) for core zone
4	Ccu=0.645 ∅cu = 23°	- ditto -	- ditto -	0.15	Soil test results (CU) for core zone
5	- ditto -	- ditto -	To be calculated by back analysis for S.F.=1.0	0.15	- ditto -
6	- ditto -	- ditto -	- ditto - for S.F.=1.2	0.15	- ditto -
7	- ditto -	- ditto -	Cd = 0	0.15	Original design condition at L.W.L.
8	-	-	- ditto -	0.15	Surface sliding condigion
9	-	-	- ditto -	0.15	- ditto - Slope gradient to be calculated for S.F.=1.0
10	-	-	Cd = 0 ∅d = 43°	0.15	- ditto -

Note : CD .... Consolidated - drained condition.
CU .... Consolidated - undrained condition.

(2) Results of Analysis

The results of the analyses performed for the cases listed in Table-8.4 are summarized in Table-8.5.

As can be seen from Table-8.5, Cases 1 and 3, both without seismic effects, give safety factors of 1.478 and 1.484, respectively, which are regarded quite satisfactory.

Table 8-5

Ambuklao Dam

Upstream Slope

Safety Factors against Sliding

Case Number	1	2	3	4	5	6	7	8	9	10
Safety Factor	1.478	0.855	1.484	0.857	-	-	1.083	0.631	-	-
ϕ value of Rockfill	-	-	-	-	*1 44°32'	*2 49°58'	-	-	-	-
Slope Gradient	-	-	-	-	-	-	-	-	*3 1:2.90	*4 1:2.50

Note: *1 K=0.15, S.F.=1.0

*2 K=0.15, S.F.=1.2

*3 K=0.15, S.F.=1.2, ϕ=40°

*4 K=0.15, S.F.=1.2, ϕ=43°

However, Cases 2 and 4 for which seismic effects were considered, as shown in Fig.-8.6, yield safety factors of 0.855 and 0.857, respectively, was below 1.0. For Case

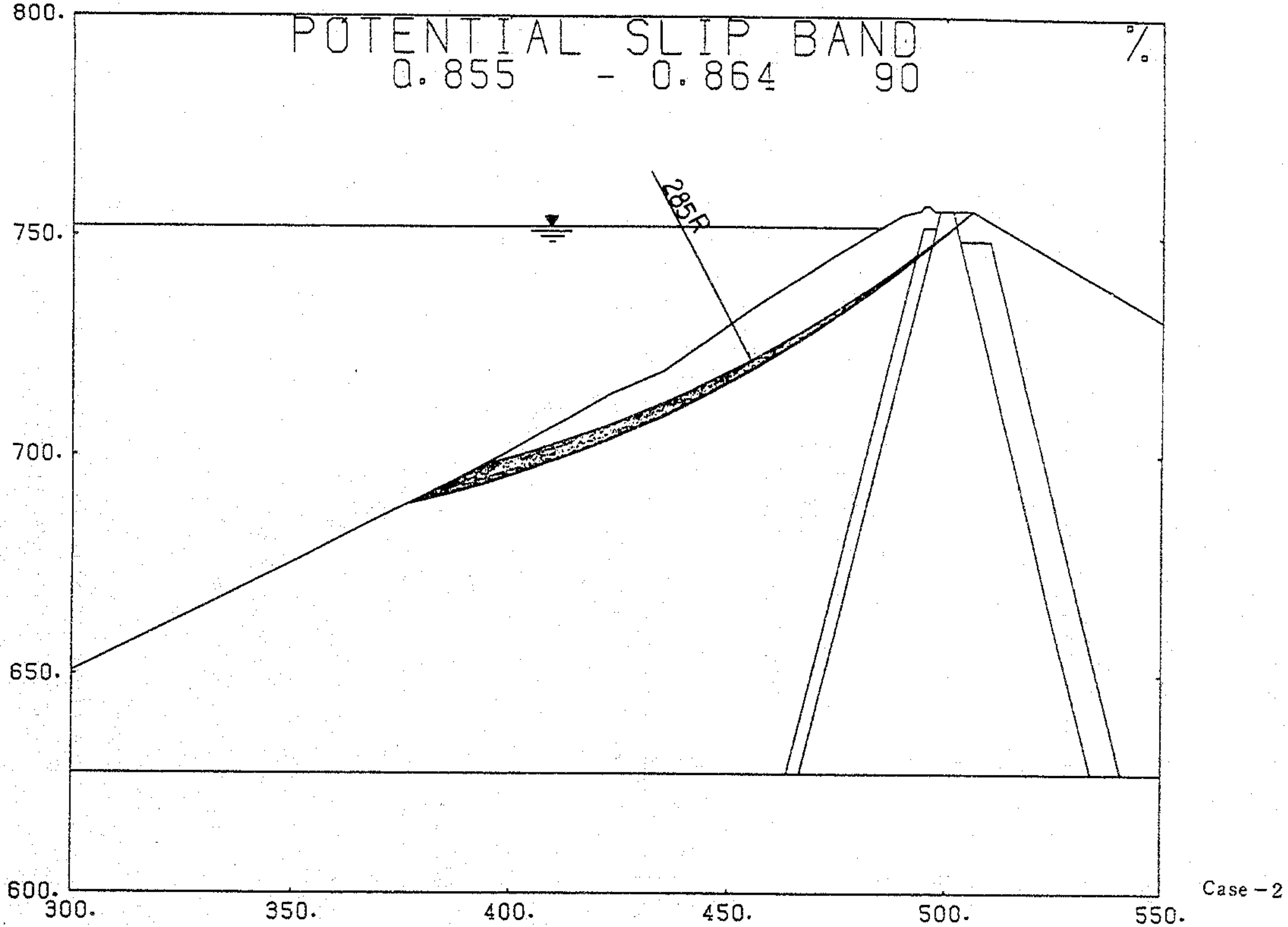


Fig − 8.6 Critical Slip Cirele on the Upstream Slope

5, the value of ϕ for Zone D rock material was obtained by reverse calculation assuming a safety factor of 1.0. The value of ϕd was found to be 44°32'. For Case 6, ϕd was found to be 49°58' assuming a safety factor of 1.2. The value of ϕd was expected to be in the range of 43° to 45° assuming that fine and well compacted materials of a low stress level were actually used, however, ϕd of 49° obtained for Case 6, was not expected.

Case 7 computed for the low reservoir level gives a safety factor of slightly over 1.0. Although the reservoir water level during the earthquake of April 1985 was at EL. 703.00 meters, this loading condition is considered to be almost the same as that of Case 7.

It is well known that the safety factor of a sliding would be minimum when the slope is analyzed as a sliding of the surface portion assuming a semi-infinite slope and that the embankment consists of materials without cohesion, such as pebbles, rock materials, etc. Case 8, for example, yields a safety factor of 0.631 which is well below 1.0. Inspection of the existing conditions of the upstream face of the dam indicates a considerable failure of the riprap at various places along the slope of the dam. The above conditions confirm the results of Case 8 for which the safety against surface sliding was found to be low.

Cases 9 and 10 were studied to determine the requirements supposing the new construction plan. The design gradient of the face will be different as affected by the characteristics of the rock materials to be obtained for the rehabilitation work. Based on the results of the field investigations, the diorite located on the mountain opposite the spillway is probably the most appropriate for use as a quarry site for production of rock materials for the dam. Since these materials are assumed to have the same properties with the rock materials used in the embankment of the dam.

Therefore, before the rehabilitation work is implemented, sampling of the materials from the proposed quarry site and triaxial compaction tests (over ϕ30 cm x h 60cm) should be performed as required to determine the slope gradient most appropriate for use in the rehabilitation work.

As mentioned above, the slope gradient of the Ambuklao Dam, if it be reconstructed, will need 1:2.50 even if the ϕ value of rock materials to be used represents 43°. However, what should be considered further are the fact that the existing dam has been in service for over 30 years and also influences given by the earthquake. It is very important who to take these factors into account in the dam rehabilitation plan.

As indicated on Fig.-8.5, the original design slope was 1:1.75. As of 1987, however, it has not been exactly kept in all the sections of the dam probably due to settlment of the dam having occurred for a long period of time after the construction.

Even if the existing slope be rehabilitated to the design slope of 1:1.75, it is judged practically convenient to adopt 1:1.80 in the execution of the slope restoration works.

In the above case, a safety factor of 1.0 against the surface sliding is obtained when the seismic coefficient (K) is 0.14. The design seismic coefficient of the

Ambuklao Dam, as aforementioned, represents K=0.15 producing a difference of 0.01 compared with the above value. It is generally recognized that there is no direct relationship between the design coefficient of seismicity and the acceleration, but through the various processes of study and considerations, the maximum accelerations can be obtained from the seismic coefficient as shown below:

In case of	K=0.15	Max. acceleration	α = 245 to 367 gal
"	K=0.14	"	α = 229 to 343 gal

In comparison of the maximum accelerations, both cases show more than 200 gal respectively, and it can be said that as far as response spectra of the Ambuklao Dam to earthquake, there is found only little differences between both cases, K=0.15 and K=0.14.

Furthermore, in case the original design slope gradient of 1:1.75 be applied, a safety factor against the slope sliding indicates 1.0 when the seismic coefficient (K) is 0.136 and the maximum accelerations (α) stands in the range of 222 to 332 gal. This case makes little differences in the response spectra with the aforesaid two (2) cases.

Since actual survey map which shows the upstream face conditions of the dam below EL.715.00 m is not available, estimate of rehabilitation costs is based on the assumed face slope from the survey map for the dam face above EL. 715.00 m as indicated in Figs.-12.20 and 12.21. Construction quantities for both slope cases of 1:1.80 and 1:1.75 were calculated on the same condition as mentioned above. Estimated construction costs and quantities are compared as follows:

	Case I	Case II
Face slope	1 : 1.80	1 : 1.75
Embankment incl. incl. riprap	230,000 m^3	188,000 m^3
Total cost (US$)	7,333,000	6,028,000
Return period of allowable earthquake	46 years	38 years

Difference in the cost is about 1,300,000 Dollars. On the other hand, probable return period of allowable earthquake is calculated by obtaining value k corresponding to the slope gradient which yield a safety factor f=1.0 at shear strength of riprap material (ϕ=43.0°). As a result, it is disclosed that the slope of 1:1.80 will be able to withstand for earthquakes of 46 probable years, which is different by 8 years as compared to the slope of 1:1.75.

As a conclusion, however, it is advised to restore the existing slope conceptually according to the original design, but adopt 1:1.80 as a practical slope gradient in view of the conveniences for field embankment work.

8.1.5. Stability Analysis of the Downstream Face of the Dam

(1) Basic Considerations

The various cases studied were selected on the basis of the earthquakes which occurred during the high reservoir water level, i.e., the cases for which the safety factor against sliding of the downstream face would be minimum. For comparison, cases without earthquake were also added to the analysis. The physical properties of the materials were used in accordance with the data given in Tables -8.1 and 8.2.

The analysis was made considering the strength of the core material determined on the basis of two different conditions of mode of load application. There were (a) consolidated drained and (b), consolidated-undrained. These conditions were considered for analysis of cases with earthquake.

The cases analyzed are shown in Table-8.6. The locations of the sections for which the analysis was made are DA10, and Sta.00 + 180 based on the survey conducted in 1985. The sections are shown in Fig.-8.7. The flow-net adopted for the analysis of the upstream face and shown in Fig.-8.3., was also used for the downstream face.

(2) Results of Analysis

The analyzed cases are listed in Table-8.6. The results of the analysis are shown in Table-8.7.

The results of the analysis for Cases 1 and 3, both without seismic effects, show safety factors of 1.347 and 1.396, respectively. These values are regarded to be

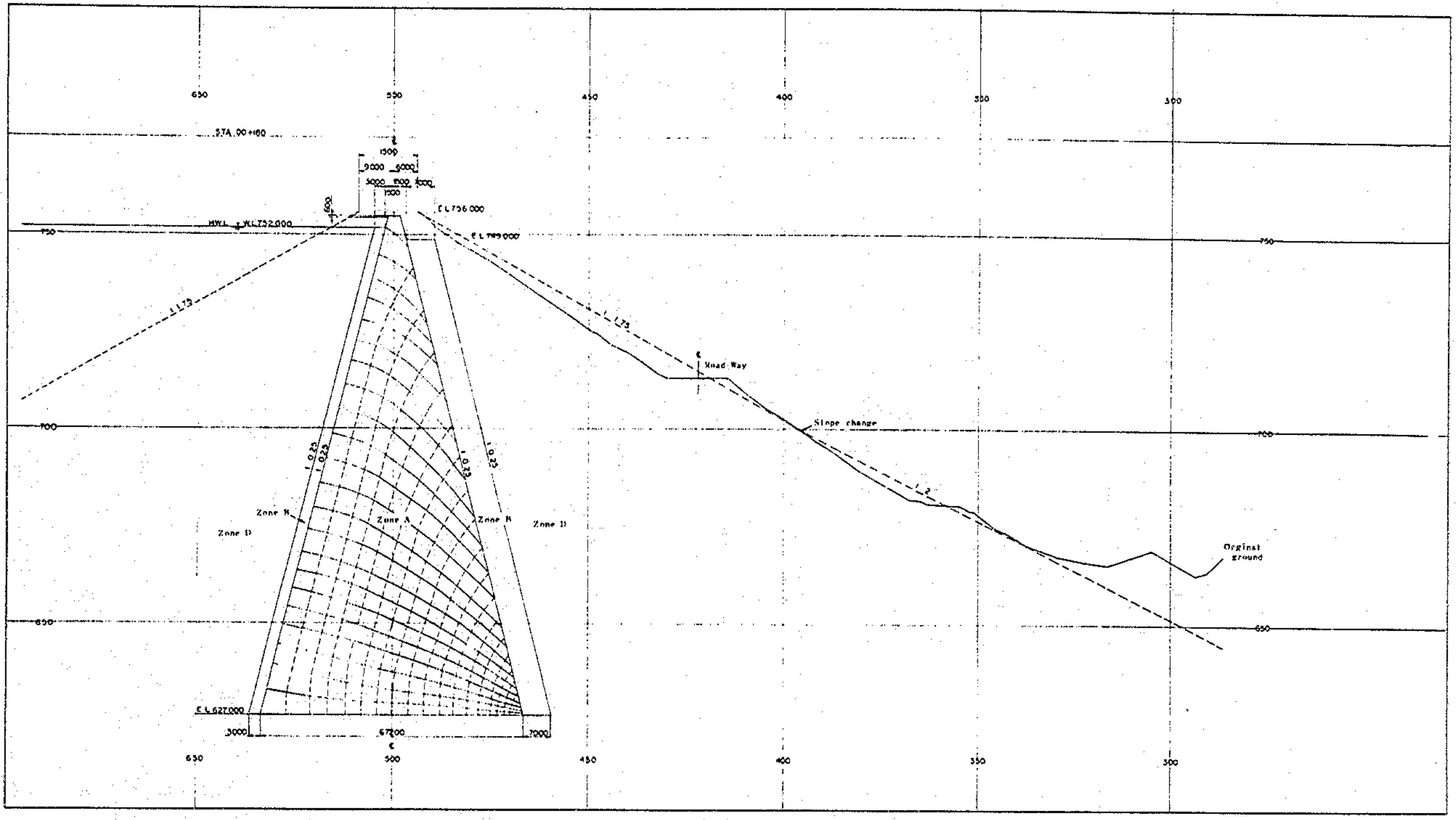


Fig — 8.7 Dam Profile and Flow-Net applied for Stability Analysis

Table 8.6

Ambuklao Dam

Downstream Slope

Data on the Analyzed Cases

Case Number	Location of Sliding Plane	Shear Strength of Materials: Core	Filter	Rockfill	Coefficiont Seismicity	Remarks
1	Point	Cd = 0 ϕd = 15°	Cd = 0 ϕd = 35°	Cd = 0 ϕd = 40°	0	Original Design Condition
2	- ditto -	Ccu = 0 ϕcu = 15°	- ditto -	- ditto -	0.15	Core zone ϕcu
3	- ditto -	Ccd=0.377 ϕcd = 29°	- ditto -	- ditto -	0	Soil test results (CD) for core zone
4	- ditto -	Ccu=0.645 ϕcu = 23°	- ditto -	- ditto -	0.15	Soil test results (CU) for core zone
5	Random	- ditto -	- ditto -	To be calculated so as to obtain S.F.=1.0	0.15	- ditto -
6	Predetermined Point	Cd = 0 ϕd = 15°	- ditto -	Cd = 0 ϕd = 40°	0.15	Original Design condition core zone (CD)
7	- ditto -	Ccd=0.377 ϕcd = 29°	- ditto -	- ditto -	0.15	Soil test result (CD) for core zone
8	Random	Ccu=0.645 ϕcu = 23°	- ditto -	- ditto -	To be calculated so as to obtain S.F.=1.0	- ditto -

Note: CD .... Consolidated and drained condition.

CU .... Consolidated and undrained condition.

satisfactory. The results for Cases 2 and 6, both of which were analyzed with the seismic conditions included, show safety factors of 0.989 and 0.984 (see Fig.-8.8.), respectively. Although they are below 1.0, considering the low value of the angle of friction used, they are also considered to be satisfactory.

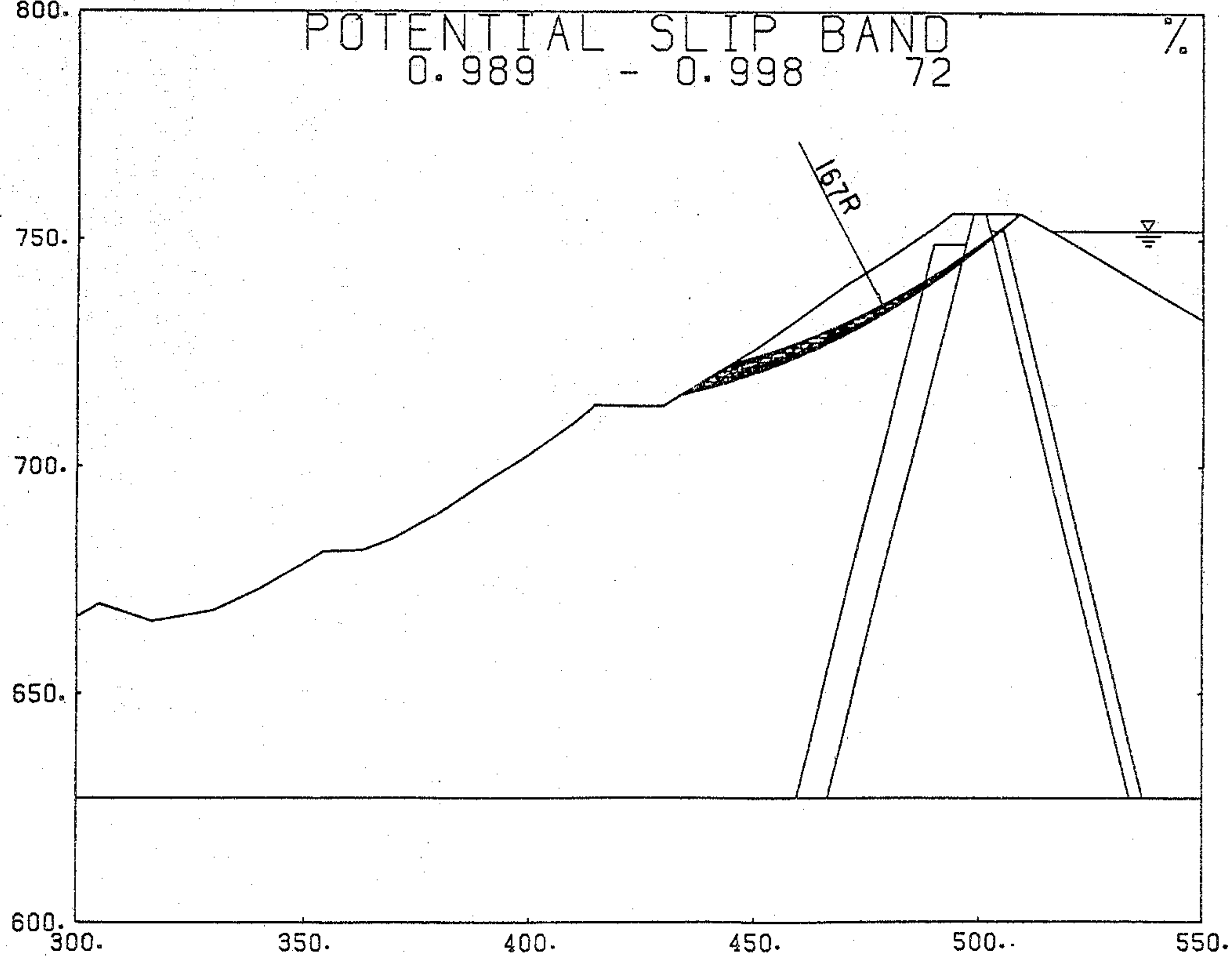
Case 7 which was based on the newly conducted soil tests has yielded a safety factor of 1.021. Comparison of the investigated cases indicate that the differences between Cases 2 and 6, and Cases 4 and 7 are minimal (see Table-8.7). This can be explained by the fact that the length of the sliding plane across the core is very small.

Table-8.7 Sliding Safety Factors on the Dam Downstream Slope

Case No.	Case-1	Case-2	Case-3	Case-4	Case-5	Case-6	Case-7	Case-8
Safety Factor	1.347	0.989	1.396	1.018	-	0.984	1.021	-
ϕ Value of Rockfill	-	-	-	-	40°32'*1	-	-	-
K Value	-	-	-	-	-	-	-	0.14*2

Note: *1 k = 0.15, S.F.=1.0
*2 ϕ Value of Rock=40°, Soil test results for core zone, S.F.=1.0

The angle of friction for the rockfill material for Case 5 was obtained by reverse calculation assuming that the safety factor should be 1.0 at K=0.15. The angle of friction obtained in this manner was found to be 40°32'. Although, no recent test values for the rockfill material in-situ were available, it is assumed from experience that the rock material used for the Ambuklao Dam could have an angle of more than 43°.



Case − 2

Fig − 8.8 Critical Slip Circle on the Downstream Slope

Case 8 was investigated to determine the earthquake coefficient corresponding to the angle of friction for the rockfill material ϕ of 40°, and the safety factor of 1.0. The corresponding earthquake coefficient "K" was 0.14.

Based on the results of the analysis for Cases 1 through 8, the stability of the downstream face of the dam is considered to be generally satisfactory, although there were cases for which the safety factor was found to be about slightly below 1.0. These were earthquake cases for which the seismic coefficient K was assumed equal to 0.15.

As discussed in Section 8.1.4, it is well known that the safety factor is minimum when the stability of the dam is analyzed as a sliding of a portion of the dam near the surface. This design approach was not yet developed in the 1950's.

The progressive failure of downstream slopes of dams is assumed to begin initially at the outer portion and then to progressively develop into sliding of the inner portion. The progress of such failure is closely related to the duration time of an earthquake as of the destructive external force.

8.1.6. Hydraulical Stability of the Dam

To determine the hydraulic stability of an embankment dam, the flow and velocity of the water seeping through the dam and its foundations must be studied.

The hydraulic stability of an embankment dam is impaired when there is an overtopping of the dam, which may occur during the initial stage of impoundment, or in case of an excessively heavy flood. No observations of any damage or partial failure of the dam or its foundations were observed during and after the occurrence of the big flood of 1976 at the Ambuklao.

Since over 30 years have passed since the completion of the construction of the dam, the current conditions of seepage flows through the embankment and dam foundations are considered to be normal and stable.

Judging from the data on the borehole water level movements in PZ-1 and PZ-2, it is clear that the water seeping through the core, exhibits normal behaviour. For the seepage through the foundations, it was found that there was an excellent correlation between the water levels in the reservoir and the amount of seepage through the foundations. It is assumed that the above fluctuations in the amount of seepage through the foundations of the dam have been repeated every year since the commissioning of the project. Therefore, the hydraulic stability of the foundations of the Ambuklao Dam is also considered satisfactory.

8.2. Stability Analysis of the Left Abutment of the Spillway

8.2.1. General

The left abutment of the spillway has a very steep natural slope. However, the abutment has an adequate thickness in the direction of the reservoir water pressure, and is considered to be a stable structure for the condition of a reservoir water pressure. Therefore, the stability of the abutment is controlled by the movement of the ground water through the bedrock.

After the occurrence of the unusual flood in 1976, a drainage system consisting of drilled holes and adits was installed in the abutment area. Therefore, it is assumed that the ground water in the bedrock of the abutment is kept at a fairly low level as compared to that before the installation of the drainage system.

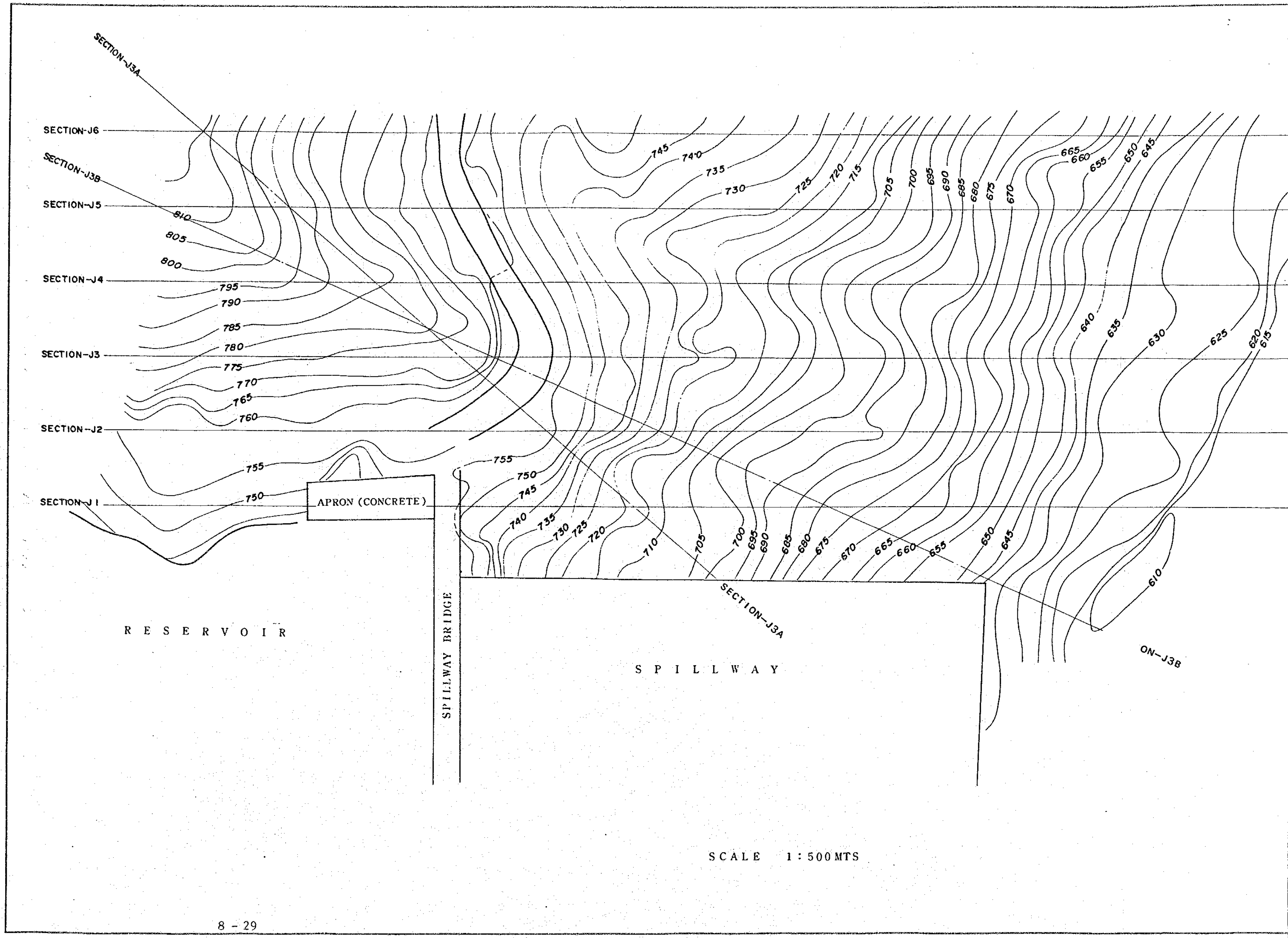
Based on the above considerations, for the stability analysis of the abutment, it was judged to be adequate to study only the stability against sliding of the steep slope of the abutment taking into the account the effects of the ground water. Besides, for safety reasons, it is recommended that periodic observations of the ground water level fluctuations in the bedrock and movement of the bedrock by rock deformeters, be carried out continuously.

Details of the stability analysis of the abutment are given in the following paragraphs.

8.2.2. Stability Analysis

The cross sections to be analyzed were selected on the basis of a ground survey carried out in 1987. Two basic sections were selected. They were (1) a cross section of a very steep slope representative of a relatively shallow steep slope in a limited area, and (2) a cross section of a steep slope representative of a larger area of the abutment. The former is designated Section J3A and is shown in Figs.8.9 and 8.10, and the latter is Section J3B and is also shown in Figs.-8.9 and 8.10. The above two cases are representative of a shallow and a deep seated slides, respectively.

For the analysis, as described above, consideration of ground water conditions was required. For the analysis of the shallow sliding, Section (1) above, the underground water conditions were not considered due to the existence of a very steep slope and the drainage mechanism on the surface. Con-


SECTION-J3A
SECTION-J6
SECTION-J3B
SECTION-J5
SECTION-J4
SECTION-J3
SECTION-J2
SECTION-J1
810
805
800
795
790
785
780
775
770
765
760
755
750
745
740
735
730
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APRON (CONCRETE)
RESERVOIR
SPILLWAY BRIDGE
SPILLWAY
ON-J3B
SCALE 1 : 500 MTS

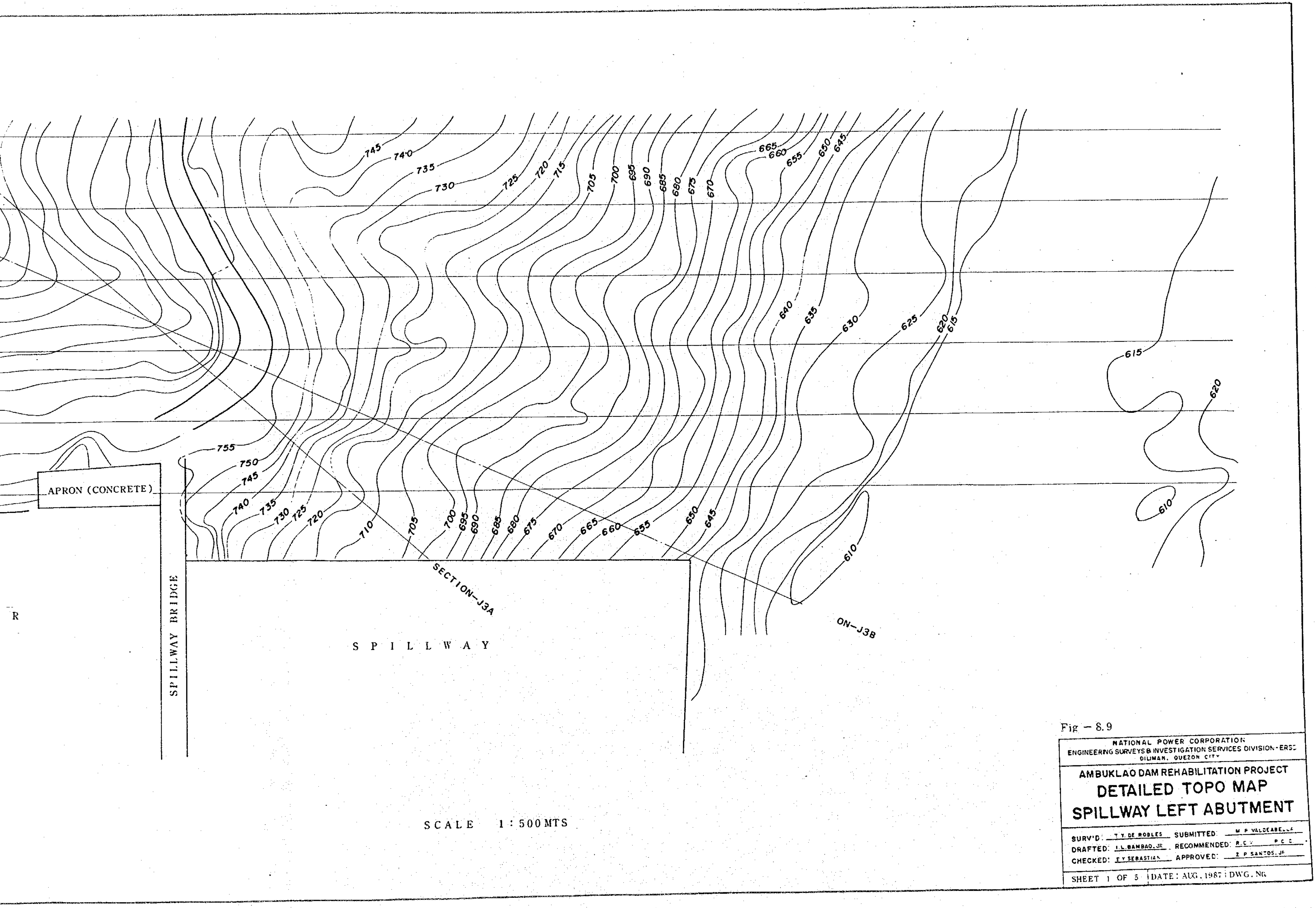
APRON (CONCRETE)
SPILLWAY BRIDGE
SPILLWAY
SECTION–J3A
ON–J3B
SCALE 1 : 500 MTS
NATIONAL POWER CORPORATION
ENGINEERING SURVEYS & INVESTIGATION SERVICES DIVISION - ERSD
DILIMAN, QUEZON CITY
AMBUKLAO DAM REHABILITATION PROJECT
DETAILED TOPO MAP
SPILLWAY LEFT ABUTMENT
SURV'D: T. Y. DE ROBLES
SUBMITTED:
DRAFTED: I. L. BAMBAO, JR
RECOMMENDED:
CHECKED: E Y SEBASTIAN
APPROVED: Z P SANTOS, JR
SHEET 1 OF 5
DATE: AUG. 1987
DWG. No.

Fig – 8.9

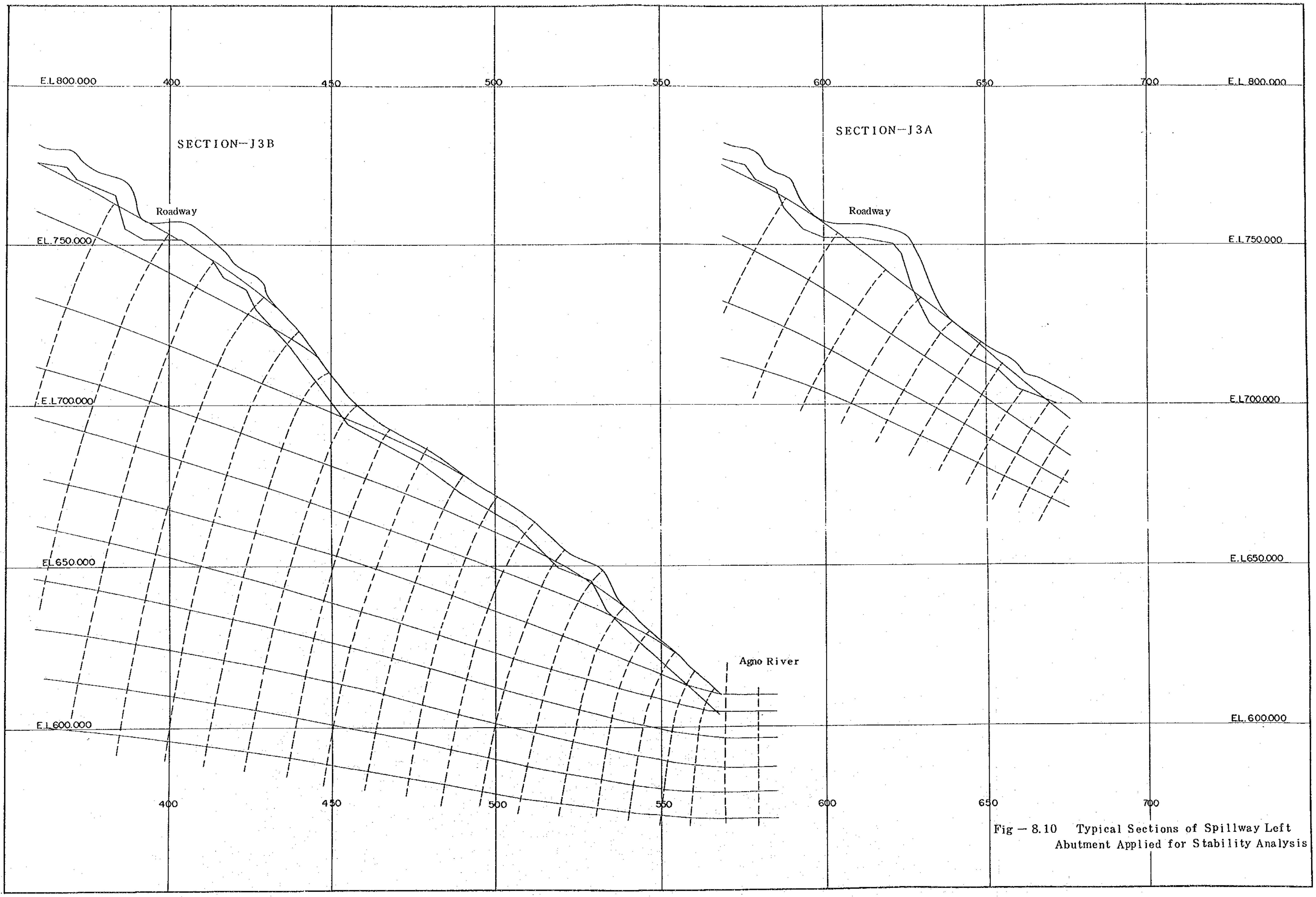

Fig - 8.10 Typical Sections of Spillway Left Abutment Applied for Stability Analysis

sidering, however, the existence of a crack in the bedrock, the seismic effect was added.

For the analysis of the deep-seated sliding, Section (2) above, 50 percent of the ground water was considered to be effective in case of earthquake, and 80 percent, in case without earthquake. It should be added to the above that earthquake and heavy rainfall normally do not occur at the same time.

Tested data on the rock characteristics were not available. Therefore, the values required for the analysis were assumed from experience. It was decided to compute the shear strength of the bedrock consisting of cohesion C and internal friction angle ϕ, for a safety factor against sliding of 1.0.

For the purpose of the analyses, the unit weight of the bedrock by volume was divided into two sections. One was for the part of the slope to about 5 m from the surface, and the other for part of the slope beyond the first 5 meters.

The following unit weights were assumed:

<u>The first 5 meters</u>:

γwet (wetted) = 2.35 g/cm^3

γsat (saturated) = 2.42 g/cm^3

<u>Beyond the first 5 meters</u>

γwet (wet) = 2.40 g/cm^3

γsat (saturated) = 2.45 g/cm^3

8.2.3. Results of Analysis

(1) Shallow Sliding Surface - Section J3A

The shear strength of the bedrock for Section J3A, corresponding to a safety factor against sliding of 1.0, is shown in Table-8.8.

It was found that for a safety factor of 1.0 and internal angle of friction of $\phi = 35°$, the required cohesion of the underlying bedrock was $C = 4.3\ kg/cm^2$ (see Fig.-8.11). The above included the earthquake effects.

Seismicity is normally not taken into consideration where stability analysis of foundation rocks is dealt with. In this case, the result shows a yield of $c=3.1\ kg/cm^2$ at $\phi 35°$.

The left abutment of the spillway geologically consists of metamorphic rocks in the upper layers underlain by diorites. The metamorphic rocks are weathered and cracked. Some cracks are well developed and easily observed from the surface.

After considering the above and studying the conditions in the field, it was decided to base the calculations for the metamorphic rocks on $C = 5\ kg/cm^2$ and $\phi = 38°$. This produced a safety factor against sliding above 1.0.

Table-8.8 Results of Analysis - Shallow Sliding Surface

Case	Conditions of Analysis		Shearing Strength		
	Coeff. of Seismicity	Seepage	$\phi = 35°$	$\phi = 45°$	$\phi = 50°$
1	0	None	$C=3.1kg/cm^2$	$C=2.3kg/cm^2$	$C=1.7kg/cm^2$
2	0.15	None	$C=4.3kg/cm^2$	$C=3.8kg/cm^2$	$C=3.4kg/cm^2$

(2) Deep Sliding Surface - Section J3B

The shear strength of the bedrock material for Section J3B as calculated to be required for a safety factor against sliding of 1.0, is shown in Table-8.9.

For this section, the sliding surface is assumed to be deep-seated and slicing through the underlying diorite of the abutment. The shearing strength of diorite, being a better rock, is assumed to be considerably higher than that of the metamorphic rock. As shown in Table-8.9, for Case 2, which considers stability of the abutment with 80 percent of pore pressure in the bedrock, it is found that the cohesion can be only $C = 5.5\ kg/m^2$ if the safety factor of 1.0 is to be maintained. The assumed sliding surface for this case is shown in Fig.-8.12.

As mentioned previously, it is a normal practice that a seismic condition is not applied for stability analysis of rock foundation. It is, however, useful that physical properties of foundation rocks are confirmed through tests. Generally, it is difficult tasks to obtain their values of high accuracy owing to variations in the scale and accuracy of tests performed and in quality of rocks as well. However, for Ambuklao Dam where drain adits have been provided fortunately, it will be possible to conduct in-situ tests at a proper scale.

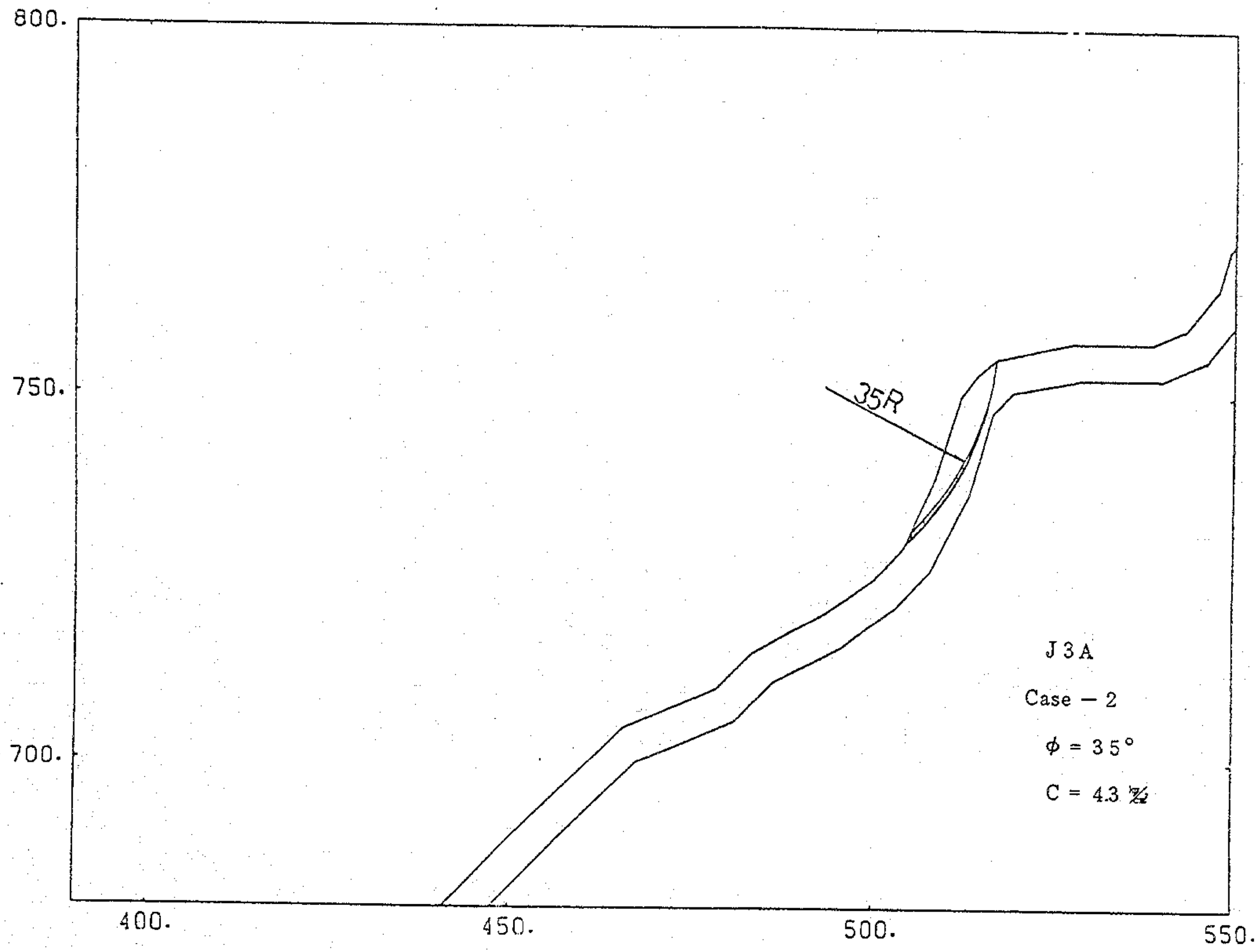

Fig — 8.11 Critical Slip Circle in Case of Shallow Slide

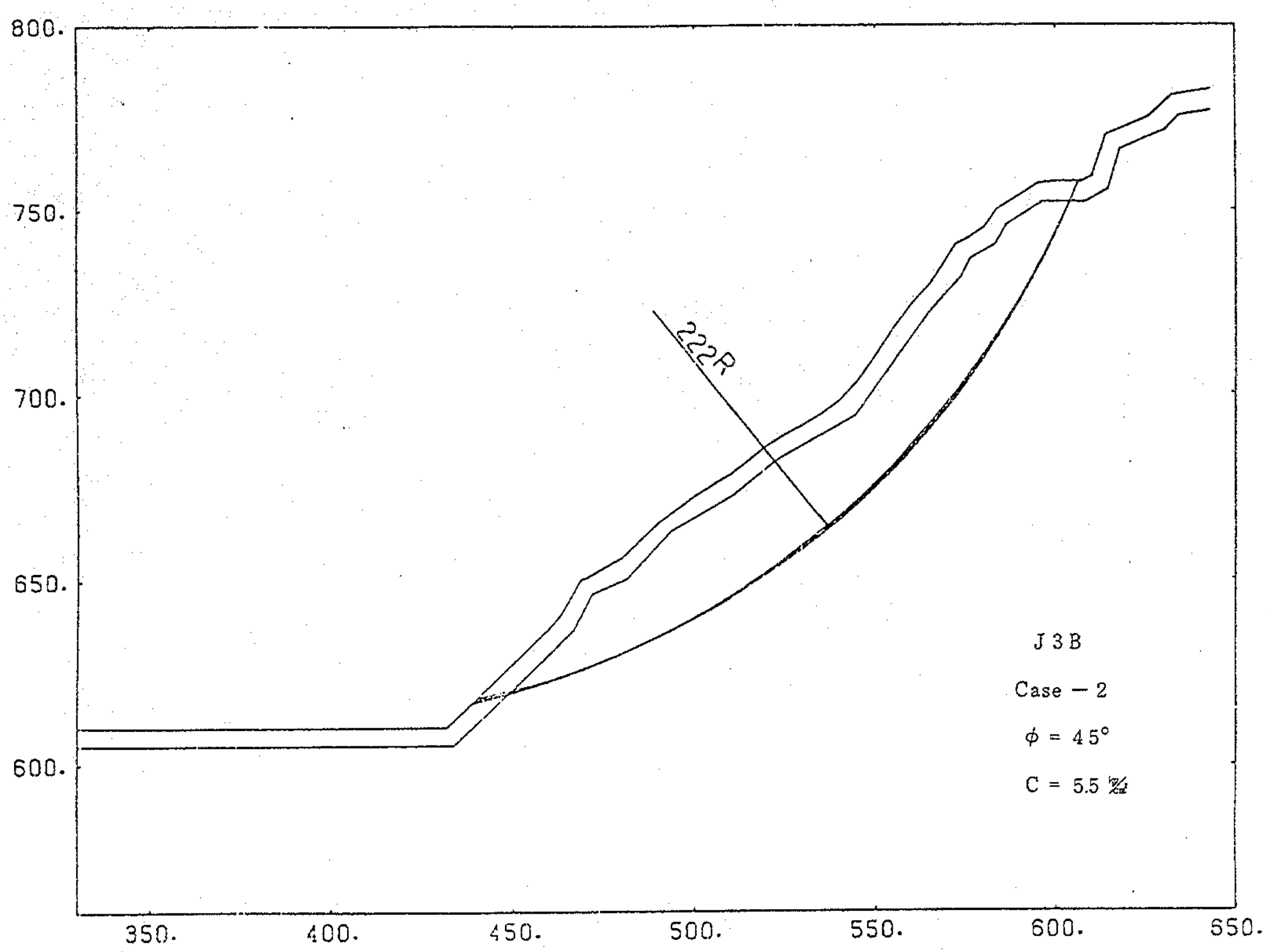

Fig — 8.12 Critical Slip Circle in Case of Deeper Slide

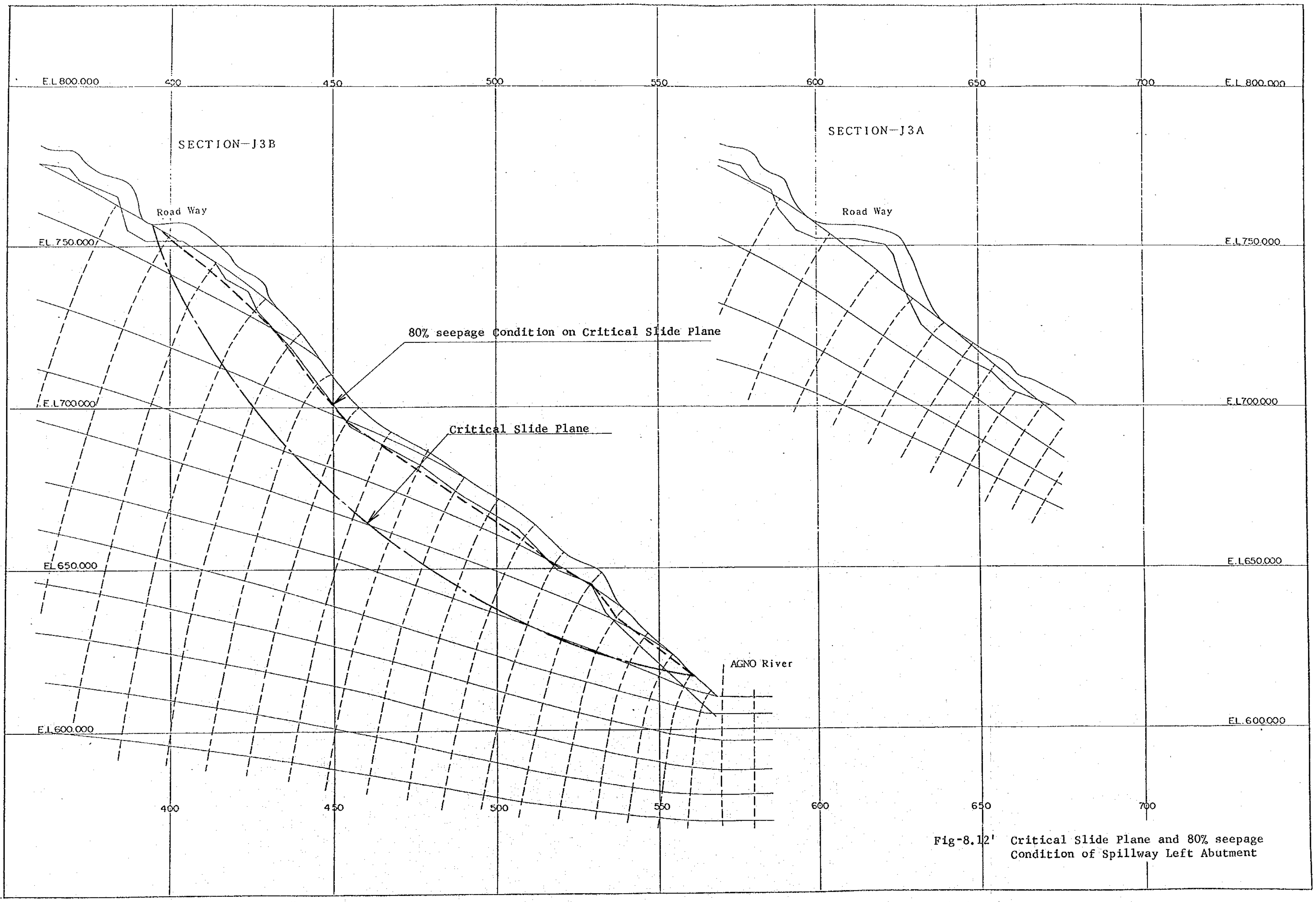

Fig-8.12' Critical Slide Plane and 80% seepage Condition of Spillway Left Abutment

Table-8.9 Results of Analysis - Deep Sliding Surface

Case	Conditions of Analysis		Shearing Strength		
	Coeff. of Seismicity	Seepage	ϕ = 35°	ϕ = 45°	ϕ = 50°
1	0	None	C=3.1kg/cm^2	at C = 0*1	at C = 0*2
2	0	80%*3	C=15kg/cm^2	C=5.5kg/cm^2	C=1.5kg/cm^2
3	0.15	50%	C=20kg/cm^2	C=10 kg/cm^2	C=5.0kg/cm^2

Note: *1 S.F. = 1.071

*2 S.F. = 1.303

*3 Case 2 shall be critical condition and referring to Fig.-8.12'

8.3. Stability Analysis of the Spillway Training Wall

8.3.1. Basic Considerations

The spillway training wall is located on the right hand side of the spillway entrance between the spillway control structure and dam embankment. It is a reinforced concrete structure of an reverse T shape. The spillway consists of five bays with dimensions as shown in the following tabulation:

Bay	Height (m)	Length (m)
1	21.00	11.59
2	21.00	14.47
3	21.00 - 16.05	10.47
4	16.05 - 11.10	10.47
5	11.10 - 6.00	15.00

The bays are separated by vertical contraction joints.

Since the watertightness of these joints was damaged by the earthquake of April 1985, a rehabilitation work on the joints had been carried out. Because of the above damage, it was decided to analyze the stability and the adequacy of the cross-section of the training wall structure.

The analyses carried out in this connection are described below.

8.3.2. Stability Analysis

(1) Conditions of Analysis

The stability analysis was carried out for Bay 1 shown in

Fig.-8.13, which is the largest of the five bays of the spillway control structure.

The stability of the training wall was investigated for overturning, sliding and bearing pressure on the foundation. Normal and seismic loading conditions were considered. In addition to the stability, adequacy of the reinforced concrete design of the structure at the critical points was also investigated. For this purpose, the amount of reinforcement and the size of cross sections of the structure at different locations were checked.

Three cases of load application were studied. They are graphically presented in Fig-8.14. The results of the analysis indicated that there are no big differences between the three cases. The type of analysis performed is indicated in Table-8.10. The geometry of the wall is shown in Fig.-8.13.

Table- 8.10

Cases of Analysis

Case No.	Earth Pressure acting on	Type of Analysis
1	Vertical Plane	Stability for overturning, sliding and bearing pressure. Normal and earthquake conditions.
2	Inclined Plane	
3	Vertical Plane, Front Footing omitted	

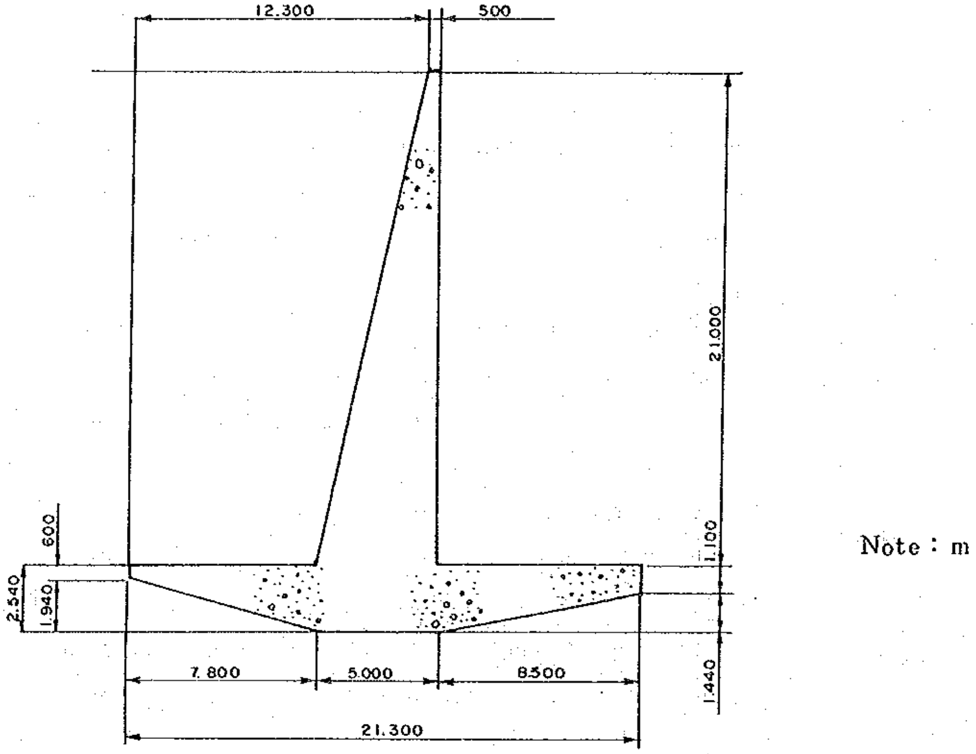

Fig — 8.13 Profile of the Training Wall (Bay 1)

P : Resultant of Earth Pressure

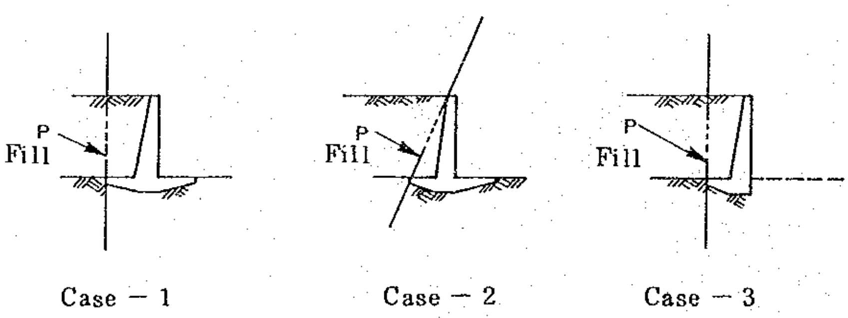

Fig — 8.14 Analysis Cases for Reversed T—shaped Retaining Wall

(2) Formulae

(a) Earth pressure

Earth pressure, normal condition

$$PE = \frac{1}{2} K_A \cdot \gamma \cdot h^2 \quad .... \quad (8\text{-}5)$$

Earth pressure, seismic condition

$$P_{EA} = \frac{1}{2} K_{EA} \cdot \gamma \cdot h^2 \quad .... \quad (8\text{-}6)$$

where,

P_E = Active earth pressure (normal condition, t/m)
P_{EA} = Active earth pressure (seismic condition, t/m)
K_A = Coefficient of active earth pressure in Coulomb's formula (normal condition)
K_{EA} = Coefficient of active earth pressure in Coulomb's formula (seismic condition)
γ = Unit weight by volume of soil (t/m^3)
h = Height of wall of backfill m

(b) Coulomb's coefficient of active earth pressure

Normal and Seismic Conditions

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta \cdot \cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\cos(\phi+\delta)\cos(\phi-\alpha)}}\right]^2} \quad ...... \quad (8\text{-}7)$$

$$K_{EA} = \frac{\cos^2(\phi-\theta_0-\theta)}{\cos\theta_0 \cdot \cos^2\theta \cdot \cos(\theta+\theta_0+\delta)\left[1+\sqrt{\dfrac{\sin(\theta+\delta)}{\cos(\theta+\theta_0+\delta)} \cdot \dfrac{\sin(\phi-\alpha-\theta_0)}{\cos(\theta-\alpha)}}\right]^2} \quad .... \quad (8\text{-}8)$$

where,

ϕ = Angle of internal friction of soil, degree
θ = Angle of the back of the wall with the vertical plane (°)

θ_0 = Seismic composite angle = $\tan^{-1} K_h$

K_h = Coefficient of horizontal seismicity

δ = Friction angle of the wall surface (°)

α = Angle of the backfill surface with the horizontal plane (°)

(c) Eccentricity of Loading

$$e = \frac{B}{2} - d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8\text{-}9)$$

where,

e = Eccentricity of loading, or the distance from the center of gravity of the base of the point of intersection of the resultant with the base.

B = Width of footing (m)

d = Horizontal distance from the point of origin (rotation) to the point of intersection of the resultant with the base This is equal to $(\Sigma M_V - \Sigma M_H) / \Sigma P_V$

ΣM_V = Summation of moments of all vertical forces about the point of origin (t-m/m)

ΣM_H = Summation of moments of all horizontal forces about the point of origin.(t-m/m)

ΣP_V = Summation of all vertical forces (Total vertical force. (t/m)

For stability, the following requirement should be satisfied:

$e \leqq B/6$

(d) Safety Factor

The safety factor against sliding was calculated using the following formula:

$$S.F. = \Sigma P_V \tan \phi_B / \Sigma P_H \quad \dots\dots\dots\dots\dots\dots \quad (8\text{-}10)$$

where,

$S.F.$ = Safety factor against sliding

ϕ_B = Coefficient of friction between concrete and foundation rock

(e) Foundation Bearing Pressure

For $e = 0$

$$q_{max} = \Sigma P_V / B \quad \dots\dots\dots\dots\dots\dots \quad (8\text{-}11)$$

For $e \leq B/6$

$$q_{max} = \Sigma P_V / B \left(1 + \frac{6 \cdot e}{B}\right) \quad \dots\dots\dots\dots\dots\dots \quad (8\text{-}12)$$

For $e > B/6$

$$q_{max} = \frac{2 \ \Sigma P_V}{3d} \quad \dots\dots\dots\dots\dots\dots \quad (8\text{-}13)$$

where, q_{max} = Maximum bearing pressure (t/m^2)

f) Allowable Bearing Loading

$$\frac{Qu}{F_B} = A' \left\{ \alpha \cdot K \cdot C \cdot Nc + K \cdot q \cdot Nq + \frac{1}{2} \gamma_1 \cdot \beta \cdot B' \cdot N\gamma \right\}$$

$$\dots\dots\dots\dots \quad (8\text{-}14)$$

where,

Q_U = Maximum bearing load of foundation (t).

F_B = Safety factor, assumed 3 for normal loading condition, 2 including seismic effects.

C = Cohesion between concrete and foundation rock (t/m^2)

q = Surcharge (t/m^2) = $_2 D_f$

A' = Effective foundation area (m^2)

$\gamma 1$ = Unit weight by volume of foundation materials (t/m^3)

$\gamma 2$ = Unit weight by volume of penetration part of foundation

B' = Effective loading area including eccentricity = B - 2e

B = Width of foundation (m)

e = Eccentricity of loading (m)

D_f = Effective depth of foundation penetration (m)

α, β = Coefficient of foundation shape (in case of a long and narrow shape = 1.0, β = 1.0)

K = Coefficient of penetration effect equal

to $1 + 0.3 \dfrac{D'f}{B'}$

Nc, Nq, Nr = Bearing force coefficient considering the resultant loading

(3) Results of Analysis

The results of the analysis described above are summarized in Table-8.11. All loading conditions with and without earthquake effects have yielded very satisfactory results. This includes the safety against overturning, sliding and allowable foundation bearing pressure.

For the foundation bearing pressure, adequate safety factors both, for normal operating and seismic conditions, were obtained assuming $C = 0.51\ kg/cm^2$ and $\phi=40°$. The above values for C and ϕ are considered to be on the safety side considering the shearing strength of the foundation. Since these safety factors are for the normal allowable bearing pressures on the foundation, the safety factors for limit bearing pressures will be 3 for normal loading condition, and 2 for seismic condition.

The allowable safety factors were assumed to be 1.50 for normal loading condition, and 1.30 for cases with earthquake.

Table- 8.11
Spillway Training Wall
Stability Analysis

Case No.	Each Pressure acting on:	Stability for					
		(1) Overturning		(2) Sliding		(3) Bearing Pressure	
		Normal Condition	With Earthquake	Normal Condition	With Earthquake	Normal Condition	With Earthquake
				S.F.	S.F.	S.F.	S.F.
1	Vertical Plane	Stable	Stable	5.13	1.77	3.25	2.28
2	Inclined Plane	Stable	Stable	3.46	1.36	2.63*1	1.82*1
3	Vertical Plane, front footing omitted	Stable	Stable	4.85	1.71	5.20*2	1.49*2

Note: *1 For C = 0.5 kg/cm^2, ϕ=30°
*2 For C = 0.5 kg/cm^2, ϕ=40°

8.3.3. Structural Analysis

(1) Vertical Stem

Structural analysis of the training wall was carried out to determine the adequacy of concrete and reinforcement design. For this purpose, a horizontal section through the stem of the wall at its connection with the footing was structurally analyzed. This is section A-A in Fig.-8.15. The earthquake effects were included. The wall was analyzed for backfill pressures as shown in Fig.-8.15.

The shearing force and the bending moment acting on Section A-A, the horizontal plane between the vertical stem of the wall and its footing, were computed to be as shown below:

- Shearing force on Section A-A

This load consists of a load due to own weight plus earth pressure, and is equal to:

= 21.66 + 155.56
= 177.22 t/m

- Bending moment on Section A-A

The bending moment on Section A-A is also caused by own weight plus earth pressure from backfill, and is equal to:

= 165.38 + 1,088.92
= 1,254.30 t-m/m

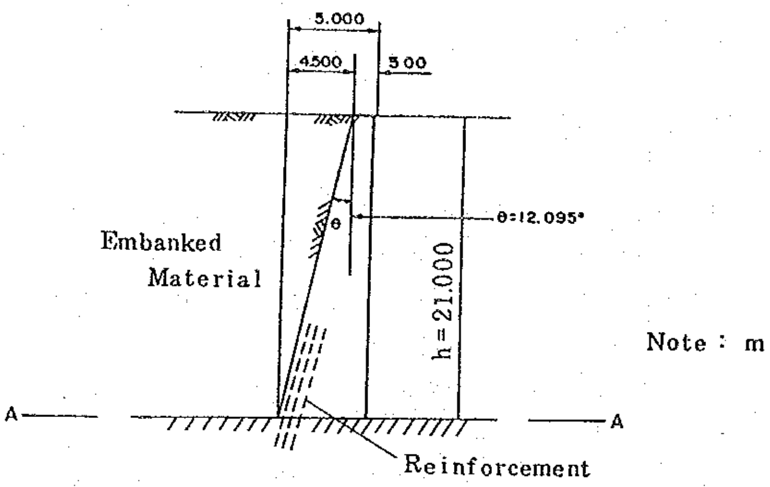

Fig-8.15 Section Wall.

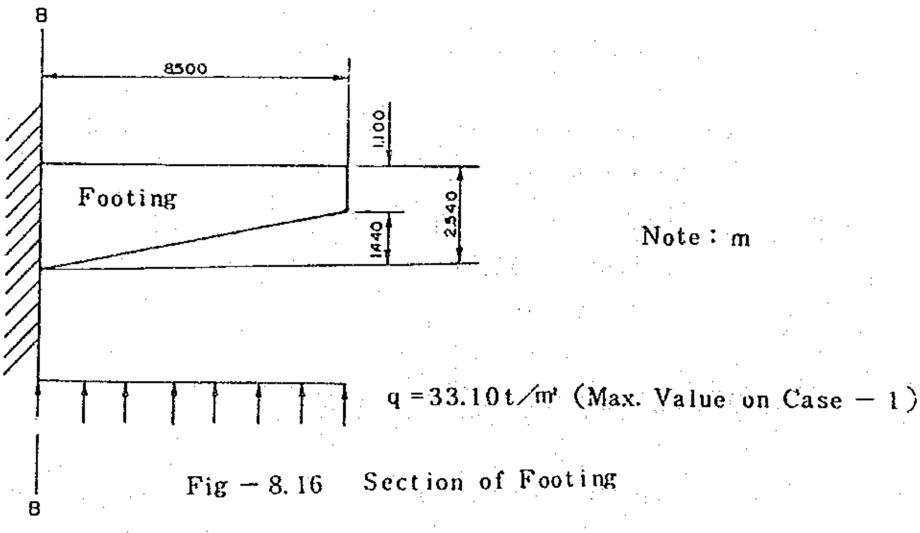

Fig − 8.16 Section of Footing

The stresses due to the above were calculated using formulae (8-15), (8-16) and (8-17), as shown below:

(a) Compressive stress:

$$\sigma c = \frac{2 \cdot M}{K \cdot j \cdot b \cdot d^2} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (8\text{-}15)$$

(b) Shearing stress:

$$\tau c = \frac{S}{b \cdot j \cdot d} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (8\text{-}16)$$

(c) Tensile stress in the reinforcement:

$$\sigma s = \frac{M}{As \cdot j \cdot d} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (8\text{-}17)$$

where,

M = Bending moment = 1254.30 t-m/m

j = $1 - \frac{K}{3} = 0.9032$

K = Neutral axis ratio = $\sqrt{2\ pn + (pn)^2} - pn$

= 0.2903

n = Young's modulus ratio = 15

p = Reinforcement ratio = $\frac{As}{b\ d}$ = 0.003959

As = Section area of reinforcement

= #10 BAR @125 mm x 3 lines

= 190.02 cm^2

b = Width of investigated concrete cross section = 100 cm

d = Effective height of above section = 500 - 20 = 480 cm

S = Shearing force = 177.22 t/m

The stresses in Section A-A of the wall were calculated using the above formulae and loading conditions. The results were:

$$\sigma_c = 41.5\ kg/cm^2,\ \tau_c = 4.1\ kg/cm^2,\ \text{and}\ \sigma_s = 1.523\ kg/m^2$$

- σ_c and τ_c are compressive and shear stress in the concrete, respectively, and
- σ_s = tensile stress in the reinforcement.

The level of the above stresses are all judged to be acceptable, and, thus, no problem is expected because the tensile stress in the reinforcement is lower than the allowable (σsa = 1,600 kg/cm^2)of the training wall.

(2) Footing

For the structural analysis of the footing, it is adequate to verify the compressive and tensile stresses of the concrete, and the tensile stresses of the reinforcement in the vertical section of the joint between the vertical wall stem and the footing. This is Section B-B shown in Fig.-8.16.

- Shearing force on Section B-B

This force consists of a load due to own weight plus foundation reaction, and is equal to:

$$= -38.68 + 281.35 = 242.67\ t/m$$

- Bending moment on Section B-B

The bending moment acting on this section consists of a bending moment due to own weight plus the bending moment due to foundation reaction.

The total moment is equal to:

$$= -142.72 + 1195.74 = 1053.02 \text{ t/m}$$

The corresponding stresses were computed using formulae (8-15), (8-16) and (8-17) shown above.

The section area of the reinforcement actually used is As = 158.34 cm^2. The reinforcement is 3 lines composed of #10 bars spaced at 150 mm. The effective height of the cross section was calculated using the cross section of the foundation as actually excavated.

The analysis produced the following results:

$\sigma c = 62.4 \text{ kg/cm}^2$, $\tau c = 7.7 \text{ kg/cm}^2$ and $\sigma s = 2,116.5 \text{ kg/cm}^2$.

The above concrete compressive stress is well within the allowable limits. The tensile stress in the reinforcement is adequately below the yield point (2,400 kg/cm^2).

Therefore, a safety factor against the yield point was obtained as 2,400/2,116.5, that is 1.13 showing more than 1.0.

As the training wall is an existing structure which has been in service for over 30 years, the above values are considered acceptable.

8.4. Comments on the Finite Element Studies for the Ambuklao Dam, Final Report, Salvador F. Reyes, Consultant, Sep. 24, 1985

After the earthquake of April, 1985, a dynamic analysis for the Ambuklao Dam had been performed by F.E.M., Finite Element Method, by Dr. Salvador F. Reyes, a private consultant from the

Philippines. The Report on the above dam analysis, entitled as shown above, was prepared and presented to NAPOCOR.

Comments on this Report are presented below:

(1) It is difficult to set up a model where distribution of the material characteristics will be the same or similar to the those of the real structure as the element distribution of the model is rough (total of 35 elements). Finer element distribution is required at the upper sections of the dam.

(2) The characteristics of the dam embankment materials were divided into two parts, one was for the core and the other for the rockfill. The model can be improved if the characteristics of the filter zone and the differences of the characteristics within the dam embankment as affected by depth, were also introduced.

(3) Characteristics of core zone materials. Although consolidation, triaxial, compaction and direct shear tests were carried out using undisturbed samples, the results of these tests should be consiously evaluated, as they were not dynamic, but static tests. There is no description in the Report on how the test results were applied. It is assumed, however, that the test results were used carefully, properly and effectively.

(4) Characteristics of rockfill materials. Poisson's ratio was fixed at 0.3. Fig.-8.17 shows the distribution of Poisson's ratio of two rockfill dams in Japan, as a reference.*1 Besides, Young's modulus E for rockfill zone was computed by the following formula:

---

*1 "Study on Stability Analysis, Design and Construction of Rockfill Dams" by Tsuguo HARADA, 1977.

$$E = K \cdot P \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8\text{-}18)$$

where,

K = Constant value = 85 (assumed)
P = Average normal stress

The value of K in formula (8-18), above, has been determined using the horizontal displacement of the dam crest caused by rising of the water level to EL 740.00 m during the period from 1955 to the beginning of 1966. Therefore, it can be assumed that this value was determined on the basis of the displacement obtained during the initial impounding. The displacement characteristics of rockfill dams between initial impounding and after repeated impounding (See Fig.-8.18), are very much different. Therefore, it is not advisable to use these data as shown above. The displacement data of the later periods should have been included. In addition, it is not clear how the average normal stress P was calculated.

(5) Although the occurrence of the crack on the dam crest is indicated by the mode of vibration, it is doubtful whether the prediction of a crack occurrence should be based on its mode above.

(6) Predominant period of a rockfill dam. If the cross section of the dam is assumed to have a triangular shape and is assumed to possess a ideal elastic body with isotropic and homogeneous, the period of first mode of shearing may be indicated by the following formula :

$$T = 2.61 \frac{H}{Vs} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8\text{-}19)$$

where,

T = Period of first mode (sec)
H = Height of dam = 129 m
Vs = Velocity of shearing wave = 500 m/sec (assumed)
V = Poisson's ratio = 0.35 (assumpted)

The period calculated by formula (8-19) is T = 0.67 sec. In addition, the period calculated by the use of the experimental formula of Messrs. Okamoto and Tamura, shown below, is T = 0.65 sec.

$$T = 0.5 \times \frac{H}{100} \quad \cdots\cdots\cdots\cdots \quad (8\text{-}20)$$

The period as calculated and shown in the above Report is T = 3.23 sec, five times bigger than the predominant periods calculated by formulae (8-19) and (8-20), above.

The shearing velocity of the dam materials calculated by formula (8-19), above, and using T = 3.23 sec., is Vs ≑ 100 m/sec. It is difficult to accept such a small value for shearing wave velocity, not only for the Ambuklao Dam which was built 30 years ago, but also for any rockfill dam at the time of impounding. Therefore, it is assumed that it must have been an error in this part of the analysis.

Doubts also exist with regard to the calculation of the second mode and the following mode.

The periods of first shearing mode in the upstream-downstream direction for embankment dams due microtremor, vibration test by blasting, earthquake observations, etc., are shown in Fig.-8.19. The value of T = 0.65 or 0.68 sec., for the Ambuklao Dam, based or the above data (see Fig.-8.19*3) appears to be appropriate. This further reinforces the view that value of T = 3.23 sec. is something out of the ordinary, and, therefore, it thus should be reevaluated.

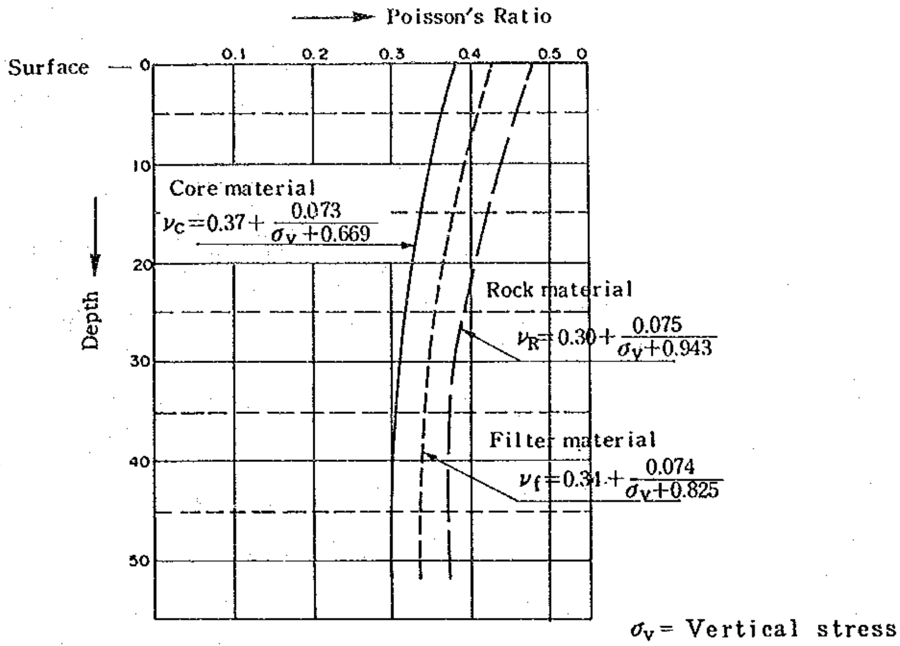

Fig – 8.17 Typical Example of Poisson's Ratio on the Existing Rockfill Dam*1

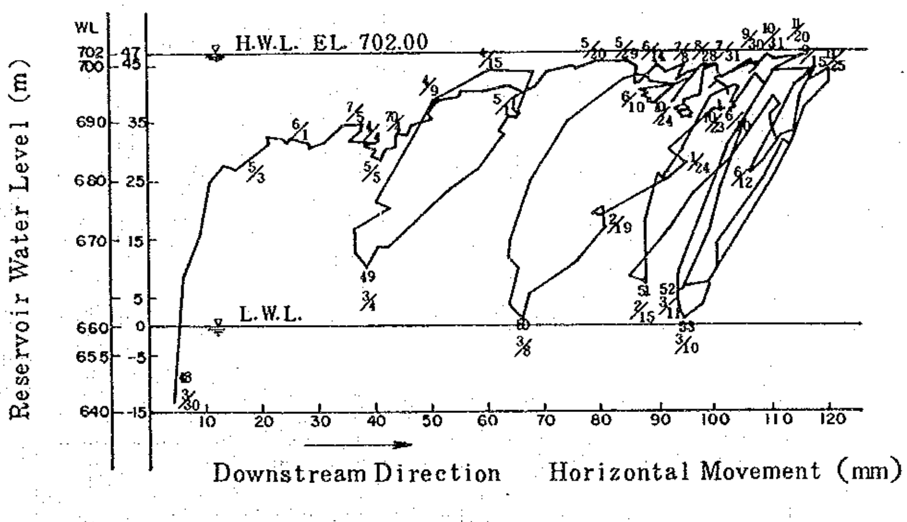

Fig – 8.18 The Plotting of Horizontal Movement at the Crest of a High Rockfill Dam*1

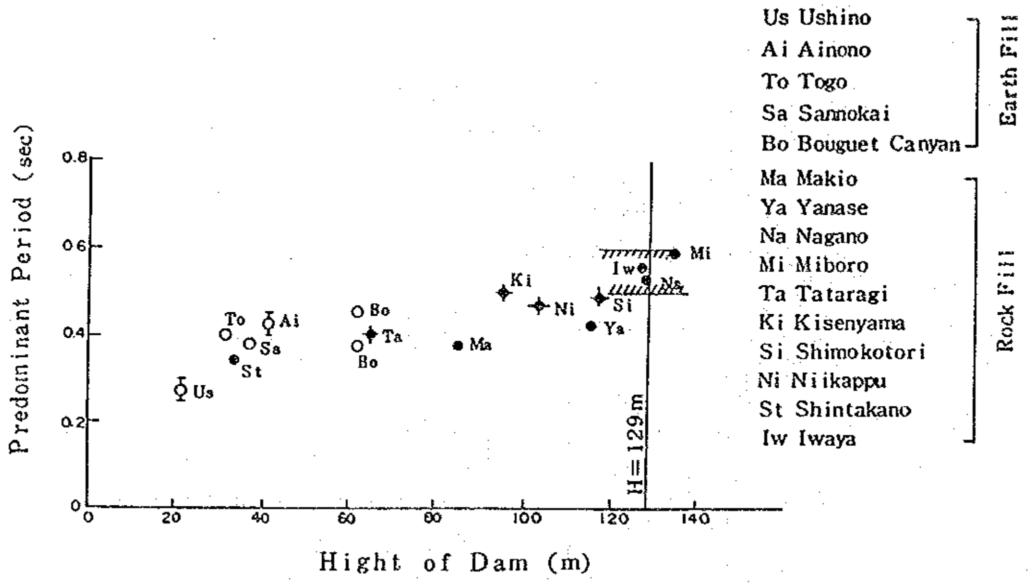

Fig – 8.19 Predominant Period of Fill Dams*2

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*2 "The Distribution Characteristics of Material Properties and Dynamic Behaviour of Rockfill Dams" by Yoshiro Sawada, eal., Central Research Institute of Electric Power Industry, Report No. 377008, Nov.1977.