THE GOVERNMENT OF THE REPUBLIC OF THE PHILIPPINES

REPORT ON STUDY

OF

ANGAT DAM REHABILITATION PROJECT

MARCH 1989

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

In response to a request from the Government of the Republic of the Philippines, the Japanese Government decided to conduct a survey on the Angat Dam Rehabilitation Project and entrusted the survey to the Japan International Cooperation Agency (JICA).

JICA sent to the Philippines a survey team headed by Dr. Ryuichi Kozuki of The New Japan Engineering Consultants, Inc. from September 1987 to October 1988.

The team held discussions on the Project with the officials concerned of the Government of the Republic of the Philippines and conducted a field survey. After the team returned to Japan, further studies were made and the present report was prepared.

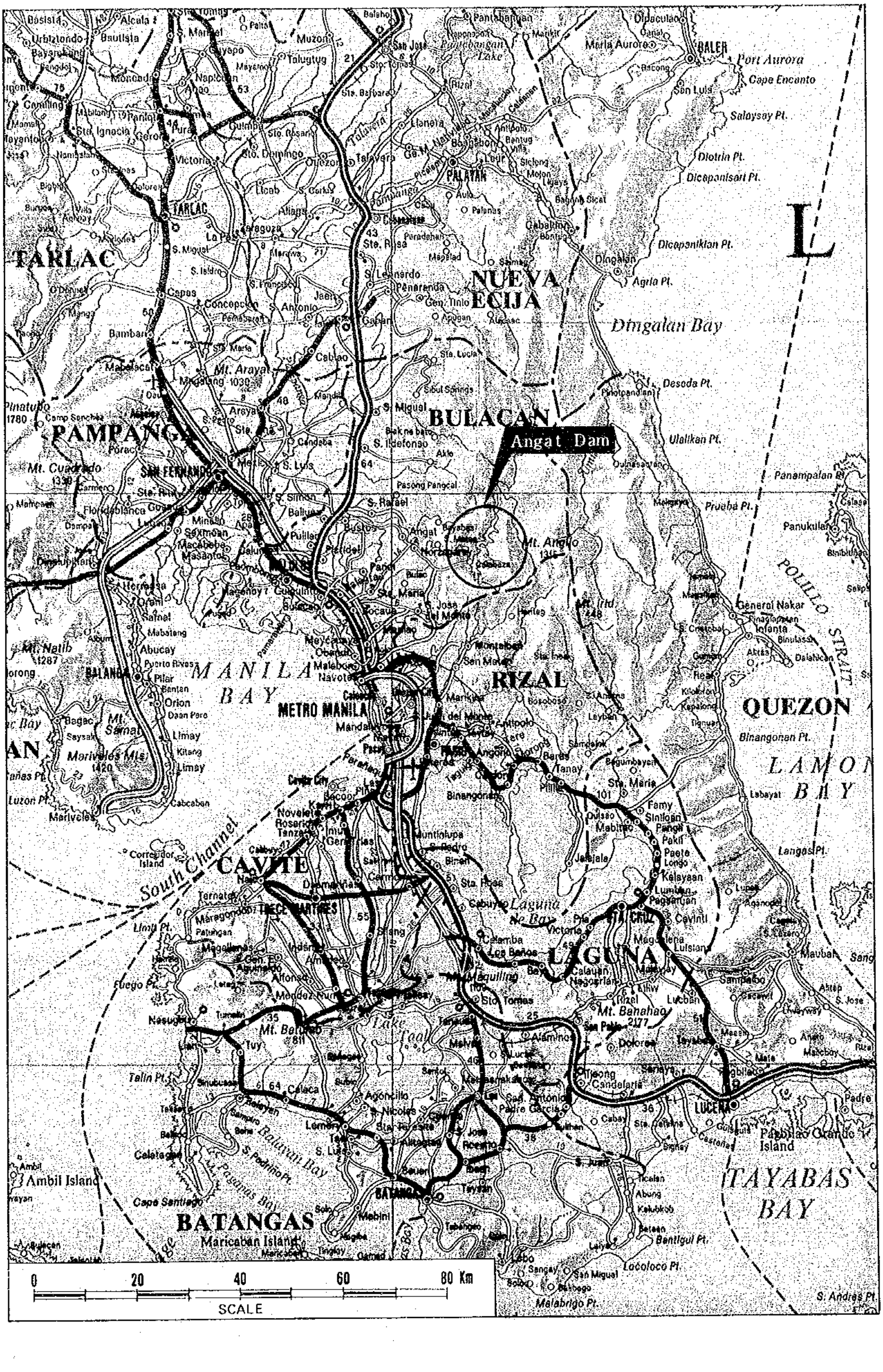
I hope that this report will contribute to the development of the Project and to the promotion of friendly relations between our two countries.

I wish to express my sincerest appreciation to the officials concerned of the Government of the Republic of the Philippines for their close cooperation extended to the team.

March, 1989

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency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Laguna de Bay
Lake Taal
Balayan Bay
Port Aurora
Cape Encanto
Mt. Arayat
Mt. Angilo
Mt. Natib
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Mt. Cuadrado
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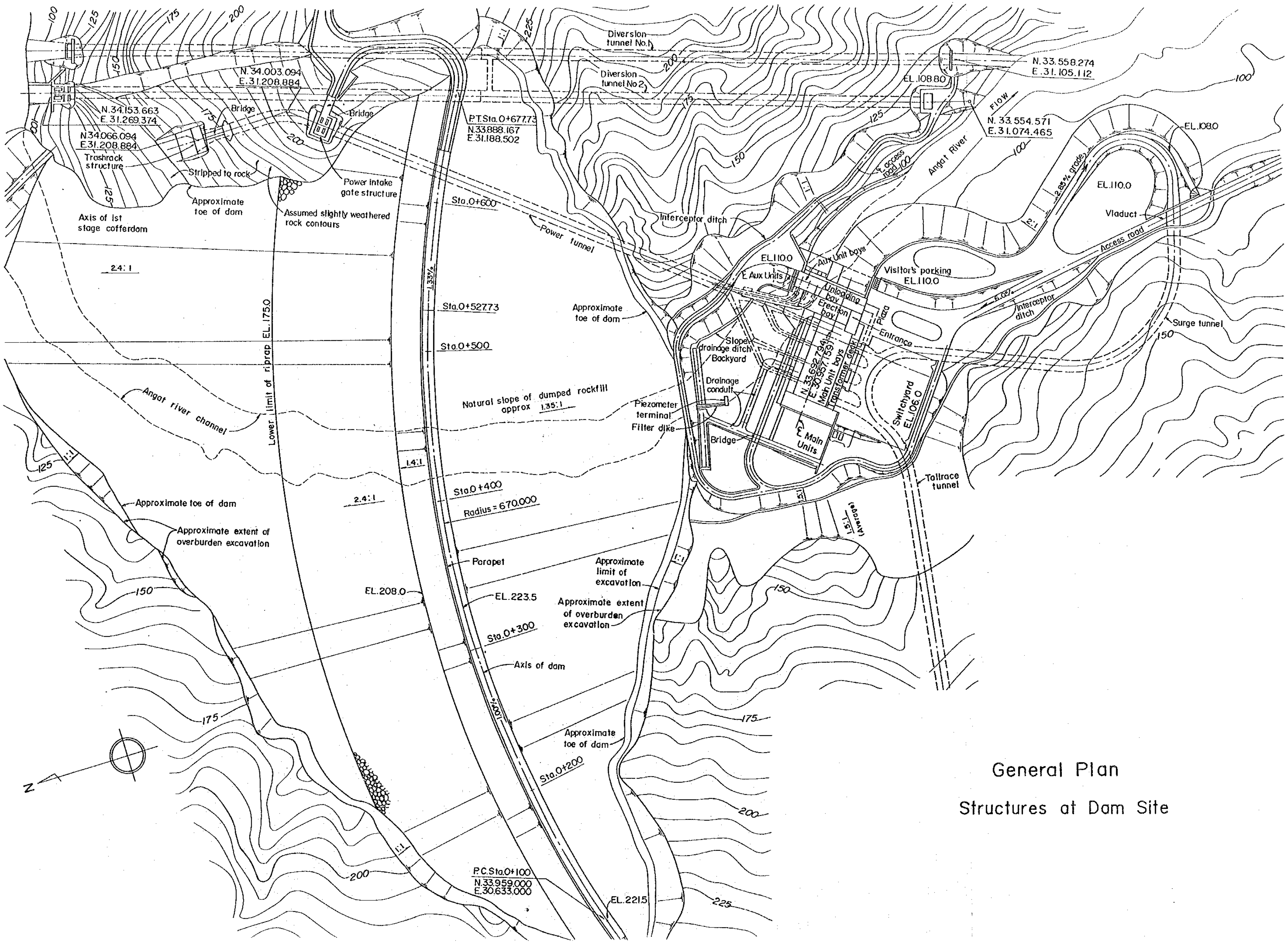
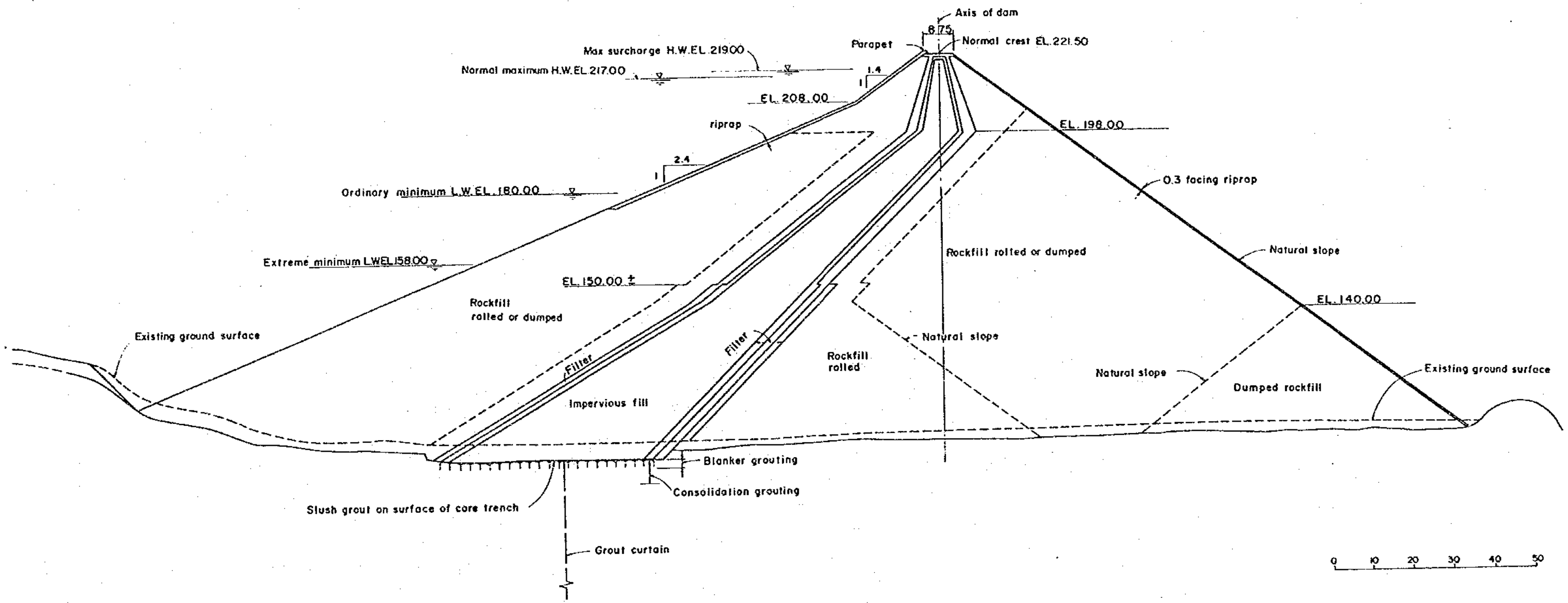


General Plan

Structures at Dam Site

TYPICAL CROSS SECTION OF THE ANGAT DAM



Angat Dam and Powerhouse

TABLE OF CONTENTS

Page

Page

Page

Page

Page

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.

1. SUMMARY

1. Summary

1.1. Preface

This is 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.

1.2. 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.

1.3. 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.

1.3.1. 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 shown in Table 1.1.

1.3.2. 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.

Table 1.1. 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.

(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 this Report as a supplement, in that it may have a serious effect on the safety of the Angat Power Plant.

This 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.

1.4. Summary of the Study

1.4.1. Adequacy of Spillway Capacity

In Section 2 of this 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 this 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 (10^6m^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.

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 x 10^6m^3, 457.6 x 10^6m^3, 560.8 x 10^6m^3, 656.7 x 10^6m^3 and 718.0 x 10^6m^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.

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.

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.

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.

1.4.2. Stability of the Dam and the Dyke

In Section 3 of this 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.

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 earthquake 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°.

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.

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.

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.

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.

1.4.3. Landsliding at the Ex-Batcher Plant Site

Section 4 of this 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 this 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.

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°.

(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.

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.

1.4.4. Seepage through the Dyke

Section 5 of this 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.

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.

1.4.5. Rehabilitation Plans

Items discussed in Section 6 of this 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 this 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.

(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.

(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.

1.4.6. Economic Analyses

Section 7 of this 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 this 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.

1.4.7. Safety Control Standards of the Dam and Associated Structures

Section 8 of this 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.

1.4.8. Safety Control System of the Dam

Section 9 of this 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.

1.4.9. Leakage from the Penstock

The last section as a supplement to this 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.

2. ADEQUACY OF SPILLWAY CAPACITY

2. Adequacy of Spillway Capacity

Adequacy of the design flood inflow to the Angat dam was reviewed in 1984 to a considerable extent by a joint team of Nippon Koei Co., Ltd., CTI Engineering Co., Ltd. and Basic Technology and Management Corporation. The spillway gate operation procedure of the dam during flood periods was re-established on the basis of the result of this review.

The Angat dam was completed in 1966. The original design flood inflow to the dam was reviewed in October 1960, prior to the commencement of the construction, by Harza Engineering Co., Chicago, Illinois, U.S.A. in association with Engineering and Development Corporation of the Philippines, and was revised based on the probable maximum flood determined by the inflow analysis. (Review Memorandum on the Angat Project, Oct., 1960, by Harza Engineering Co. and Engineering and Development Corporation of the Philippines).

The 1984 review of the design flood inflow mentioned above was made based on this 1960 Harza - EDCOP Review Memorandum, and on the additional data collected since then (1960 through 1982). The probable maximum flood was also re-examined by making some modifications to the calculation method.

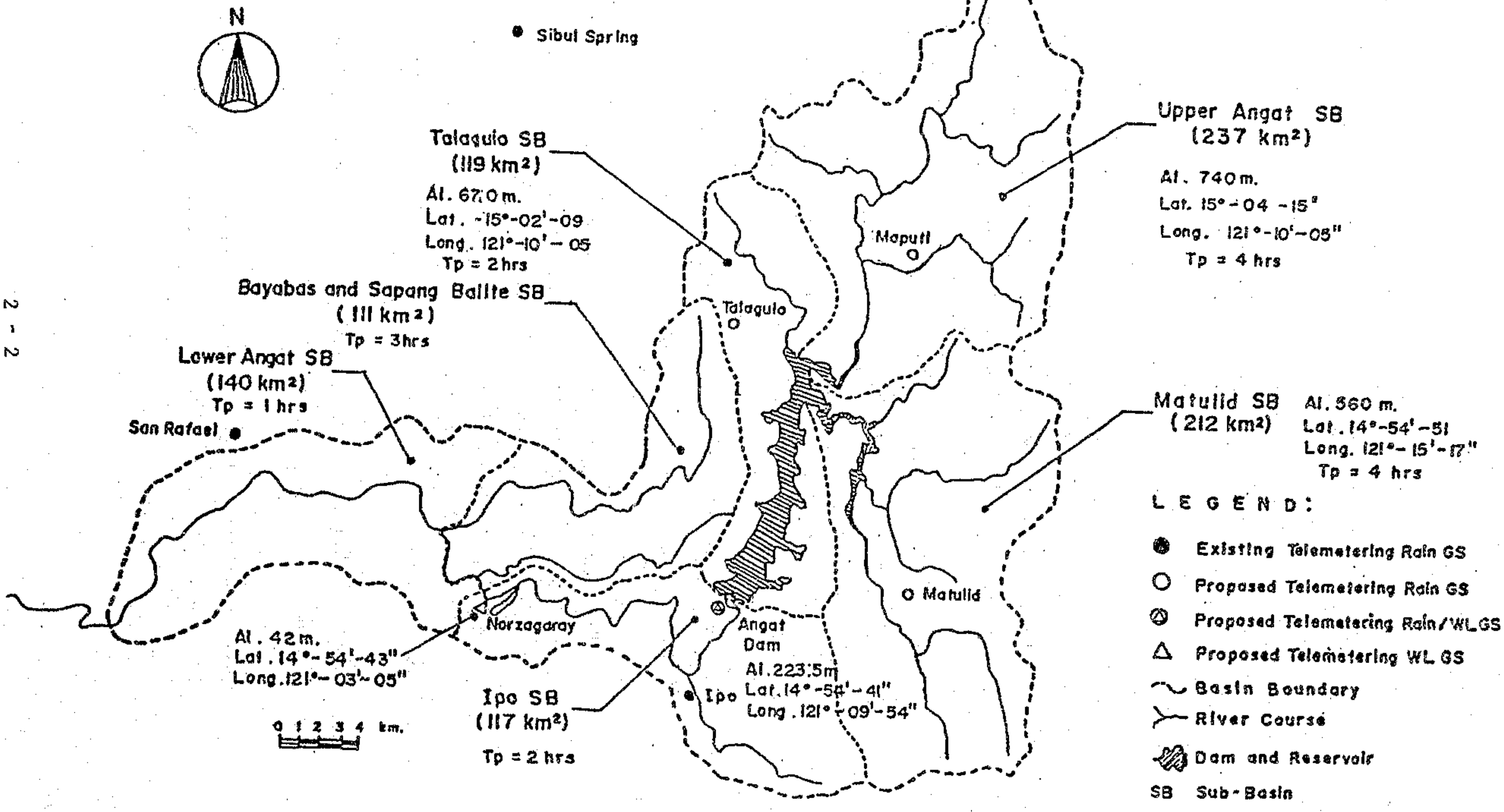
This Report is to re-examine the adequacy of the design flood inflow, using the additional data collected since the 1984 review (1982 through 1987), thereby to check whether it would be necessary to revise the result of the 1984 review.

2.1. Angat Dam Basin

The Angat dam basin is as shown in Fig.2.1 and has a drainage area of 568 km^2.

Also shown in the Figure are the locations of the rainfall gauging stations. The oldest of these stations is Norzagaray station,

Fig. 2.1 Angat River Basin



which has continued recording monthly rainfall since 1925 and daily rainfall since 1962.

The monthly inflow data are also available at the Norzagary station for the period from 1968 through 1982, and the maximum inflow was 960 x $10^6 m^3$/month or 358 m^3/sec, while the minimum inflow was 3.4 x $10^6 m^3$/month or 1.27 m^3/sec.

2.2. Rainfall Analysis

The probable precipitation can be obtained from the rainfall data taken at the Norzagaray gauging station over the period from 1925 to 1986. During the 61-year period, no data were taken in 1953 and in the period from 1940 through 1947. Except such years of non-observation, monthly rainfall data are available throughout the period. Daily rainfall data are also available since 1962.

2.2.1. Probable Annual Rainfall at the Norzagaray Site

Table A 2.1 as attached to the Appendix, containing associated Tables and Figures, shows the annual rainfall data at the Norzagaray gauging station over the period from 1925 to 1986, except years of non-observation.

Fig. A.2.1 as attached to the Appendix shows the results of calculations using the Logarithmic-Normal Distribution, Moment and Pearson III Distribution Methods on the basis of the data shown in Table A 2.1. In the Figure, the annual rainfall is shown on the abscissa, while the return periods shown on the ordinate.

It can be said from the Figure that the results by the Pearson III Distribution Method seem to be more probable in this area for the smaller return periods, but those by the Log-Normal

Distribution Method or the mean values of those by the Moment and Pearson III Distribution Methods seem to be more probable for the larger return periods (10 years or longer).

Shown in Table 2.1. as attached hereto are the probable annual rainfall calculated by different methods for each return period.

2.2.2. Probable Monthly Rainfall at the Norzagaray Site

The rainfall pattern in this area differs considerably by season, dry and rainy, as it is situated in the Torrid Zone. Tabulated in Tables A 2.2.1 through A 2.2.12 as attached to the Appendix are the monthly rainfall data taken at the Norzagaray site.

The probable monthly rainfall was calculated for evaluation of the difference in the monthly rainfall pattern from statistical point of view. Shown in Figs. A 2.2.1 through A 2.2.12 as attached to the Appendix is the probable monthly rainfall calculated by month. Used for the calculation were the Log-Normal Distribution, Moment and Pearson III Distribution Methods, as were the cases with the probable annual rainfall. As is seen in the Figures, the results by the Pearson III Distribution Method seem to be most probable.

Table 2.2 shows the probable rainfall by month for each return period. It well indicates the rainfall pattern throughout a year. For the period from January to April, there is few rainfall. May is a transient period from the dry to the wet. The rainy period starts from June and ends in December. Rainfall in this period peaks in July and August, then gradually decreases toward December. The dry period shifts rather suddenly to the rainy period, but the rainy period shifts gradually to the dry period.

Table 2.1 Probable Annual Rainfall at the Norzagaray Gauging Station

(mm)

Return Period (Yrs)	Log-Normal Distribution Method	Moment Method	Pearson III Distribution Method
Expected	1488	1471	969
2	3069	3069	3401
5	3987	4004	3878
10	4572	4602	3990
20	5119	5162	4026
50	5813	5874	4060
100	6327	6403	4067
200	6838	6928	4069

Table 2.2 <u>Probable Monthly Rainfall at the Norzagaray Gauging Station</u>

(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

This pattern of rainfall implies that it may be possible to store water to the fullest possible extent during the peak rainy period of July and August, while controlling the water level by discharging water through the spillway, but it becomes necessary to take utmost care to control the water level after it has reached HWL in August, in that the rainfall is, since then, on the gradual decrease toward November.

The change by month of the probable monthly rainfall is shown in Fig.2.2, with the probable rainfall for return period of 10, 20 and 100 years on the ordinate, and the month on the abscissa.

2.3. Inflow Analysis

Shown in Table 2.3 are the annual maximum flood inflow (m^3/sec) over the 30-year period from 1957 to 1987 and the daily maximum runoff ($10^6 m^3$) throughout each year of the same period. The daily maximum runoff was calculated on the single day, and two, three, four and five consecutive days.

The annual maximum flood inflow during the 18-year period from 1957 to 1973 was back-calculated from the measured daily maximum runoff, utilizing the equation on the relationship between the daily maximum runoff throughout a year and the annual maximum flood inflow of the corresponding year.

The annual maximum flood inflow since then was derived from the hourly operation records on the Angat dam and power plant.

The annual maximum flood inflow during the 30-year period was 5,650 m^3/sec which occurred in 1978.

Fig.2.2 Change in the Probable Monthly Rainfall by Month at the Norzagaray Site

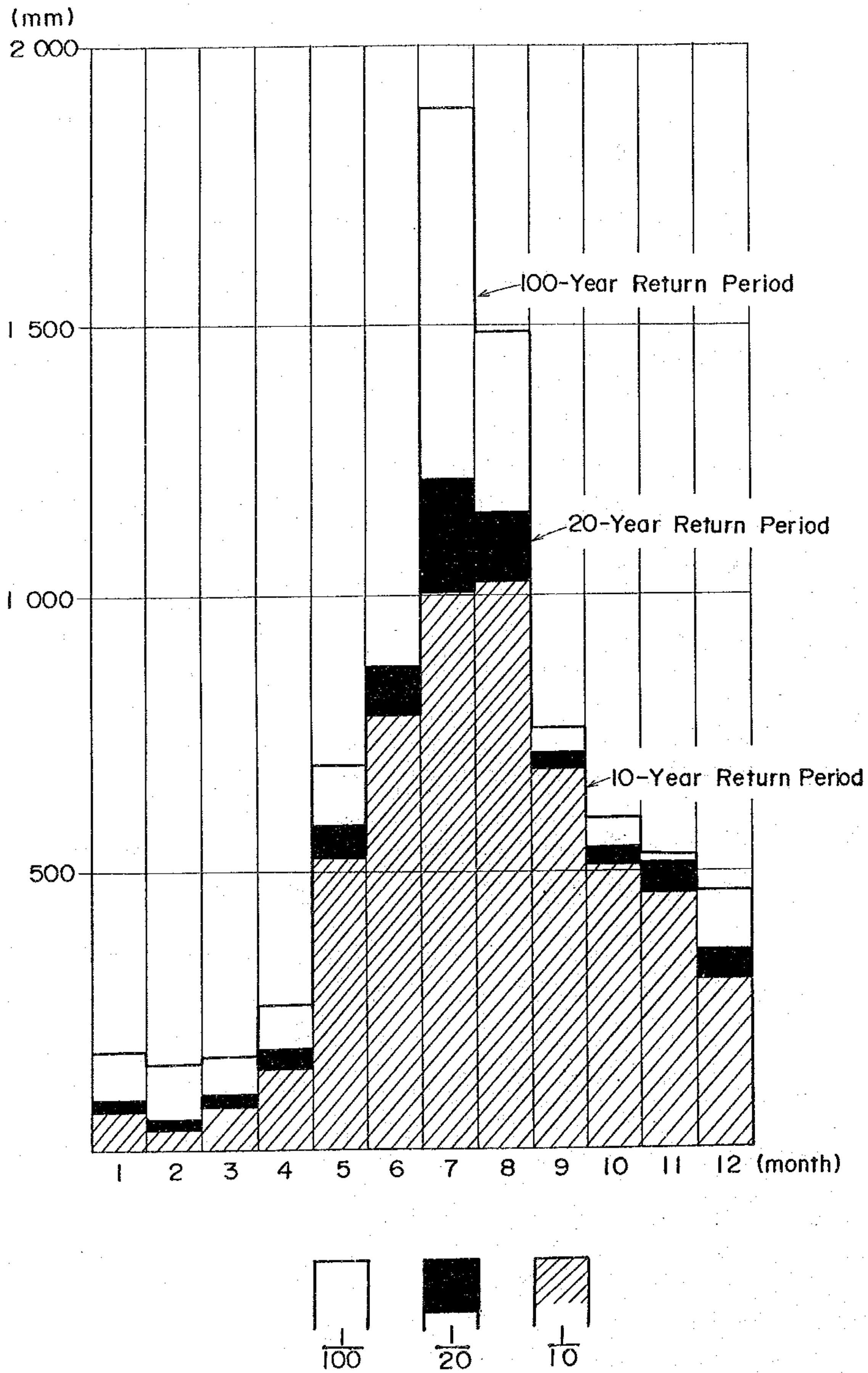


Table 2.3 Annual Maximum Inflow and Daily Maximum Runoff

Year	Annual Maximum Inflow (m^3/sec)	Daily Maximum Runoff (10^6 x 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						

2.3.1. Probable Inflow

The single day's, two, three, four and five consecutive days' maximum runoff throughout a year shown in Table 2.3 indicate in themselves the respective inflows to the Angat dam.

Tabulated in Tables 2.4.1 through 2.4.5 are the probable inflows calculated for various return periods. These are shown in Figs. A 2.3.1 through A 2.3.10 as attached to the Appendix, with the inflow on the abscissa and the return period on the ordinate. The Log-Normal Distribution, Moment and Pearson III Distribution Methods were used for the calculations. As shown in these Figures, it seems that the results by the Pearson III Distribution Method are most probable.

In these Tables and Figures, data are shown for two different periods; one from 1957 through 1982 and the other from 1957 through 1987. The data for the 1957-1982 period were those used for the 1984 joint team study for reviewing the adequacy of the design flood inflow and for establishing the gate operation procedure. The data for the 1957-1987 period were those newly prepared for the Study this time.

If there is no fundamental nor substantial difference between the data for two different periods, it indicates that the probable flood inflow established by the 1984 review should not necessarily be revised at this stage.

As can be seen from Tables 2.4.1 through 2.4.5, the data for the 1957-1982 period are greater than those for the 1957-1987 period, irrespective of the calculation method and of the return period.

This indicates that the design flood inflow determined by the 1984 review based on the data for the 1957-1982 period should be well justifiable for the use without any revision.

Table 2.4.1 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 2.4.2 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 2.4.3 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 2.4.4 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 2.4.5 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

As is seen in Tables 2.4.1 through 2.4.5, the resutls calculated by the Pearson III Distribution method are in most cases greater than those obtained by the other two methods. Therefore, the study for this Report was made using the data for the 1957-1982 period calculated by the Pearson III Distribution Method.

2.3.2. Peak Inflow

As are the cases with the daily maximum inflow (single day's, two, three, four and five consecutive days'), the probability evaluation can be made as to the annual peak inflow as given in Table 2.3. Table 2.5 shows the results of calculations of the probable inflow for various return periods, by the use of three different methods (Log-Normal Distribution, Moment and Pearson III Distribution). These are shown in Fig.2.3, with the return period on the ordinate and the inflow on the abscissa.

As is seen in Table 2.5, the results calculated by the Pearson III Distribution Method tend to be increasingly greater than those obtained by the other two methods, as the return period becomes greater. Therefore, the study for this report was made using the data calculated by the Pearson III Distribution Method, as were the cases with the daily inflow.

2.4. Probable Flood Hydrograph

The original design flood inflow to the Angat dam was determined by NAPOCOR in 1960 by using the Creager's envelop curve, and the peak flood inflow was set at 9,080 m^3/sec and the total runoff for the consecutive 3-1/2 days at 740 x $10^6 m^3$.

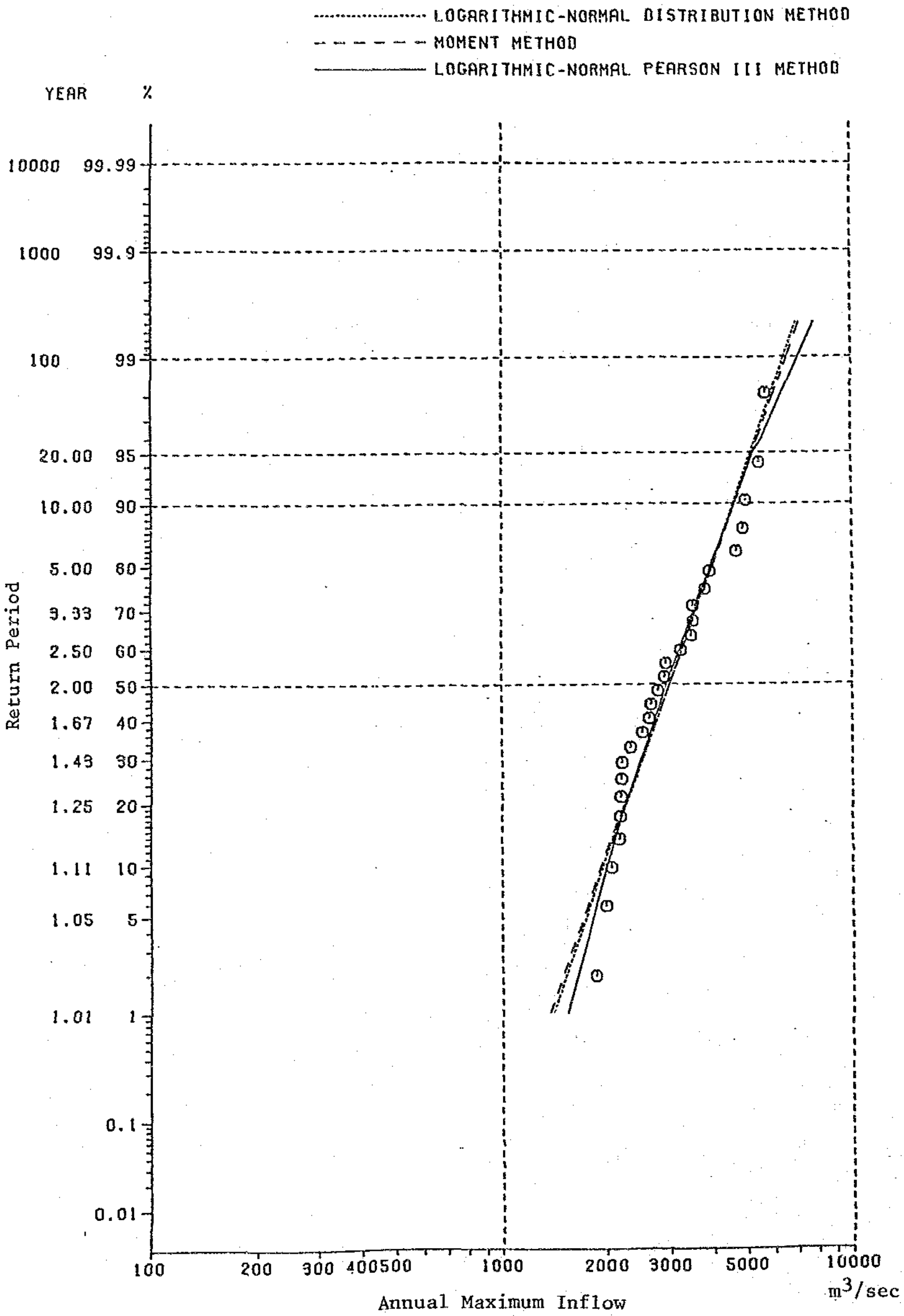
These values were, however, reviewed in August 1960 for designing the spillway. As the result, a bigger peak value of 12.075 m^3/sec and a smaller volume of 598 x $10^6 m^3$ for the consecutive 3-1/4 days' runoff were adopted. The revision was made using the unit hydrograph instead of the Creager's envelop curve.

Table 2.5. Probable Annual Inflows to the Angat Dam

(m^3/sec.)

Return Period (Yrs)	Log-Normal Distribution Method	Moment Method	Pearson III Distribution Method
2	2990	2990	2926
5	3938	3966	3905
10	4548	4597	4601
20	5122	5193	5202
25	5303	5382	5532
50	5856	5958	6267
100	6402	6529	7037
200	6947	7099	7850

Fig. 2.3 Probable Annual Inflow to the Angat Dam



These design values were further reviewed by Harza Engineering Co. in October, 1960, on the basis of the probable maximum flood determined by the probable maximum precipitation. As the result, it was recommended that the peak flood inflow be set at 7,500 m^3/sec, while the total runoff be set at 1,141 x $10^6 m^3$ for the consecutive 6-1/4 days and 923 x $10^6 m^3$ for the consecutive 3-1/4 days in the vicinity of the peak time. The peak flood inflow recommended by Harza is smaller than that determined by NAPOCOR in August 1960, but the total runoff is greater than the NAPOCOR's.

Some modification was further given to the flood inflow in October 1984 by the joint team of Nippon Koei Co., CTI Engineering Co. and Basic Technology and Management Corporation. This was for the purpose of establishing the reservoir operation procedure during flood periods as part of the "Flood Forecasting and Warning System for Dam Operation Project". The modification was based on the revised probable maximum flood due to some improvement given to the calculation method.

Table 2.6 shows a comparison of the probable maximum flood inflow (m^3/sec) and the daily runoff ($10^6 m^3$), obtained by the Harza review (1960) and the joint team review (1984). The table also shows the probable inflow (m^3/sec) for the various return periods calculated from the data given in Table 2.3 to compare with the corresponding values for the 1984 review.

As shown in the Table, the peak value of the probable inflow calculated for the review this time is greater than that for the 1984 review upon which the reservoir operation procedure was established, while there exists no substantial difference of the probable daily runoff between the two reviews.

The pattern of the probable inflow was prepared, using the peak value given in Table 2.6, and the daily runoff for any consecutive days based on the same return period as given in Tables 2.4.1 through 2.4.5.

Table 2.6 Comparison of Annual Peak Inflow and Daily Maximum Runoff among Three Reviews (1960, 1984 and 1987)

		Annual Peak Inflow (m^3/sec.)			Single Day's Maximum Runoff (10^6 m^3)		
		Harza Review (1960)	Joint Team Review (1984)	Review This Time (1987)	Harza Review (1960)	Joint Team Review (1984)	Review This Time (1987)
Flood Frequency	PMF	7500	8400	-	471	524	-
	1/200	-	-	7850	-	-	339
	1/100	-	5400	7036	-	384	295
	1/50	-	4400	6267	-	260	253
	1/25	-	3590	5532	-	225	214
	1/20	-	-	5202	-	-	197
	1/10	-	2580	4600	-	161	167
	1/5	-	-	3905	-	-	132
	1/2	-	-	2926	-	-	87

For instance, the pattern of the 100-year flood inflow was prepared based on the peak value of 7,036 m^3/sec, and the daily runoff of 294.6 x 10^6m^3, 400.8 x 10^6m^3, 488.5 x 10^6m^3, 567.2 x 10^6m^3, and 619.6 x 10^6m^3, for the single day, two, three, four and five consecutive days, respectively.

For simplifying the calculation processes, it was assumed that day-to-day change in the flood inflow except the peak day would be linear.

Tabulated in Table 2.7 is a basic pattern of the probable inflow by node and flood frequency. The position of each node in the pattern is as shown in Fig.2.4.

As is seen in the above Table and Figure, the third day's inflow pattern would produce a dominant effect on the rise of the reservoir water level. In order to check the possible change in the effect on the rise of the water level, a comparative study was made on two cases with different peak positions for the same 200-year flood inflow. One is the normal case in which the peak inflow would occur sixty hours after the flood inflow began (hereinafter referred to as Case 1). The other is the alternative case in which the peak inflow occurs six hours earlier than the normal case (hereinafter referred to as Case 2). The shift of the nodes on the third day in Case 2 is shown on Table 2.7. The peak inflow would occur at the x_4 position (t=54 hrs.) in this case.

Fig. 2.5 shows the pattern of the probable flood inflow by return period. Tables 2.8.1 and 2.8.2 show the relation between the flood inflow and passing of time. The flood inflow data shown in the Tables are those only after 48 hours have passed since the flood inflow began. This is because there would be no problems during the first 48 hours, even if a flood of the PMF magnitude should occur and the flood inflow would be wholly discharged downstream with no storage in the reservoir, as it would be still

Table 2.7 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 2.8.1 Relation Between Flood Inflow and Passing of Time (200-Year Flood Inflow)

(m^3/sec.)

Passing of Time	(1) 200-Yr Flood Inflow	(2) Case 1 (1)x1.2 *	(3) Case 2	Passing of Time	(1) 200-Yr Flood Inflow	(2) Case 1 (1)x1.2 *	(3) Case 2
48	1500	1800	1800	69	2250	2700	2280
49	1800	2160	3070	70	2000	2400	2120
50	2100	2520	4340	71	1750	2100	1960
51	2400	2880	5610	72	1500	1800	1800
52	2700	3240	6880	73	1475	1770	1770
53	3000	3600	8150	74	1450	1740	1740
54	3300	3960	9420	75	1425	1710	1710
55	4058	4870	8658	76	1400	1680	1680
56	4817	5780	7896	77	1375	1650	1650
57	5575	6690	7134	78	1350	1620	1620
58	6333	7600	6372	79	1325	1590	1590
59	7092	8510	5610	80	1300	1560	1560
60	7850	9420	4848	81	1275	1530	1530
61	7041	8449	4500	82	1250	1500	1500
62	6233	7480	4152	83	1225	1470	1470
63	5425	6520	3804	84	1200	1440	1440
64	4617	5540	3456	85	1175	1410	1410
65	3803	4564	3108	86	1150	1380	1380
66	3000	3600	2760	87	1125	1350	1350
67	2750	3300	2600	88	1100	1320	1320
68	2500	3000	2440				

Note : * A 20% increase over the 200-year flood inflow is normally used in Japan for determining the design flood inflow for rockfill dam.

Table 2.8.2 Relation Between Flood Inflow and Passing of Time by Return Period

(m^3/sec)

Passing of Time (Hrs)	Return Periods (Years)						Passing of Time (Hrs)	Return Periods (Years)					
	200	100	50	25	20	10		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

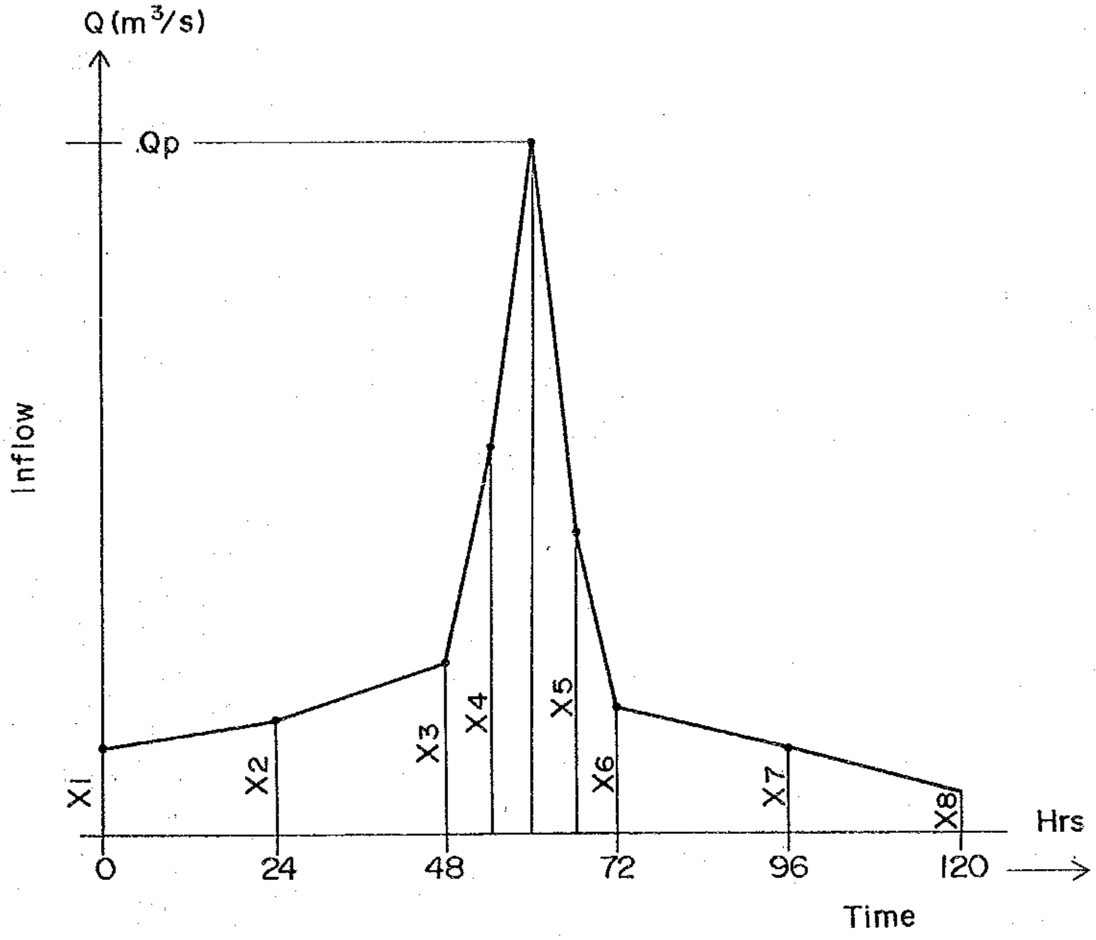
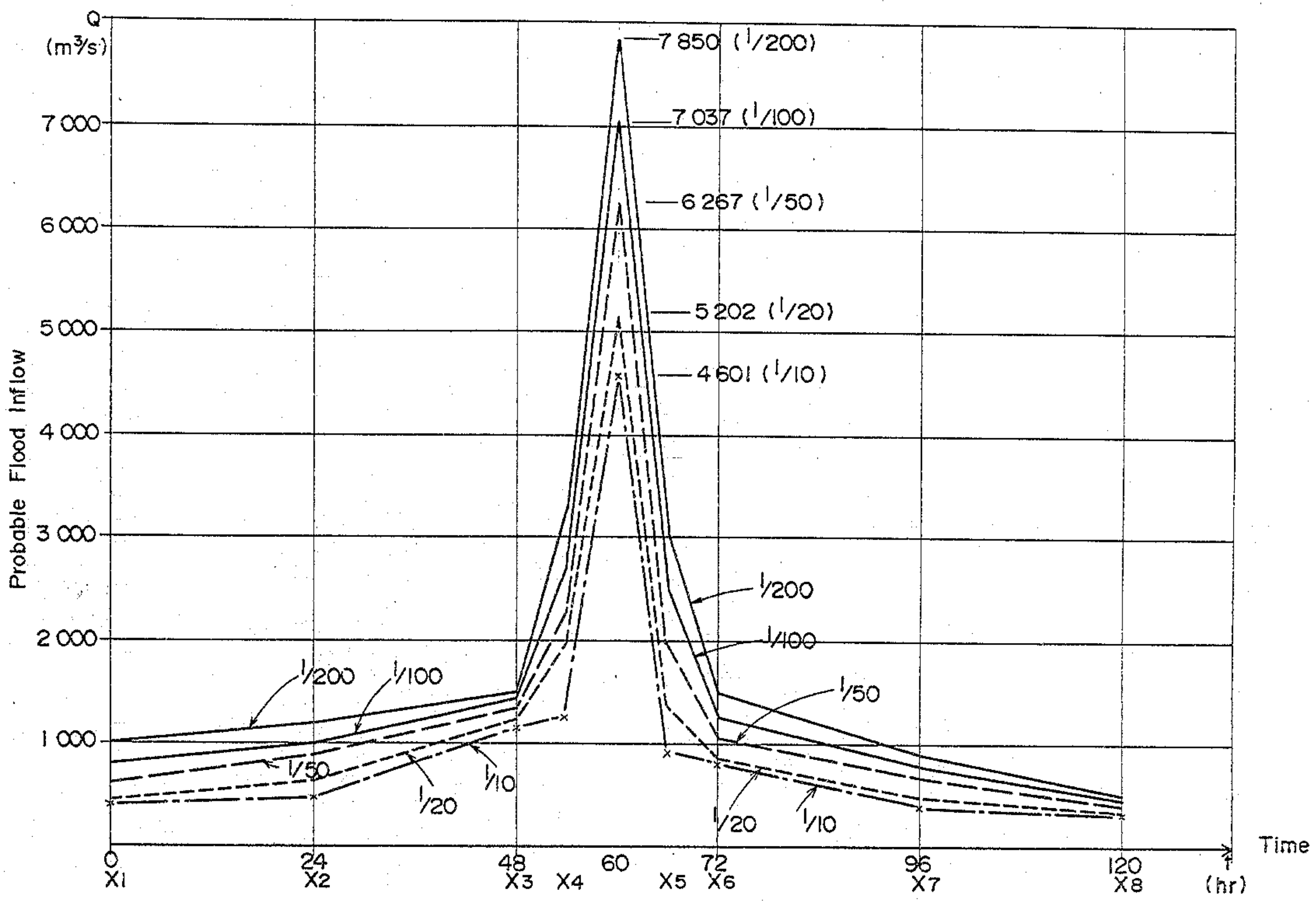


Fig. 2.4 Basic Pattern of the Probable Inflow

Fig. 2.5 Patterns of the Probable Flood Inflow by Return Period



within the maximum limit of the discharge rate that assures no damages to the downstream basin. Another reason is that the effect of a flood inflow on the reservoir water level should be examined only as to the cases where a flood occurs at times of operating the dam with the water level at EL 212 m (HWL), for instance, and the flood inflow would increase to a certain level (3 x 789 m^3/sec = 2,367 m^3/sec. as described in Sect. 2.5.3. below), with which the water level may no longer be kept at the same EL 212 m, even with all spillway gates fully open.

2.5. Adequacy of Spillway Capacity

2.5.1. Dimensions of the Spillway Structures

Principal dimensions and characteristics of the spillway structures and the reservoir are as shown below:

1)	Dam crest elevation	:	EL 221.5 m
2)	Elevation of the top of spillway gate	:	EL 217.0 m
3)	Elevation of the top of wave deflector of spillway gate	:	EL 217.6 m
4)	Elevation of the crest of overflow weir of spillway	:	EL 202.0 m
5)	Number of spillway gates	:	3
6)	Height of spillway gate	:	15 m
7)	Width of spillway gate	:	12.5 m
8)	Operation speed of spillway gate	:	0.3 m/min.
9)	Reservoir drainage area	:	598 km^2
10)	Reservoir surface area at normal high water level	:	23 km^2
11)	Normal high water level	:	EL 212.0 m

2.5.2. Restrictions on Gate Operation

It is most essential in operating spillway gates during the flood period that the utmost care be taken to keep the downstream basin from being inundated and to protect inhabitants from any possible damages.

The study for "Flood Forecasting and Warning System for Dam Operation Project" made by the joint team in 1984 was to establish the criterion for the permissible rate of flood discharge to the downstream basin, based on the result of unsteady flow analyses of flood waves to be generated by the spillway gate discharge.

The permissible rate of flood discharge was determined so as to keep the rate of increase in the water level at the Norzagaray gauging station downstream of the Angat dam at lower than 30 cm per 30 minutes.

Shown in Fig.2.6 is the permissible rate of flood discharge determined by such criterion. The Figure indicates that it is possible to increase flood discharge at the rate of 52.8 m^3/30 min. from t=0 (starting with 183 m^3/sec. for the release through turbines) to t=3 (500 m^3/sec.). In other words, the rate of flood discharge would come up to 500 m^3/sec. three hours after the start of discharge. The Figure also indicates that the rate would rise to 800 m^3/sec. from 500 m^3/sec. in 1-1/4 hours.

A study made this time was to examine in more detail the relation between the permissible rate of flood discharge and the flood inflow.

The results of the study are shown in Table 2.9. Given for comparison in the Table are the hourly inflow of the PMF magnitude flood to the Angat dam (Qi), the hourly increment of the flood inflow ($\frac{dQi}{dt}$), and the permissible rate of flood

Fig.2.6. Permissible Rate of Increase of Flood Discharge

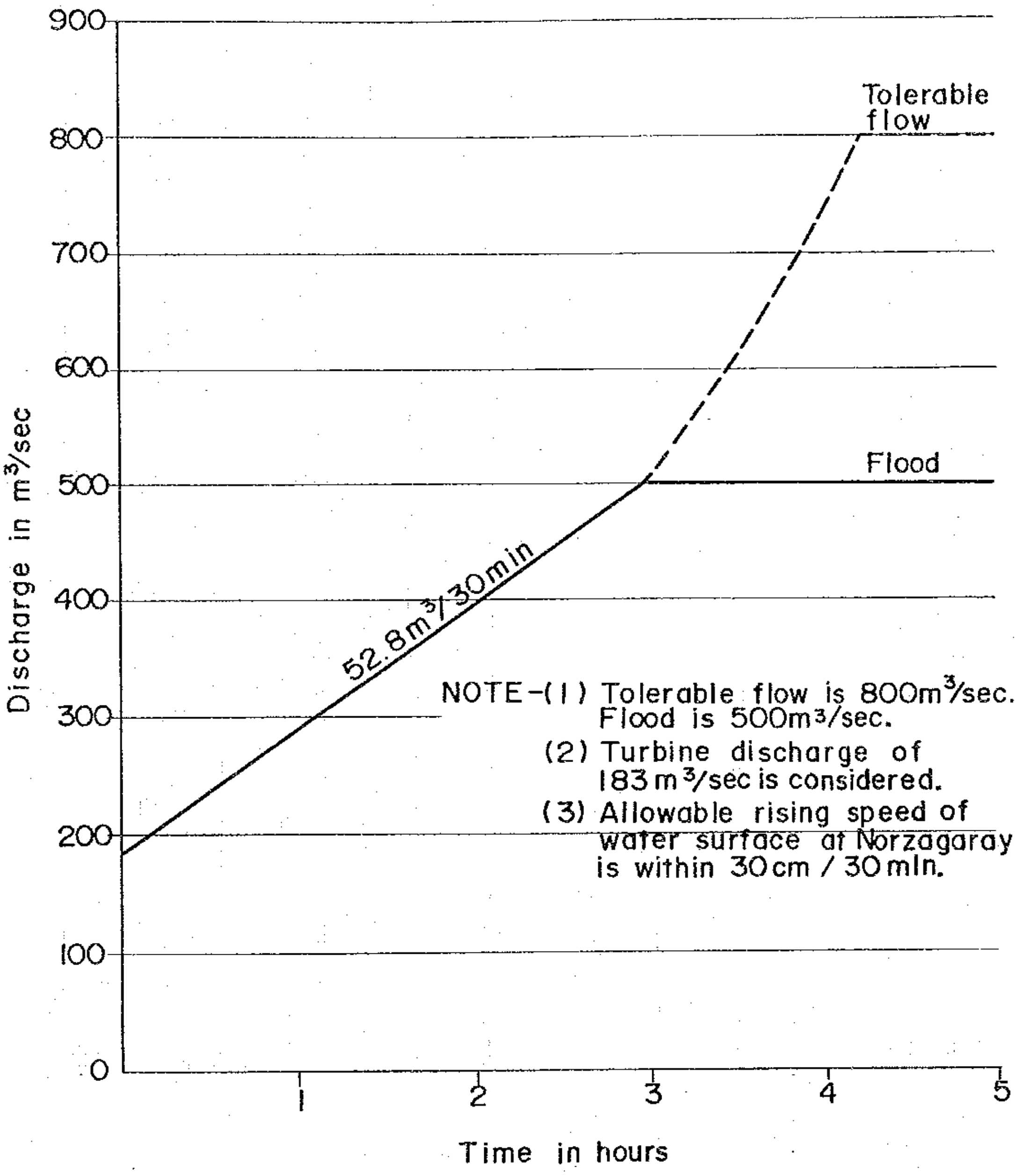


Table 2.9 Relation Between Flood Inflow and Permissible Rate of Flood Discharge

(t/hr)

Passing of Time (Hr)	Hourly Inflow of PMF Magnitude Flood (Qi)	Hourly Increment of Flood Inflow $\left(\frac{dQi}{dt}\right)$	Permissible Rate of Flood Discharge $\left(\frac{dQi}{dt}\right)$	Passing of Time (Hr)	Hourly Inflow of PMF Magnitude Flood (Qi)	Hourly Increment of Flood Inflow $\left(\frac{dQi}{dt}\right)$	Permissible Rate of Flood Discharge $\left(\frac{dQi}{dt}\right)$
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				

discharge ($\frac{dQo}{dt}$), which can meet the limit of the rate of increase in the water level (30 cm/30 min.) at the Norzagaray site.

The water level - discharge curve at the Norzagaray site can be expressed as:

$$Qo = aH^2 + bH + C$$

Where, at the Norzagaray site:

a = 83.501
b = -235.4
c = 301.1

The permissible rate of flood discharge ($\frac{dQo}{dt}$) can be written as:

$$\frac{dQo}{dt} = (2aH + b)\frac{dH}{dt}$$

Where, $\frac{dH}{dt}$ can be replaced with Ko, or the permissible rate of increase in the water level.

Then,

$$\frac{dQo}{dt} = 2\sqrt{a} \cdot \sqrt{Qo + \frac{b^2 - 4ac}{4a}} \cdot Ko$$

$$= 11\sqrt{Qo - 135} \cdot Ko$$

Since $\frac{dQo}{dt}$ is the hourly rate of increase, $\frac{dH}{dt}$ was shown as 0.6 (0.3 m/0.5 hr.).

It is obvious from Table 2.9 that, even if a flood of the PMF magnitude should occur and the flood inflow would be discharged downstream without any flow control, the rate of increase in the water level at the Norzagaray site could be well within the limit of 30 cm/30 min. unless any intentional gate operation is done to decrease the reservoir water level.

Since the flood discharge through the spillway forms a regime of unsteady flow in the downstream, it may not necessarily be reasonable to determine the adequacy of the spillway capacity only on the basis of the water level - discharge curve at the Norzagaray site. But, there are no substantial discrepancies in the result of evaluation made on the assumption that the regime would be of steady flow because of the extreme slowness of its change.

It is therefore considered that the rate of discharge through the Angat spillway may be subject to the discharging capacity of the spillway gates rather than to the requirement for the harmless discharge to the downstream basin, unless any intentional gate operation is done to decrease the reservoir water level.

2.5.3. Spillway Discharge Rating Curve

Fig. 2.7 is the discharge rating curve of the Angat spillway, showing the relation between the reservoir water level in front of the spillway gates and the spillway discharge by gate opening position.

Table 2.10 shows the relation between the reservoir water level and the maximum discharge per gate before free flow condition and after free overflow condition at full gate opening.

2.5.4. Maximum Limit of Water Level

The height of the non-overflow section of a dam differs by dam type and provision of spillway gates. It is a general practice in Japan to determine the elevation of the top of the non-overflow section of the fill type dam provided with spillway gates by adding at least one meter to the larger one of the

Fig. 2.7 Discharge Rating Curve of Spillway

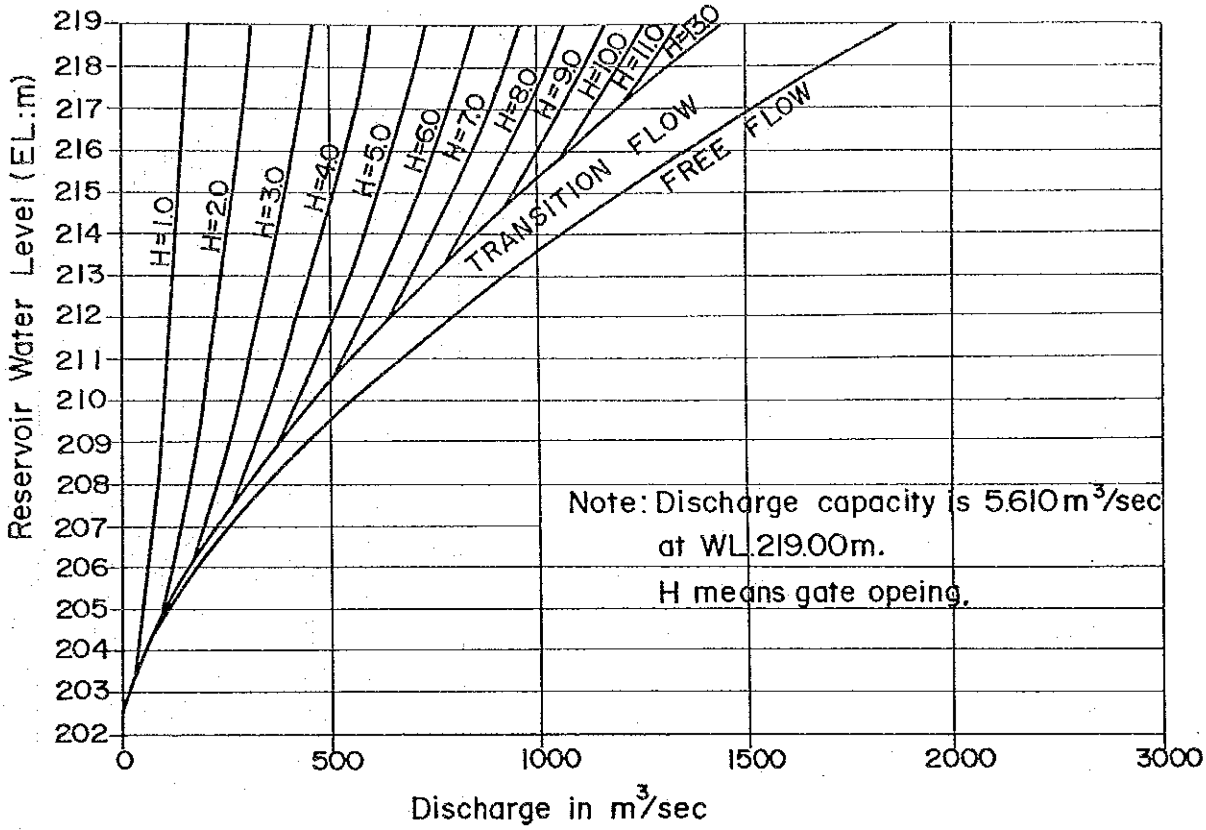


Table 2.10 Spillway Discharging Capacity Per Gate

(m^3/sec)

Reservoir Water Level	Maximum Discharge Immediately Before Free Flow Condition	Discharge in Free Flow Condition at Full Gate Opening
207	229	261
208	301	348
209	379	446
210	464	551
211	553	666
212	648	789
213	748	920
214	853	1060
215	961	1207
216	1074	1362
217	1191	1523
218	1313	1693
219	1437	1870

values determined by the following two equations:

(i) $Hn + hw + he + 0.5$, or
$Hn + 2$, if $hw + he$ is smaller than 1.5.

(ii) $Hd + hw + 0.5$, or
$Hd + 1$, if hw is smaller than 0.5.

Where,

Hn = Normal water level
hw = Height of wave above the reservoir water surface due to winds at the design flood water level, and can be expressed as:

$$hw = 0.00086\, V^{1.1} \cdot F^{0.45}$$

Where,

V = Wind velocity (m/sec), average of 10 minutes.
F = Fetch (m), the maximum distance from the dam to the opposite bank at the design flood water level.

he = Height of wave above the reservoir water surface due to earthquakes with the design seismic coefficient used for calculation of the dam structures at the normal water level, and can be expressed as:

$$he = \frac{1}{2} \cdot \frac{k\tau}{\pi} \sqrt{gHo}$$

Where,
k = Seismic coefficient
τ = Seismic period
Ho = Depth of reservoir water

hd = Design flood water level

For the Angat dam,

HWL = EL 212 m
F = 10,000 m
V = 25 m/sec.
K = 0.15 g
τ = 1 sec.
Ho = 108 m (HWL 212 m - Dam invert EL 104 m)

Accordingly,
hw ≑ 2 m
he ≑ 0.8 m

Therefore, the dam crest elevation to meet the requirement by Equation (i) is:

EL 212 m + 2 m (hw) + 0.8 m (he) + 0.5 m + 1.0 m = EL 216.3 m

Since the dam crest is at EL 221.5 m, the dam still has a sufficient freeboard.

The maximum limit of the water level for this dam during the probable flood (with the 200-year flood inflow multiplied by 1.2) must be lower than:
EL 221.5 m - 1 m - 0.5 m - 2 m (hw) = EL 218.0 m,
according to the requirement by Equation (ii).

Therefore, the design flood water level should be at EL 218.0 m.

2.5.5. Examination of Design Flood Inflow

As the design flood inflow, the 20 % increase over the 200-year flood inflow was applied to the case of the Angat dam, in accordance with the corresponding provision formulated by the

Ministry of Construction of Japan.

Hourly flood inflow used for the examination is as given in Table 2.8.1. The pattern of the flood inflow is as shown in Fig.2.8.

(1) Gate Operation Modes

It was assumed that there would occur a flood inflow at times when the dam is being operated with the water level at a certain elevation, for instance at H.W.L. (EL 212 m), and the following modes of gate opening operation against the flood inflow would be taken:

- To start gate opening operation to maintain the reservoir water level at the same elevation, or
- To control gate opening operation, when needed, to increase the water level to a certain elevation.
- To fully open all gates to discharge water in free flow condition when it became no longer possible to control the rise of the water level by gate opening operation.
- To keep all gates fully open until the flood is over and the water level comes down to HWL or a level close to HWL.
- Then to start gate opening operation again to control water discharge.

Calculations of the spillway discharge and change in the reservoir water level were made as to the following four cases:

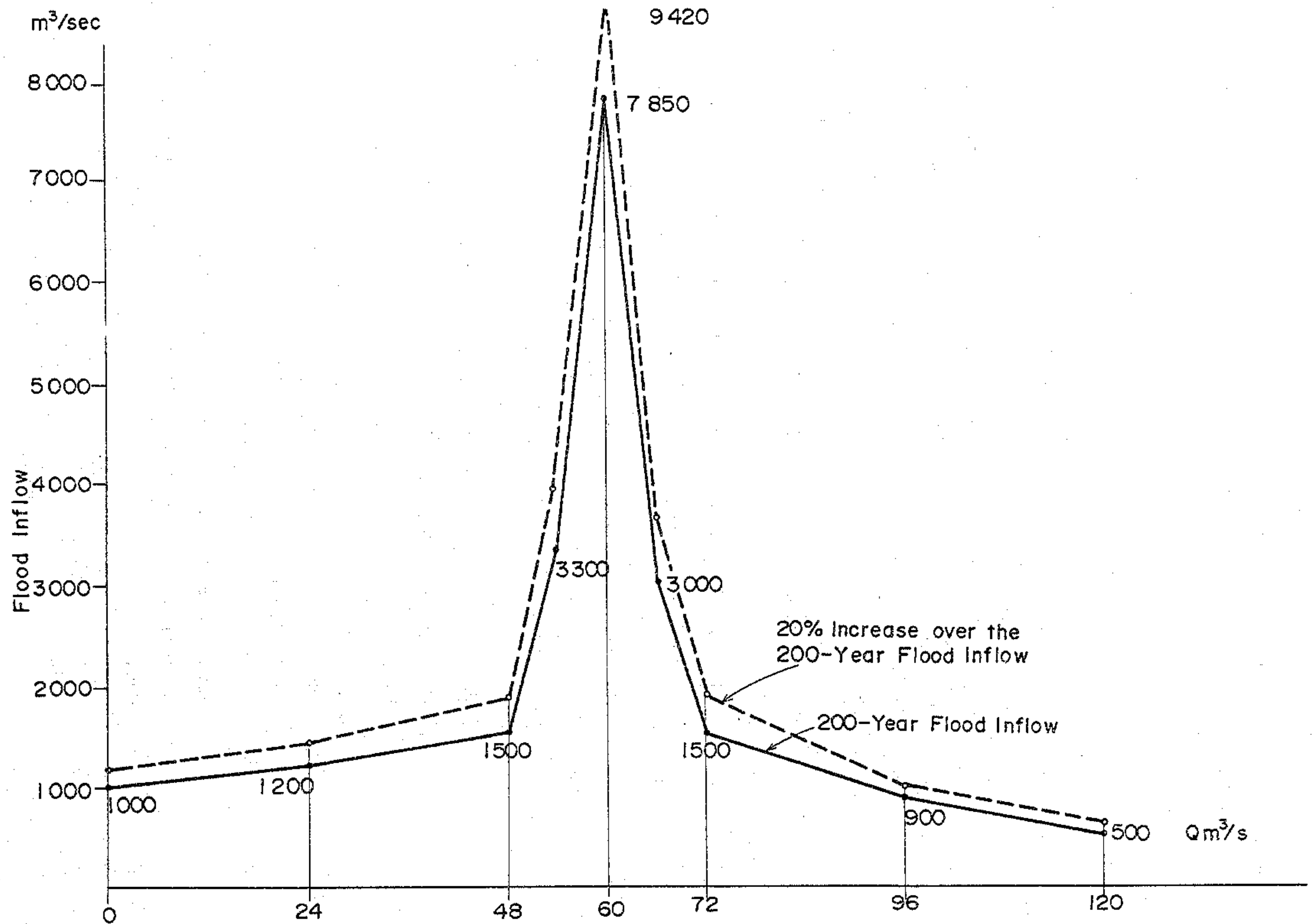


Fig.2.8 Pattern of Flood Inflow to the Angat Dam

	Reservoir water level before starting gate opening operation	Reservoir water level at full gate opening
1.	EL 210 m	EL 210 m
2.	EL 211 m	EL 211 m
3.	EL 212 m (HWL)	EL 212 m
4.	EL 212 m (HWL)	EL 213 m

It is extremely difficult to continue gate opening operation while keeping the water level constantly at HWL (EL 212 m), unless exact data on ever-changing rate of inflow are attained. Besides, there should occur some errors in the process of gate opening operation.

This is the reason why the calculations were made of the case in which gate opening operation is done so as to make the full gate opening discharge almost equal to the inflow, while keeping the water level in the range from EL 212 m to 213 m.

(2) Gate Discharging Capacity

The spillway discharging capacity per gate is as given in Table 2.10. This can be expressed in the following equations with the water level(H) measured from EL 170 m (as a base line) as a parameter:

$$Qoc = a1H^2 + a2H + a3$$
$$Qo_F = b1H^2 + b2H + b3$$

Where,

Qoc = Maximum discharge to be controlled by gate opening operation.

Qo_F = Discharge in free flow condition at full gate opening.

Given below are the values for a1, a2, a3, b1, b2 and b3 in the above equations, obtained by the method of least squares from those shown in Table 2.10 multiplied by 3 (number of gates):

a1 : 7.08941
a2 : -306.277
a3 : 2,304.889
b1 : 12.0764
b2 : -635.1713
b3 : 7,743.836

Compatibility of the values determined by such simplified method can be confirmed by the comparison shown in Table 2.11. It is, therefore, well justifiable that the gate discharging capacity may be expressed as:

$$Qoc = 7.0894\,H^2 - 306.277\,H + 2,304.889$$
$$Qo_F = 12.0764\,H^2 - 635.1713\,H + 7,743.836$$

(3) Calculation Method

The relation between the reservoir water surface area (F) and the water level (H) measured from EL 170 m (as a base line) can be expressed as:

$$F = [0.0014286H^2 + 0.20629H + 10.100] \times 10^6 m^2$$

The flood outflow from the reservir and the change in the water level in conjunction therewith can be obtained by the following equation:

$$[\frac{QIi + QI(i+1)}{2}] \cdot \Delta t$$

$$= [\frac{Qoi + Qo(i+1)}{2}] \cdot \Delta t + [\frac{Fi + F(i+1)}{2}][H(i+1) - Hi]$$

Table 2.11 Verification of the Simplified Method to Obtain Spillway Discharging Capacity

(m^3/sec)

Reservoir Water Level	Water Level Measured From EL 170m	Maximum Discharge to be Controlled by Gate Operation (Q_{OC})		Discharge in Free Flow Condition at Full Gate Opening (Q_{OF})	
		Table 2.11	Calculated	Table 2.11	Calculated
EL m 212	m 42	1944	1947	2367	2369
213	43	2244	2243	2760	2761
214	44	2559	2553	3180	3176
215	45	2883	2878	3621	3616
216	46	3222	3217	4086	4080
217	47	3573	3570	4569	4568
218	48	3939	3938	5079	5080
219	49	4311	4319	5610	5616

Where,

QIi = Flood inflow to the reservoir (i) hours after the flood inflow began.

$QI(i+1)$ = Flood inflow to the reservoir (i+1) hours after the flood inflow began.

Qoi = Outflow from the reservoir (i) hours after the flood inflow began.

$Qo(i+1)$ = Outflow from the reservoir (i+1) hours after the flood inflow began.

Hi = Water level measured from EL 170 m (i) hours after the flood inflow began.

$Hi+1$ = Water level measured from EL 170 m (i+1) hours after the flood inflow began.

Δt = Time interval between (i) and (i+1) hours.

The above equation can be simplified by replacing $H(i+1)$ with $Hi + \Delta Hi$ and omitting orders higher than $(\Delta Hi)^2$. When Δt is assumed to be one hour, then $H(i+1)$ will be the water level one hour after Hi.

The result of the calculations can be shown as:

$$Hi+1 = Hi + \frac{\frac{QIi+QI(i+1)}{2} - (12.0764Hi^2 - 635.1713Hi + 7,743.836)}{0.396865Hi^2 + 69.38376Hi + 2,488.1944}$$

(4) Result of Calculations

Shown in Figs. 2.9 and 2.10 are the change in the reservoir water level and the inflow to and outflow from the reservoir during the flood with frequency of 1/200

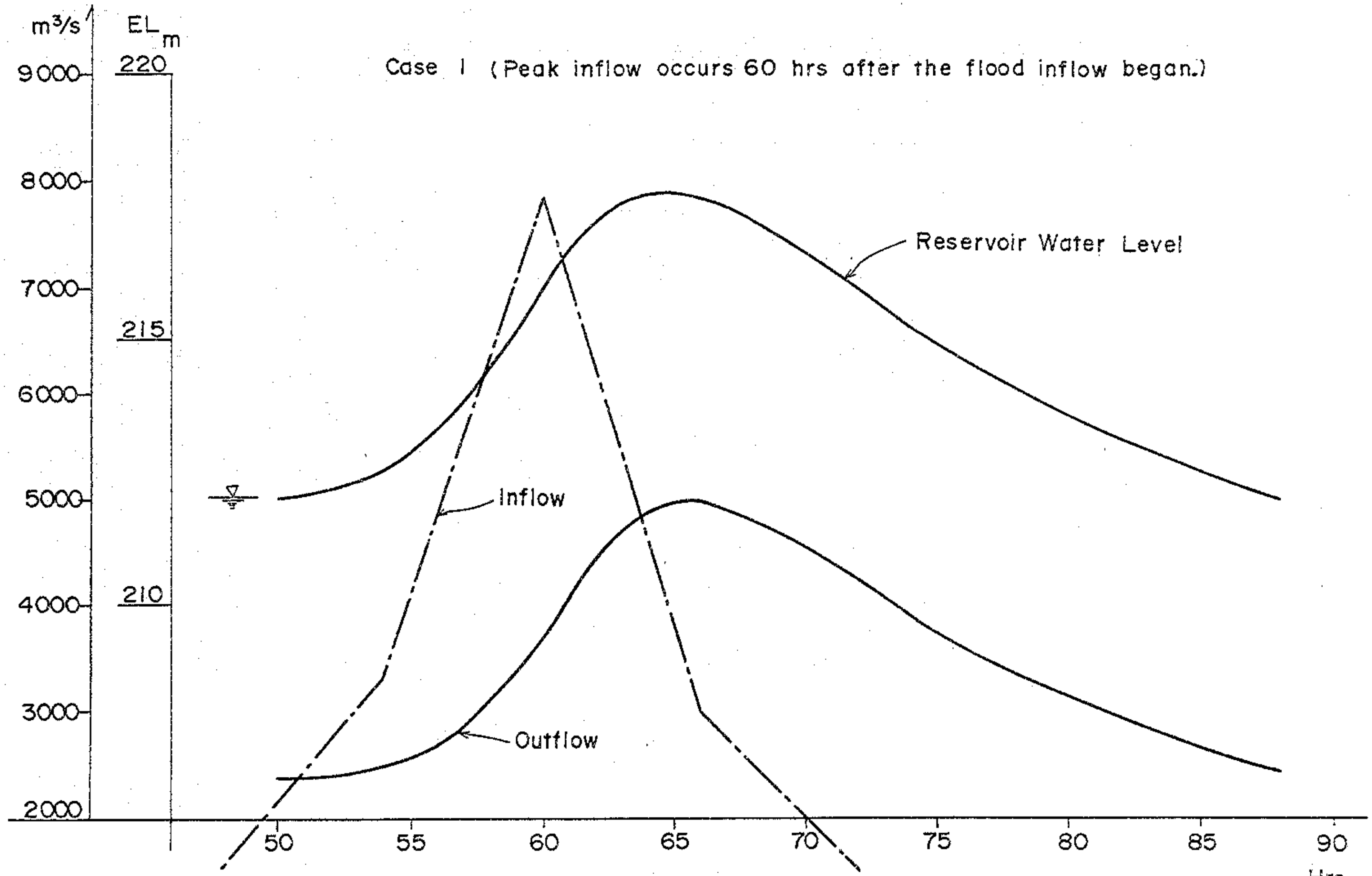


Fig. 2.9 Change in the Reservoir Water Level and the Inflow to and Outflow from the Dam during the Flood with a 200-Year Return Period Multiplied by 1.2

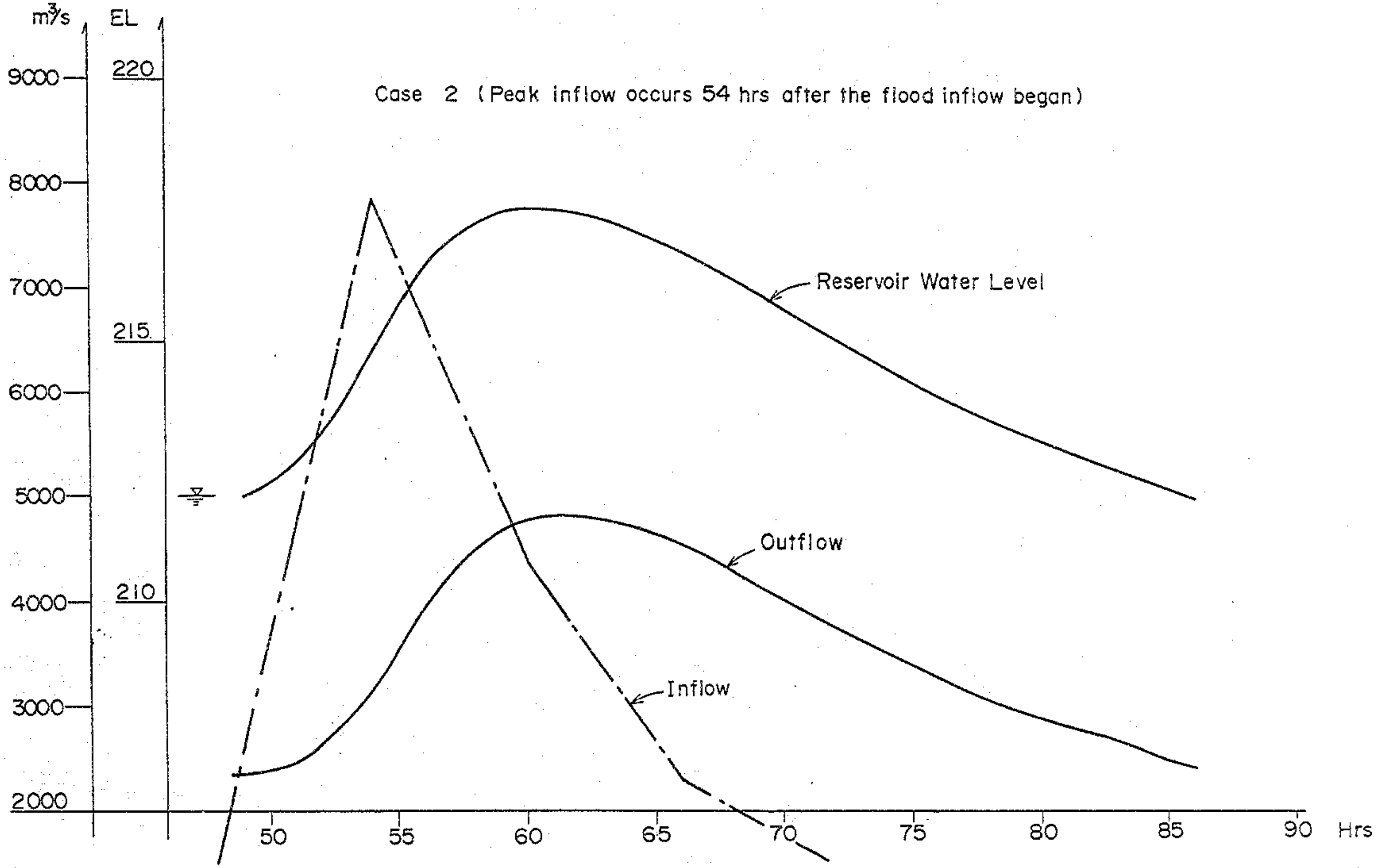


Fig. 2.10 Change in the Reservoir Water Level and the Inflow to and Outflow from the Dam during The Flood with a 200-Year Return Period Multiplied by 1.2

multiplied by 1.2, assuming that it would occur when the water level is at EL 212 m, and the inflow (Qi) would be wholly discharged downstream by gate opening operation, in so far as Qi is smaller than 2,367 m^3/sec., but be left uncontrolled with all gates fully open, after Qi becomes larger than 2,367 m^3/sec.

Fig. 2.9 shows the case in which the peak inflow would occur 60 hours after the flood inflow began (as referred to as Case 1 in Sect. 2.4), and Fig. 2.10 shows the case in which the peak inflow would occur six hours earlier than the time of Case 1 (as referred to as Case 2 in Section 2.4). A chain line in the Figures shows the flood inflow.

Table 2.12 shows the maximum reservoir water level, the maximum rate of discharge and the ratio of peak outflow to peak inflow during floods for various return periods.

On the basis of the above calculations, the following conclusions can be made:

i) The spillway has a sufficient discharging capacity against the design flood inflow, if the following steps for gate opening operation are taken properly:

- To set the reservoir water level at EL 212 m before the flood inflow began.

- To keep the water level at the same elevation by gate opening operation after the flood inflow.

- To fully open all gates when the inflow exceeds the rate of 2,367 m^3/sec.

Table 2.12 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

(Table 2.12 continued)

Flood Inflow for Various Retuen 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

These steps may allow the water level to rise to EL 217.8 m, but this level is still lower than the maximum limit of EL 218 m.

ii) Even in case where the reservoir water level may not be kept at EL 212 m due to improper gate opening operation, and would, instead, rise to EL 213 m, the maximum water level at full gate opening with the inflow in excess of the rate of 2,760 m^3/sec, would be still within a controllable limit, as it would exceed the maximum limit of EL 218 m by only 0.12 m.

iii) In case when there occurs the PMF magnitude flood inflow, the maximum reservoir water level would be at much higher elevation than that in the case of the design flood inflow. The maximum water level would come up to EL 219.35 m in case when the water level at full gate opening is at EL 213 m, and to 219.20 m even in case when the water level at full gate opening is at EL 210 m. Therefore, it may be difficult to curb the rise of the maximum water level against the PMF magnitude flood inflow, even by setting the water level at full gate operation at a reduced elevation.

However, the PMF magnitude is too severe to be considered as the design condition for the spillway, and it may not be necessary to consider any specific freeboard for the rise of the water level by such magnitude flood inflow. If so, the maximum permissible water level in this case should be regarded as:

Dam crest elevation (EL.221.5 m) - the height of wave due to winds (2.0 m) = EL.219.5 m.

This level permits no overflow even in case of the PMF magnitude flood inflow.

It was confirmed from the above studies that the existing spillway of the Angat dam may prove effective for disposing of the probable flood inflow, though it may not be sufficient enough in its capacity. Also confirmed from the studies were the following points:

i) The maximum water level is lower in Case 2 pattern (in which the peak inflow would occur earlier) than in Case 1 normal pattern.

ii) The bigger the design flood inflow is, the smaller the ratio of the peak outflow to the peak inflow becomes, or the greater effect on the flood control by the dam and reservoir can be expected.

iii) Increase or decrease in the water level at full gate opening, if in a minor measure, may have little effect on the increase or decrease in the maximum water level. For instance, lowering the water level at full gate opening by one meter may make the maximum water level lower by only 30 cm.

3. STABILITY OF THE DAM AND THE DYKE

3. Stability of the Dam and the Dyke

3.1. Design and Construction Records

Construction of the Angat dam commenced 27 years ago in 1961. The dam was completed in 1967. The power plant started its operation in September, 1967, according to the record.

At the inception of this Study, efforts were directed to collect the design and construction records and other relevant data on the project as much as possible from the NAPOCOR head office and the Angat power plant. However, adequate records on the design of the dam were not available after all, and no construction records were available except a few as-built drawings of the dam (a plan and a cross section on the center portion).

3.2. Basic Design Concept

The final design conditions as actually used for construction, including details of changes in such conditions during construction, are not known. However, it appears from the as-built drawings that no substantial changes were made to the design conditions.

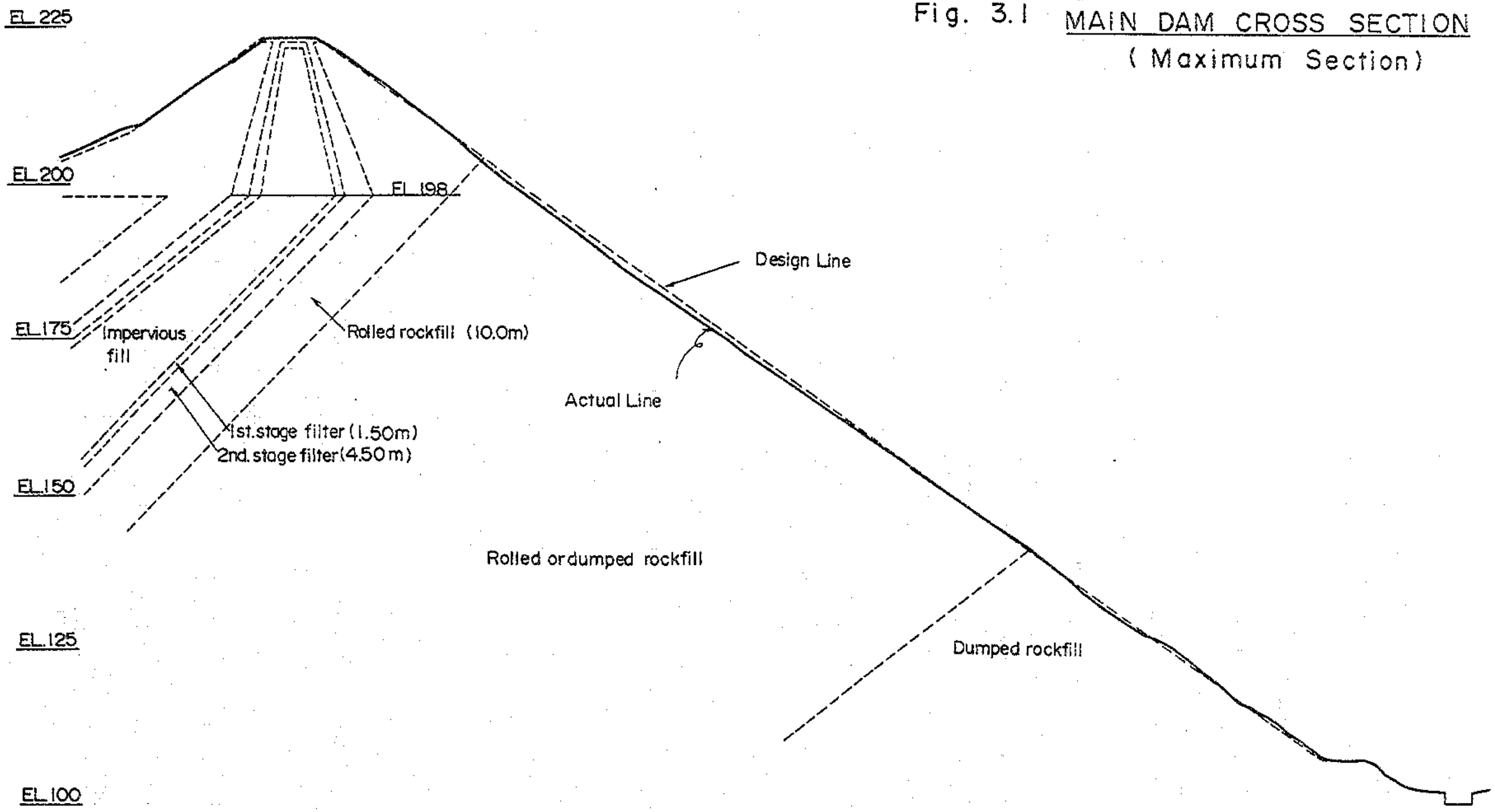
The main characteristics of the main dam and the dyke are as mentioned below:

3.2.1. Main Dam

The Angat dam is a rockfill dam with an impervious inclined core, and 131 m high. It is principally constructed of dumped rockfill. Two layers made up of coarse and fine filter are provided both upstream and downstream of the core zone.

The cross section of the dam on the center portion is as shown in Fig.3.1. Indicated in dotted lines on the Figure is the

Fig. 3.1 MAIN DAM CROSS SECTION
(Maximum Section)



cross section taken from the design drawing, and given in solid lines is the cross section based on the field measurement made for this Study.

As is seen in the Figure, the solid line cross section is in substantial agreement with the dotted line cross section, despite the fact that the measurement of the upstream zone was made only at the crest portion, as the reservoir water level was then at HWL. It appears from this agreement that the dam was constructed as originally designed and has not suffered from any damages since then.

The upstream and downstream face slopes of the dam are as follows:

Upstream face slope

EL 222.0 m - EL 208.0 m
Design : 1 : 1.4
Actual : 1 : 1.4

EL 208.0 m or lower
Design : 1 : 2.4
Actual : 1 : 2.4 (Est.)

Downstream face slope

EL 220.0 m or lower
Design : natural slope (1 : 1.4 Est.)
Actual : 1 : 1.4

3.2.2. Dyke

The dyke is a rockfill dam with an inclined core, and is 52 m high. The construction is similar to the main dam. It is principally constructed of dumped rockfill. Two layers made up of coarse and fine filter are provided both upstream and downstream of the core zone.

Shown in Figs. 3.2 through 3.4 are the cross sections of the dyke on the three measurement lines, A-A', B-B' and C-C'. Indicated in dotted lines on the Figures are the cross sections taken from the design drawing, and given in solid lines are those based on the results of the field measurement conducted for this Study.

As is the case with the main dam, it appears that the dyke was constructed as originally designed and has not suffered from any damages since then.

The upstream and downstream face slopes of the dyke are as follows:

Upstream face slope

EL 222.0 m - EL 212.0 m
Design : 1 : 1.4
Actual : 1 : 1.4

EL 212.0 m - EL 190.0 m
Design : 1 : 2.2
Actual : 1 : 2.2 (Est.)

EL 190.0 m - EL 175.0 m
Design : Natural slope
Actual : 1 : 1.4 (Est.)

Downstream face slope

EL 222.0 m - EL 135.0 m
Design : Natural slope (1 : 1.4 Est.)
Actual : 1 : 1.45

EL 135 m Design : Horizontal
Actual : (Horizontal Est.)

3.3. Historical Records after Construction

There are no records on the dam behavior after its completion, because no measurement instruments were installed at the time of construction. However, no damages were found out by the survey made this time.

Fig. 3.2 DYKE CROSS SECTION A - A'

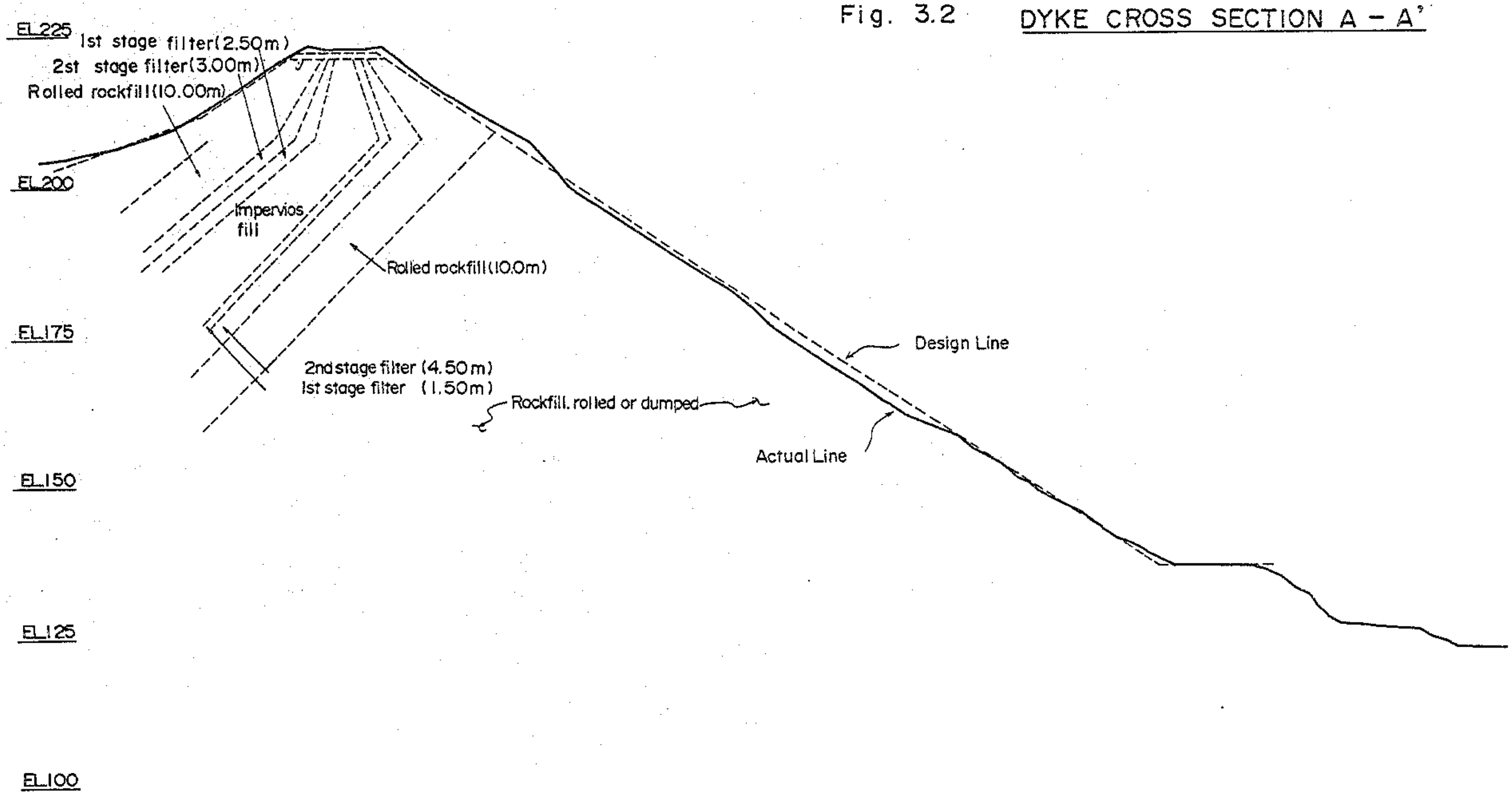
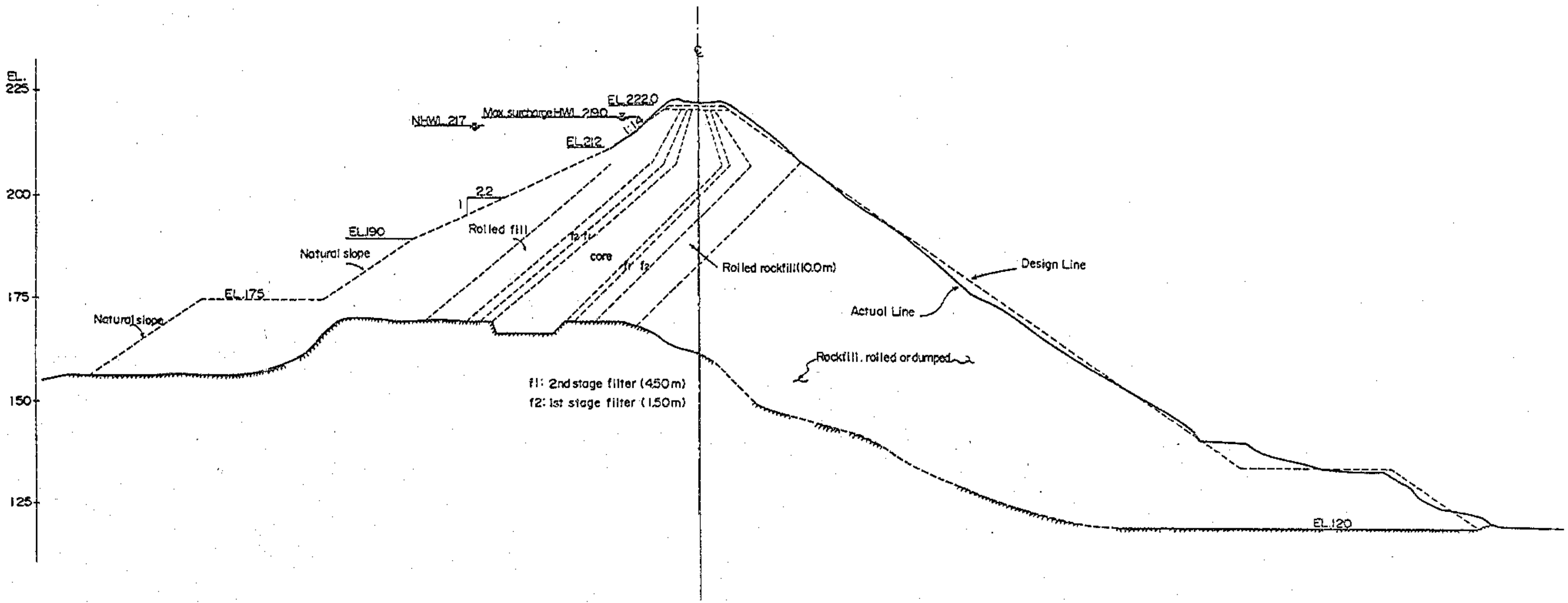


Fig. 3.3 Dyke Cross Section B - B'
(Maximum Section)



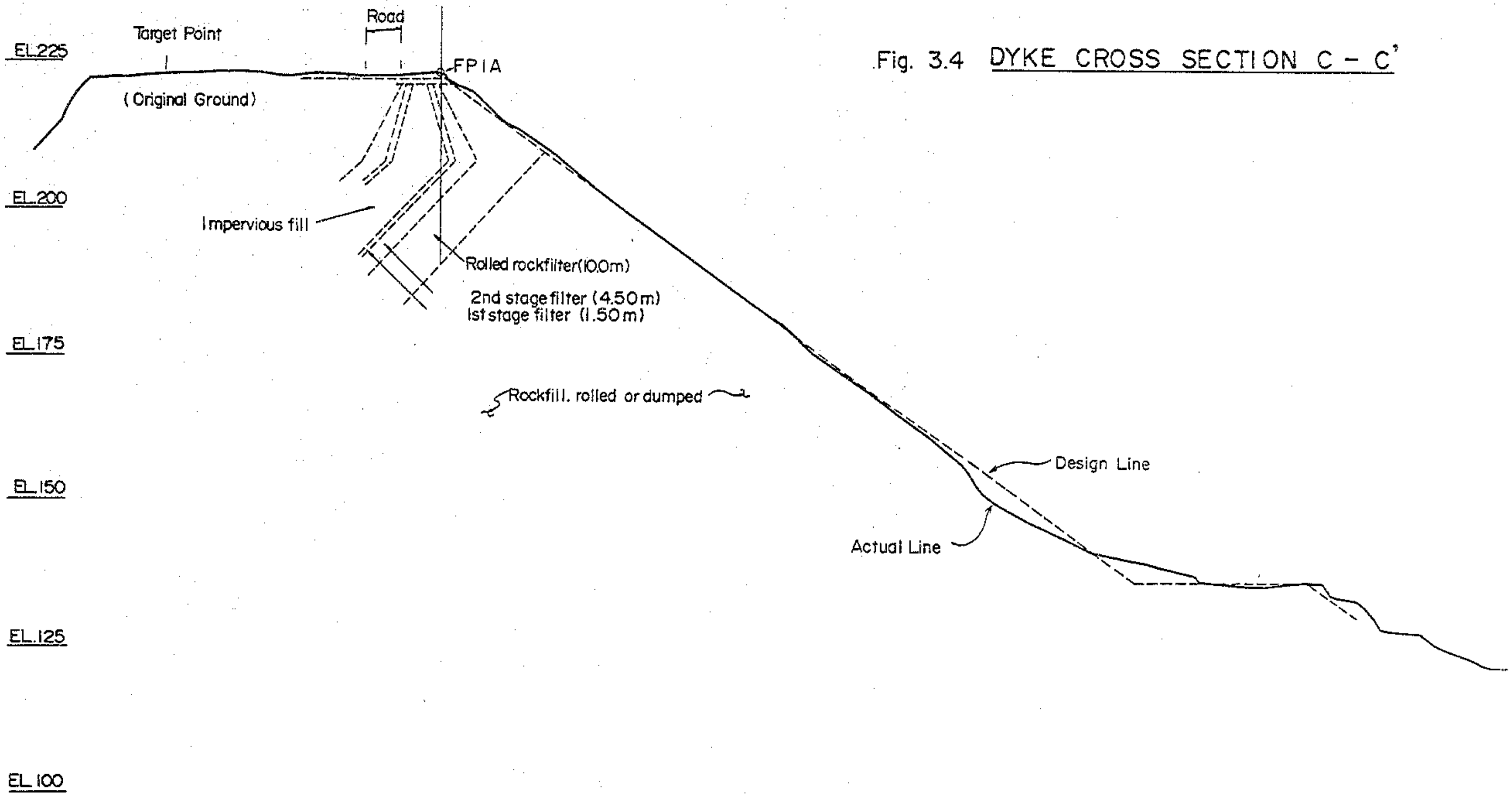


Fig. 3.4 DYKE CROSS SECTION C - C'

Given below is a comparison with the Ambuklao and the Binga dams of the earthquake activities estimated by the Okamoto's method using the historical records on the earthquake in the Philippines.

Unit: Gal

Return Period	Angat	Ambuklao	Binga
50	136	150	144
100	158	184	178
200	174	213	206
500	188	241	235
1000	195	257	251

As is seen in the above table, the earthquake acceleration at the Angat dam site in the past is less than that at the Ambuklao and Binga dam sites, and the Angat site seems to be better than the other two from the seismic statistical point of view. It should be noted that the number in the table shows the maximum amplitude of acceleration and may not necessarily be identified as the static, horizontal seismic coefficient (Kh) to be used for the design of dam structures. It appears, however, that there are no such defects as jeopardizing the safety of the dam, judging from the observation of outward appearances.

3.4. Stability Analysis

3.4.1. Outline of the Analysis

It is in no way possible to confirm how the stability analysis was made for the Angat dam at the design stage, because no relevant data are available. In making analysis of the stability for this Study, therefore, the methodology usually applied to fill type dams was used. The cross section for the analysis was taken from the design drawing, since it is almost identical with the measured cross section.

3.4.2. Input Variables for the Analysis

(1) Dam Cross Section

A typical cross section of the main dam and the dyke was chosen for the stability analysis. For the dyke, a section with the maximum height was selected. Those cross sections are as shown in Figs.3.1 and 3.3.

(2) Physical Properties of Embankment Materials

Since no data on the physical properties of the embankment materials are available, the analysis for this Study was made of the following variables based on the data used for the Ambuklao and Binga dams, and usually used for fill type dams:

Internal Friction Angle and Cohesion:

	Internal Friction Angle ϕ(°)	Cohesion C (ton/m^2)
Rockfill materials	43	0
	45	0
Filter materials	35	0
Core materials	20	
	25	
	30	

Unit Weight (ton/m^3):

	γd (Dry condition)	γt (Wet condition)	γsat (Saturated condition)
Rockfill materials	1.9	2.0	2.1
Filter materials	1.9	2.0	2.1
Core materials	1.8	1.9	2.0

(3) Loading Conditions

The loading conditions for the analysis were set as follows:

Dam Face	Water Level at	Earthquake Effect (Kh = 0.15 g)
Upstream	HWL	Without
Upstream	HWL	With
Upstream	LWL	With
Upstream	Abrupt drawdown from HWL to LWL	Without
Upstream	Abrupt drawdown from HWL to LWL	With
Downstream	HWL	Without
Downstream	HWL	With

3.4.3. Judgement Criterion for the Stability Analysis

In making the stability analysis, the safety factor against sliding under the above loading conditions was set, as the criterion for judgement, at 1.2 or greater without the earthquake effect, and 1.0 or greater with the earthquake effect.

The internal friction angles of rockfill, filter and core materials were assumed as shown in the preceding subsection. The cohesion strength of core materials with the internal friction angles of 20°, 25° and 30° was estimated so that it may satisfy the above safety factor.

The stability analysis was made by checking if the core materials should in actuality have the cohesion strength thus estimated.