No. 9

THE GOVERNMENT OF THE REPUBLIC OF THE PHILIPPINES

REPORT ON STUDY

OF

ANGAT DAM REHABILITATION PROJECT

SUMMARY

MARCH 1989

JAPAN INTERNATIONAL COOPERATION AGENCY

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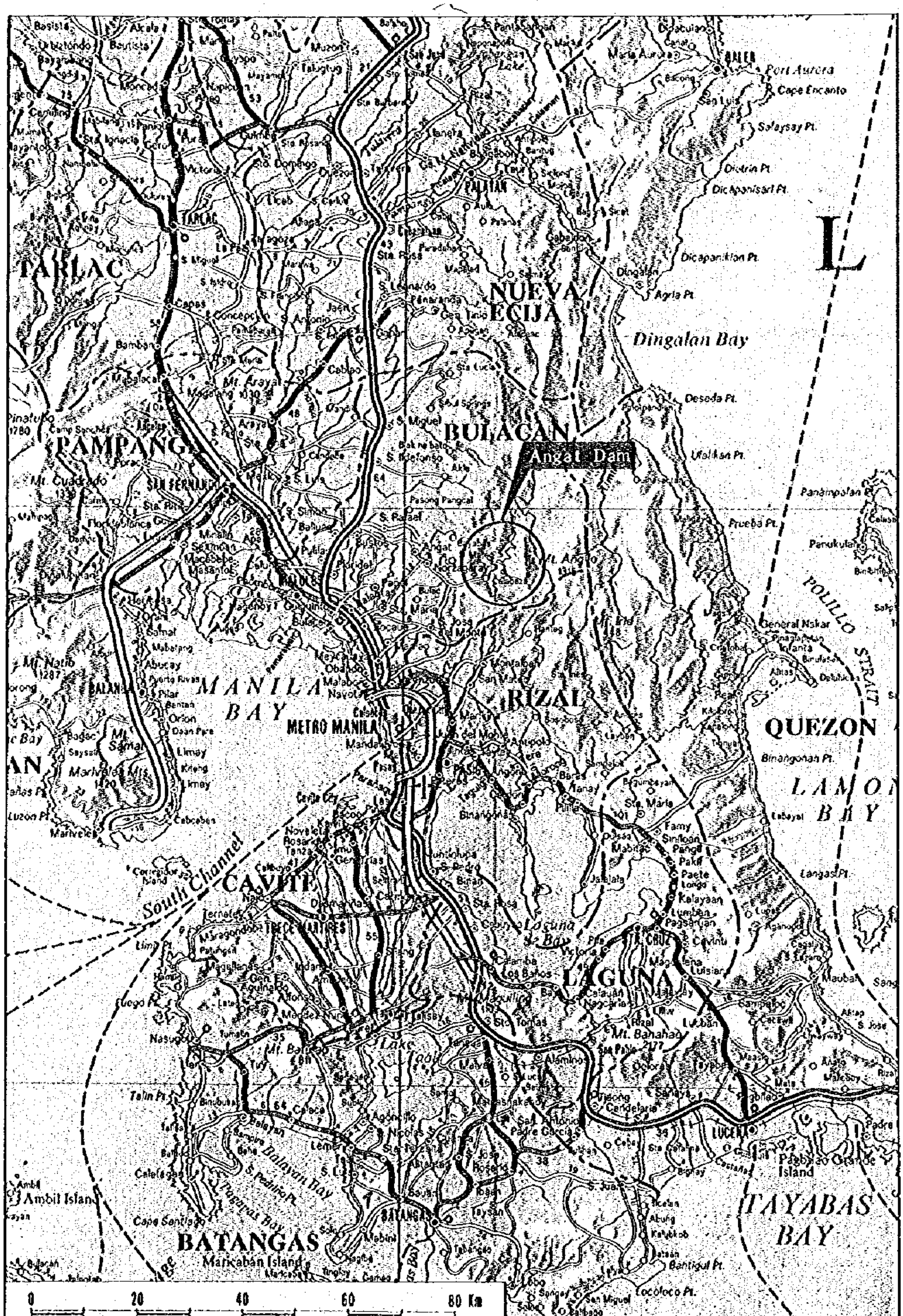
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BATANGAS
Laguna de Bay
Dingalan Bay
LAMON BAY
TAYABAS BAY
POLILLO STRAIT
South Channel
Balayan Bay
Pagapas Bay
Port Aurora
Cape Encanto
Dingalan
General Nakar
Infanta
Mt. Angat
Mt. Arayat
Mt. Natib
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Lucena
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Ambil Island
Pagbilao Grande Island
San Fernando
Tarlac
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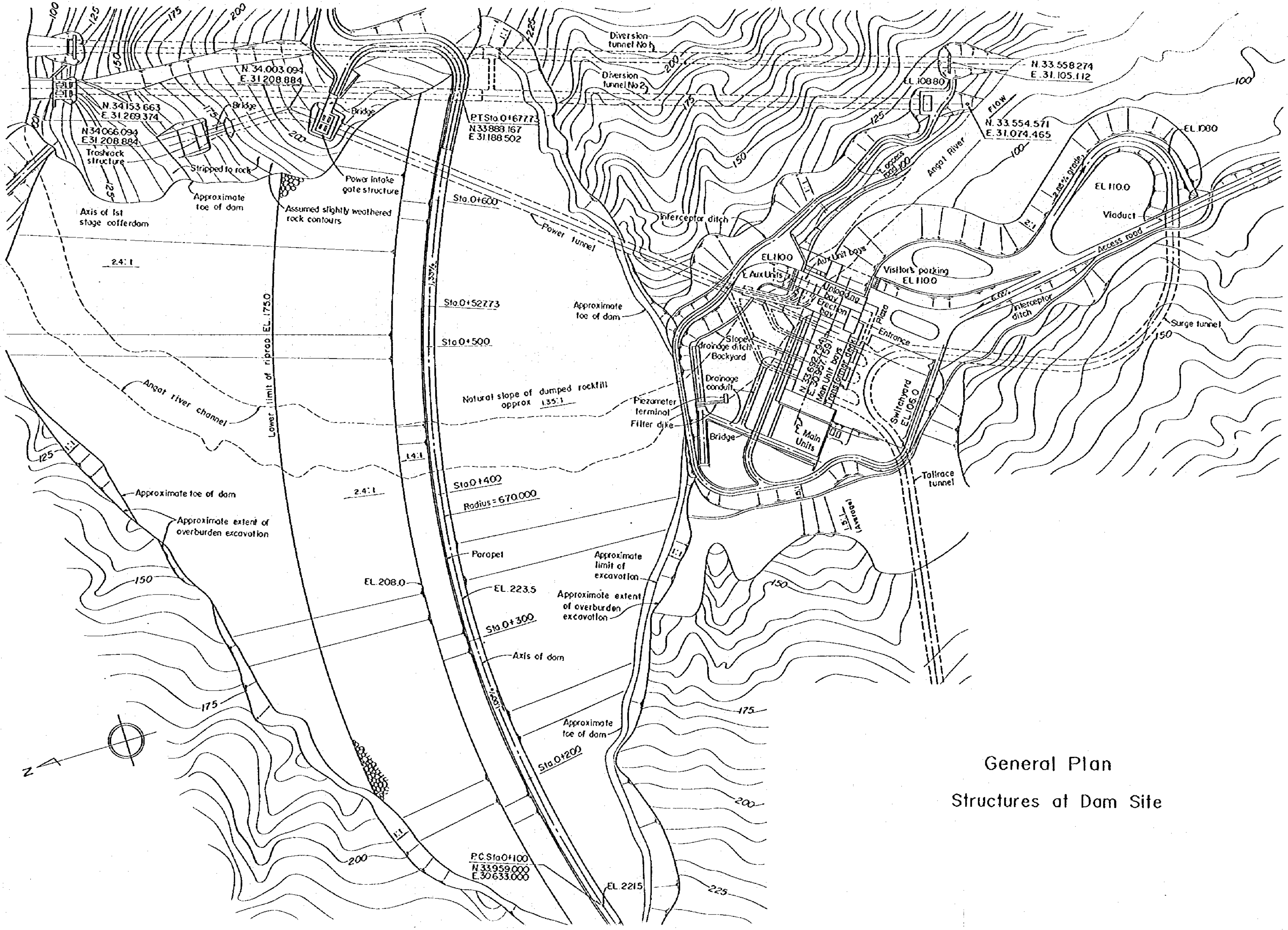
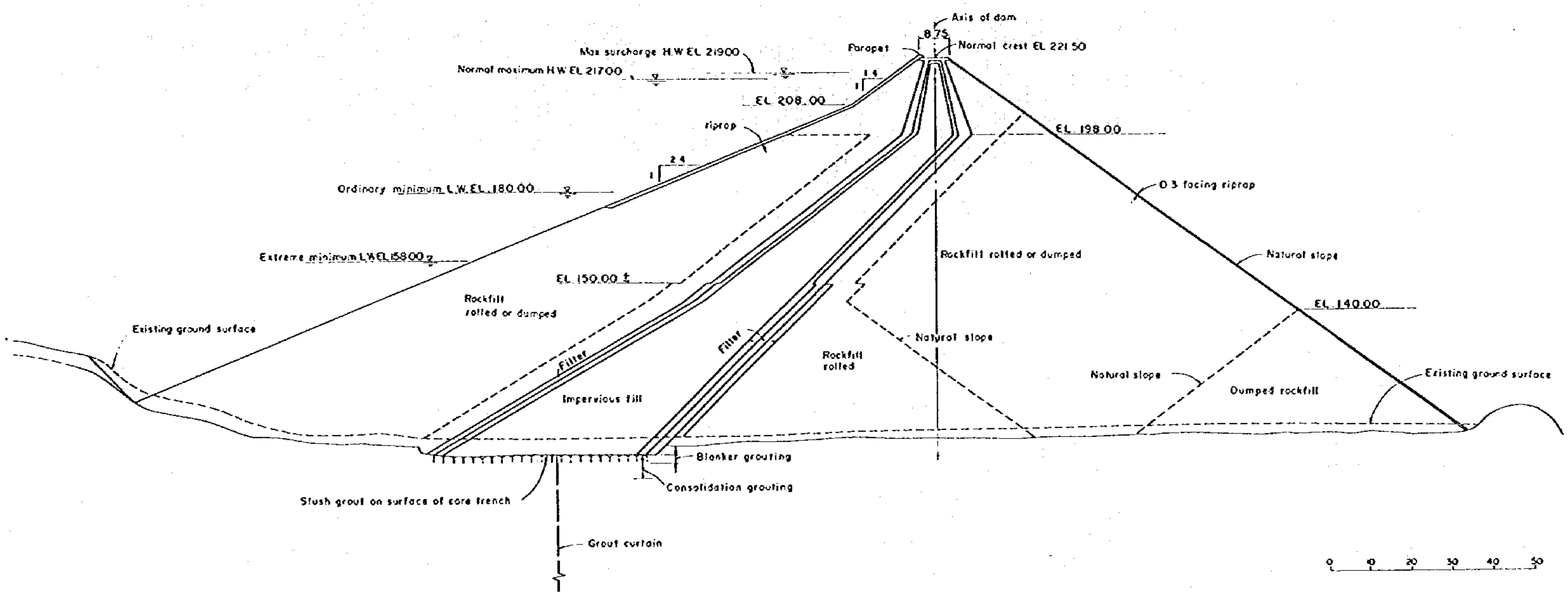


General Plan

Structures at Dam Site

TYPICAL CROSS SECTION OF THE ANGAT DAM



Angat Dam and Powerhouse

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CONCLUSIONS AND RECOMMENDATIONS

Conclusions and Recommendations

1. Conclusions

Studies for the Angat Dam Rehabilitation Project were made on the following five problems.

i. Discharging capacity of the spillway.

ii. Stability of the dam and the dyke.

iii. Possibility of landsliding at the ex-batcher plant site.

iv. Seepage through the dyke.

v. Leakage from the penstock.

The pertinent conclusions from the results of the studies are as follows:

i. Discharging Capacity of the Spillway:

. The spillway of the Angat dam is capable of spilling flood inflows of whatever magnitude, insofar as the high water level (HWL) of the reservoir is held at EL 212 m.

ii. Stability of the Dam and the Dyke:

. About two decades have passed since the dam and the dyke were built, but no remarkable damages have been found out since then.

. The dam and the dyke pose no particular safety problems under static condition.

. There is a possibility of local sliding on the slopes with a steeper gradient, if and when an earthquake with a large seismic intensity occurs. The sliding, if occurred, however, would be limited only to the portion close to the surface of the slopes, and no catastrophic damage is likely to occur in its consequence.

. It is, therefore, considered unnecessary to take an immediate action for any rehabilitation work.

iii. Possibility of Landsliding at the Ex-Batcher Plant Site:

. A large-scale landslide occurred in 1986 at the fill-up ground for the concrete mixing plant for construction work, but the ground has remained unstable since then. Under such unstable condition particularly against rainfall, it is very likely that a landsliding would re-occur, if a heavy rainfall hits the area again.

. It is, therefore, mandatory to take an immediate action for any protective measure against possible landsliding.

iv. Seepage through the Dyke:

. The loss of water due to seepage through the dyke is estimated to be about 261,000 m^3 a year, or 8.3 liters per second, and the resultant loss of energy production is estimated to be 56,100 kWh a year at most.

. Seepage of water would predominantly be through the left bank abutment of the dyke at elevations higher than EL 190 m.

. It appears that seepage of water through the dyke is currently in the stable state with no prospect of augmentation in the immediate future.

. Hence, it is not considered necessary to take an immediate action for any rehabilitation work.

v. Leakage from the Penstock:

. It is believed that the rate of water leakage from the penstock into the surrounding mountain masses would have increased with time. The leakage rate as of 1982 is estimated to be 0.7 m^3 per second.

. The locations of leakage are approximated to be in the vicinity of the bifurcation point of the penstock and of the connection with the concrete tunnel.

. It is mandatory to make repair at the soonest possible convenience.

2. Recommendations

2.1. Rehabilitation Plans

Items discussed in this Report for the necessity of rehabilitation are as follows:

i. Protective measure against possible landsliding at the ex-batcher plant site.

ii. Preventive measure against seepage through the dyke.

iii. Repair of the penstock to eliminate leakage.

Recurrence of a large-scale landsliding at the ex-batcher plant site may produce a serious effect on the water supply to Metro Manila and a far-reaching consequence to the social activities. The protective measure against possible landsliding should, therefore, be given top priority. The cost estimate for the slope reformation work considered as the protective measure is estimated to be US$437,000, and the time required for the work to be about nine months.

The present state of water seepage through the dyke is not considered dangerous in terms of the structural stability. Besides, the benefit that can be derived from the implementation of the preventive measure will be too small against the cost worth expending for the measure. It is, therefore, not considered necessary to take an immediate action for any rehabilitation work. However, in an event that the amount of seepage tends to increase due to an earthquake or other natural phenomena that may break the present state of equilibrium, an appropriate rehabilitation work should be implemented. The cost estimate for grouting work considered as the preventive measure is estimated to be about US$380,000.

Leakage from the penstock poses a very serious problem for the safety of the Angat power plant. It is, therefore, absolutely necessary to make internal inspections at an earliest time as possible to pinpoint the exact locations of leakage and identify the magnitude of defects, so as to establish the procedures for repair.

2.2. Safety Control of the Dam Structure

The dam and the dyke are wooded with trees and plants. These trees and plants are not only obstructive to visual inspections, monitoring and maintenance works, but harmful to the dam and dyke structures as movements of their trunks and roots by strong winds may loosen the embankments. They must be felled or grubbed for proper implementation of inspections and maintenance work, as well as for the safety of the dam and dyke structures.

2.3. Monitoring Activities

It is very essential to continue the monitoring activities that have been put in practice. It is particularly necessary to keep vigilant watches on the behavior of seepage through the dyke, as there is a possibility of augmentation of the seepage rate due to occurrence of an earthquake, etc. that may break the present state of equilibrium, though it is currently in the stable state with no prospect of augmentation in the immediate future.

The dam and dyke slopes may be locally unstable under condition with the earthquake effect with an unusually large seismic intensity. In an event that an earthquake of greater than 70 gal occurs, it is mandatory to make the additional inspection and slope survey to check if the slopes have undergone any changes or damages.

For the Angat power plant, it is particularly desirable to post on a permanent basis a civil engineering staff who is responsible for inspection and maintenance of the dam and the associated civil structures.

SUMMARY

1. Preface

This is the summary of the Final Report on the Study for Rehabilitation Project of the Angat Dam owned and operated by National Power Corporation of the Republic of the Philippines (hereinafter referred to as NAPOCOR).

The principal objectives of the Study for Rehabilitation Project (hereinafter referred to as the Study) are to investigate the actual state of the dam and its associated facilities, to formulate the additional field survey and dam monitoring works, to supervise such works to be implemented by NAPOCOR, to formulate the standards for safety control of the dam, and to prepare optimum rehabilitation programs from technical, economic and financial points of view, within the specified study period of about 17 months from September 1987.

At the inception of the Study, the scope of work included the field investigation of leakage from the penstock and the turbine draft tube, in addition to the investigation of the landslide at the ex-batcher plant site and seepage through the dyke. The leakage of the penstock was then considered most critical in urgency and importance among the investigation items, but it has, after all, been left uninspected, because of no chances for reasons on the NAPOCOR side to dewater the penstock throughout the specified study period, and none of the detail of the leakage and the damages of the penstock has been identified. From this point of view, it can be said that the Study has yet to be finalized with essential items being left uninspected.

2. Background of the Study

The Angat Hydroelectric Power Plant is located some 35 km northeast of Manila City, Philippines. The dam and the associated facilities were designed by Harza Engineering Co., Chicago, Illinois, U.S.A. The plant was completed in 1966 and commissioned in 1967.

The Angat dam is a rockfill dam with an inclined impervious core, 131 m high and 368 m long in crest. The reservoir with a catchment area of 568 km^2 has an effective storage capacity of 850 million m^3. Utilizing water in the reservoir, the power plant is capable of generating electricity at the rated capacity of 228 MW and is in operation as one of the main power sources to serve peak loads in the Luzon Grid.

After about two decades have passed since its commissioning, the dam has posed problems in safety control, and rehabilitation plans for water leakage and landsliding have been worked out from time to time.

More attentions have been directed to the safety control problems, as more serious incidents arose one after another, including a large landslide of the ground at and around the ex-batcher plant site, which occurred in August 1986, leakage from the penstock and the turbine draft tube, and seepage through the left abutment of the dyke. These were considered to be in urgent need of rehabilitation or repair.

While making their own investigations, NAPOCOR made an official request through the Government of the Republic of the Philippines to the Government of Japan for the technical assistance of the rehabilitation program.

In view of the urgency and importance of the program, the Japan International Cooperation Agency (hereinafter referred to as JICA), responsible for implementation of technical cooperation programs of the Government of Japan, responded to the request in sending a pre-study mission to the dam site in February 1987, started a full-scale investigation in September 1987, and has finalized the investigation to work out solutions to the various problems.

3. Scope of Work of the Study

At the inception, the Study was principally directed to carry out the work as itemized below:

(1) Investigation and inspection of seepage through the dyke built on the left bank of the main dam.

(2) Investigation and inspection of the landslide which occurred at the ex-batcher plant site on the terrain of the left bank of the dam.

(3) Investigation and inspection of leakage from the penstock and the turbine draft tube, and assessment of the present state of power plant equipment and facilities.

Of the above three, however, item (3) had to be excluded from the scope of work, because it became in no way possible for reasons on the Philippine side to make internal inspections of the penstock after dewatering.

The Study was conducted both in the field and in Japan.

3.1. Field Investigations

(1) Field Investigations by the JICA Study Team

Field investigations were made by the JICA Study Team during a period from September 17 to November 3, 1987. The Team members who joined the investigations were as follows:

Name	Period
(Dr.) Ryuichi Kozuki, Team Leader	Sept.17 - Oct.16, 1987
Yutaka Matsui, Civil Engineer (Design)	Oct. 5 - Nov. 3, 1987
Ikuo Kozu, Civil Engineer (Design)	Sept.17 - Nov. 3, 1987 (Dec. 1 - Dec.30, 1987 as a supervisor of the field survey and monitoring works)
Megumu Kawahara, Geologist	Sept.17 - Oct. 4, 1987
Masaki Takahashi, Electric Engineer (Electro-Mechanical)	Oct. 4 - Oct.14, 1987
Nobuo Ohno, Mechanical Engineer (Metalwork)	Oct. 4 - Oct.14, 1987

The principal activities of the Team members for the field investigations were as follows:

(1) Presentation of the Inception Report to NAPOCOR and discussion on the Report with the NAPOCOR staff.

(2) Collection of information and data related to the Study.

(3) Inspection of the dam and dyke structures.

(4) Inspection of the landslide at the ex-batcher plant site.

(5) Inspection of the spillway and the associated structures.

(6) Preparation of the additional field survey and monitoring programs and the technical specifications therefor.

(7) Surface inspection of the penstock route and outward inspection of power plant equipment and facilities.

(8) Inspection of the existing monitoring systems.

(2) Additional Field Survey and Monitoring Works

The additional field survey and monitoring works programmed by the JICA Study Team based on the results of their field investigations have been carried out by NAPOCOR. To supervise these field survey and monitoring works, the Team members (Ikuo Kozuki and Tamotsu Fujiwara) visited the Angat project site four times during the specified study period.

The details of the additional field survey and monitoring works that have been undertaken by NAPOCOR are as shown below.

Additional Field Survey and Monitoring Works

Location	Work Item		Unit	Quantity	Date of Completion*
Dyke Area	Survey	Plane survey	m^2	65,000	February 1988
		Cross section	m	750 (3 sections)	February 1988
	Drilling	Left bank	hole	4 (140 m)	October 1988
		Center	hole	3 (70 m)	October 1988
	Displacement measurement		point	3	March 1988
	Leakage water measurement		location	1	March 1988
Landslide Area	Survey	Plane survey	m^2	151,000	February 1988
	Drilling	Geological investigation	hole	6 (180 m)	July 1988
		Installation of inclinometer	unit	6	July 1988
	Soil test	Pit sampling	pit	3	December 1987
		Test	set	3	December 1987
Penstock Area	Survey	Plane survey	m^2	50,000	February 1988
	Drilling	Geological investigation	hole	1 (75 m)	October 1988
		Installation of piezometer	hole	1	November 1988
	Leakage water measurement		location	2	March 1988
Dam area	Plane survey	Cross section	m	300	March 1988
	Displacement measurement		point	6	March 1988
	Seismograph		point	1	April 1988
	Crack meter	Spillway	set	1	April 1988

* Date of completion of the additional field survey work and installation of the monitoring systems.

3.2. Studies and Analyses in Japan

Studies and analyses were made based on the various data obtained from the field investigations and surveys undertaken for the Study since September 1987, as well as on the results of the previous investigations made independently by NAPOCOR.

Items discussed in this Report are:

(1) Investigation to check if the spillway capacity is adequate.

(2) Investigation to ensure the stability of the dam and the dyke.

(3) Investigation of the possibility of landsliding at the ex-batcher plant site.

(4) Investigation of seepage through the dyke.

In addition, the result of studies on the leakage from the penstock based on the data collected by NAPOCOR from their own investigations was also included in the Report as a supplement, in that it may have a serious effect on the safety of the Angat Power Plant.

The Report also proposes the implementation plans for the rehabilitation project based on the result of investigations, together with the economic analyses thereof, and recommends the system to maintain safety control of the dam structure and its associated facilities.

4. Summary of the Study

4.1. Adequacy of Spillway Capacity

In Section 2 of the Report, the adequacy of the spillway capacity is discussed. The adequacy of the spillway capacity had been reviewed several times, and the latest one was done in 1984.

To review the adequacy of the spillway capacity, it is necessary to:

(1) Establish a basic pattern of the flood inflow to the dam during the flood period,

(2) Determine the maximum limit of rise in the water level in the reservoir during the flood period, and

(3) Confirm if the outflow from the dam during the flood period would not exceed the maximum limit of discharge without causing any damages to the downstream basin.

In determining the design flood inflow for rockfill dam, it is a normal practice in Japan to use a 20% increase over the 200-year flood inflow. The probable maximum flood inflow has also to be taken into consideration.

In the Report, the probable inflow was calculated based on the annual maximum inflow (m^3/sec) over the 30-year period from 1957 to 1987, as well as on the single day's and two, three, four and five consecutive days' maximum runoff ($10^6 m^3$) throughout each year of the same period, and the basic pattern of the probable inflow for various return periods (200, 100, 50 years, etc.) was determined based thereon. (Refer to Fig. 1.1.)

As a result of these calculations, it was found that the peak inflow for the 200-year return period would be 7,850 m^3/sec. and the single day's and two, three, four and five consecutive days' maximum runoff for the same return period would be $339.3 \times 10^6 m^3$, $457.6 \times 10^6 m^3$, $560.8 \times 10^6 m^3$, $656.7 \times 10^6 m^3$ and $718.0 \times 10^6 m^3$, respectively.

The 20% increase over the 200-year flood inflow with such inflow patterns as above was applied to the case of the Angat dam as the design flood inflow, and the maximum permissible water level at this particular flood inflow was assumed to be at EL.218 m, with the high water level (HWL) set at EL.212 m, taking into account the height of wave above the reservoir water surface due to winds, and the minimum required freeboard of 1.5 m for fill type dam. (Refer to Fig. 1.2.)

Calculations of the rise in the water level were made on the various assumptions for the design flood inflow.

The results indicate that, if the gate operation is done in such manner as the flood inflow be discharged to keep the same water level of EL.212.0 m, and be discharged in free flow condition at full gate opening after the flow rate exceeds 2,400 m^3/sec, the water level would rise to EL.217.80 m. This is, however, still below the maximum permissible limit of EL.218.0 m.

The results also indicate that, even if the water level is kept at EL.213.0 m, and the gate operation is done in such a manner as the flood inflow be discharged in free flow condition at full gate opening after the flow rate exceeds 2,800 m^3/sec, the water level would rise only to EL.218.12 m. This is a little over the maximum permissible limit of EL.218.0 m, but this much of excess may not pose any particular problems in dam operation. (Refer to Fig. 1.3. and Table 1.1.)

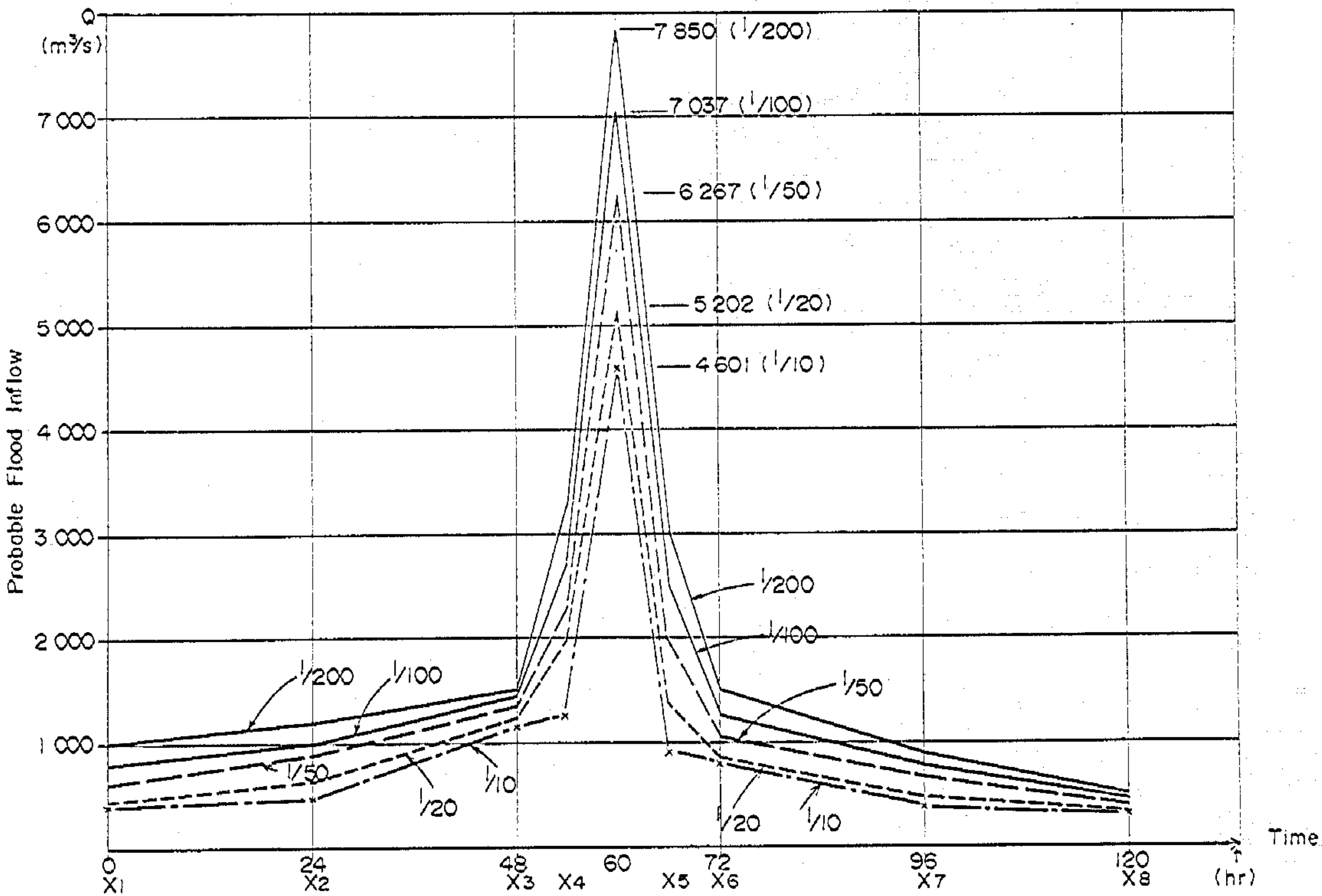
Calculations of the rise in the water level were also made on the PMF magnitude flood inflow estimated on the basis of the data prepared by the 1984 review team.

The results indicate that, if the gate operation is done in the manner similar to the above, then the water level would rise to EL.219.35 m. This is in excess of the maximum permissible limit by 1.35 m. However, the PMF magnitude is too severe to be considered as the design criteria, and it may not be necessary to consider any specific freeboard for such magnitude of flood inflow, but may be considered enough to keep the freeboard at lower than non-overflow section. If so, the maximum permissible limit in this case should be regarded as EL.219.5 m (dam crest elevation minus the height of wave due to winds), and this implies that the present spillway capacity may be enough to spill the flood inflow even of such magnitude. (Refer to Table 1.1.)

It was also confirmed that, even if the flood inflow is discharged downstream through the spillway by such gate operation as mentioned above, the rate of increase in the water level at the Norzagaray site could be well within the limit of 30 cm/30 min. that assures no damages to the downstream basin.

The pertinent conclusion from the results of the above studies is that the existing spillway capacity of the Angat dam may meet the requirements of the respective Japanese standards, though not sufficient enough.

Fig. 1.1 Patterns of the Probable Flood Inflow by Return Period



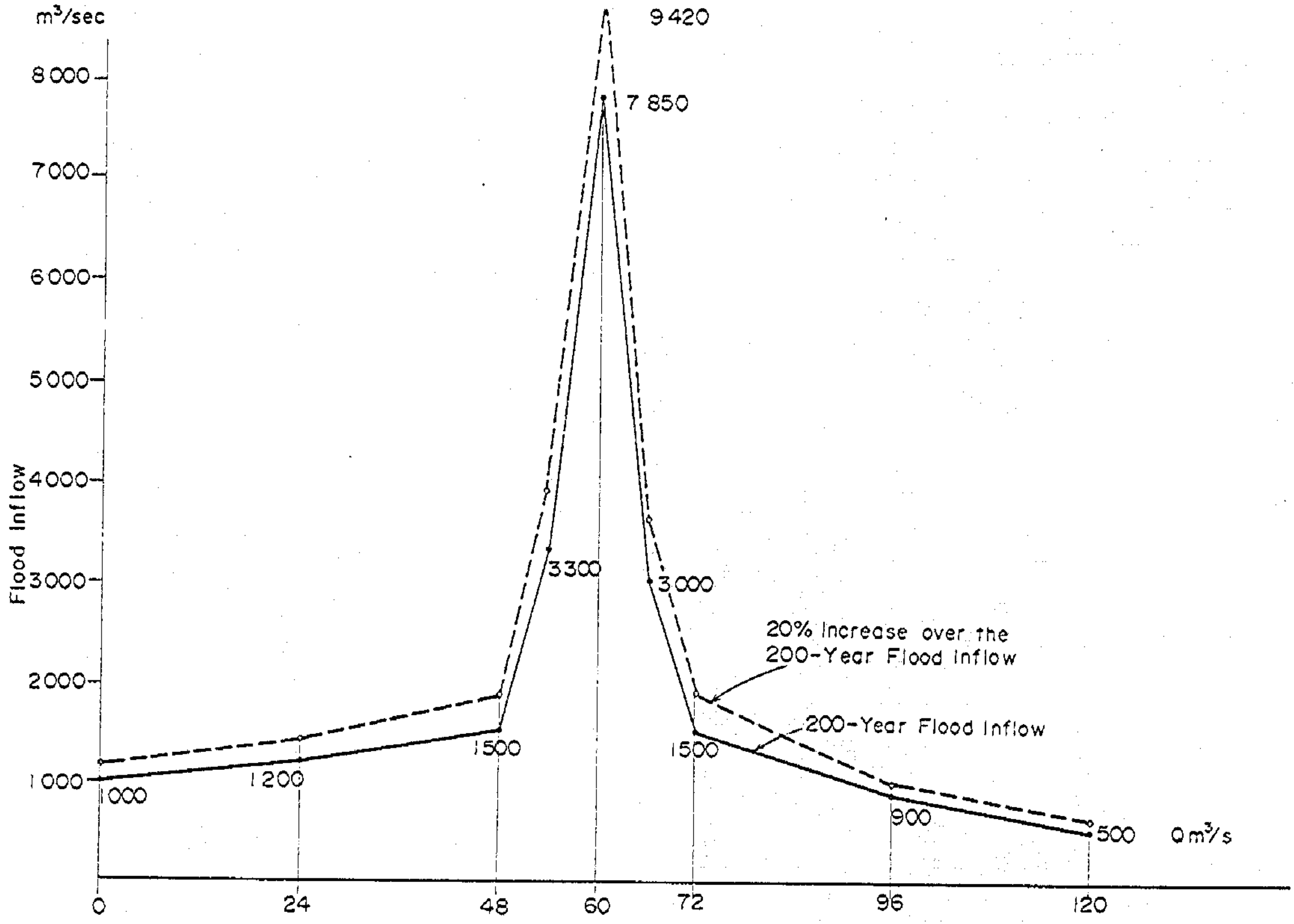


Fig.1.2 Pattern of Flood Inflow to the Angat Dam

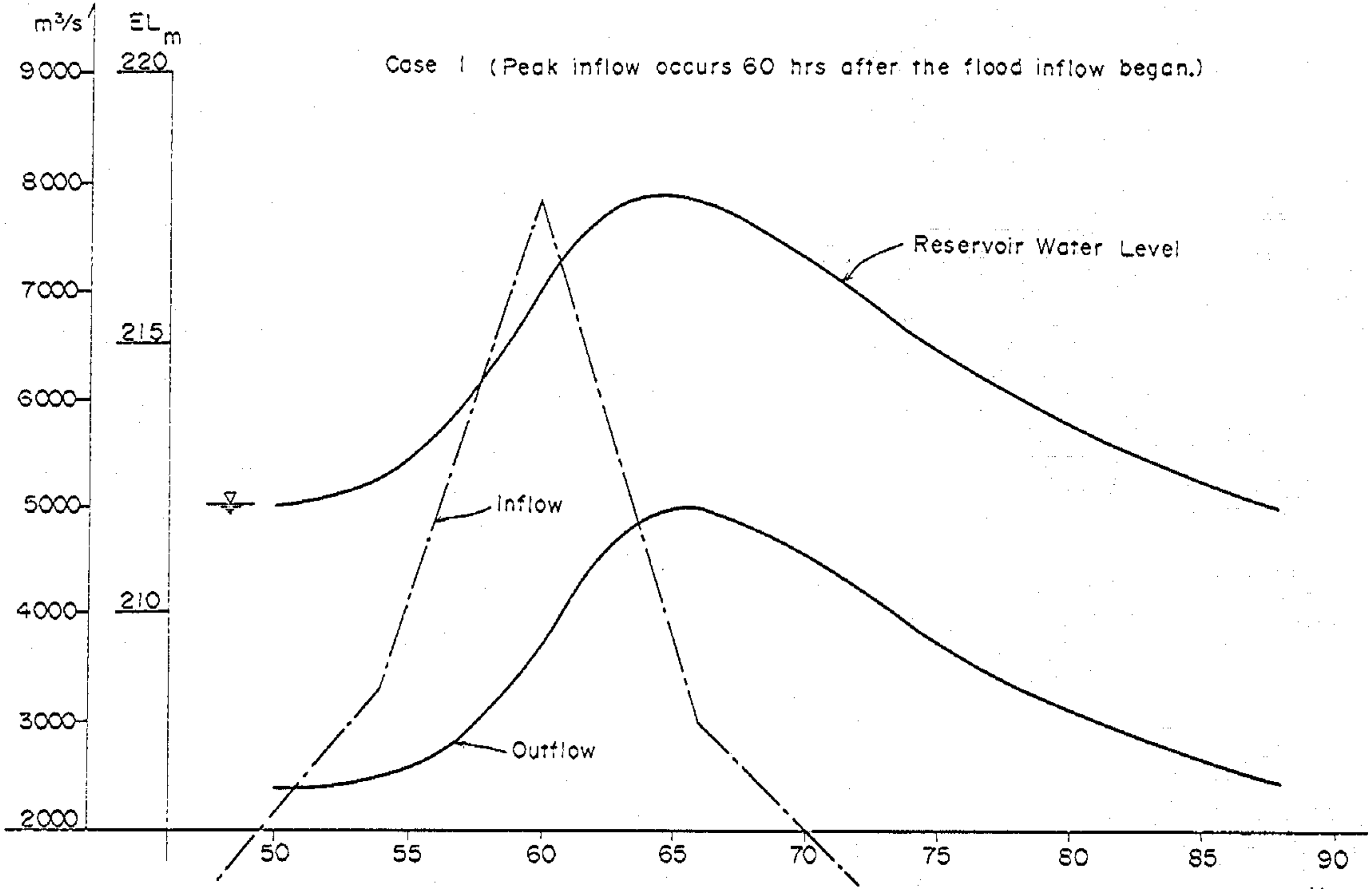


Fig. 1.3 Change in the Reservoir Water Level and the Inflow to and Outflow from the Dam during the Flood with a 200-Year Return Period Multiplied by 1.2

Table 1.1 Ratio of Peak Outflow to Peak Inflow during Floods for Various Return Periods

Design Flood Inflow

Return Period	When Peak Occurs	Water Level before Flood Inflow (EL-m)	Water Level with All Gates Full Open (EL-m)	Max. Water Level (EL-m)	Max. Discharge (m^3/sec.)	Ratio of Max. Outflow to Max. Inflow
200-Year x 1.2	60 hrs after inflow started	212	213	218.12	5143	0.55
	- do -	212	212	217.80	4977	0.53
	- do -	211	211	217.51	4826	0.51
	- do -	210	210	217.23	4684	0.50
	54 hrs after inflow started	212	213	217.87	5012	0.53
	- do -	212	212	217.49	4815	0.51

PMF Magnitude Flood Inflow

Return Period	Flood Inflow	Water Level before Flood Inflow (EL-m)	Water Level with All Gates Full Open (EL-m)	Max. Water Level (EL-m)	Max. Discharge (m^3/sec.)	Ratio of Max. Outflow to Max. Inflow
PMF	PMF Pattern	212	213	219.35	5810	0.69
	- do -	212	212	219.29	5774	0.69
	- do -	211	211	219.24	5747	0.68
	- do -	210	210	219.20	5725	0.68

4.2. Stability of the Dam and the Dyke

In Section 3 of the report, the stability of the dam and the dyke is discussed. The stability analysis of the main dam and the dyke was made on various loading conditions. As to the reservoir operating conditions, HWL, LWL and an abrupt reservoir drawdown from HWL to LWL were considered. The earthquake effect with Kh value of 0.15 g was also considered as the additional loading condition. (Refer to Table 2.1.)

The judgement of the safety was made by the safety factor against sliding. The criterion for the judgement was set at 1.2 or greater under normal loading conditions without the earth-quake effect, and at 1.0 or greater under loading conditions with the earthquake effect. The results of the analysis were as follows:

With regard to the stability of the upstream face slope of the dam, it was found that the loading conditions under which the safety factor against sliding becomes smallest are the reservoir operation at HWL accompanied with the earthquake with Kh of 0.15 g, except the case of an abrupt reservoir drawdown from HWL to LWL. It was also found that the safety factor would be smaller at the LWL operation than at the HWL operation in case of the sliding line assumed to pass through the core zone.

The properties of the core materials are unknown because of lack of data and records during the construction. But, judging from the results of analyses of residual soils, and of the core materials used for the Binga and Ambuklao dams that were built earlier than the Angat dam, the internal friction angle (ϕ) and the cohesion (C) of the core materials for the Angat dam can be assumed to be in the range of 25° and 6 tons/m^2 to 30° and 4 tons/m^2, respectively.

If ϕ value of the rockfill and filter materials is assumed to be 43° and 35°, respectively, the safety factor against sliding of the dam would be greater than 1.0, even under the loading conditions (LWL with the earthquake effect) which may cause the sliding to pass through the core zone, in so far as ϕ value of the core materials is larger than 25°.

If ϕ value of the rockfill materials is assumed to be larger than 45°, the safety factor against sliding would be greater than 1.0, in so far as ϕ value of the core materials is larger than 22°. (Refer to Fig. 2.1.)

Since it is a general practice to use the rockfill materials with ϕ value of larger than 43°, and the core materials with ϕ value of nearly 30° for rockfill dams, the Angat dam, if built in such materials, would be safe against sliding under any loading conditions, except the case of an abrupt reservoir drawdown from HWL to LWL.

Even under the loading conditions of an abrupt reservoir drawdown from HWL (EL 217 m) to LWL (EL 180 m), the safety factor against sliding would be 1.2, if no earthquake effect is taken into consideration. However, it would become smaller than 1.0, if the earthquake effect is taken into consideration, in case where ϕ value of the rockfill materials is assumed to be 43°, while ϕ value of the core materials to be smaller than 29°, or ϕ value of the rockfill materials is assumed to be 45°, while ϕ value of the core materials to be smaller than 26°.

Such loading condition (an abrupt drawdown from HWL to LWL) is, however, too severe and unrealistic for the Angat dam, because it has a considerably large reservoir surface area. Besides, it is not likely that an earthquake with such a large seismic coefficient of 0.15 g would occur just at the time when the reservoir water level is down to LWL. It was therefore still considered safe that the safety factor would become even smaller than 1.0 in such an event. (Refer to Tables 2.2 and 2.3.)

With regard to the downstream face slope of the dam, it appears that no changes have taken place on the dam since its completion from the fact that the measured gradient of 1:1.4 is identical with the design drawing. (Refer to Figs. 2.2 and 2.3.)

The safety factor against sliding of the downstream face slope would be larger than 1.2 in static condition, when assuming ϕ value of the rockfill materials is 43°, but would come down to 0.964 or smaller than 1.0 under the loading conditions with the earthquake with Kh of 0.15 g. The sliding, if happened, however, would be limited only to the portion close to the slope surface of the dam. Though it may not necessarily be said that the slope is absolutely safe against any large earthquake, the occurrence of such earthquake may not lead to a disruption of the dam. The stability analysis of the slope profile after removing the earth slidden due to the earthquake indicates that the safety factor against further sliding would be 1.0 in static condition. (Refer to Fig. 2.4.)

The pertinent conclusion from the above analysis is that it is not necessary to do anything to the existing Angat dam structures under the present circumstance, and any rehabilitation work should be implemented only when a large Kh earthquake would occur, and unfortunately, it would cause any surface layer sliding on the downstream face of the dam. The dam stability may be retained by rebuilding the embankment to make the gradient gentler (from 1:1.4 to 1:1.46).

The results of the stability analysis of the dyke were as follows:

The average gradient of the upstream face slope of the dyke is 1:1.8, a little steeper than the corresponding gradient of the main dam. Therefore, the safety factor against a large-scale sliding would be smaller than that of the main dam, but could still keep 1.0 or larger even under the earthquake loading condition.

There is a possibility of a small-scale sliding on the portion with a gradient of 1:1.4, similarly to the case of the main dam, when assuming ϕ value of the rockfill materials would be 43°, but there is no possibility of such a large-scale sliding that may pass through the core zone. Therefore, any actions, if necessary, for the rehabilitation program should be taken as is the case with the main dam. The safety factor could be maintained at greater than 1.0 even under the loading conditions of an abrupt drawdown from HWL to LWL, accompanied with the occurrence of an earthquake, because the slope of the dyke with the gradient of 1:1.4 is shorter in length than the corresponding slope of the main dam.

The gradient of the downstream face slope of the dyke is 1:1.45, which is gentler than the corresponding gradient of 1:1.40 of the main dam. Therefore, if the same materials are used for the embankment, the dyke is stabler than the main dam against sliding.

The results of the stability analysis indicate that the dyke would be stable against sliding even under the earthquake loading with Kh of 0.15 g, if the rockfill materials with ϕ value of 43° and the filter materials with ϕ value of 35° are used.

Based on the results of the stability analysis of the main dam and the dyke, the following conclusions can be made:

(1) The worst effect on the stability of the main dam and the dyke is the earthquake loading, which has an overriding influence upon the judgement for the stability analysis.

(2) The gradient of the upstream face slope of the dyke is on the average steeper than the corresponding slope of the main dam, but the gradient of the downstream face of the main dam is a little steeper than the corresponding slope of the dyke.

(3) The main dam has a minor problem on the downstream face slope against the earthquake, but it is considered unnecessary to do anything to the existing structures, since a sliding, if happened due to an earthquake with Kh value of 0.15 g, would be limited only to the portion close to the surface of the slope.

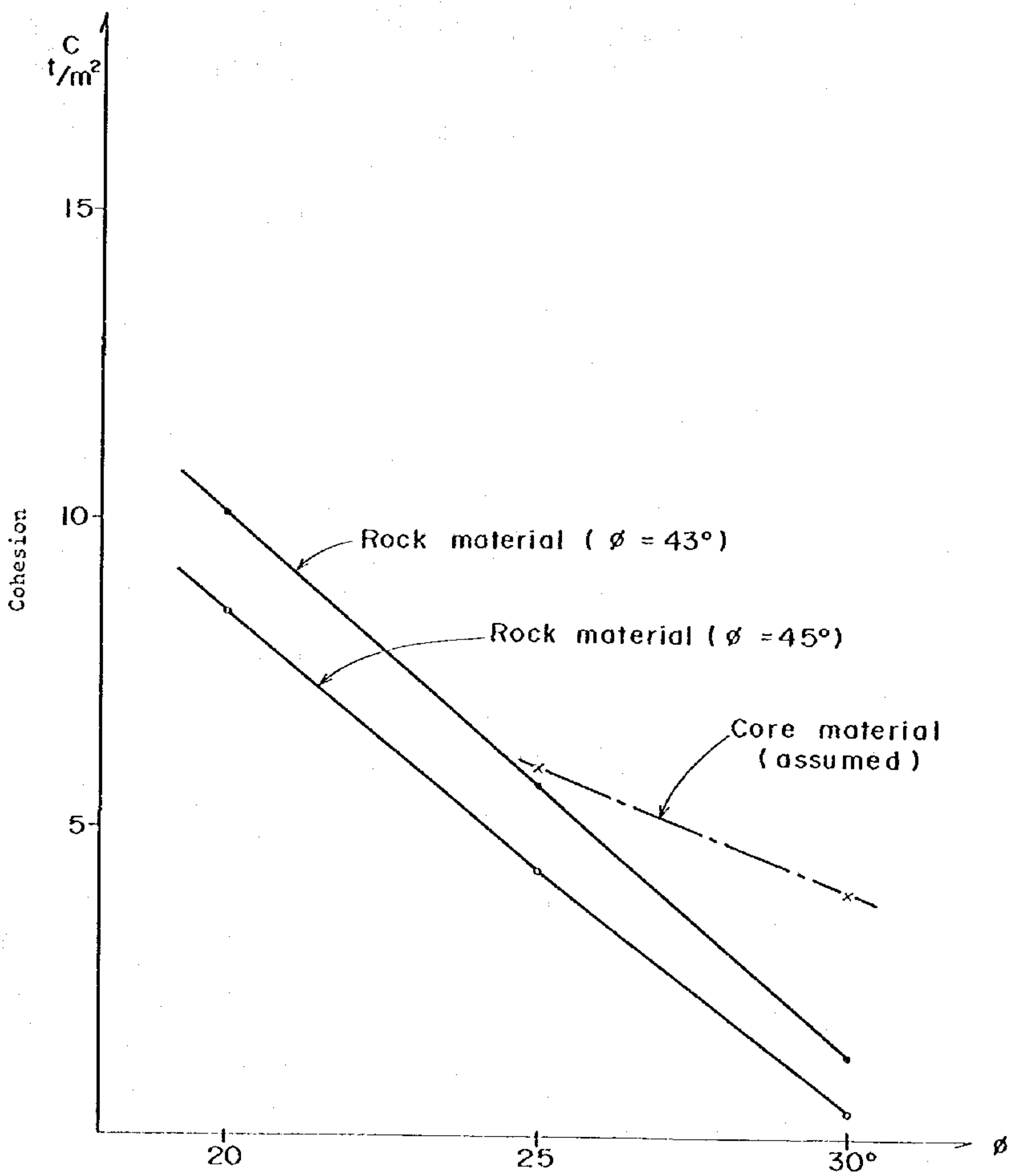
(4) There is a possibility of local damages on the slopes of the dam and the dyke when there occurs an earthquake with Kh of greater than 0.07 g. It is therefore necessary to keep patrolling inspections to check if both slope faces of the dam and the dyke have undergone any damages.

Table 2.1. Loading Conditions for Stability Analysis of the Upstream Face Slope of the Dam

Case	Dam Face	Water Level at		Earthquake Effect (Kh = 0.15g)	
1	Upstream	HWL		Without	
2	Upstream	HWL			With
3	Upstream		LWL		With
4	Upstream	Abrupt drawdown from HWL to LWL		Without	
5	Upstream	Abrupt drawdown from HWL to LWL			With
6	Downstream	HWL		Without	
7	Downstream	HWL			With

Fig.2.1 Relation Between ϕ and C Values of Core Materials Necessary to Keep the Required Safety Factor

(Loading Conditions - Group I)



Internal Friction Angle

Table 2.2. Safety Factor Against Sliding Lines Passing through Rockfill Zone of the Upstream Face Slope of the Dam

Case	Calculated Safety Factor		Required Safety Factor
	Rockfill Material ϕ=43°	Rockfill Material ϕ=45°	
1	1.31	1.40	1.2
2	0.74 (0.069)	0.80 (0.087)	1.0
3	0.96 (0.13)	1.03	1.0

Note: The parenthesized figure represents the seismic coefficient shown in g with the safety factor assumed to be 1.0.

Table 2.3. Safety Factor Against Sliding Lines Passing through Rockfill and Filter Zones of the Upstream Face Slope of the Dam (Assuming that ϕ value of filter materials would be 35°)

Case	Calculated Safety Factor				Required Safety Factor
	Against Large Scale Sliding		Against Small Scale Sliding		
	Rockfill ϕ=43°	Rockfill ϕ=45°	Rockfill ϕ=43°	Rockfill ϕ=45°	
1	2.163	2.266	1.364	1.455	1.2
2	1.156	1.313	0.860 (0.10)	0.919 (0.12)	1.0
3	1.118	1.172	1.065	1.140	1.0

Note: The parenthesized figure represents the seismic coefficient shown in g with the safety factor assumed to be 1.0.

Fig. 2.2 MAIN DAM CROSS SECTION
(Maximum Section)

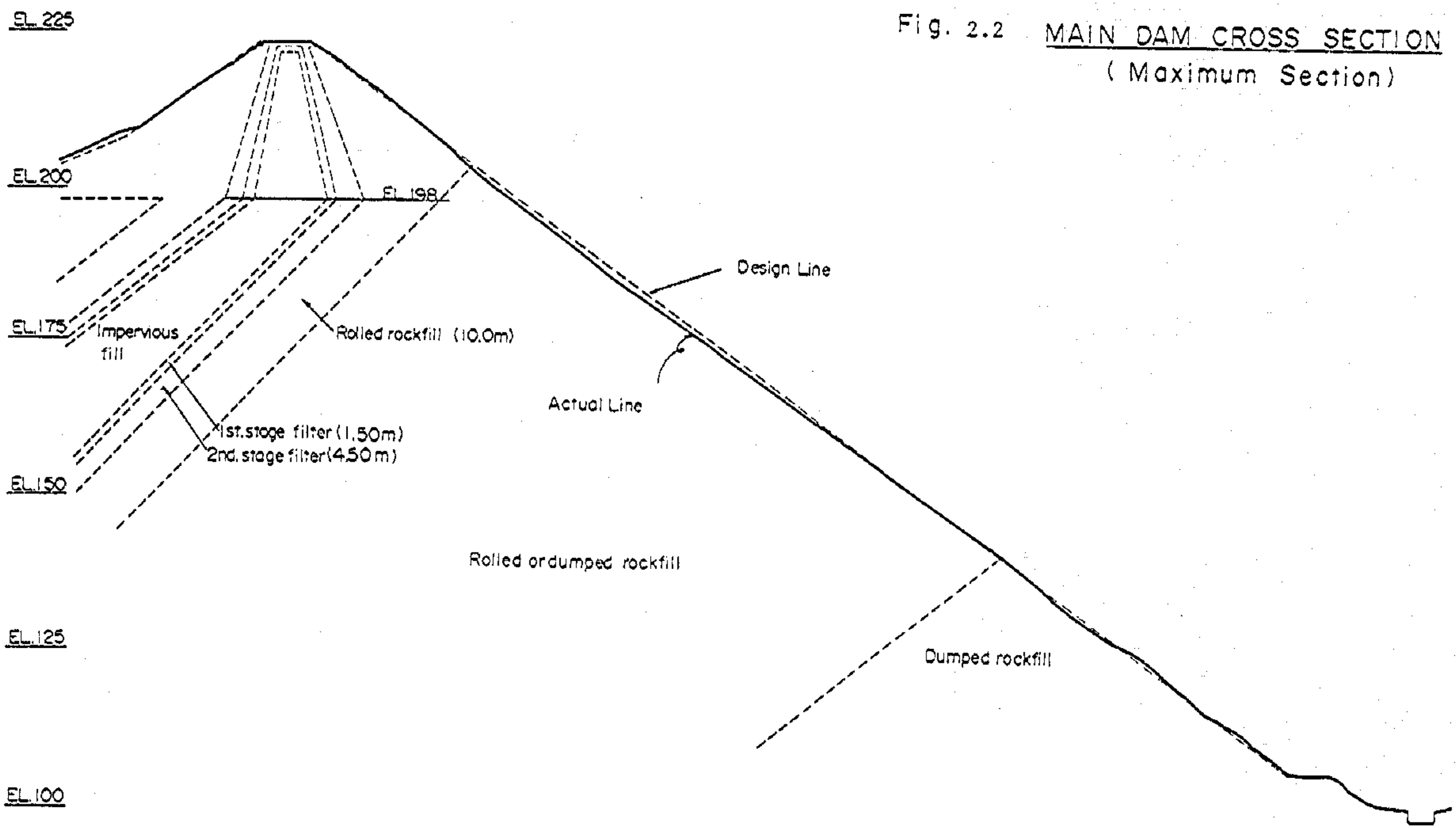


Fig. 2.3 Dyke Cross Section B - B' (Maximum Section)

Fig. 3.3 Dyke Cross Section B - B' (Maximum Section)

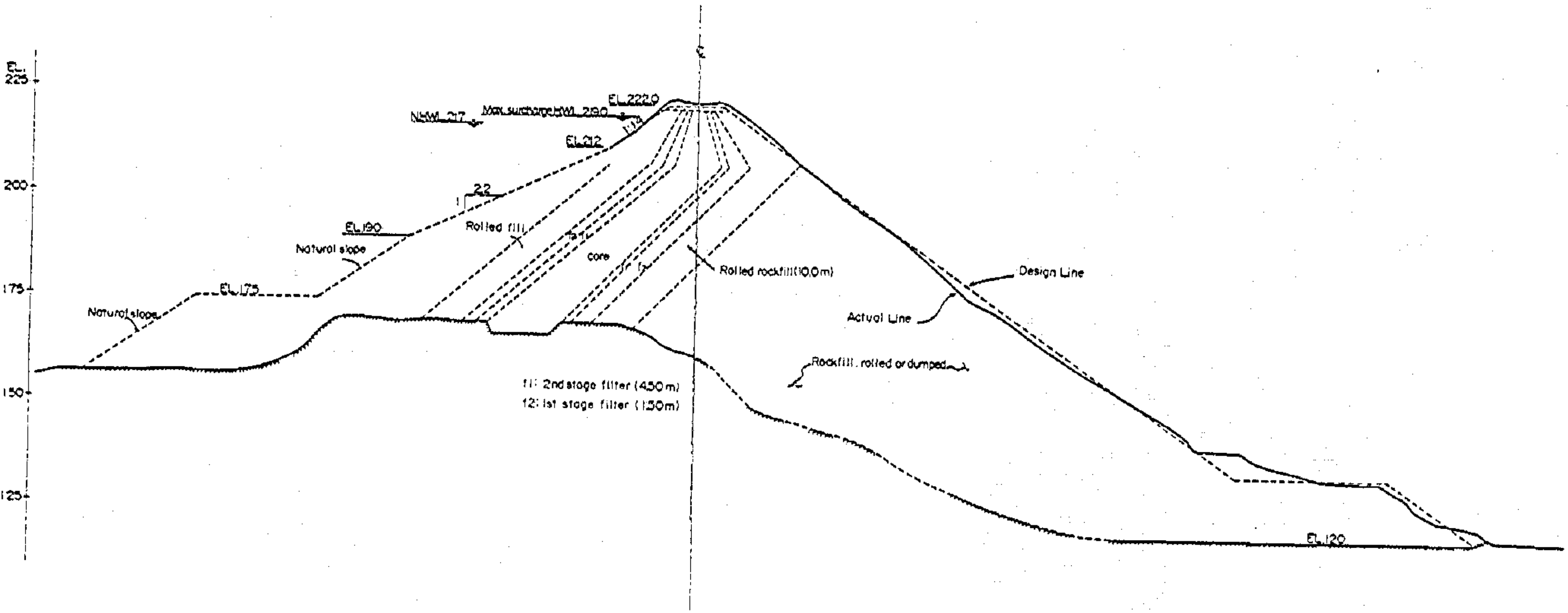
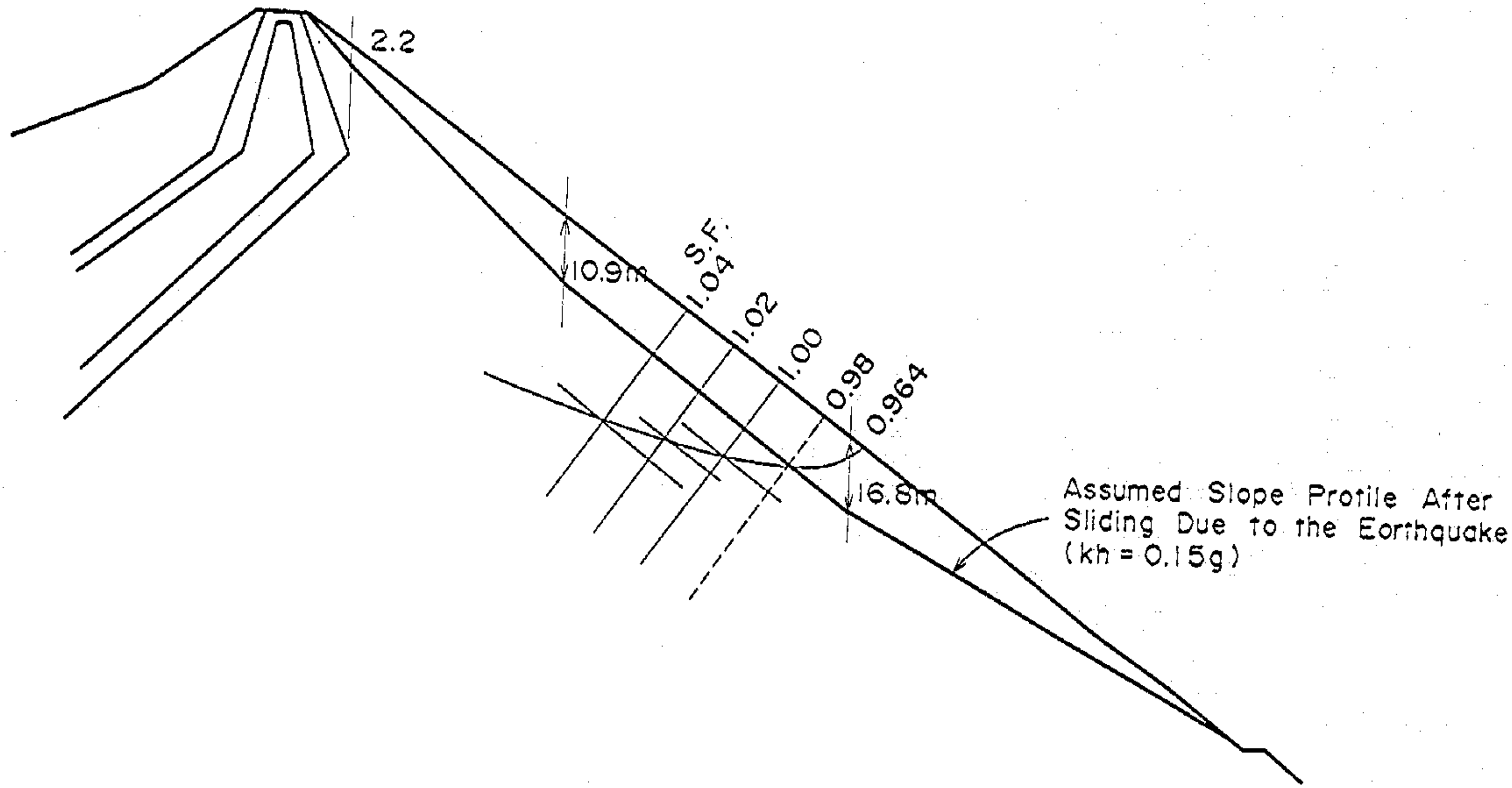


Fig. 2.4 Change By Depth in the Safety Factors Against Sliding of the Downstream Face Slope of the Main Dam During Earthquake (ø Value of Rockfill Materials Assumed at 43°)



4.3. Landsliding at the Ex-Batcher Plant Site

Section 4 of the Report deals with the analyses on the possibility of landsliding at the ex-batcher plant site.

A large-scale landslide occurred at the ex-batcher plant site in 1986, and it blocked a stream flow of the Angat river. Any recurrence of landsliding may cause disruption of water discharge into the Ipo dam, and may produce a serious effect on the water supply to Metro Manila.

Field investigations have been continued by using inclinometers and drill holes for measurements of the following: (1) how high is the ground water table in the mountain masses, (2) whether there is any sign of recurrence of landsliding, and if so, how deep the sliding depth would be. However, data on (2) were not available in time for the preparation of the Report, mainly because of some defects in the instruments.

Analyses were, therefore, made on the basis of the results of the standard penetration tests previously conducted by NAPOCOR and the topographic survey made for this Study, as well as of the data taken from measurements using drill holes provided for inclinometers.

As the result, the following were made clear:

(1) Physical Properties of Mountain Masses

It is very likely that the physical properties of the mountain masses would be dissimilar by depth, and become greater as a depth from the surface becomes deeper into the mountain. The cohesion strength (c) may be 3 tons/m^2 at the ground surface, 8 tons/m^2 at a depth of 6 m and 12 tons/m^2 at a depth of 10 m, if the internal friction angle (ϕ) is assumed to be 10°, and 1 ton/m^2 at the ground surface, 4 tons/m^2 at a depth of 6 m and 7 tons/m^2 at a depth of 10 m, if ϕ value is assumed to be 20°, in so far as the mountain masses are at the elevations higher than EL.180 m. (Refer to Fig. 3.1.)

The cohesion strength (c) of the mountain masses at the elevations lower than EL.180 m can be assumed to be 15 tons/m^2 or larger, if ϕ value is assumed to be at least 30°, and when N values of the penetration test are greater than 50.

(2) Possibility of Recurrence of Landsliding

Analyses on the possibility of landsliding were made of three assumption cases, A, B and C. Case A is to assume that the ground would be impregnate with water up to the surface. Case B is to assume that the ground water would be at lower elevations, but an earthquake with Kh of 0.15 g would occur. Case C is to assume that an earthquake with Kh of 0.15 g would occur while the ground is impregnate with water up to the surface.

Analyses on the landsliding possibility in Cases A and B were made on condition that the safety factor against sliding would be 1.2 or greater, while analyses in Case C were made on condition that the safety factor against sliding would be 1.0 or greater.

As the result, it was found that the mountain masses at the elevations higher than EL.180 m would be stable against landsliding if ϕ value is assumed to be 10°, but would be unstable (with the safety factor against sliding lower than 1.0) in Case A (when the ground water would come up to the surface) or in Case C (when an earthquake would occur in Case A), if ϕ value is assumed to be 20°. (Refer Fig. 3.2.)

(3) Protective Measures against Possible Landsliding

Solutions to be considered as the protective measures against possible landsliding are:

a) To provide a retaining structure by driving piles into the ground, thereby to increase a shearing resistance against sliding.

b) To excavate the shoulder portion of the slope, thereby reducing a load of earth prone to landsliding and attenuating a sliding force, and at the same time to fill up the ground at the bottom portion of the slope, thereby increasing a shearing resistance against sliding.

As a result of the comparative studies, it was confirmed that the solution (b) proves to be more effective for protection against landsliding than the solution (a), even on an extremely adverse condition that the ground at elevations higher than EL 180 m is assumed to have an internal friction angle (ϕ) of 20°.

It was also confirmed that the solution (b), if implemented in such manner as to cut the ground at elevations higher than EL.215 m to make the slope gentler, and to fill up the ground at elevations lower than 215 m to keep it from sliding, will ensure the safety of the slope, even in an extreme case where an earthquake with Kh of 0.15 g would occur at any time when the ground is impregnate with water up to the surface due to heavy rainfall. (Refer to Figs. 3.3, 3.4 and 3.5.)

It is also recommended to provide drain holes, each with a diameter of 75 mm, at the elevations of EL 215 m and 225 m, and surface drain ditches, in order to reduce ground water level and to protect the surface from erosion by rainfall, so as to make doubly sure of the safety against sliding.

Fig. 3.1 Relation Between C and ϕ Values for Analyses on the Possibility of Landsliding

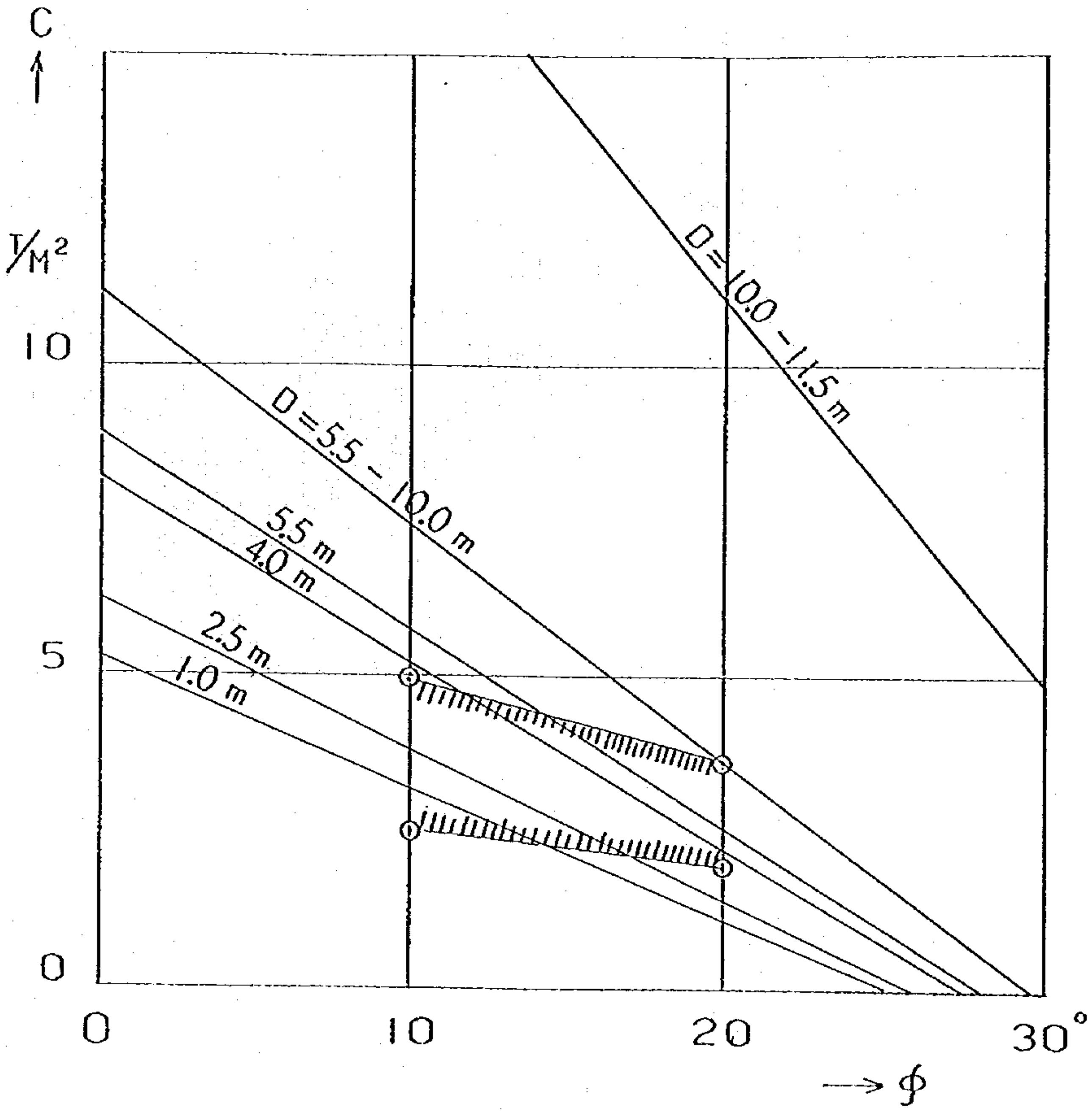


Fig.3.2 Probable Sliding Lines of Possible Landslides

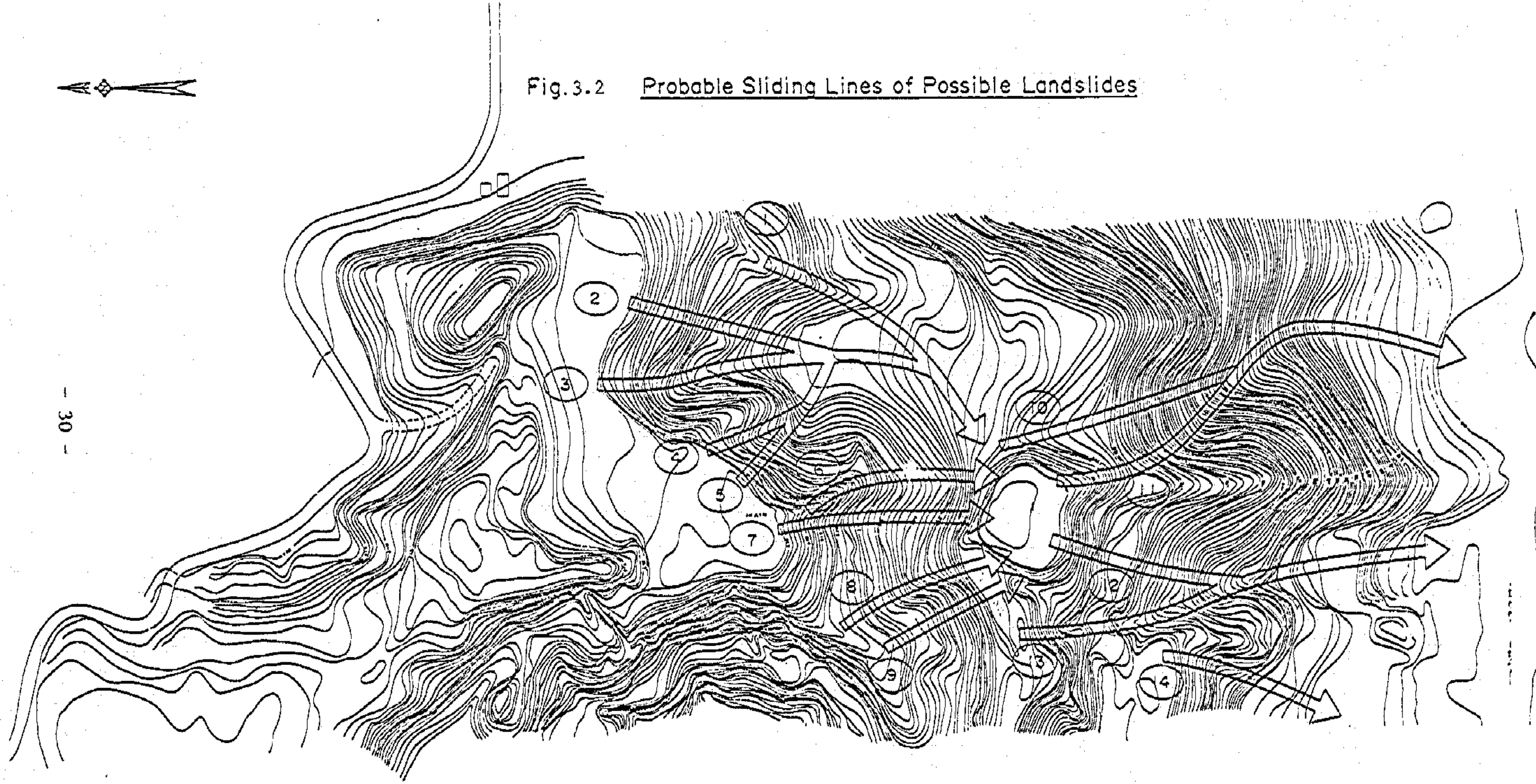


Fig. 3.3 Rehabilitation Plan at the Ex-Batcher Plant Site

Fig. 3.3

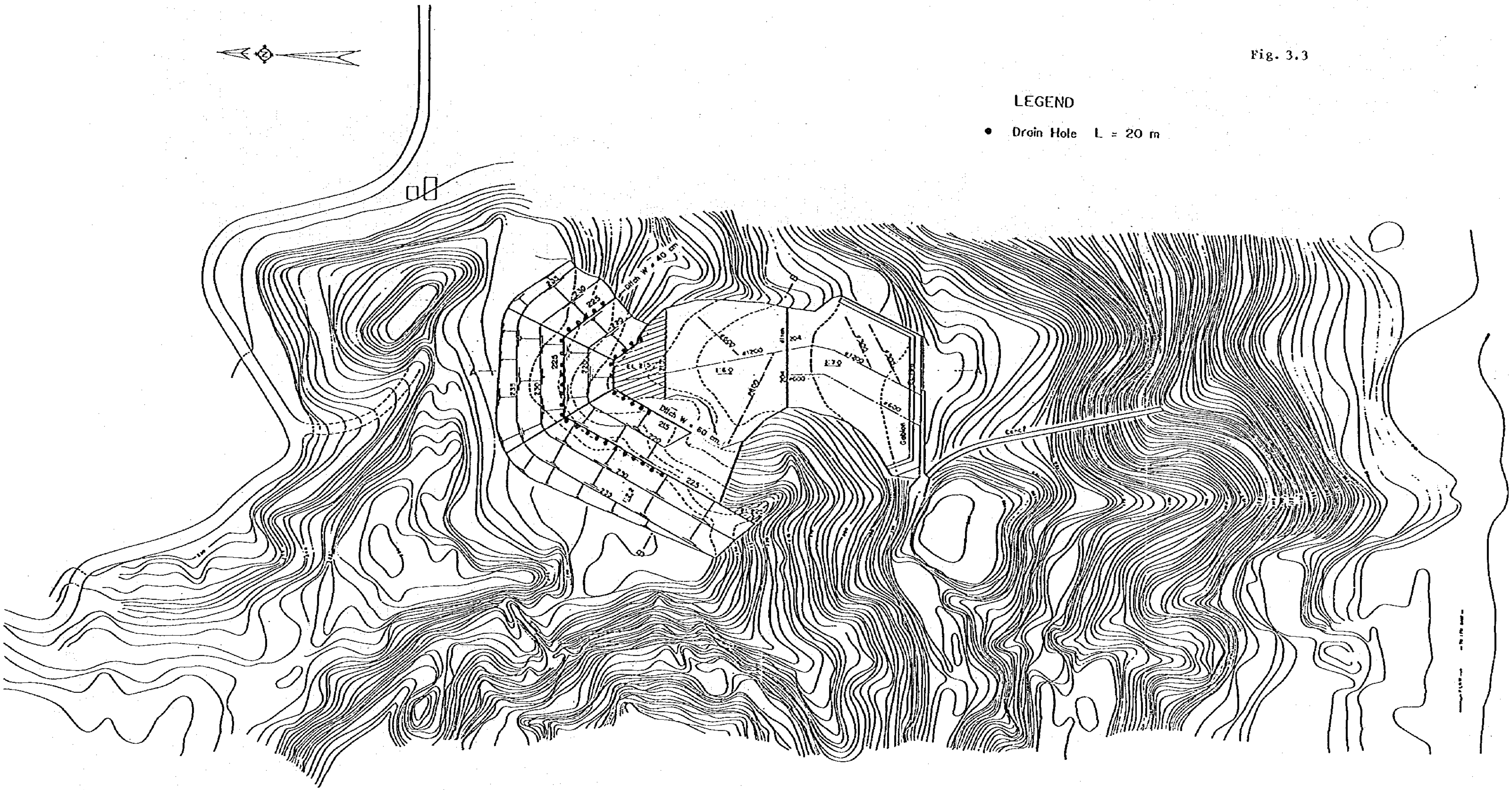


Fig. 3.4 Longitudinal Profile after Slope Reformation (Cross Section A-A)

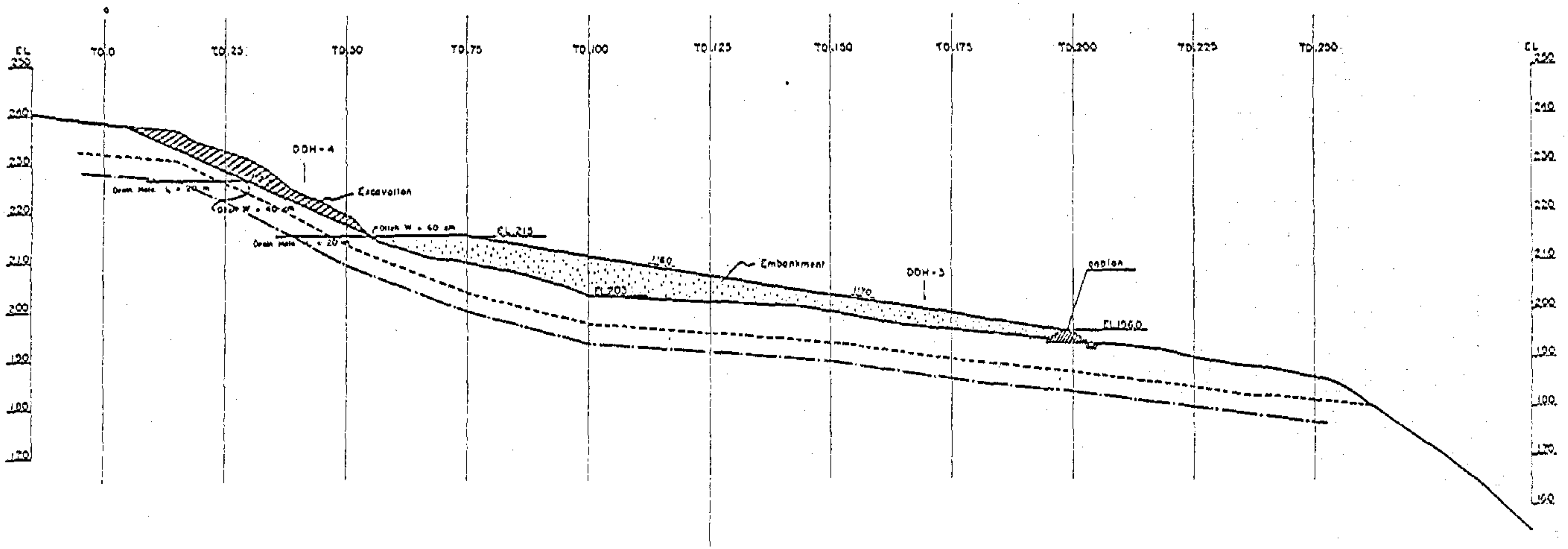
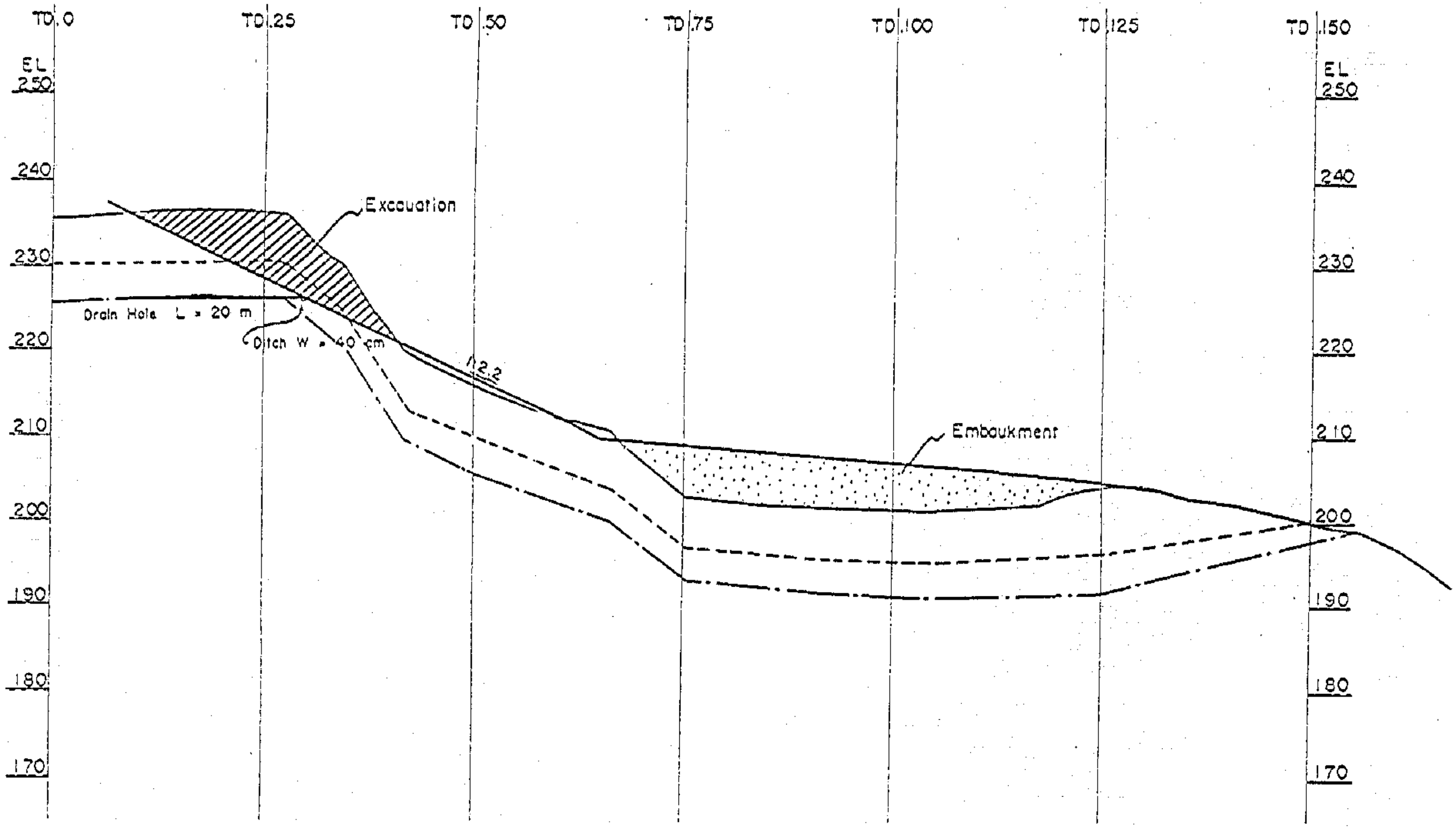


Fig. 3.5 Longitudinal Profile after Slope Reformation (Cross Section B-B)



4.4. Seepage through the Dyke

Section 5 of the Report deals with the studies on seepage through the dyke built on the left bank of the main dam. This investigation is one of the essential items for this Study.

Quantitative analyses of the seepage problem can be made using the data taken at the measurement weirs installed by NAPOCOR at three locations downstream of the dyke. Measurement has been continued since 1986, but the analyses for this Study were made on the basis of the measured data for a period from January 1987 to October 1988.

The amount of seepage measured at these three weirs consists of inflows from two sources; seepage through the dyke and rainfall. The seepage through the dyke is supposed to be directly influenced by the reservoir water level on the day when the measurement was done, but the inflow due to rainfall is supposed to come partly from surface runoff, partly from seepage through the adjoining mountain masses on an intermediate-term basis, and partly from seepage through the adjoining mountain masses on a long-term basis.

Therefore, it should be construed that the measured amount of seepage is influenced not only by rainfall on the same day when the measurement was done but by rainfall on days before the measurement was done. In this Study, the amount of rainfall influencing the seepage rate was considered to be comprised of the amount of rainfall on the same day, on the day before, on two days before, on three days before, ... on 35 days before the measurement was undertaken.

The measured amount of seepage was sorted out by the above components, using the least square method.

The location of the measurement weirs is indicated as SW-1, SW-2 and SW-3 in the Report. SW-1, located at the left bank of the dyke at EL 138 m, is to principally measure the amount of seepage probably through the left bank side of the dyke. SW-2, located immediately downstream of the center of the dyke at EL 114 m, is to principally measure the amount of seepage probably through the central portion of the dyke. SW-3, located at some 80 meters downstream of SW-2 at EL 110 m, is to measure the amount of seepage through the dyke as a whole. (Refer to Fig. 4.1.)

The results of calculations for the analyses can be summarized as follows:

(1) The maximum rate of seepage supposedly through the dyke during the period in which the data were available is estimated as follows:

At SW-1 : 19 ℓ/sec.
At SW-2 : 11 ℓ/sec.
AT SW-3 : 30 ℓ/sec.

(2) Irrespective of the location of the weirs, most of the measured amount of seepage during the dry period (December through May) is believed to be influenced by the reservoir water level, while most of that during the rainy period (June through November) is believed to be influenced by rainfall.

(3) Judging from the relation of the measured amount of seepage among the three weirs, it is very probable that seepage through the dyke would mostly flock to SW-1 and SW-2. (The amount of seepage through the right bank side of the dyke, if any, is considered negligible.)

(4) It can be estimated that the elevations of seepage through the dyke would be at EL 190 m or higher. It can also be estimated from the measurement of ground water level at drill holes provided downstream of the dyke that seepage through the dyke would predominantly come from the left bank side of the dyke, and would decrease with distance toward the center of the dyke.

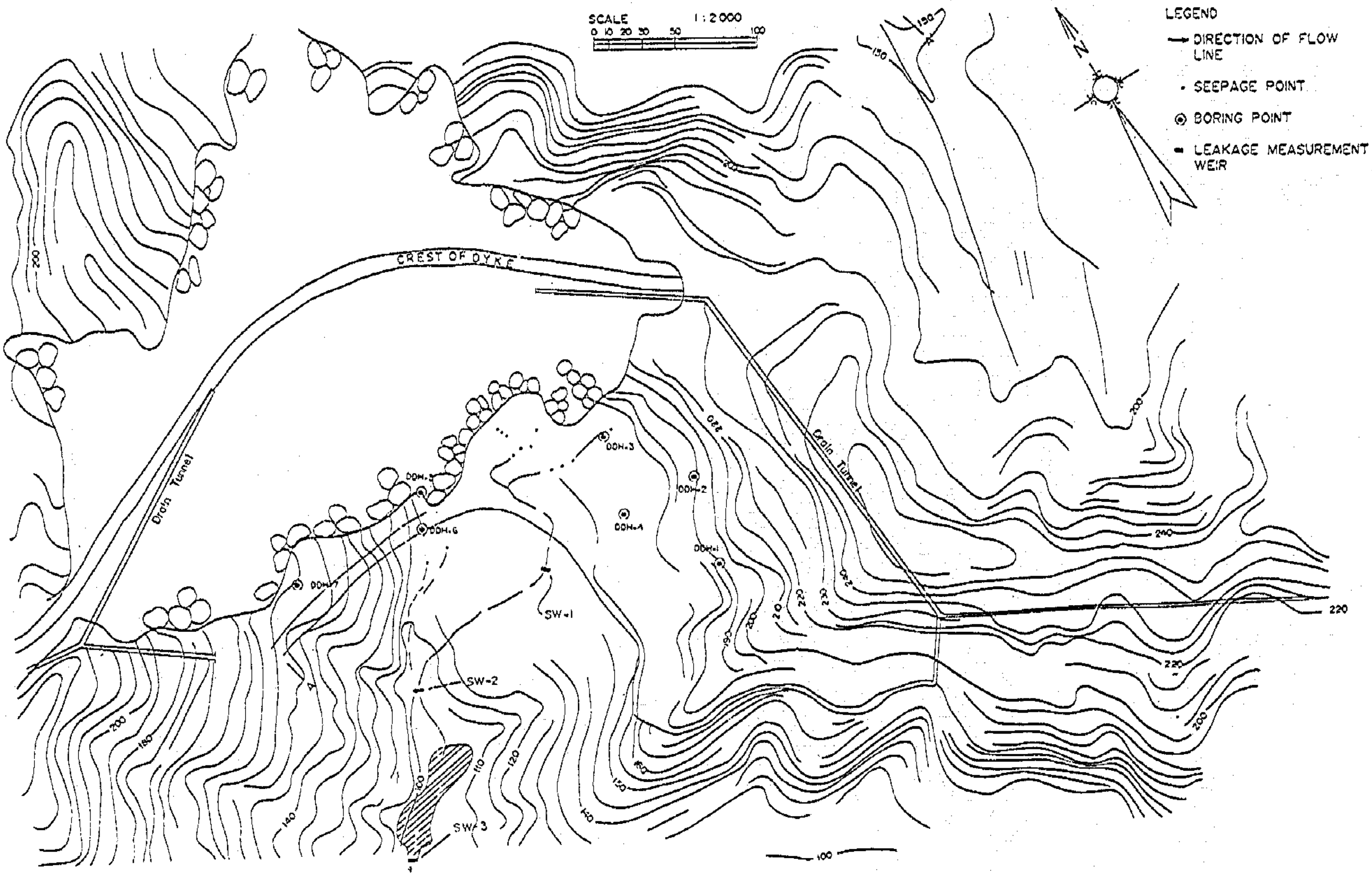
(5) The loss of water throughout the year due to seepage through the dyke is estimated to be about 261,000 m^3/year (159,000 m^3/year at SW-1, and 102,000 m^3/year at SW-2) or 8.3 ℓ/sec. at the average rate, and the resultant loss of energy production is estimated to be 56,100 kWh a year at most.

If it is planned to grout the dyke as a solution to reduce the amount of seepage, it is considered adequate only to cover a section of mountain masses around the left bank abutment toward the center of the dyke at elevations higher than EL.190 m.

However, the solution by means of grouting may not be so effective from the economical point of view, in that the estimated losses of water due to seepage through the dyke would not be so substantial and so would be the resultant losses of energy output, while it is difficult to stop seepage completely by this method.

The state of seepage through the dyke seems to remain unchanged at present, but it may change if some outer forces such as an earthquake of large magnitude would act on the dyke and surrounding mountain masses. It is therefore very essential to continue monitoring of seepage on a long-term basis.

Fig.4.1 Location of Measurement Weir SW - 1, SW - 2, SW - 3



4.5. Rehabilitation Plans

Items discussed in Section 6 of the Report for the necessity of rehabilitation are as follows:

i. Review of the spillway capacity.

ii. Analyses of the stability of the dam and the dyke.

iii. Investigation of the possibility of landsliding at the ex-batcher plant site.

iv. Investigation of seepage through the dyke.

v. Investigation of leakage from the penstock.

The necessity of rehabilitation for each item is discussed in detail in the corresponding Sections of the Report. The summary of the discussions is as follows:

(1) The spillway capacity is adequate, and no rehabilitation is necessary to be given under the present circumstances.

(2) There may be cases where the safety factor of the slopes of the dam and the dyke against sliding could come down below 1.0 in the event that a large earthquake (with Kh of 0.15g) would occur, but such unstable zones may be limited only to the portions close to the surface or confined locally, and the resultant sliding, if happened, may not produce any serious effect on the stability of the entire structures.

The conclusion that can be derived from the analyses is that it is not necessary to implement any rehabilitation plan until there occurs such a large Kh earthquake that may cause any surface layer sliding. The plan may and should be worked out only when such a phenomenon arises.

(3) Since any recurrence of large-scale landsliding at the ex-batcher plant site may produce a serious effect on the water supply to Metro Manila, it is recommended to provide protective measure against possible landsliding, including reformation of the slope at the site and provision of drain holes and drain ditches. The cost estimate for the slope reformation work is estimated to be US$437,000. (Refer to Table 5.1. and Fig. 5.1.)

(4) The present state of seepage through the dyke is not considered dangerous in terms of the structural stability. As a solution to reduce the amount of leakage, it is recommended to provide curtain grouting. The cost estimate for the grouting work is estimated to be US$380,000. (Refer to Fig. 5.2. and Table 5.2.)

(5) A particular emphasis is placed on the necessity of rehabilitation of the penstock, but it is not possible to work out a detailed repair program, because circumstances did not permit any internal inspections with the penstock being dewatered.

Table 5.1 Cost Estimate of the Slope Reformation Work

Description	Specification	Unit	Quantity	Unit Price (Peso) F . C	Unit Price (Peso) L . C	Amount (Peso) F . C	Amount (Peso) L . C	Amount (Peso) Total
1. Clearing and grubbing		m^2	16,000	32	34	512,000	544,000	1,056,000
2. Excavation - 1		m^3	17,600	38	34	668,800	598,400	1,267,200
3. Excavation - 2		m^3	2,700	64	63	172,800	170,100	342,900
4. Embankment		m^3	19,200	18	11	345,600	211,200	556,800
5. Trench excavation		m^3	4,900	60	61	294,000	298,900	592,900
6. Gabion		m^3	1,545	68	570	105,060	880,650	985,710
7. Cobble stone		m^3	1,950	68	559	132,600	1,090,050	1,222,650
8. Concrete		m^3	710	196	1,629	139,160	1,156,590	1,295,750
9. Reinforcement		ton	33	521	24,500	17,193	808,500	825,693
10. Concrete piping	ϕ 1,200	m	150	200	604	30,000	90,600	120,600
11. — do —	ϕ 600	m	150	72	216	10,800	32,400	43,200
12. — do —	ϕ 300	m	60	50	154	3,000	9,240	12,240
13. Boring	ϕ 75	m	700	1,000	800	700,000	560,000	1,260,000
14. P.V.C Piping	ϕ 50	m	700	8	43	5,600	30,100	35,700
Total						Peso 3,136,613	6,480,730	9,617,343
						US$(Equiv) 142,573	294,579	437,152

Fig. 5.1 Work Schedule of the Angat Dam Rehabilitation Project

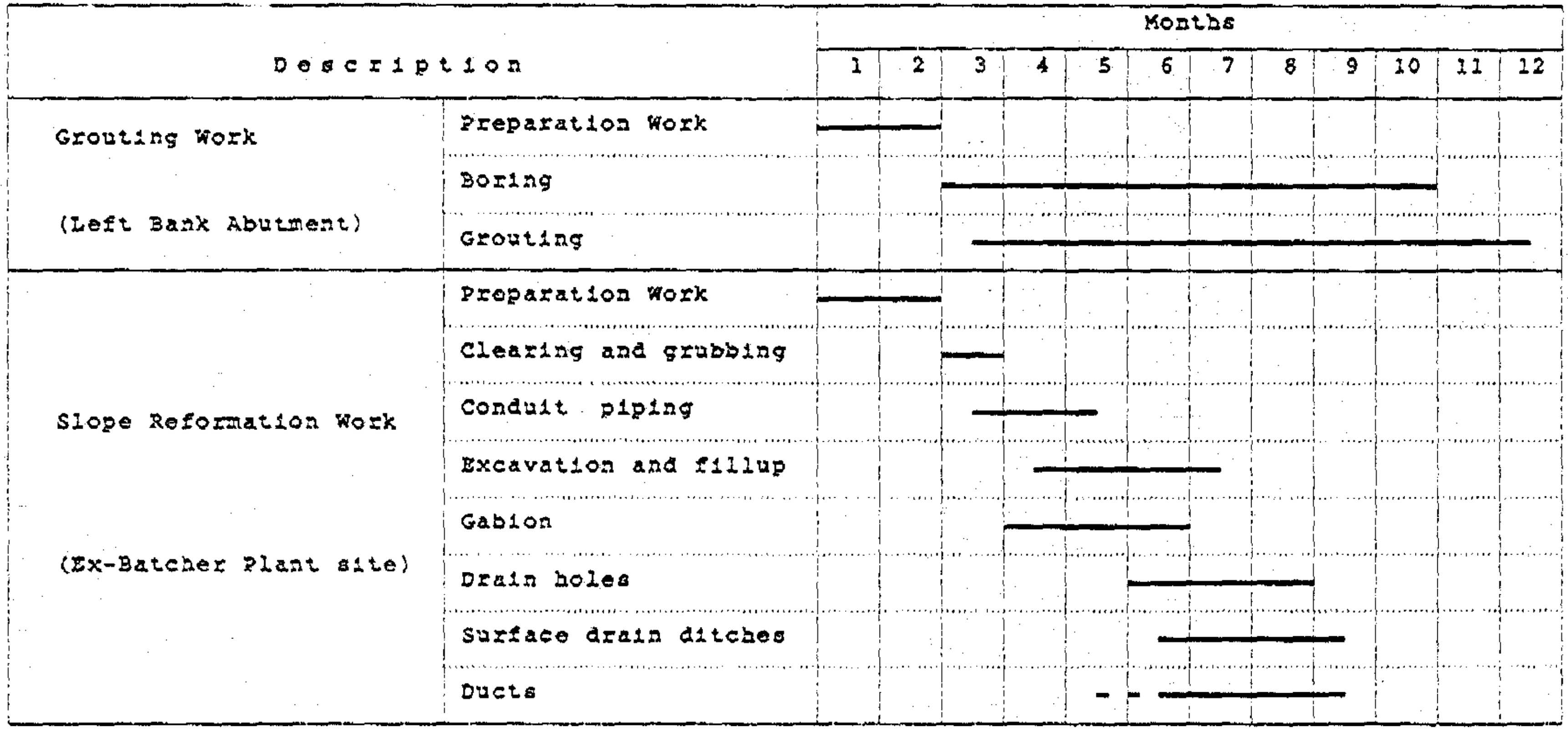


Fig. 5.2 Grouting for Prevention of Seepage Through the Dyke (Left Bank Abutment)

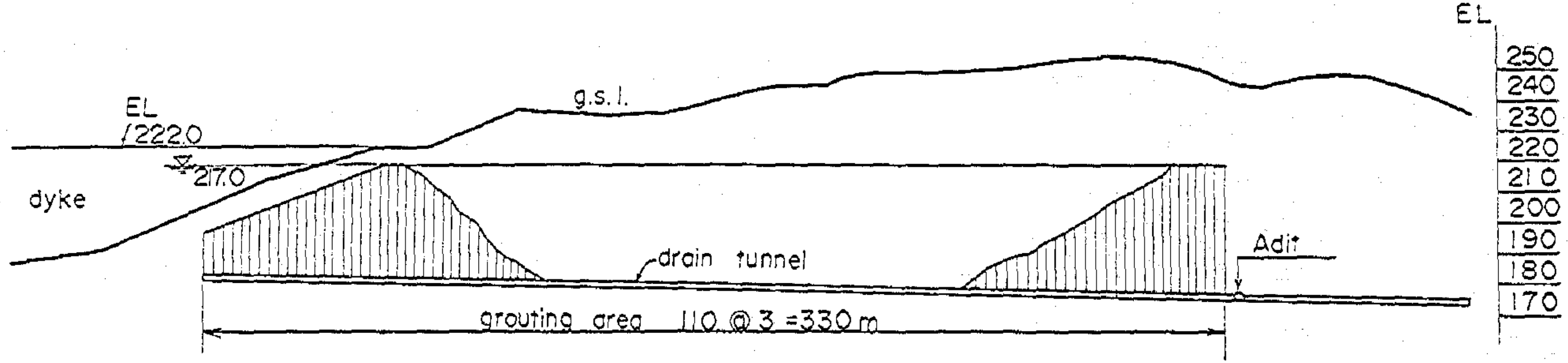


Table 5.2 Cost Estimate of the Grouting Work

Description	Specification	Unit	Quantity	Unit Price (Peso)		Amount (Peso)		
				F . C	L . C	F . C	L . C	Total
1. Boring (Non-core)	ϕ56	m	2,770	924	726	2,559,480	2,011,020	4,570,500
2. 〃 (Core recovery)	ϕ56	m	900	1,188	792	1,069,200	712,800	1,782,000
3. Cement Grouting		ton	560	350	2,898	196,000	1,622,880	1,818,880
4. Mortal Grouting		m^3	50	674	2,739	33,700	136,950	170,650
Total						Peso 3,858,380	4,483,650	8,342,030
						US$(Equiv) 175,381	203,802	379,183

4.6. Economic Analyses

Section 7 of the Report deals with economic analyses and appraisal of the rehabilitation plans. Considered necessary or desirous to be undertaken for the Angat Dam rehabilitation Project are the protective measure against possible landsliding at the ex-batcher plant site, and the preventive measure against seepage through the dyke.

The protective measure against possible landsliding at the ex-batcher plant site should be given top priority, because any recurrence of large-scale landsliding may produce a serious effect on the water supply to MWSS, and a far-reaching consequence to the social activities.

The benefit that can be derived from this rehabilitation plan should be evaluated from the standpoint of securing social stability, and it is extremely difficult to evaluate the benefit numerically. Therefore, the plan was not given any detailed economic analysis in the Report.

Since water seeping through the dyke is flowing into the river upstream of the Ipo dam which is provided with intake facilities for water supply to MWSS, the seepage problem is not producing any ill effect on the water supply. Hence, the benefit from the preventive measure against seepage through the dyke should be evaluated only in terms of the resultant recovery of power production.

The monetary loss of power production due to seepage of water was evaluated on the basis of would-be revenues from sales at the discount rate of 14%, and the cost worth expending for the measure was calculated. As the result, it was found that the cost should be held down to the amount equivalent to US$19,310 at most.

On the other hand, the cost for the grouting work is estimated to be as much as US$380,000. Nevertheless, it is believed next to impossible to stop seepage completely by this means.

Therefore, unless seepage through the dyke is increasing, it is not considered necessary to provide the grouting work at this point in time. It is, however, recommended to continue monitoring of seepage and to have an accurate knowledge of changes in the behavior, so as to determine the most appropriate time for implementation of the grouting work.

4.7. Safety Control Standards of the Dam and Associated Structures

Section 8 of the Report deals with items of monitoring to be done continuously and on a regular basis for the safety control of the Angat dam, and the method of judgement on the monitoring results.

Monitoring activities to be continued on a regular basis are as follows:

(1) Check the deformations of the dam and the dyke by collimation survey.

(2) Check the amount of seepage through the dyke.

(3) Check the possibility of landsliding at the ex-batcher plant site by using inclinometers.

(4) Check the amount of leakage from the penstock.

(5) Check the water quality at the intake and leakage measurement weirs.

4.8. Safety Control System of the Dam

Section 9 of the Report discusses what system to be established for the safety control of the Angat dam.

The Angat Power Plant plays not merely an important part in supplying electricity to the Luzon grid but a critically important role in maintaining water supply to Metro Manila. It is, therefore, essential to make assurance of the dam's safety control doubly sure.

It is recommended in particular for the Angat Power Plant to post a civil engineering staff who is capable of taking emergency steps as the demand arises, because the plant is located relatively distant from the Northern Luzon Regional Center which is responsible for maintenance and operation of the Angat plant and other four hydro power plants in the Northern Luzon Area.

4.9. Leakage from the Penstock

The last section as a supplement to the Report deals with the studies on leakage of water from the penstock. The result is as summarized below:

(1) It appears that the locations of leakage remained unchanged between the measurements in 1979 and 1982. This implies that no new, additional leakage would have taken place during the period from 1979 to 1982.

(2) The locations of leakage are approximated to be in the distance between 50 and 70 m and between 100 and 160 m from the sinking shaft, or in the vicinity of the bifurcation point and the connection of the penstock with the concrete tunnel.

(3) It is estimated that the leakage rate at the 1982 measurement would have increased by 10-40% at least and by 20-80% at most over the leakage rate of the 1979 measurement.

(4) The permeability of the mountain mass is estimated at about 10^{-4} m/sec.

(5) The rate of leakage into the surrounding mountain mass as of 1982 is estimated at about 0.7 m^3/sec.

The above is estimated from the studies made without internal inspection of the penstock, and it is still unknown where the exact locations of leakage are and how large is the extent of defects, nor is it known whether there would exist cracks prone to a serious damage to the penstock. Accordingly, it is not possible to determine what solutions should be taken as the measure for repair or rehabilitation. Should there be any such cracks, there is a possibility of further crack formation from their tips due to stress concentration, thereby posing a risk of further increase in the leakage rate.

Therefore, it is absolutely necessary to make internal inspections to identify the following items to establish the repair measures at the soonest possible convenience:

(1) Establishment of the repair method by welding, based on the results of chemical analyses of penstock materials.

(2) Confirmation of the actual locations of leakage and a close check of the damages.

(3) Check of the plate thickness of the penstock and the structure of the bifurcation.

(4) Establishment of the repair procedure to keep the penstock from disruption by ground water pressure when the penstock is dewatered during repair or after repair.

In order to make internal inspections of the penstock, it is necessary to shut down both of the main and the auxiliary power plants and dewater the penstock. To put the auxiliary plant out of service may, however, produce a serious effect on the water supply service to Metropolitan Waterworks and Sewerage System (MWSS), because it releases water through the turbines to the Angat river upstream of the Ipo dam, the water intake facilities for MWSS.

Only means to make it possible to dewater the penstock while keeping releasing water for MWSS is to bypass the penstock by discharging water through the outlet tunnels provided at the bottom of the dam. These were originally used as the diversion tunnels for dam construction. One of them was later equipped with a valve for discharging water, and the other was converted to an access passage to the valve chamber. These outlet tunnels are, however, in no way usable because they are now under water due to the rise of water level of the Angat river as a result of raising of the Ipo dam height and deposits of volume of slidden earth on the river basin caused by the large-scale landslide that occurred in 1986.

It is therefore recommended as a solution to break through the situation that the outlet of the access tunnel be closed to keep off water from the river, and an inclined shaft be provided to get into the tunnel and the valve chamber, thereby to regain the original function of the outlet tunnel.

REFERENCE TABLES AND FIGURES

LIST OF TABLES AND FIGURES

Fig. R1 Angat River Basin

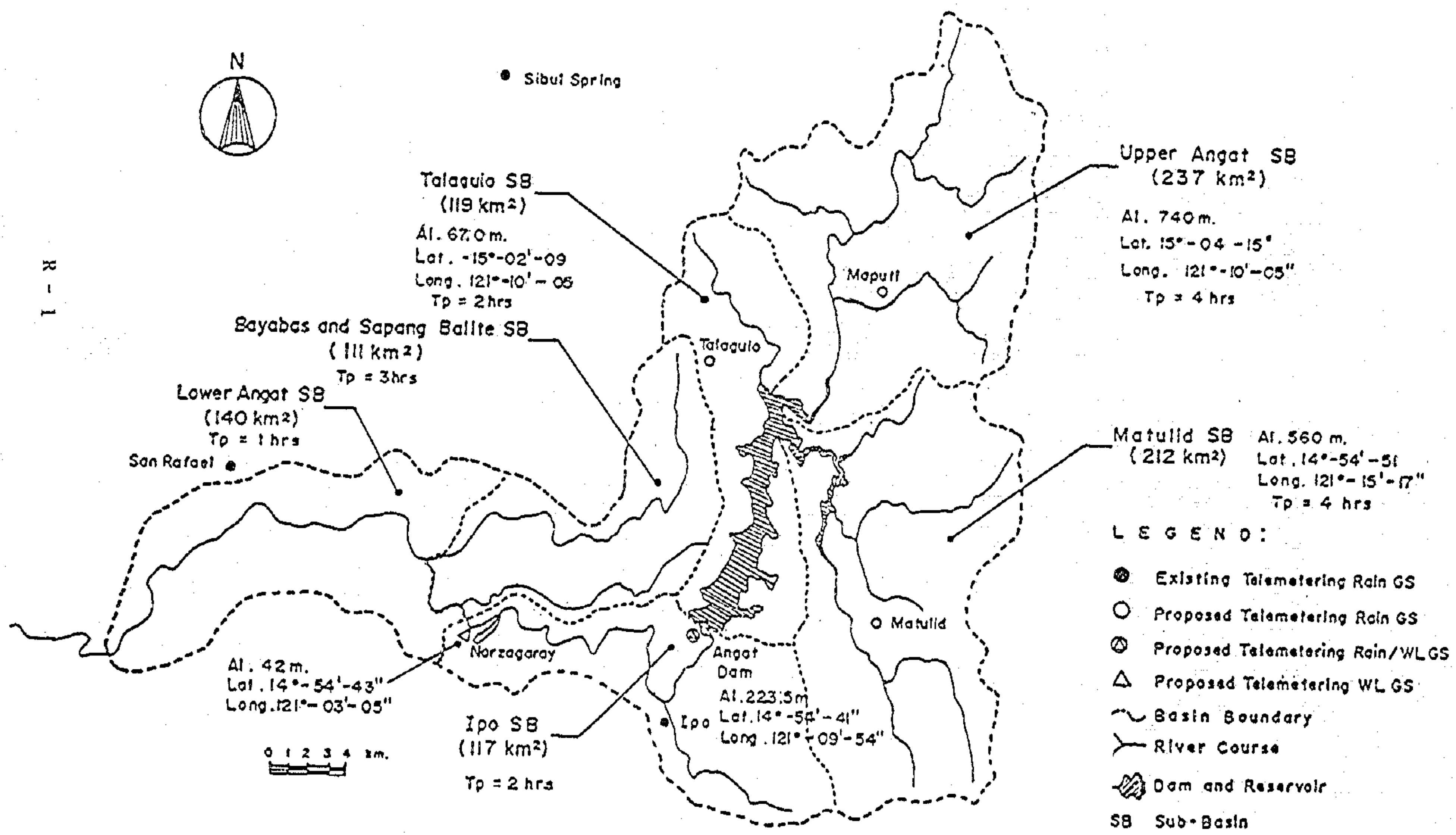


Fig. R2 Change in the Probable Monthly Rainfall by Month at the Norzagaray Site

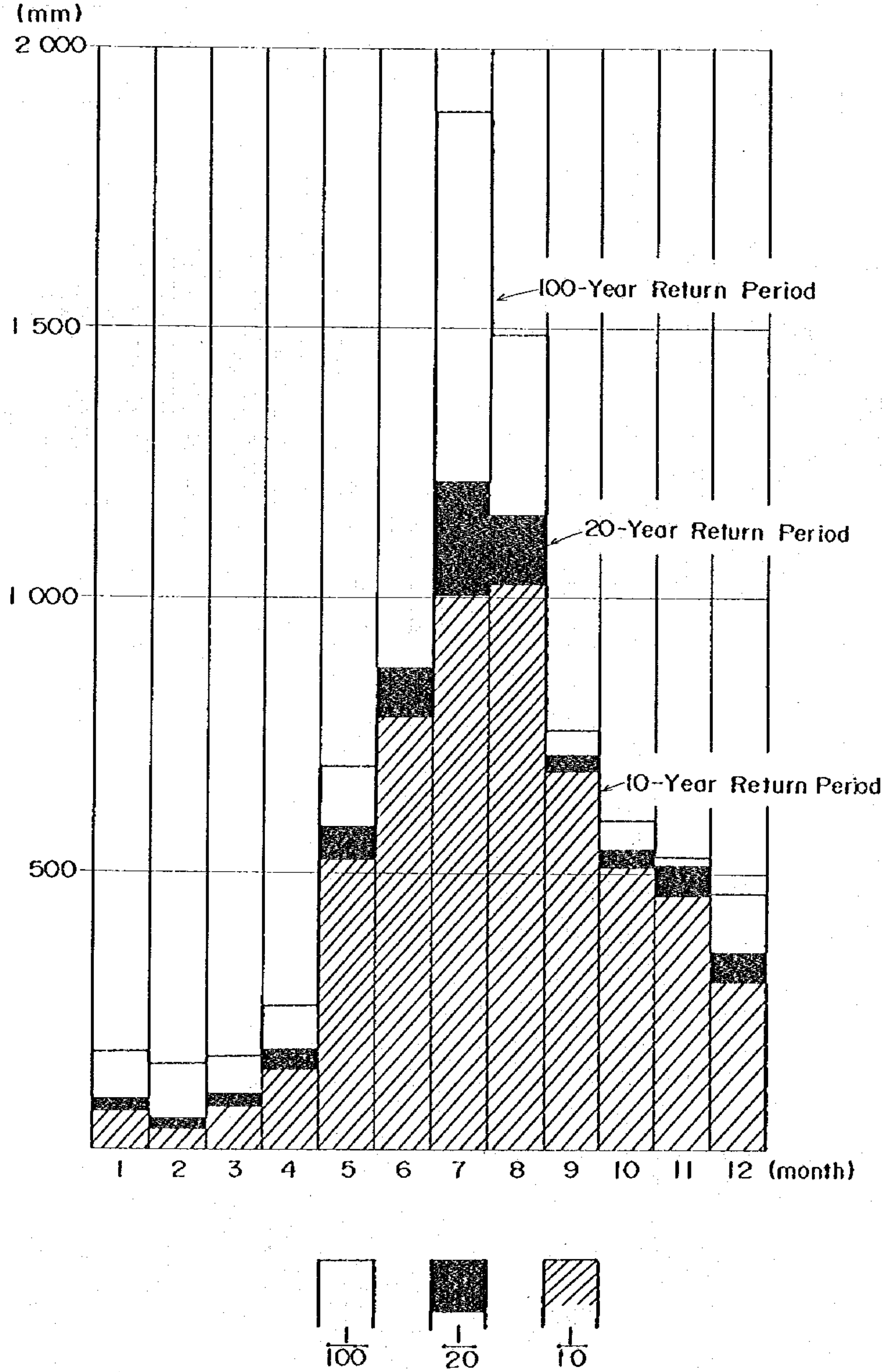


Table R1 Probable Monthly Rainfall at the Norzagaray Gauging Station

(mm)

Return Period (Years)	Average	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Expected	31.3	0.7	0.4	0.4	0.4	3.3	60.5	114.0	120.4	42.3	20.4	12.3	0.3
2	227.5	17.0	7.2	20.6	39.8	212.8	420.0	463.9	564.3	438.9	297.7	215.3	82.6
5	381.0	43.4	21.2	53.5	101.6	420.3	652.7	770.3	850.6	616.9	450.4	375.3	215.8
10	460.8	67.8	37.9	80.6	146.2	526.1	783.7	1004.3	1024.1	683.2	513.1	461.7	300.5
20	532.5	91.3	57.6	103.3	179.2	586.2	873.1	1212.9	1153.0	714.0	544.6	516.5	358.8
50	633.8	138.8	107.0	144.1	231.7	663.2	1007.3	1600.0	1357.0	749.1	582.3	593.4	431.7
100	696.0	175.0	155.5	170.4	260.3	694.0	1079.5	1885.0	1479.7	760.5	595.7	630.0	465.8
200	756.4	214.7	219.3	195.4	284.3	715.3	1141.2	2192.0	1592.7	767.7	604.1	658.7	490.8

Table R2 Annual Maximum Inflow and Daily Maximum Runoff

Year	Annual Maximum Inflow (m^3/sec)	Daily Maximum Runoff ($10^6 \times m^3$)				
		Single Day	Two Consective Days	Three Con-Consective Days	Four Consective Days	Five Consective Days
1957	3,483	102.6	144.8	189.8	207.2	216.7
1958	2,499	61.2	106.5	125.3	139.9	154.3
1959	2,324	51.8	103.7	124.3	136.4	146.8
1960	3,476	119.3	174.8	222.6	270.6	298.8
1961	2,644	64.6	86.1	109.2	120.6	133.5
1962	4,643	189.8	284.5	371.4	418.3	451.0
1963	1,993	32.4	59.3	78.4	87.6	93.2
1964	2,762	76.8	95.5	108.6	126.5	141.6
1965	2,192	47.0	84.6	109.9	125.0	137.5
1966	2,179	46.4	84.3	108.5	124.4	133.9
1967	3,882	137.4	210.1	217.5	223.5	225.0
1968	2,177	51.2	89.6	124.8	146.5	156.0
1969	2,615	69.7	96.9	119.7	156.8	180.9
1970	2,159	86.7	119.6	143.9	156.4	167.0
1971	5,396	86.4	140.3	191.5	226.1	274.4
1972	2,914	85.7	159.9	234.4	287.6	333.7
1973	2,882	83.9	144.5	168.0	180.6	185.1
1974	4,845	196.3	308.2	355.2	398.1	415.9
1975	2,058	78.6	142.3	168.4	188.9	203.3
1976	3,439	138.8	225.0	311.2	381.4	438.8
1977	2,205	110.9	131.3	138.9	144.4	162.6
1978	5,650	274.5	314.8	340.6	356.4	374.7
1979	1,860	66.2	93.5	110.9	136.3	151.0
1980	3,211	134.7	195.3	225.3	246.9	258.6
1981	3,760	112.9	164.1	182.6	194.5	203.3
1982	4,939	98.3	124.4	134.6	138.3	143.4
1983		42.9	62.4	81.2	93.3	99.9
1984		107.6	165.5	203.0	241.7	252.0
1985		129.5	171.5	196.0	218.2	233.3
1986		82.8	132.5	158.3	174.7	196.0
1987						

Table R3.1 Probable Inflow Runoff to the Angat Dam (Single Day)

(10^6 m^3)

Calculation Method	Data Period \ Return Period (Yrs)	2	5	10	20	25	50	100	200
Log-Normal Distribution	1957 ∿ 1982	88.7	133.5	165.2	197.1	207.5	240.4	274.4	309.8
	1957 ∿ 1987	88.0	131.6	162.3	193.1	203.1	234.7	267.3	301.2
Moment	1957 ∿ 1982	88.7	134.9	167.9	201.2	212.1	246.6	282.5	319.9
	1957 ∿ 1987	88.0	132.7	164.5	196.3	206.8	239.7	273.8	309.2
Pearson III Distribution	1957 ∿ 1982	87.3	132.7	166.8	197.2	214.4	253.0	294.6	339.3
	1957 ∿ 1987	86.6	130.8	163.8	193.2	209.7	246.8	286.6	329.4

Table R3.2 Probable Inflow Runoff to the Angat Dam (Two Consecutive Days)

(10^6 m^3)

Calculation Method	Data Period \ Return Period (Yrs)	2	5	10	20	25	50	100	200
Log-Normal Distribution	1957 ∿ 1982	136.0	194.5	234.4	273.6	286.1	325.5	365.4	406.3
	1957 ∿ 1987	134.3	191.9	231.2	269.7	282.1	320.7	360.0	400.1
Moment	1957 ∿ 1982	136.0	196.3	237.7	278.5	291.7	332.8	374.8	417.8
	1957 ∿ 1987	134.3	193.3	233.9	273.7	286.6	326.7	367.7	409.6
Pearson III Distribution	1957 ∿ 1982	133.2	193.0	237.2	276.4	298.4	348.0	400.8	457.6
	1957 ∿ 1987	132.4	190.9	233.1	269.8	290.2	335.3	382.9	433.2

Table R3.3 Probable Inflow Runoff to the Angat Dam (Three Consecutive Days)

(10^6 m^3)

Calculation Method	Data Period \ Return Period (Yrs)	2	5	10	20	25	50	100	200
Log-Normal Distribution	1957 ∿ 1982	165.9	234.8	281.6	327.2	341.8	387.4	433.6	480.7
	1957 ∿ 1987	163.7	230.9	276.4	320.7	334.9	379.0	423.7	469.2
Moment	1957 ∿ 1982	165.9	236.9	285.5	333.0	348.2	396.0	444.4	494.0
	1957 ∿ 1987	163.7	232.6	279.5	325.3	340.0	385.9	432.4	479.9
Pearson III Distribution	1957 ∿ 1982	161.4	232.3	285.8	333.6	360.6	422.1	488.5	560.8
	1957 ∿ 1987	160.4	229.3	279.5	323.9	348.6	404.2	463.1	526.1

Table R3.4 Probable Inflow Runoff to the Angat Dam (Four Consecutive Days)

(10^6 m^3)

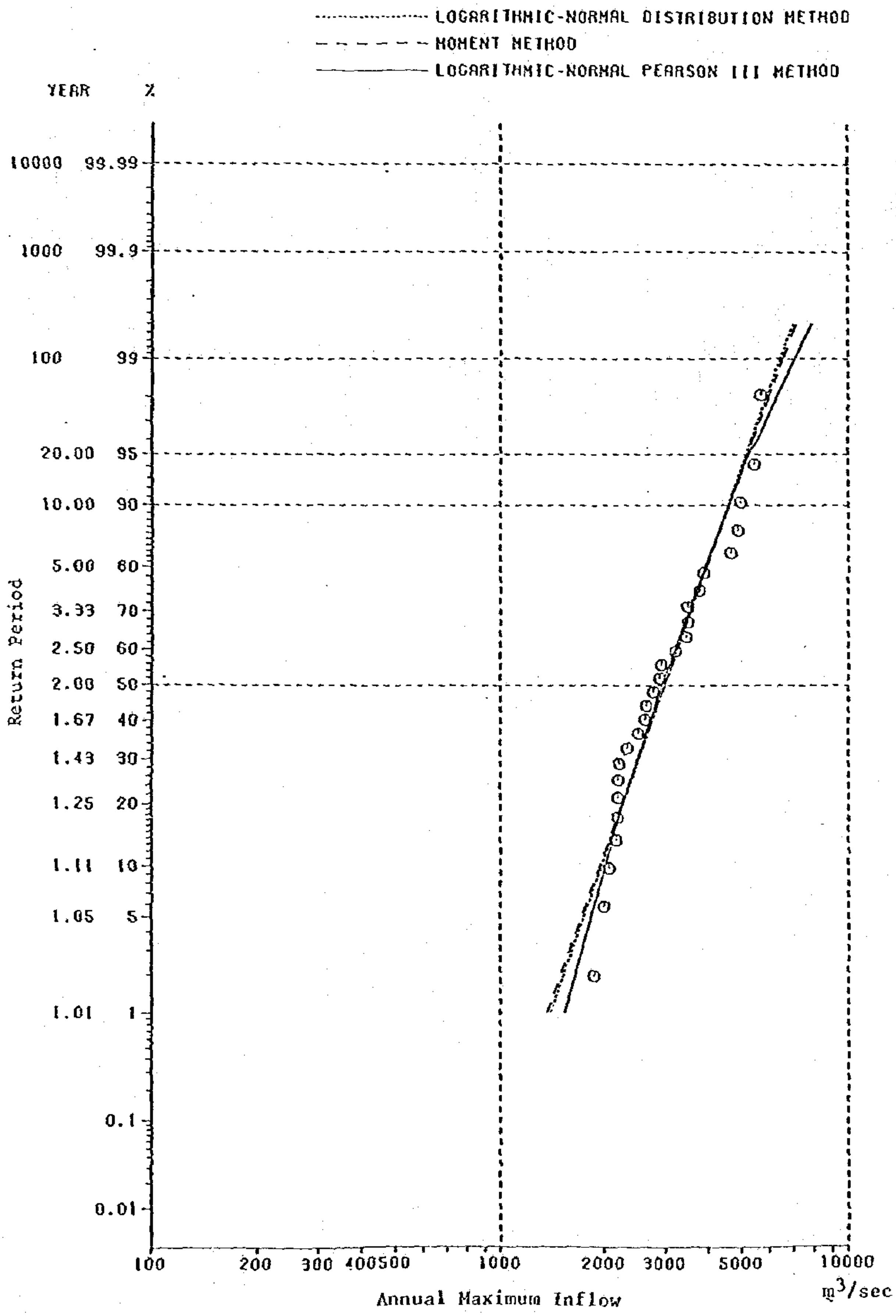
Calculation Method	Data Period \ Return Period (Yrs)	2	5	10	20	25	50	100	200
Log-Normal Distribution	1957 ~ 1982	187.0	264.8	317.5	368.9	385.4	436.8	488.9	541.9
	1957 ~ 1987	184.8	260.7	312.0	362.0	378.0	427.8	478.2	529.5
Moment	1957 ~ 1982	187.0	267.1	321.9	375.4	392.6	446.4	501.1	556.9
	1957 ~ 1987	184.8	262.6	315.6	367.2	383.8	435.6	488.0	541.6
Pearson III Distribution	1957 ~ 1982	180.7	261.1	323.0	379.6	411.6	485.8	567.2	656.7
	1957 ~ 1987	179.9	258.0	316.6	369.0	398.5	465.6	538.1	616.8

Table R3.5 Probable Inflow Runoff to the Angat Dam (Five Consecutive Days)

(10^6 m^3)

Calculation Method	Data Period \ Return Period (Yrs)	2	5	10	20	25	50	100	200
Log-Normal Distribution	1957 ~ 1982	203.0	288.0	345.7	402.0	420.1	476.5	533.6	591.8
	1957 ~ 1987	200.4	283.1	339.1	393.6	411.0	465.5	520.6	576.7
Moment	1957 ~ 1982	203.0	290.6	350.5	409.2	428.0	487.0	547.0	608.3
	1957 ~ 1987	200.4	285.2	342.9	399.3	417.4	473.9	531.3	589.9
Pearson III Distribution	1957 ~ 1982	196.1	284.0	351.7	413.8	448.9	530.2	619.6	718.0
	1957 ~ 1987	195.1	280.1	344.0	401.3	433.4	506.8	586.0	672.1

Fig. R3 Probable Annual Inflow to the Angat Dam



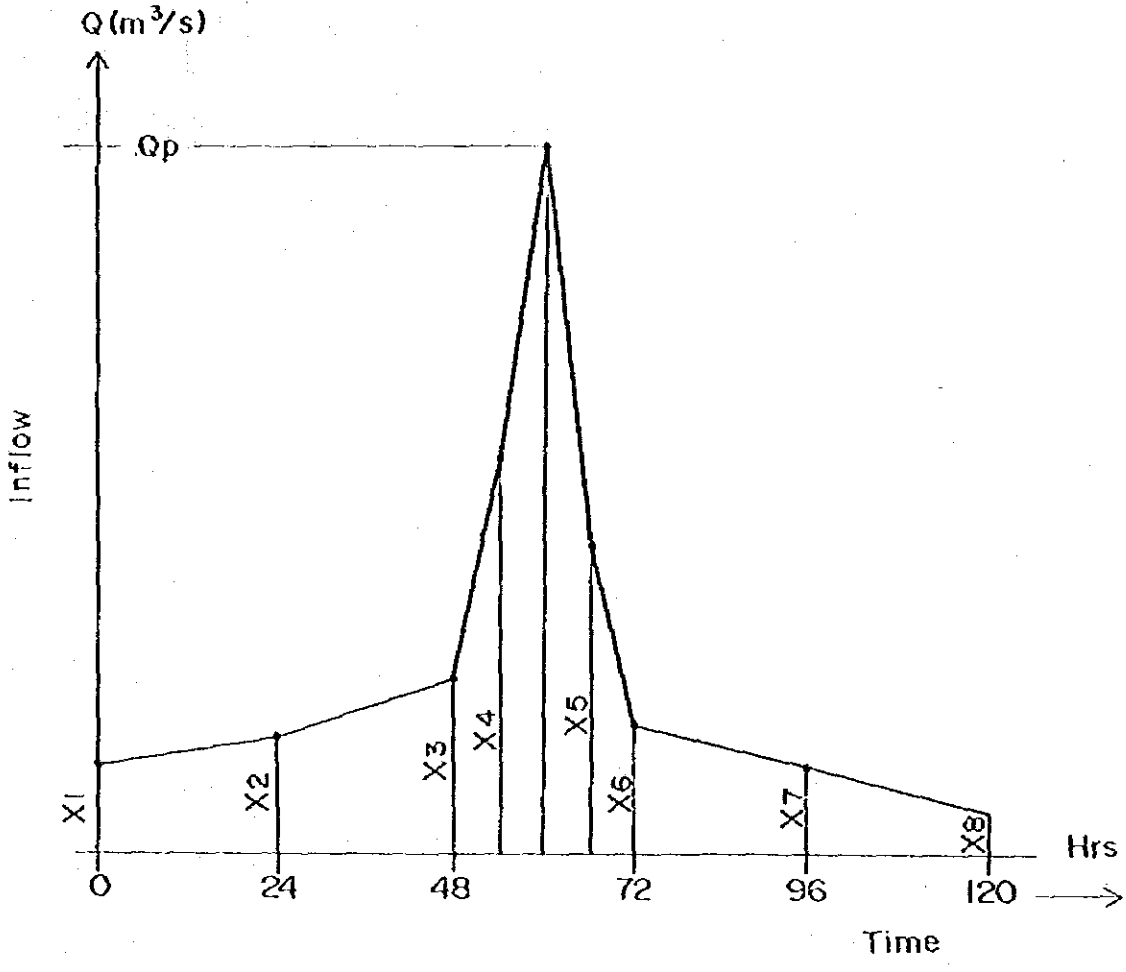


Fig. R4 Basic Pattern of the Probable Inflow

Table R4 Basic Pattern of the Probable Inflow by Node and Flood Frequency

(m^3/sec)

	Flood Frequency \ Node / Passing of Time (hrs)	X1	X2	X3	X4	Qp	X5	X6	X7	X8
		0	24	48	54	60	66	72	96	120
Case 1	1/200	1,000	1,200	1,500	3,300	7,850	3,000	1,500	900	500
	1/100	800	1,000	1,450	2,700	7,037	2,500	1,260	780	450
	1/50	600	890	1,320	2,300	6,267	1,950	1,050	660	400
	1/25	500	680	1,260	1,800	5,532	1,520	930	500	350
	1/20	450	620	1,240	1,500	5,202	1,370	860	460	330
	1/10	400	456	1,164	1,250	4,601	910	760	370	300
Case 2	1/200	1,000	1,200	1,500	7,850	4,040	2,300	1,500	900	500

Note : Case 2 shows the peak Inflow to occur six hours earlier than the normal case (Case 1). Qp in this case occurs at the X4 position (t = 54 hrs)

Table R5 Relation Between Flood Inflow and Passing of Time by Return Period (m^3/sec)

Passing of Time (Hrs)	Return Periods (Years) 200	100	50	25	20	10	Passing of Time (Hrs)	Return Periods (Years) 200	100	50	25	20	10
48	1500	1450	1320	1260	1240	1164	68	2500	2087	1650	1323	1200	860
49	1800	1658	1483	1350	1283	1178	69	2250	1880	1500	1225	1115	835
50	2100	1867	1647	1440	1327	1193	70	2000	1673	1350	1127	1030	810
51	2400	2075	1810	1530	1370	1207	71	1750	1467	1200	1028	945	785
52	2700	2283	1973	1620	1413	1221	72	1500	1260	1050	930	860	760
53	3000	2492	2137	1710	1457	1236	73	1475	1240	1034	912	843	744
54	3300	2700	2300	1800	1500	1250	74	1450	1220	1018	894	827	728
55	4058	3423	2961	2422	2117	1809	75	1425	1200	1001	876	810	711
56	4817	4146	3622	3044	2734	2367	76	1400	1180	985	858	793	695
57	5575	4869	4284	3666	3351	2926	77	1375	1160	969	840	777	679
58	6333	5591	4945	4288	3968	3484	78	1350	1140	953	823	760	663
59	7092	6314	5606	4910	4585	4043	79	1325	1120	936	805	743	646
60	7850	7037	6267	5532	5202	4601	80	1300	1100	920	787	727	630
61	7041	6285	5548	4863	4563	3986	81	1275	1080	904	768	710	614
62	6233	5229	4828	4195	3925	3371	82	1250	1060	888	751	693	598
63	5425	4773	4109	3526	3286	2756	83	1225	1040	871	733	677	581
64	4617	4016	3389	2857	2647	2140	84	1200	1020	855	715	660	565
65	3803	3260	2669	2189	2009	1525	85	1175	1000	839	697	643	549
66	3000	2500	1950	1520	1370	910	86	1150	980	823	679	627	533
67	2750	2293	1800	1421	1285	885	87	1125	960	806	661	610	516
							96	900	780	660	500	460	370

Table R6 Relation Between Flood Inflow and Permissible Rate of Flood Discharge

(t/hr)

Passing of Time (Hr)	Hourly Inflow of PMF Magnitude Flood (Qi)	Hourly Increment of Flood Inflow ($\frac{dQi}{dt}$)	Permissible Rate of Flood Discharge ($\frac{dQi}{dt}$)	Passing of Time (Hr)	Hourly Inflow of PMF Magnitude Flood (Qi)	Hourly Increment of Flood Inflow ($\frac{dQi}{dt}$)	Permissible Rate of Flood Discharge ($\frac{dQi}{dt}$)
1st Day				3rd Day			
0	100			0	2855	71	574
1	145	45	92	1	2909	54	579
2	145	0	92	2	2968	59	585
3	145	0	92	3	3032	64	592
4	179	34	103	4	3098	66	599
5	217	38	113	5	3167	135	606
6	265	48	125	6	3250	83	614
7	401	136	154	7	3342	92	623
8	503	102	211	8	3415	73	630
9	598	95	237	9	3525	110	640
10	668	70	254	10	3640	115	651
11	720	52	266	11	3741	101	660
12	763	43	276	12	3851	110	671
13	799	36	283	13	3967	116	681
14	830	31	290	14	4090	123	692
15	857	27	296	15	4305	215	710
16	879	22	300	16	4483	178	725
17	899	20	304	17	4661	178	740
18	967	68	317	18	4985	324	766
19	1008	41	325	19	5270	285	788
20	1048	40	332	20	5555	285	809
21	1154	106	351	21	5899	344	835
22	1236	82	365	22	6251	352	860
23	1311	75	377	23	6697	446	891
2nd Day				4th Day			
0	1366	55	386	0	7519	822	945
1	1410	44	393	1	7587	68	950
2	1467	57	401	2	7836	249	965
3	1514	47	408	3	8413	577	1000
4	1553	39	414	4	7961	-452	
5	1598	45	421	5	7426	-535	
6	1632	34	426	6	6807	-619	
7	1663	31	430	7	6279	-528	
8	1707	44	436				
9	1744	37	441				
10	1786	42	447				
11	1819	33	451				
12	1910	91	463				
13	1987	77	473				
14	2055	68	482				
15	2202	147	500				
16	2313	111	513				
17	2411	98	525				
18	2484	73	533				
19	2543	59	540				
20	2612	69	547				
21	2682	70	555				
22	2737	55	561				
23	2784	47	566				

Fig. R5 Discharge Rating Curve of Spillway

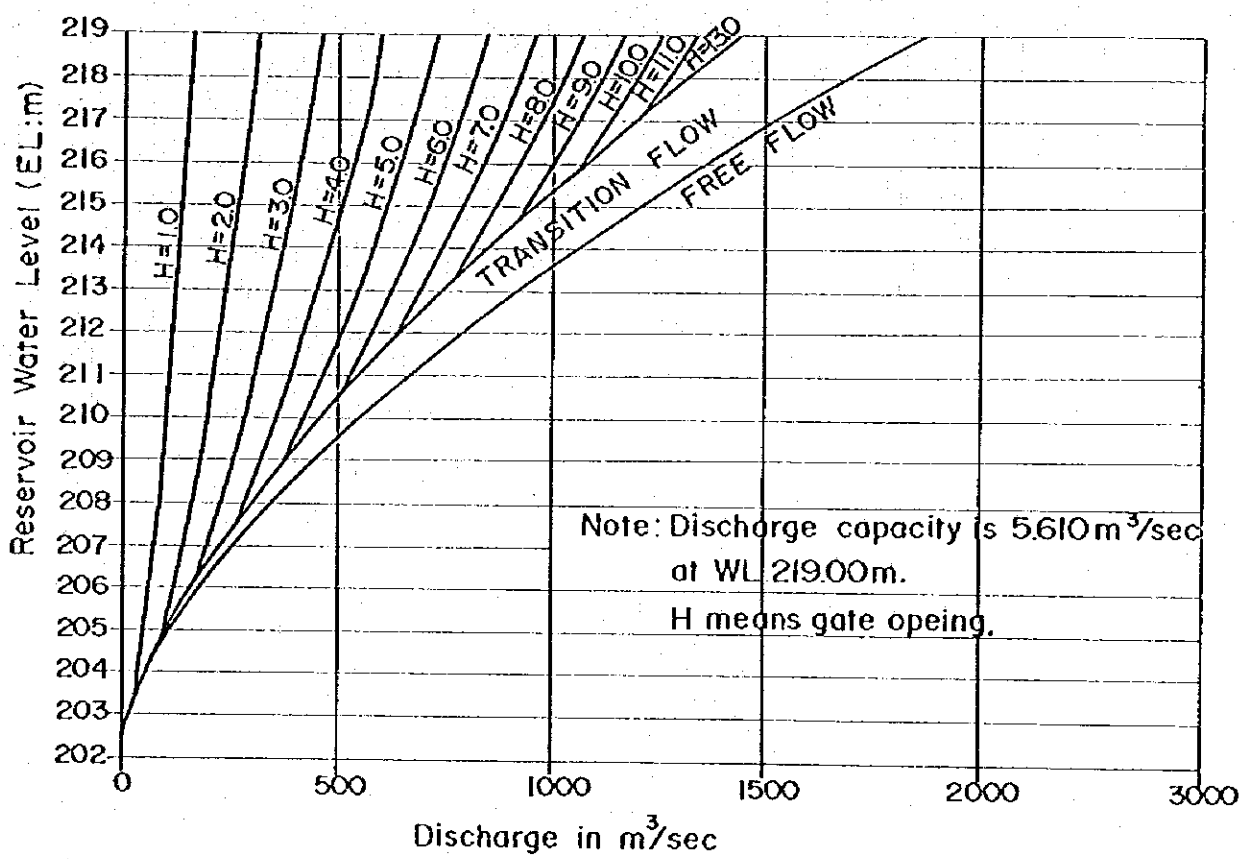
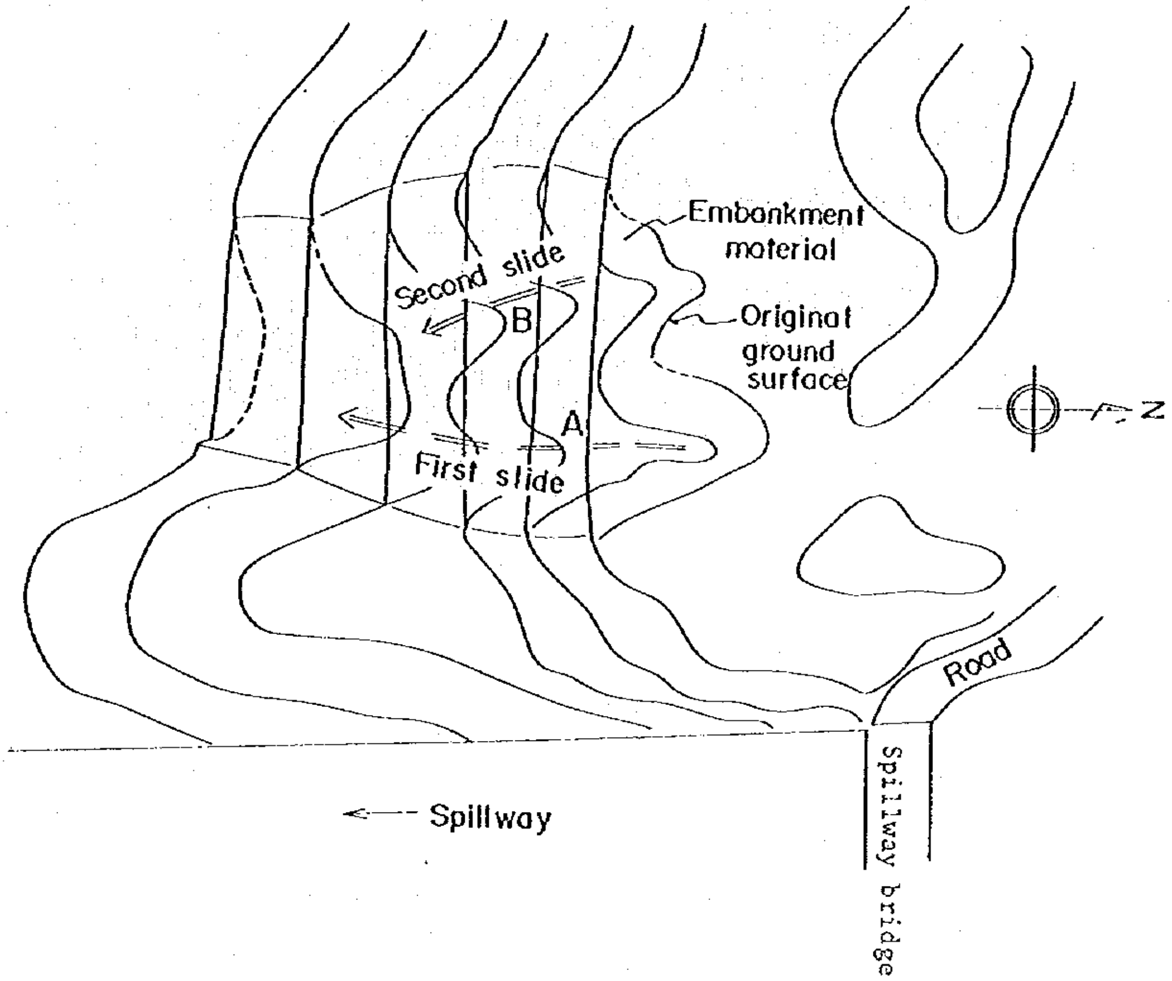


Table R7 Ratio of Peak Outflow to Peak Inflow during Floods for Various Return Periods

Return Period (Yrs)	When Peak Occurs	Water Level before Flood Inflow (EL-m)	Water Level with All Gates Full Open (EL-m)	Max. Water Level (EL-m)	Max. Discharge (m^3/sec.)	Ratio of Max. Outflow to Max. Inflow
200	60 hrs after inflow started	212	213	216.73	4432	0.56
100	- do -	212	213	215.86	4011	0.57
50	- do -	212	213	215.16	3688	0.59
20	- do -	212	213	214.24	3282	0.63
10	- do -	212	213	213.77	3078	0.67
200	- do -	212	212	216.33	4239	0.54
100	- do -	212	212	215.40	3801	0.54
50	- do -	212	212	214.68	3471	0.55
20	- do -	212	212	213.64	3025	0.58
10	- do -	212	212	213.15	2821	0.61

Fig. R6 Movement of Earth at the 1986 Landslide



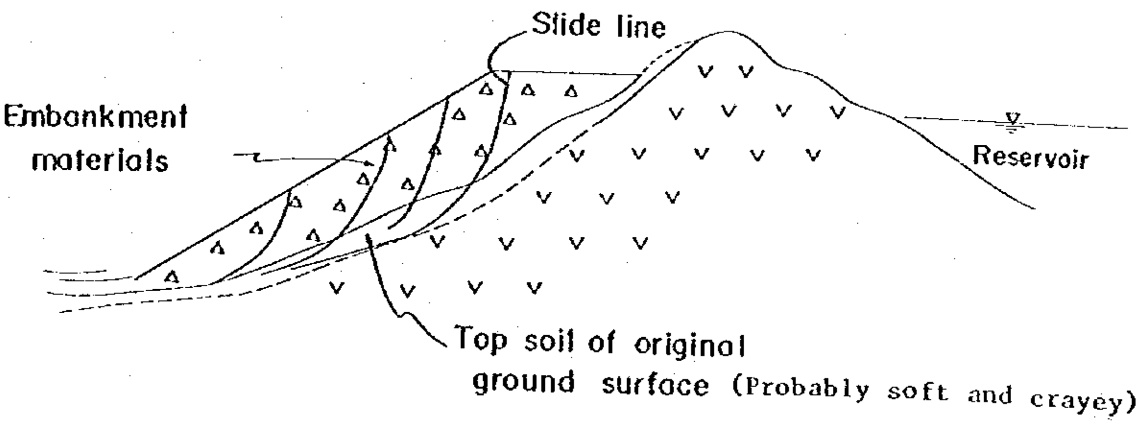


Fig. R7 ESTIMATED SLIDING DIRECTIONS OF AUG. 1986 LANDSLIDE

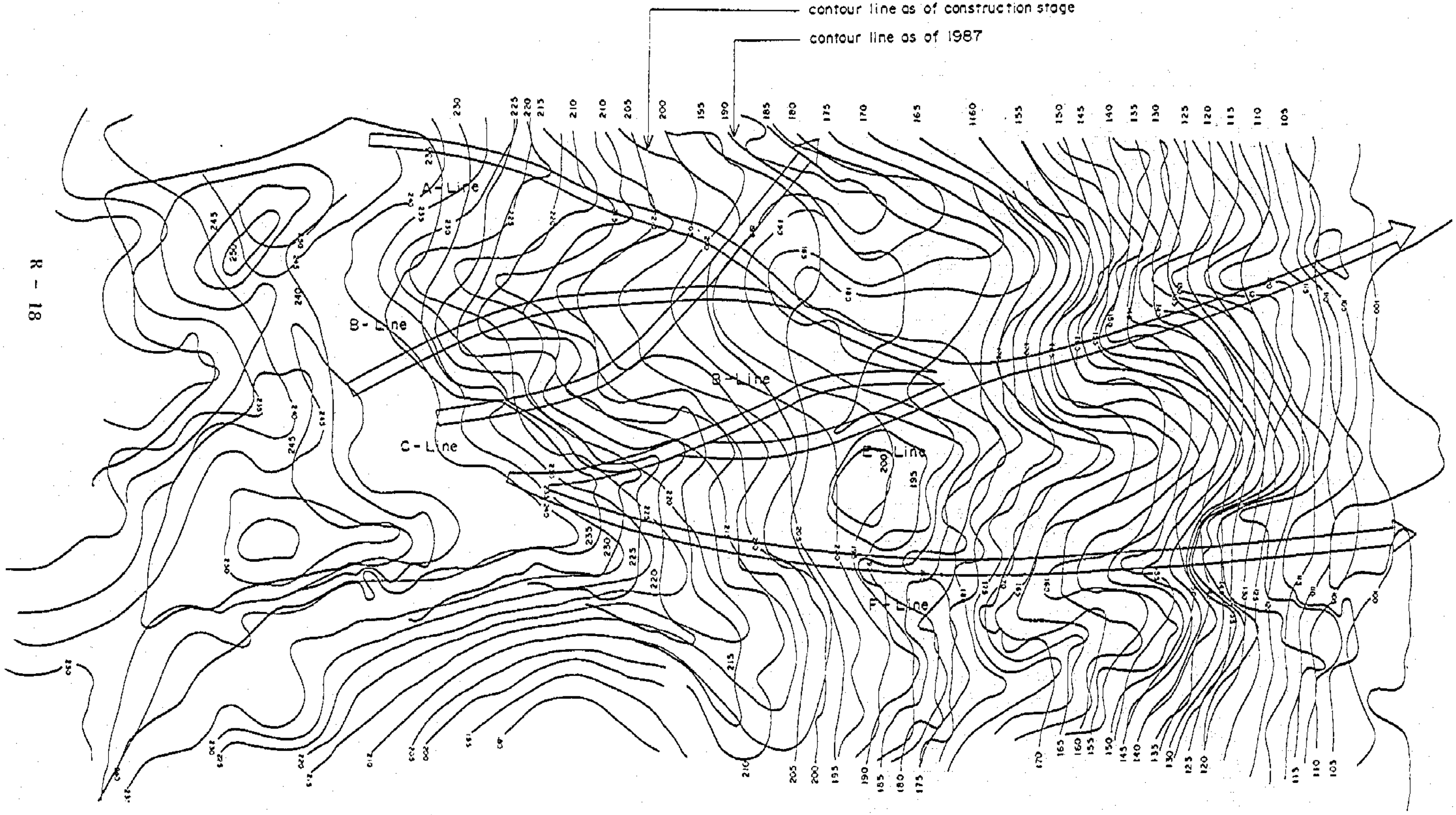


Fig. R8 LONGITUDINAL PROFILES ALONG THE ESTIMATED SLIDING LINES

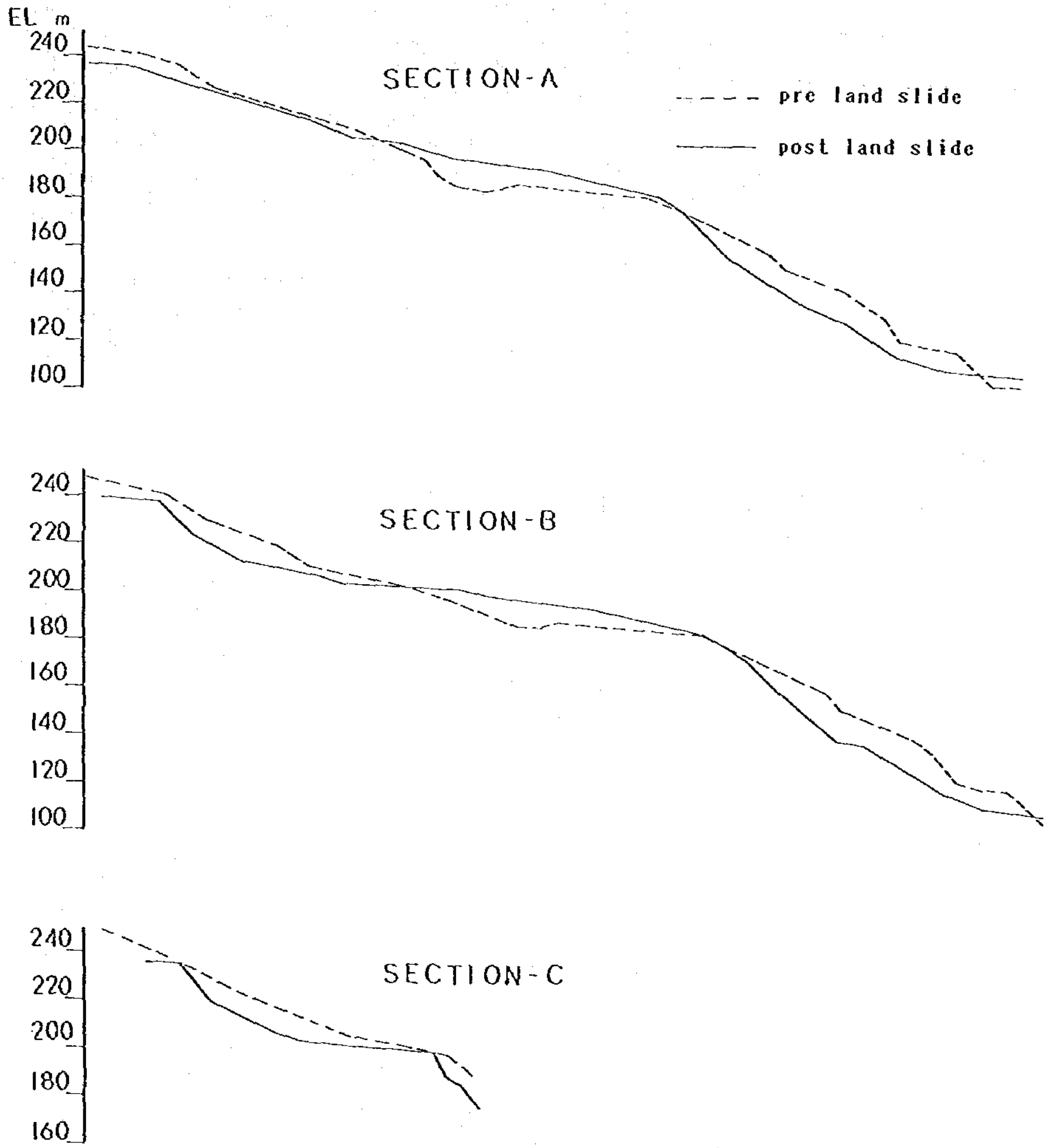


Fig. R8 LONGITUDINAL PROFILES ALONG THE ESTIMATED SLIDING LINES

(Continued)

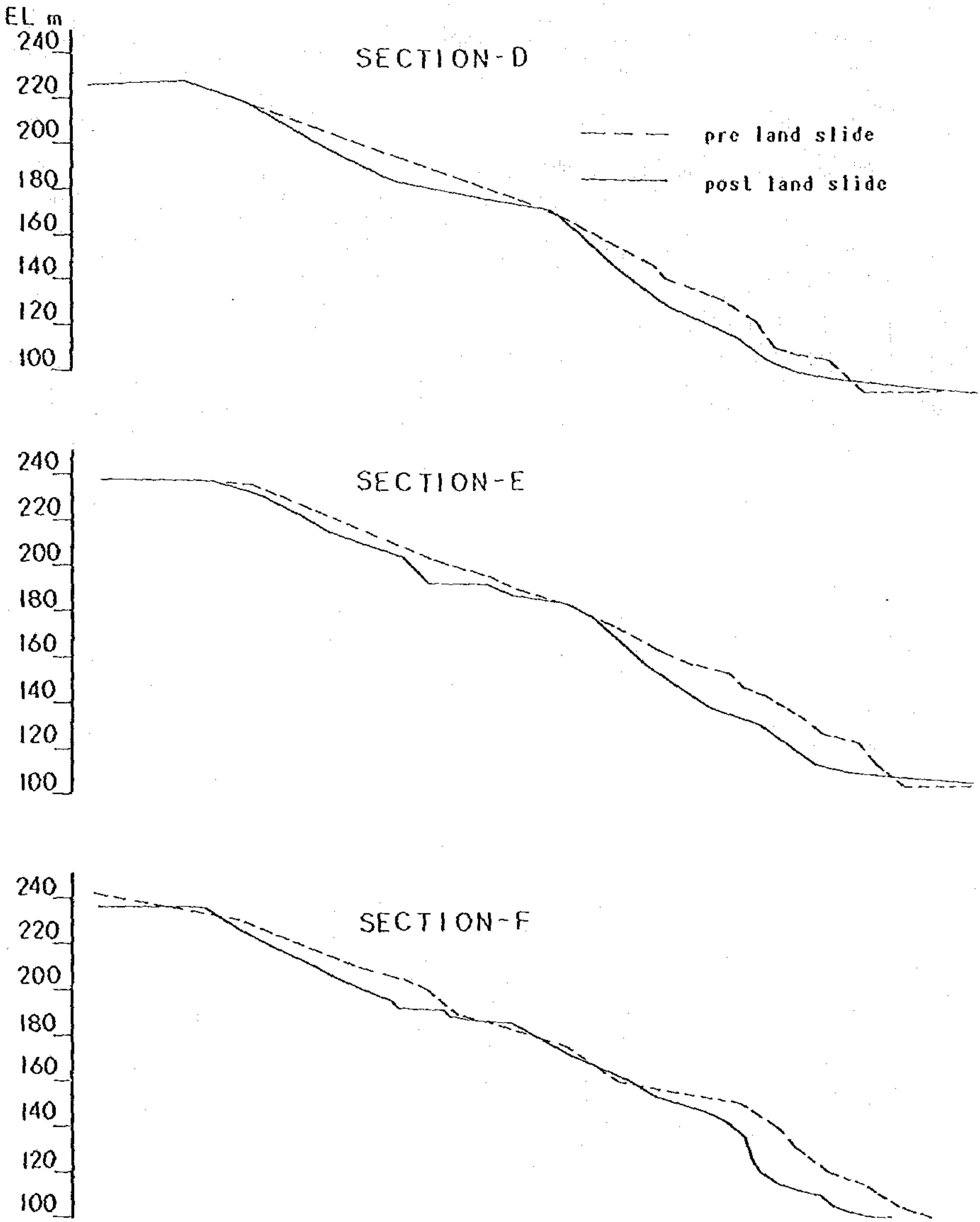


Table R8 Relation Between C and Ø Values of the Mountain Mass Obtained from the Back Analyses (Ø as Parameters)

Assumption	Upper		Lower	
	ϕ	C kg/cm²	ϕ	C kg/cm²
Landslide would occur under condition with no groundwater.	10°	0.15~0.30	10°	0.54~0.65
	20°	0.02~0.05	20°	0.08~0.34
	—	—	30°	0 ~0.05
Landslide would occur under condition with groundwater up to the surface.	10°	0.25~0.50	10°	0.77~0.96
	20°	0.19~0.36	20°	0.67~0.84
	30°	0.10~0.20	30°	0.55~0.67

Table R9.1 Results of Analysis on Possibilities of Landsliding Calculations in Case A (Underground Water: Up to the Ground Surface; Seismic Force: Zero)

	Probable Sliding Line	(1) Least Safety Factor Against Sliding	(2) Starting & Ending Points of the Sliding Line	(3) Radius of the Sliding Line (m)	(4) Sliding Resistance of the Sliding Surface (ton)	(5) Sliding Force of the Sliding Line (ton)	(6) Short Shearing Resistance of the Sliding Line (ton)
In case of $\phi = 10°$	No. 2	1.781	48 - 22	67.67	442.17	248.20	-193.96
	No. 3	1.748	60 - 32	76.05	517.81	296.24	-221.57
	No. 5	1.344	40 - 19	61.92	388.81	289.30	-99.51
	No. 7	1.871	53 - 1	72.63	2684.73	1434.72	-1250.02
	No. 9	1.866	25 - 9	90.24	212.30	113.77	-98.53
	No.12	1.075	42 - 5	145.49	5974.05	5555.86	-418.20
	No.14	1.454	50 - 6	70.65	2199.69	1512.49	-687.20
In case of $\phi = 20°$	No. 2	0.997	48 - 22	67.67	247.43	248.20	+0.77
	No. 3	1.003	56 - 33	44.83	251.57	250.73	-0.84
	No. 5	0.711	40 - 19	52.36	205.61	289.30	+83.69
	No. 7	1.075	49 - 8	143.77	272.60	253.64	-18.95
	No. 9	0.846	25 - 9	90.24	96.23	133.77	+17.53
	No.12	1.073	42 - 5	145.49	5959.09	5555.86	-403.24
	No.14	1.440	50 - 6	70.65	2178.33	1512.49	-665.84

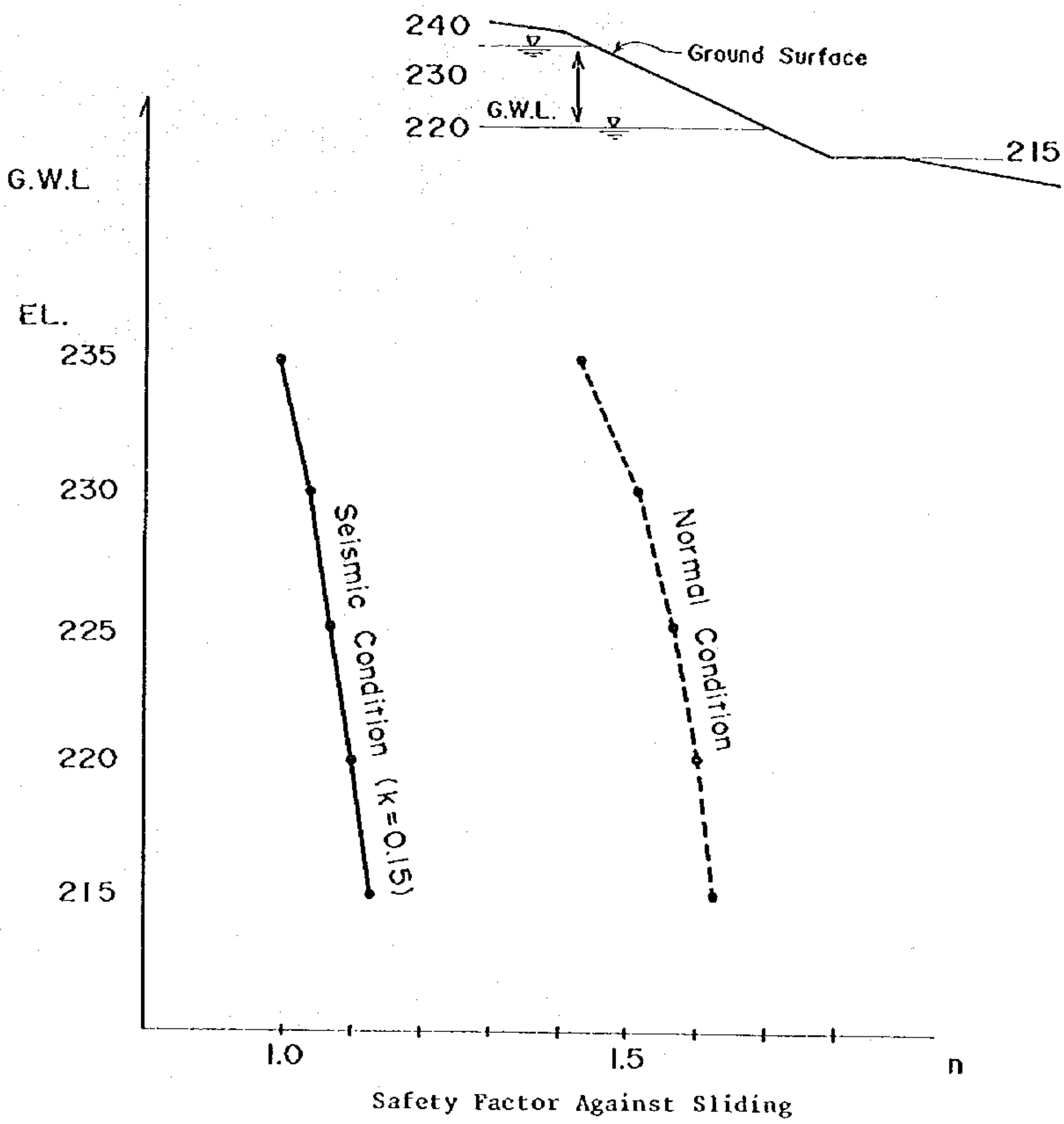
Table R9.2 Analysis on Possibilities of Lendsliding Calculations in Case B (Underground Water: None; Seismic Force: 0.15g)

		(1)	(2)	(3)	(4)	(5)	(6)
	Probable Sliding Line	Least Safety Factor Against Sliding	Starting & Ending Points of the Sliding Line	Radius of the Sliding Line (m)	Sliding Resistance of the Sliding Surface (ton)	Sliding Force of the Sliding Line (ton)	Short Shearing Resistance of the Sliding Line (ton)
In case of $\phi = 10°$	No. 2	1.680	50 - 22	95.10	514.82	306.46	-208.37
	No. 3	1.624	60 - 32	76.05	566.43	348.76	-217.67
	No. 5	1.366	40 - 19	61.92	429.28	314.19	-115.09
	No. 7	1.750	53 - 12	142.60	765.53	437.45	-328.08
	No. 9	1.824	25 - 9	90.24	228.27	125.14	-103.13
	No.12	1.432	42 - 5	160.42	8314.90	5807.47	-2507.43
	No.14	1.754	51 - 11	95.83	2504.03	1427.74	-1076.29
In case of $\phi = 20°$	No. 2	1.154	48 - 22	67.67	329.25	285.23	-44.02
	No. 3	1.160	56 - 33	44.83	335.26	288.90	-46.36
	No. 5	0.920	40 - 19	61.92	289.16	314.19	+25.03
	No. 7	1.206	50 - 4	243.83	463.04	383.84	-79.20
	No. 9	1.032	25 - 9	90.24	129.21	125.14	-4.06
	No.12	1.429	42 - 5	160.42	8301.02	5807.47	-2493.55
	No.14	1.744	51 - 11	95.83	2490.51	1427.74	-1062.77

Table R9.3 Analysis on Possibilities of Landsliding Calculations in Case C (Underground Water: Up to the Ground Surface; Seismic Force: 0.15g)

	Probable Sliding Line	(1) Least Safety Factor Against Sliding	(2) Starting & Ending Points of the Sliding Line	(3) Radius of the Sliding Line (m)	(4) Sliding Resistance of the Sliding Surface (ton)	(5) Sliding Force of the Sliding Line (ton)	(6) Short Shearing Resistance of the Sliding Line (ton)
In case of ϕ = 10°	No. 2	1.366	50 - 22	95.10	465.08	340.51	-124.57
	No. 3	1.317	60 - 32	76.05	510.24	387.51	-122.73
	No. 5	1.094	40 - 19	61.92	382.04	349.10	-32.94
	No. 7	1.350	53 - 1	72.63	2563.62	1898.63	-664.99
	No. 9	1.505	25 - 9	90.24	209.29	139.05	-70.24
	No.12	0.803	42 - 4	150.07	5757.37	7173.84	+1416.47
	No.14	1.113	50 - 7	70.75	2158.15	1939.36	-218.78
In case of ϕ = 20°	No. 2	0.738	48 - 22	67.67	233.88	316.92	+83.05
	No. 3	0.742	56 - 33	44.83	238.30	321.00	+82.70
	No. 5	0.549	40 - 19	61.92	191.64	349.10	+157.46
	No. 7	0.774	49 - 8	143.77	258.75	334.44	+75.70
	No. 9	0.647	25 - 9	90.24	90.02	139.05	+49.03
	No.12	0.800	42 - 4	150.07	5742.57	7173.84	+1431.27
	No.14	1.099	51 - 4	82.06	2008.72	1828.53	-180.20

Fig. R9 Relation Between Ground Water Level and Safety Factor Against Sliding



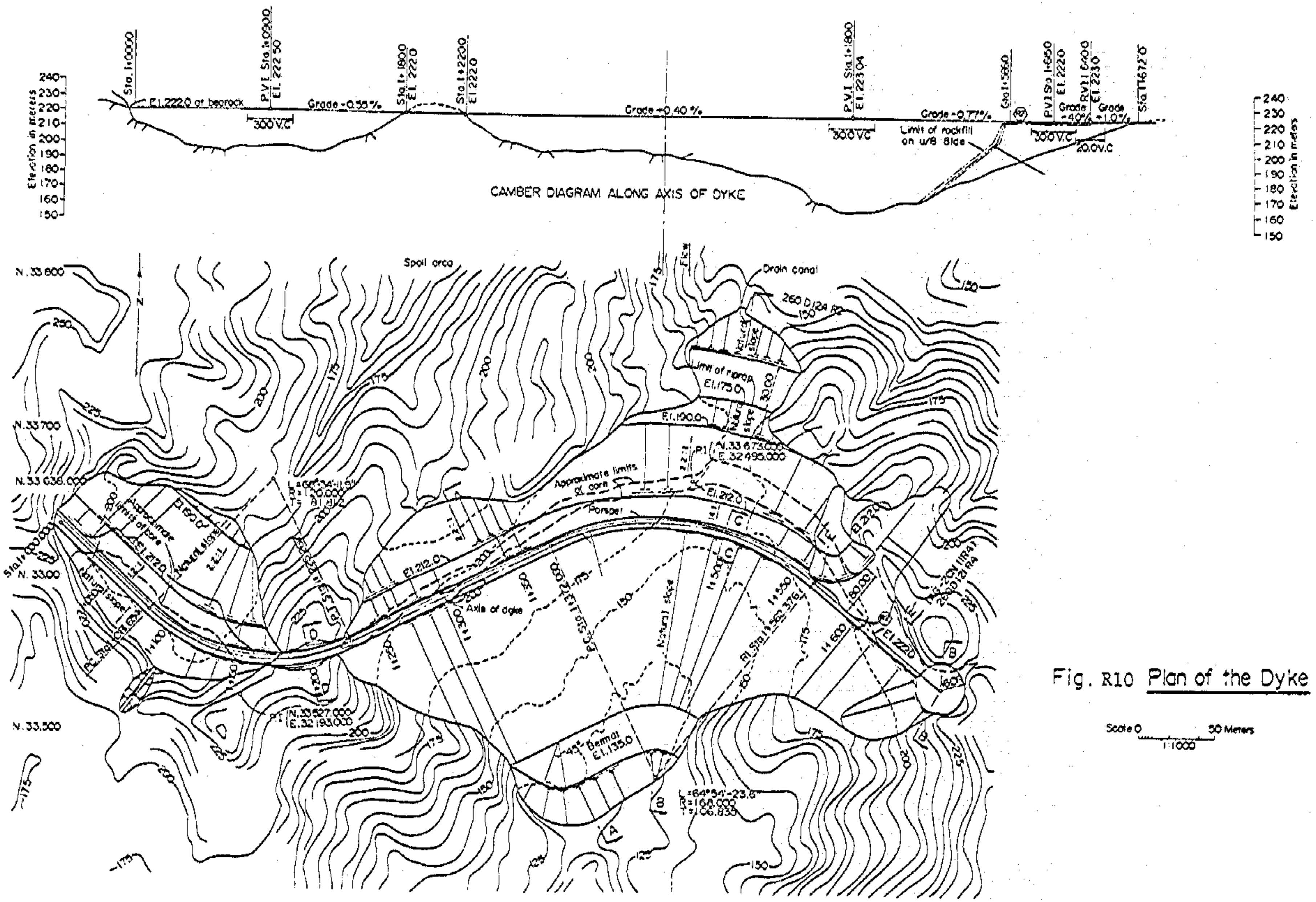


Fig. R10 Plan of the Dyke

Scale 0 50 Meters 1:1000

Fig. R11 Typical Cross Section of the Dyke

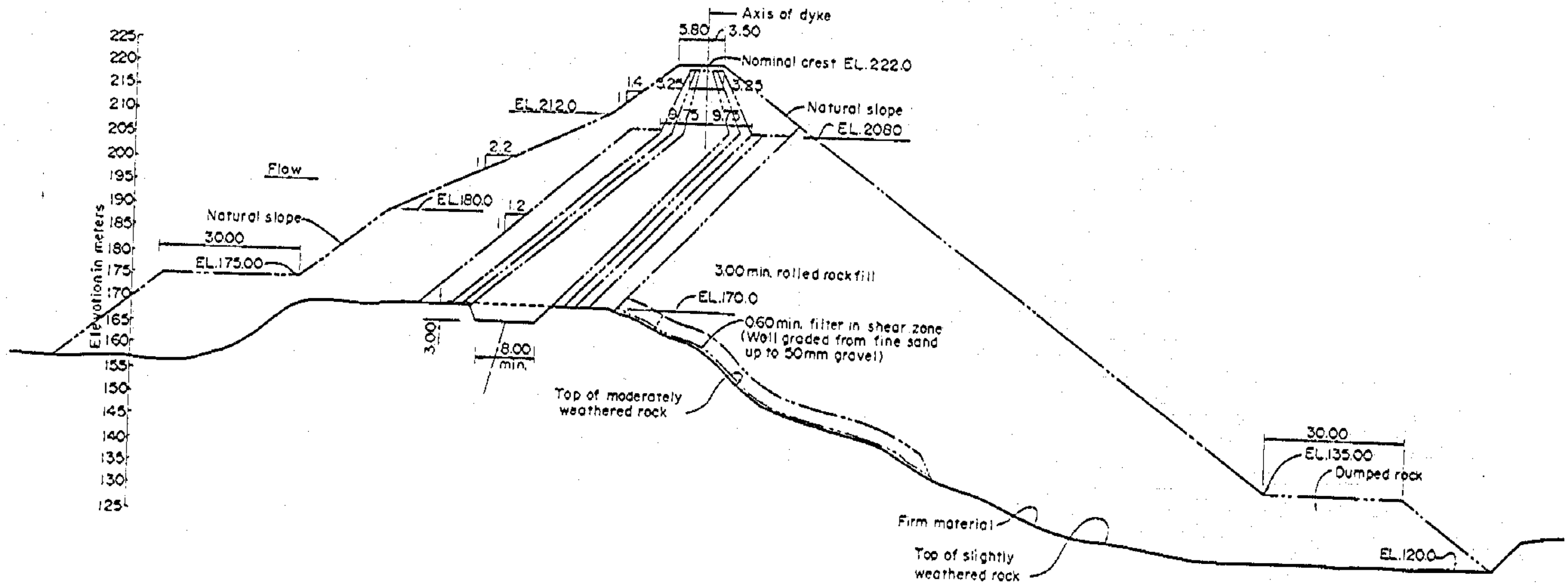


Table R10 Comparison of Measured and Calculated Amount of Seepage Through the Dyke

(Data Taken During Months in 1987 and 1988)

(m^3)

	SW-1				SW-2				SW-3			
	Measured	Calculated			Measured	Calculated			Measured	Calculated		
		Total	Influenced by			Total	Influenced by			Total	Influenced by	
			Rain	RWL			Rain	RWL			Rain	RWL
JAN	51,586	50,361	588	49,773	33,818	29,735	2,477	27,258	68,913	74,230	3,440	70,790
FEB	31,445	35,750	18	35,732	16,870	20,673	64	20,609	60,557	51,498	168	51,330
MAR	20,990	21,974	41	21,933	14,485	15,290	111	15,179	29,637	32,930	106	32,824
APR	6,128	5,576	878	4,698	8,891	9,899	2,517	7,382	11,445	11,547	2,426	9,121
MAY	1,663	691	584	107	2,873	3,388	1,636	1,752	3,747	4,993	2,047	2,946
JUN	1,218	3,508	3,404	104	2,226	9,680	9,680	0	3,722	12,023	9,172	2,851
JUL	1,259	2,569	2,462	107	5,123	8,427	8,427	0	9,887	13,341	10,395	2,946
AUG	3,677	3,961	3,854	107	12,675	11,523	11,523	0	23,764	14,478	11,532	2,946
SEP	2,970	2,669	2,565	104	9,361	9,236	9,236	0	13,952	13,595	10,744	2,851
OCT	1,259	1,313	1,206	107	3,187	4,236	4,213	23	6,023	8,076	5,130	2,946
NOV	6,711	8,810	2,099	6,711	9,562	12,771	5,423	7,348	14,155	17,617	5,749	11,868
DEC	38,481	41,377	2,286	39,091	30,482	30,817	8,201	22,616	58,049	65,173	8,986	56,187
Total	167,387	178,559	19,985	158,574	149,553	165,675	63,508	102,167	303,851	319,501	69,895	249,606

Fig. R12.1 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-1 RAINFALL ADJUST:-0 mm DEV= 2.3)

SOLUTION: A(1)=0.044 x 1 A(2)=0.018 x R00 A(3)=0.013 x R01 A(4)=0.018 x R02 A(5)=0.016 x R03 A(6)=0.002 x R04 A(7)=0.007 x R05 A(8)=0.003 x R10 A(9)=0.000 x R20 A(10)=0.001 x R35
A(11)=0.000 x (R.W.L.-180) A(12)=0.000 x (R.W.L.-185) A(13)=0.000 x (R.W.L.-190) A(14)=0.975 x (R.W.L.-195)

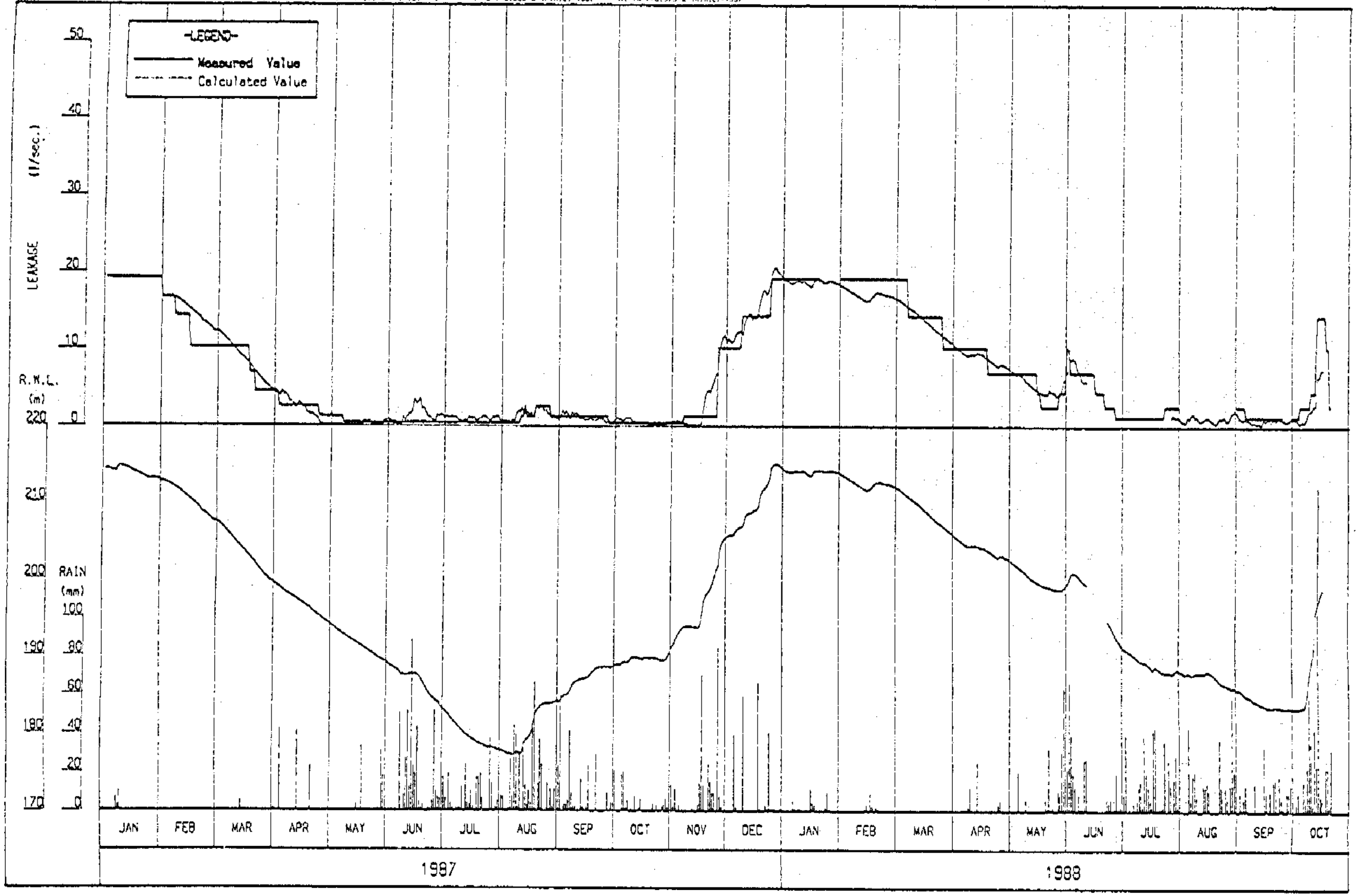


Fig. R12.2 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-2 RAINFALL ADJUST:-0 mm DEV= 3.14)

SOLUTION: A(1)=0.020 x R00 A(2)=0.026 x R01 A(3)=0.025 x R02 A(4)=0.026 x R03 A(5)=0.023 x R04 A(6)=0.015 x R05 A(7)=0.013 x R10 A(8)=0.009 x R20 A(9)=0.003 x R35
A(10)=0.000 x (R.W.L.-180) A(11)=0.000 x (R.W.L.-185) A(12)=0.424 x (R.W.L.-190) A(13)=0.000 x (R.W.L.-195)

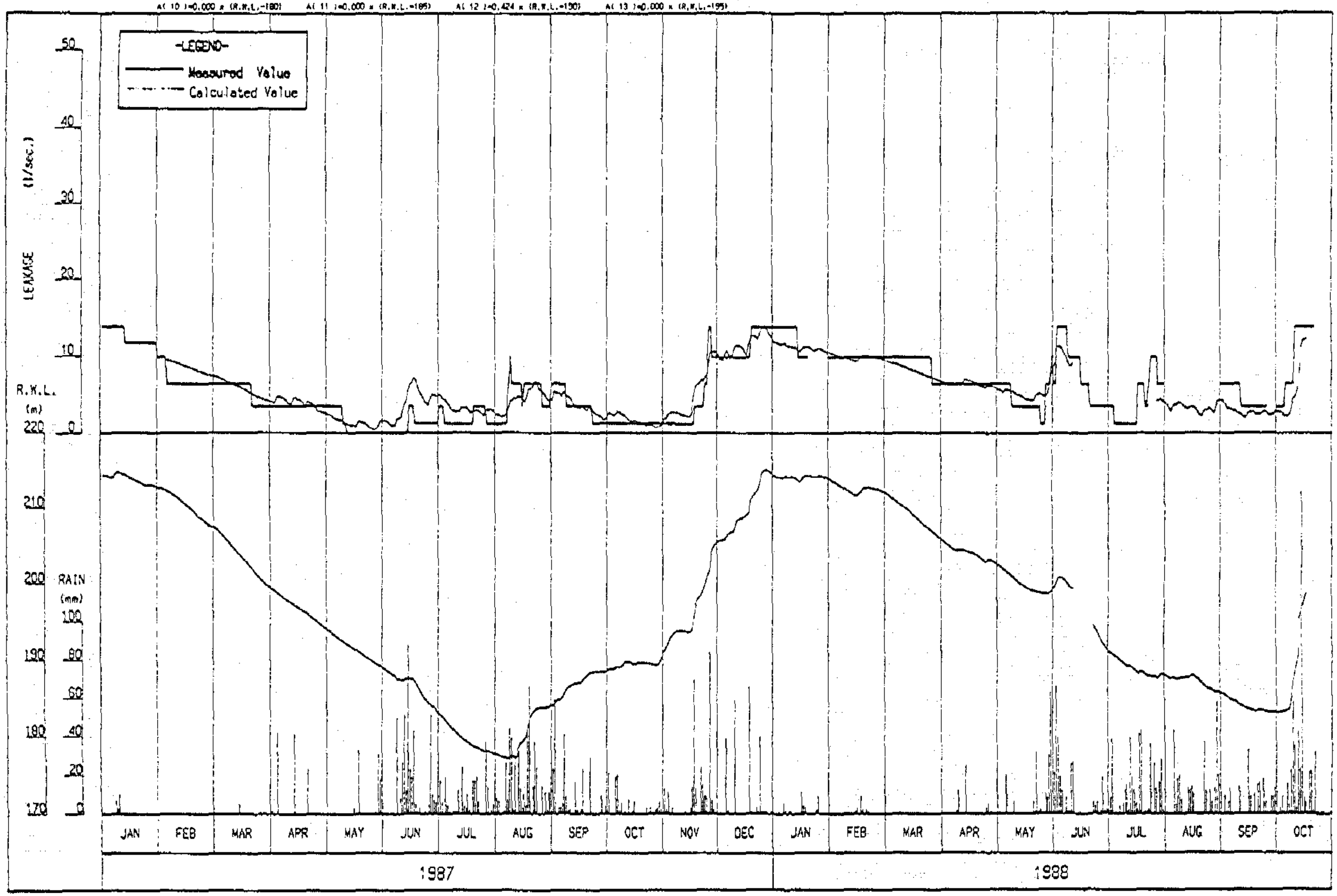


Fig. R12.3 Comparison Between Measured and Calculated Amounts of Seepage Through the Dyke (POINT :SW-3 RAINFALL ADJUST:-0 mm DEV= 10.4)

SOLUTION: A(1)=1.103 x 1 A(2)=0.033 x R00 A(3)=0.019 x R01 A(4)=0.029 x R02 A(5)=0.016 x R03 A(6)=0.016 x R04 A(7)=0.014 x R05 A(8)=0.017 x R10 A(9)=0.003 x R20 A(10)=0.009 x R35
A(11)=0.000 x (R.W.L.-180) A(12)=0.000 x (R.W.L.-185) A(13)=0.000 x (R.W.L.-190) A(14)=1.332 x (R.W.L.-195)

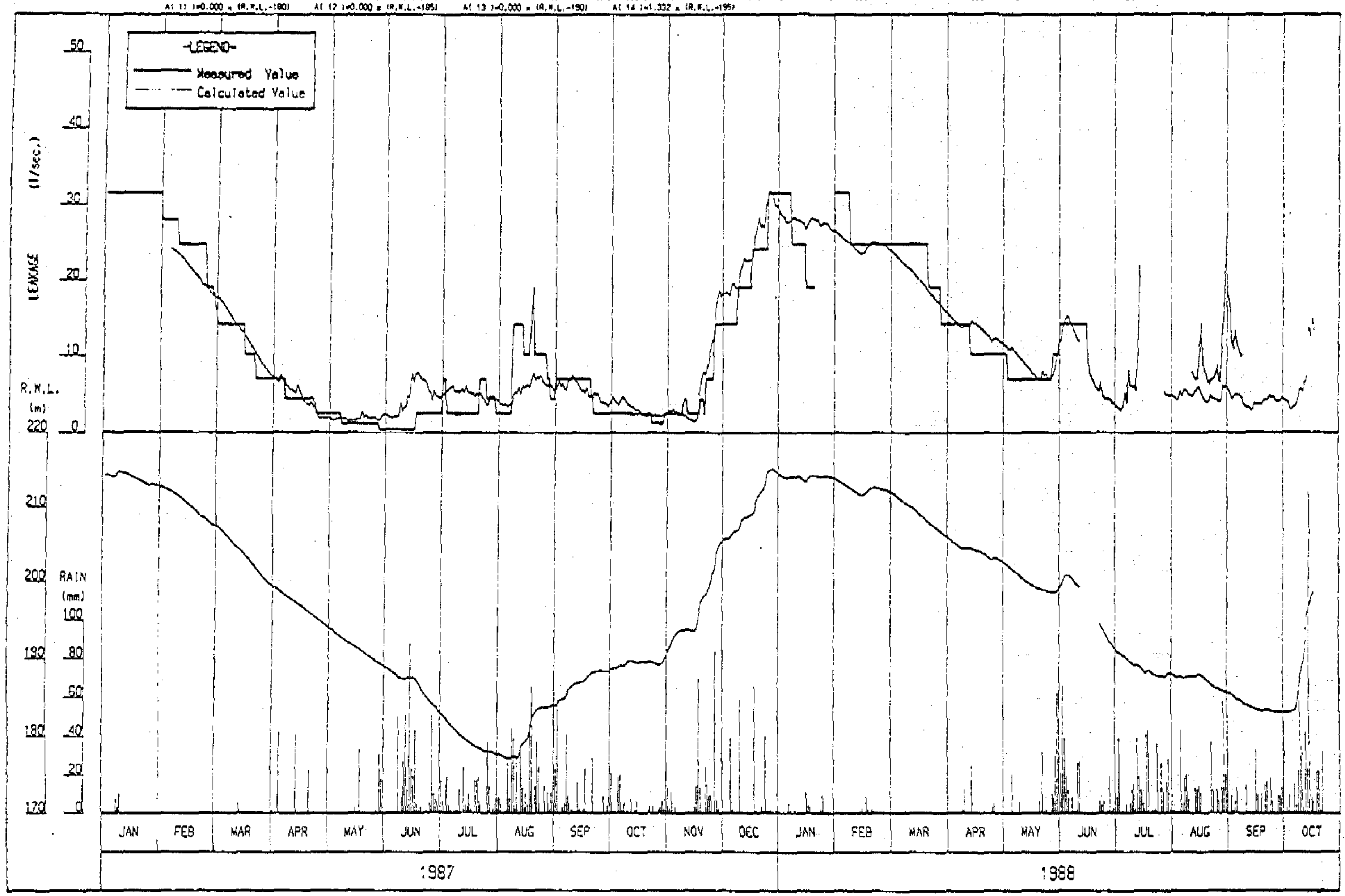


Table R11 Average Amount of Seepage through the Dyke

(m^3/min.)

Season	Weir	Seepage probably from the reservoir	Runoff probably from rainfall	Total
Dry (Dec. thru May)	SW-1	0.5759 (97)	0.0167 (3)	0.5926
	SW-2	0.3608 (86)	0.0571 (14)	0.4179
	SW-3	0.8493 (92)	0.0653 (8)	0.9146
Wet (June thru Nov.	SW-1	0.0275 (32)	0.0593 (68)	0.0868
	SW-2	0.0280 (13)	0.1846 (87)	0.2126
	SW-3	0.1005 (33)	0.2006 (67)	0.3011
Total	SW-1	0.3017 (89)	0.0380 (11)	0.3397
	SW-2	0.1944 (62)	0.1208 (38)	0.3152
	SW-3	0.4749 (78)	0.1330 (22)	0.6079

Note: The parenthesized shows percentage of the total.

Fig. R13.1 Comparison Between Measured and Calculated Values of Borehole Water Levels

(BO.NO:DYK-2)

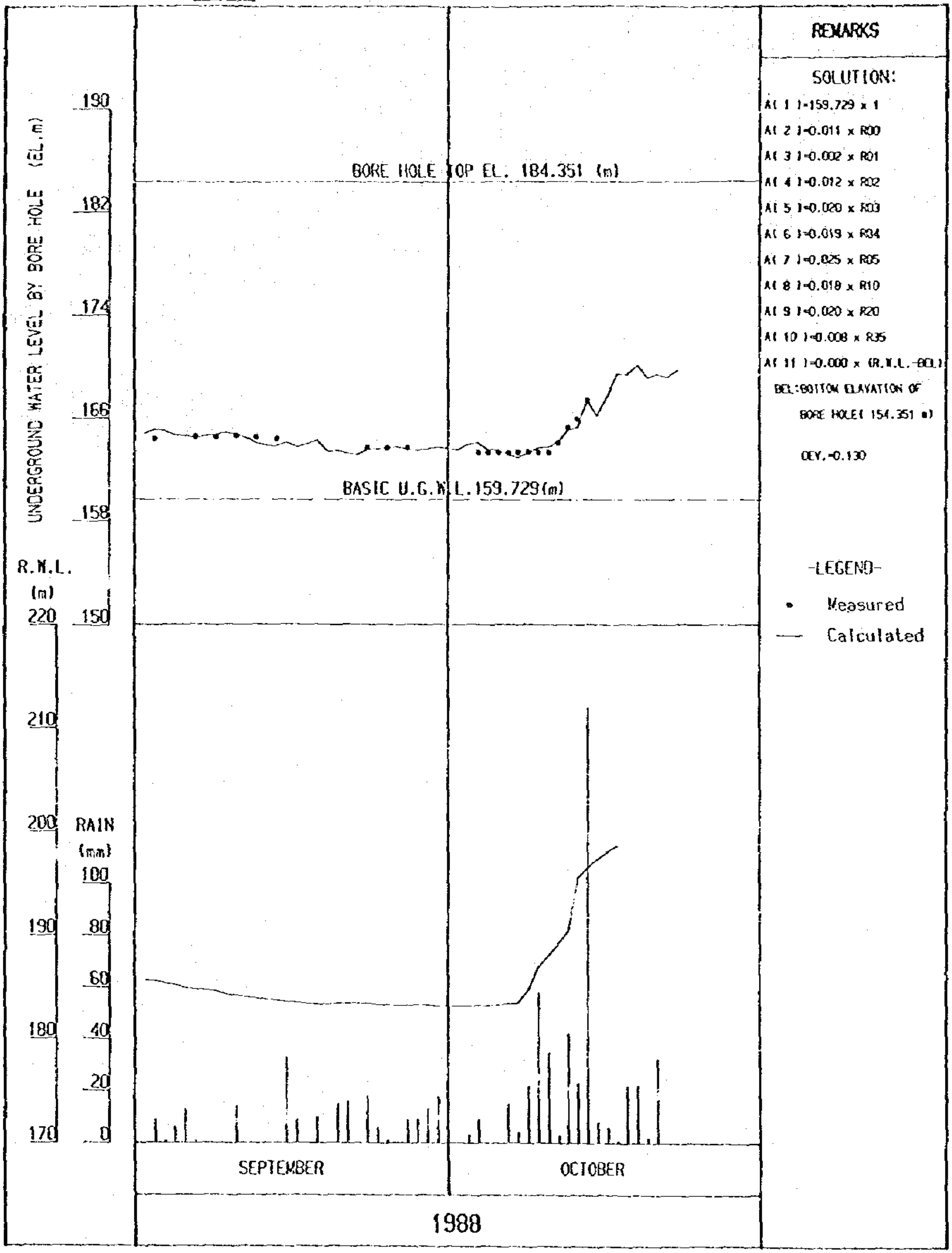


Fig. R13.2 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-4)

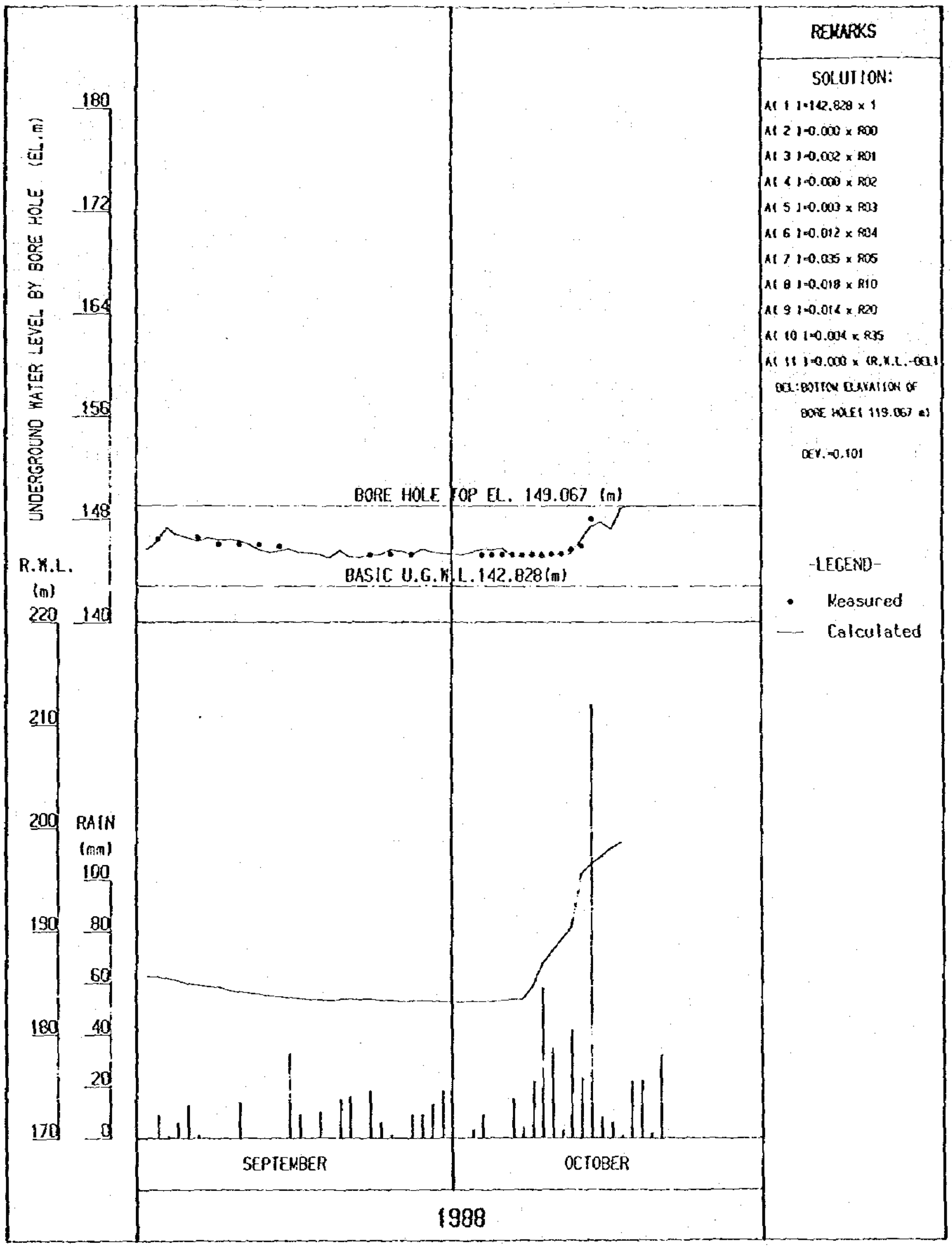


Fig. R13.3 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-5)

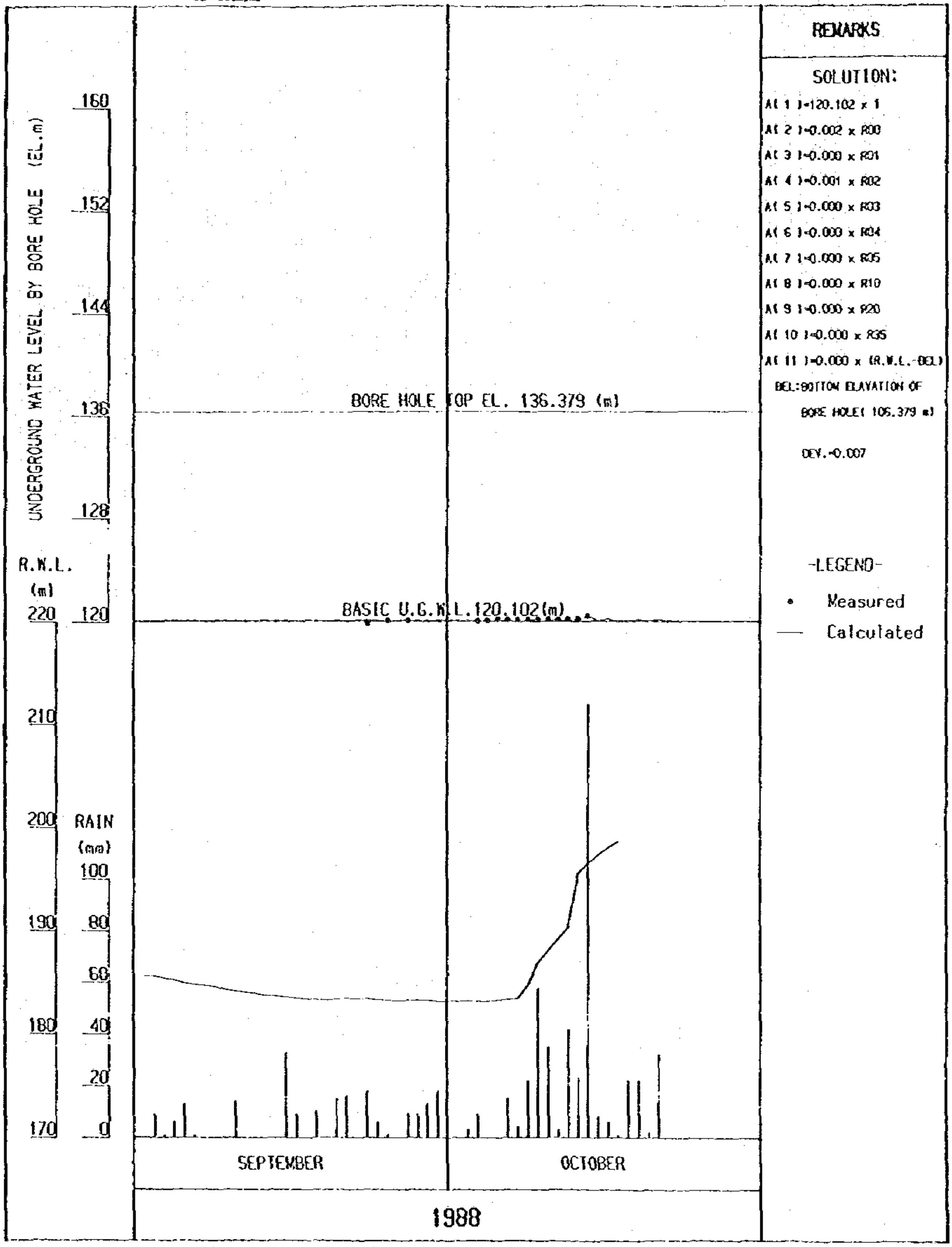


Fig. R13.4 Comparison Between Measured and Calculated Values of Borehole Water Levels (BO.NO:DYK-6)

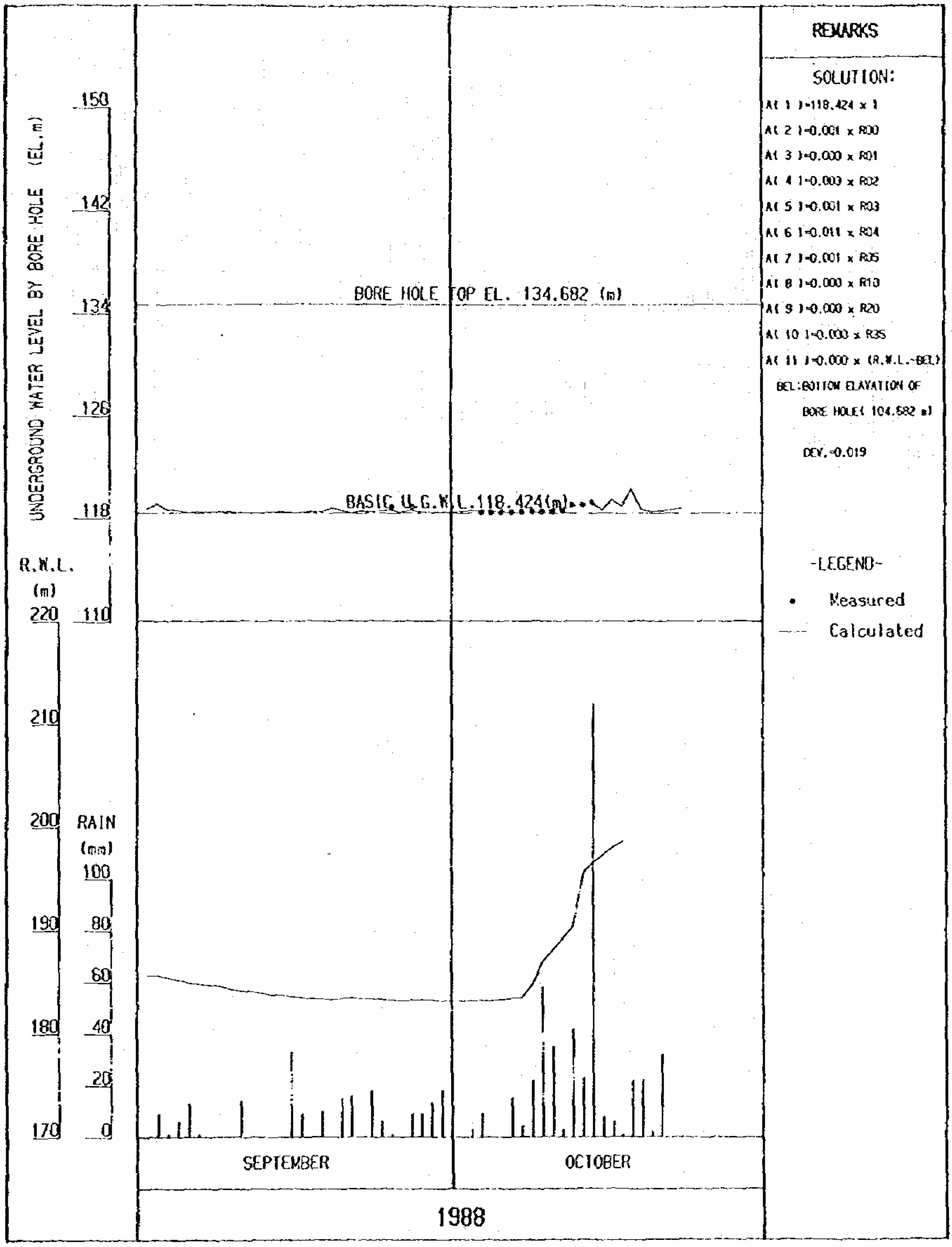


Table R12 Monthly Generation of Angat Hydroelectric Power Plant for the Year 1987

Month	Auxiliary Plant (Water Supplied to MWSS)	Main Plant (Water Supplied to NIA)
January	17,229 MWh	33,794 MWh
February	14,815	29,549
March	15,376	24,260
April	13,802	14,710
May	13,743	-
June	12,564	13,258
July	13,529	4,056
August	14,959	-
September	15,068	7,426
October	17,201	4,069
November	17,630	31,170
December	18,785	33,696
Total	184,701	195,988

Table R13 Monthly Water Requirements for Angat Hydroelectric Power Plant for the Year 1987

Month	Auxiliary Plant	Main Plant
January	69.47 10^6m^3	104.46 10^6m^3
February	62.75	94.35
March	69.58	80.35
April	67.23	51.84
May	70.42	0
June	70.11	51.84
July	76.90	16.07
August	77.25	0
September	72.63	25.92
October	76.72	13.39
November	73.52	98.50
December	76.08	104.46
Total	862.66	641.18

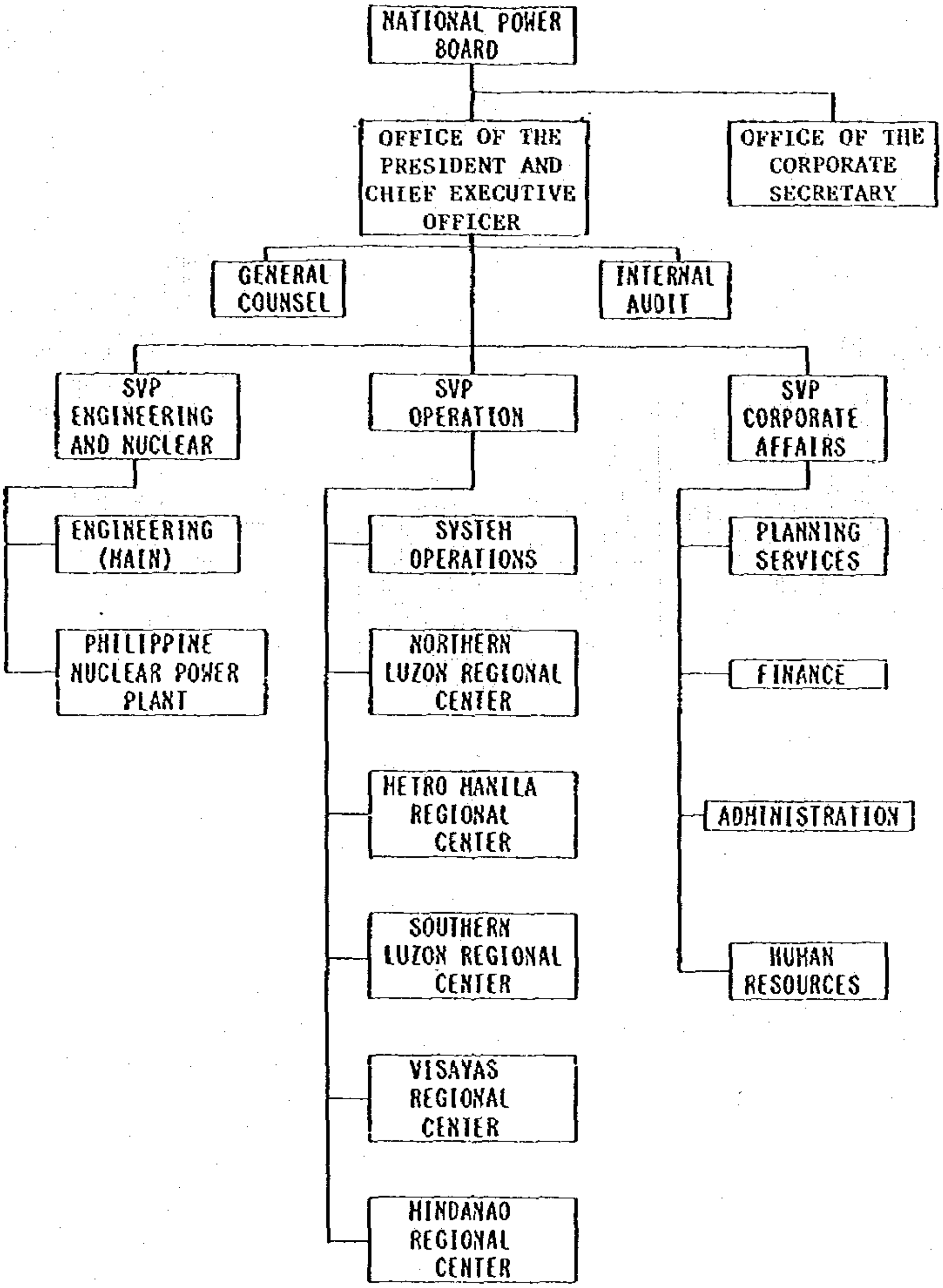
Table R14 Installed Capacity and Annual Energy Output of the Luzon Power System (1986)

Type of Power Source	Installed Capacity		Annual Energy Output	
	(MW)	(%)	(GWh)	(%)
Hydro, total	1,226	29.8	2,956	20.0
- Pumped storage	300	7.3	211	1.4
- Reservoir	895	21.8	2,643	17.9
- Run-of-river	31	0.7	102	0.7
Oilfired	1,925	46.8	6,328	42.9
Coalfired	300	7.3	1,572	10.7
Geothermal	660	16.1	3,900	26.4
Grand Total	4,111	100.0	14,756	100.0

Peak Demand : 2,435 MW

Fig. R14.1 ORGANIZATION CHART

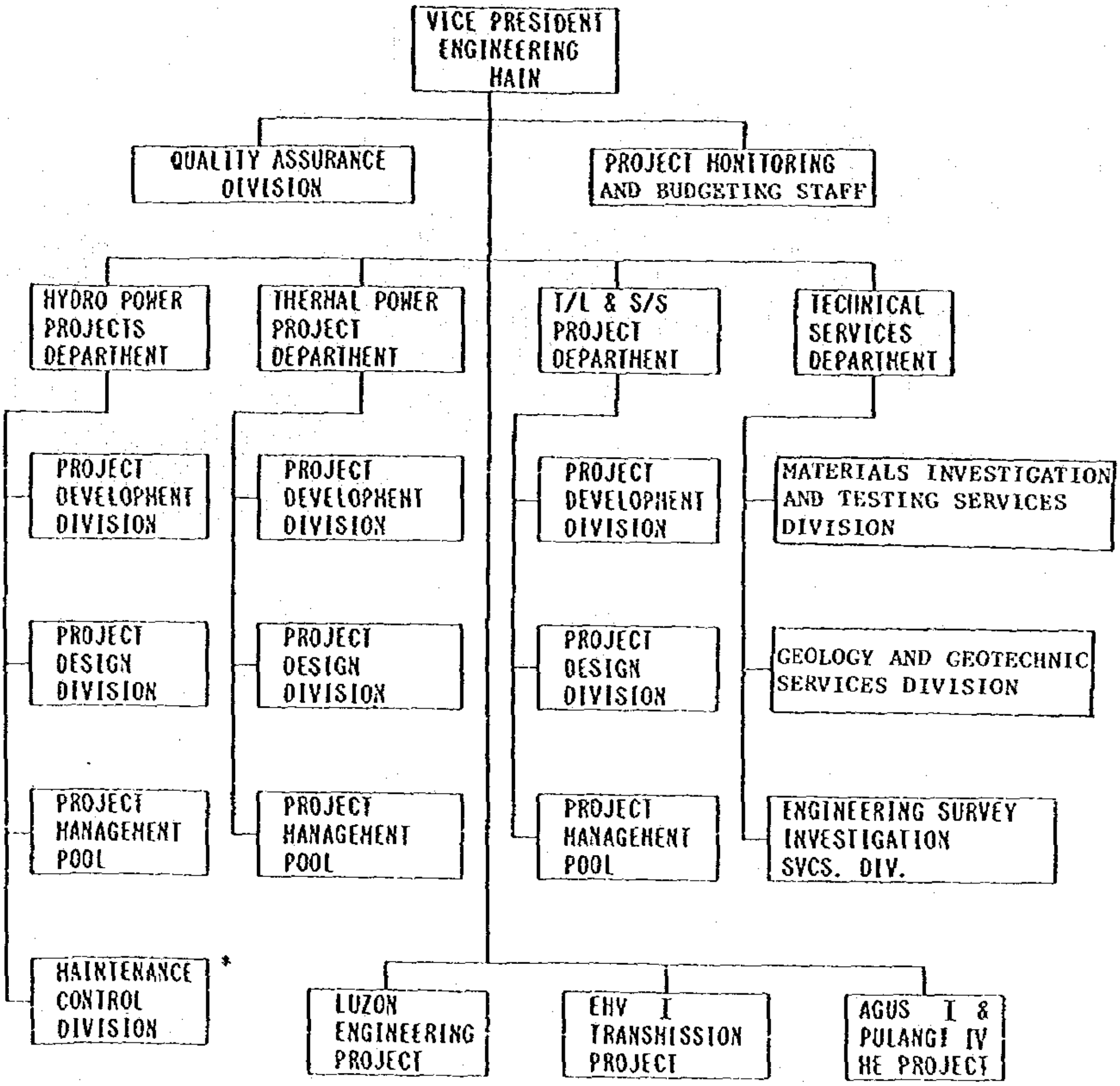
NAPOCOR



Total Number of Personnel : 10,819 (As of Dec. 31, 1987)

Fig. R14.2 ORGANIZATION CHART

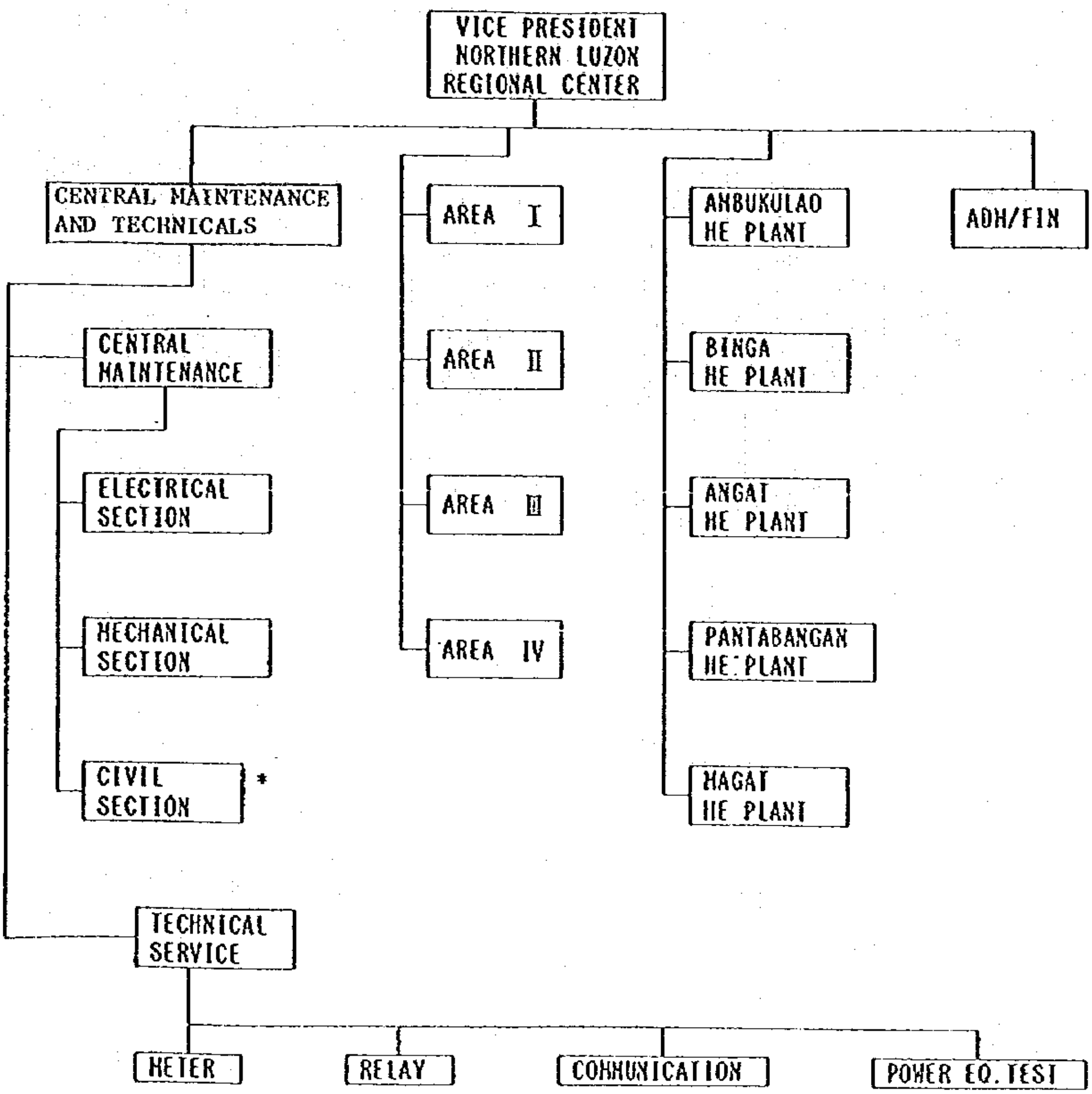
HEAD OFFICE ENGINEERING - MAIN



* A division recommended to be newly established.

Fig. R14.3 ORGANIZATION CHART

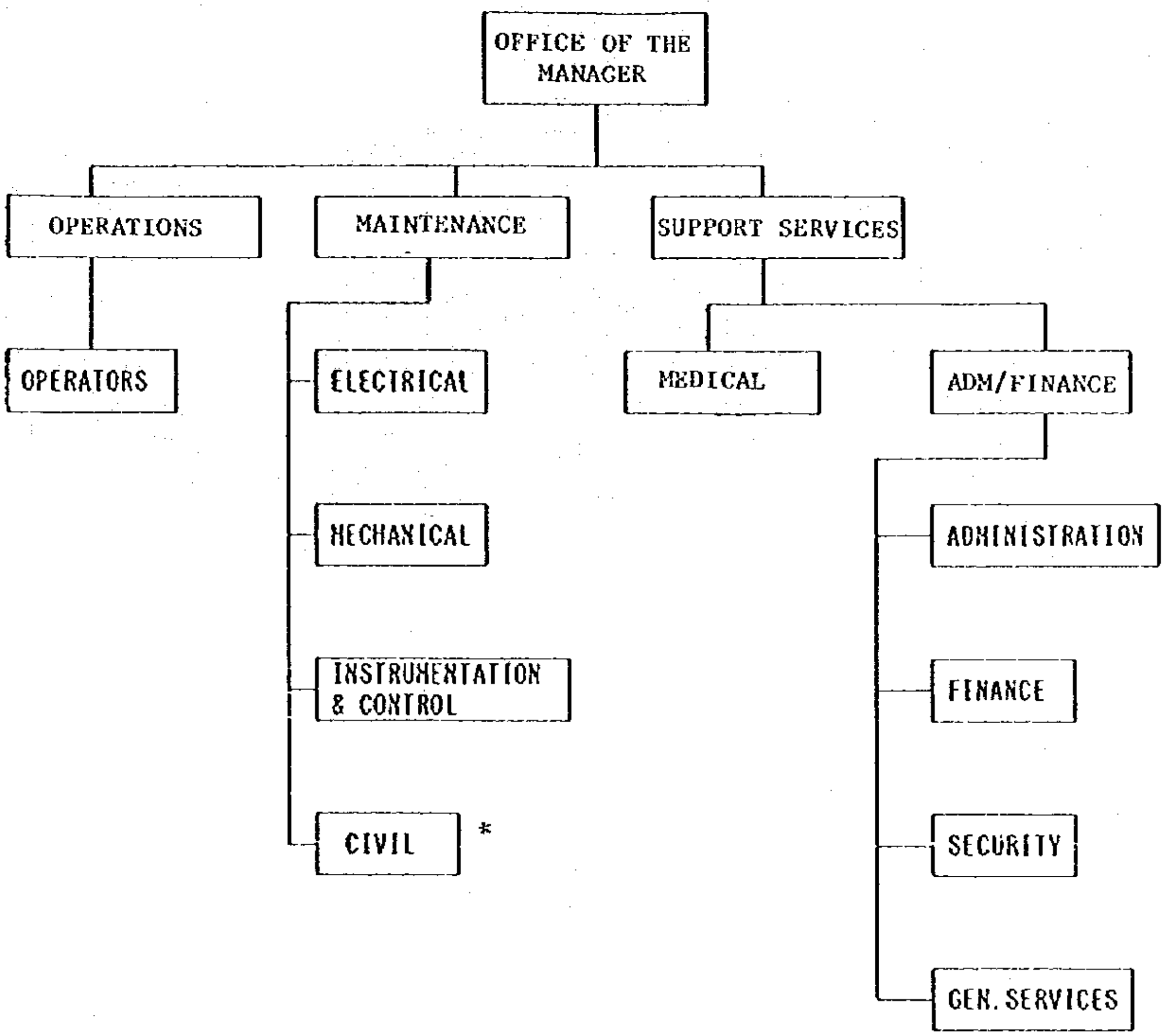
NORTHERN LUZON REGIONAL CENTER



* A section recommended to be newly established.

Fig. R14.4 ORGANIZATION CHART

ANGAT HYDROELECTRIC POWER PLANT



* A group recommended to be newly established.