No. 35

# THE GOVERNMENT OF THE REPUBLIC OF THE PHILIPPINES

# REPORT ON STUDY

# OF

# BINGA DAM REHABILITATION PROJECT

FEBRUARY 1989

JAPAN INTERNATIONAL COOPERATION AGENCY

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# P R E F A C E

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

JICA sent to the Philippines a study team headed by Mr. Motoyuki Doi, The New Japan Engineering Consultants, Inc. on four occasions during the period from September, 1987 to October, 1988.

The team held discussions with concerned officials of the Government of the Philippines and conducted field surveys at the Binga dam site. 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 concerned officials of the Government of the Philippines for their close cooperation extended to the team.

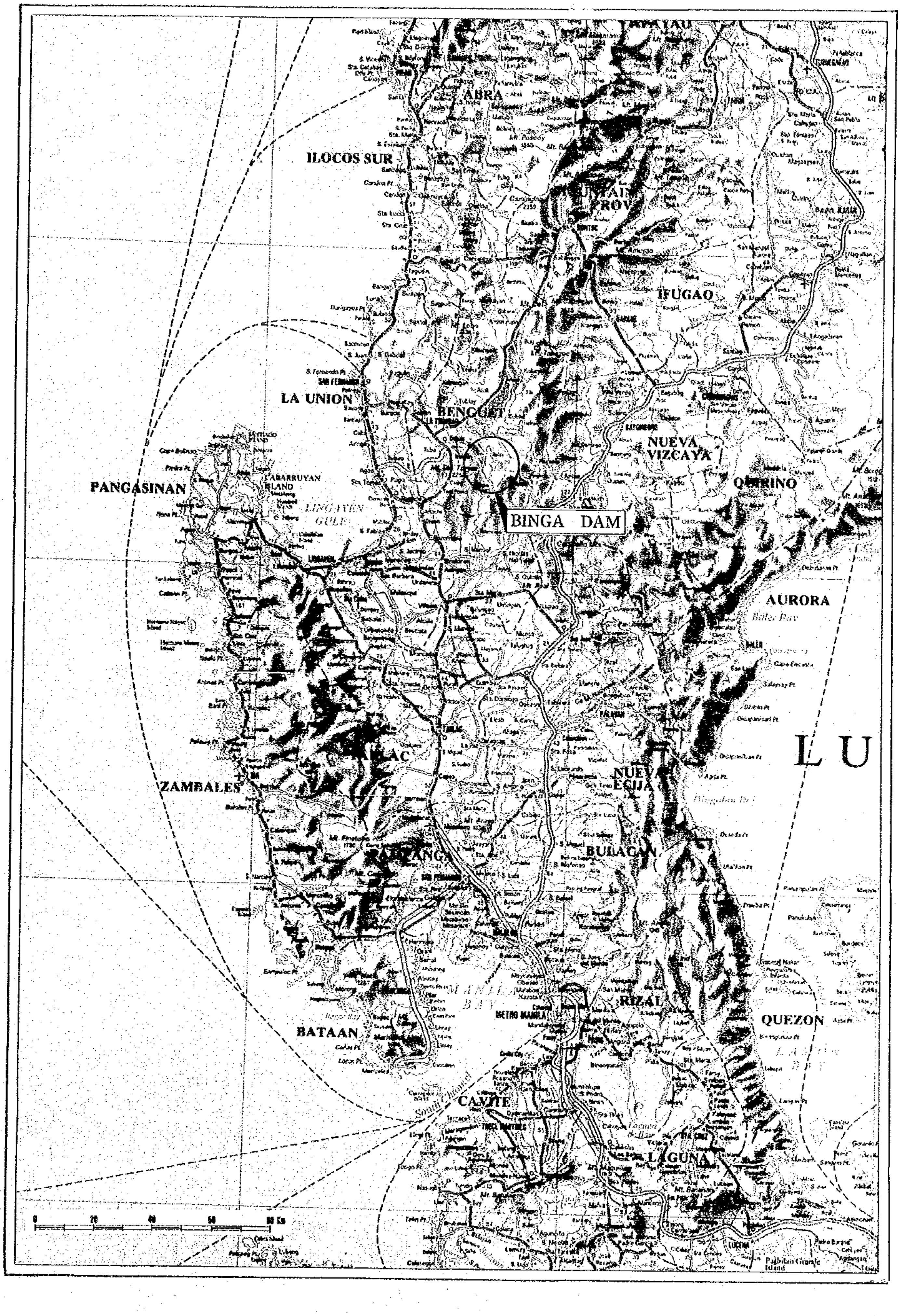
February, 1989

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation
Agency

LOCATION MAP, SITE PHOTOGRAPHS AND

GENERAL PLAN OF THE DAM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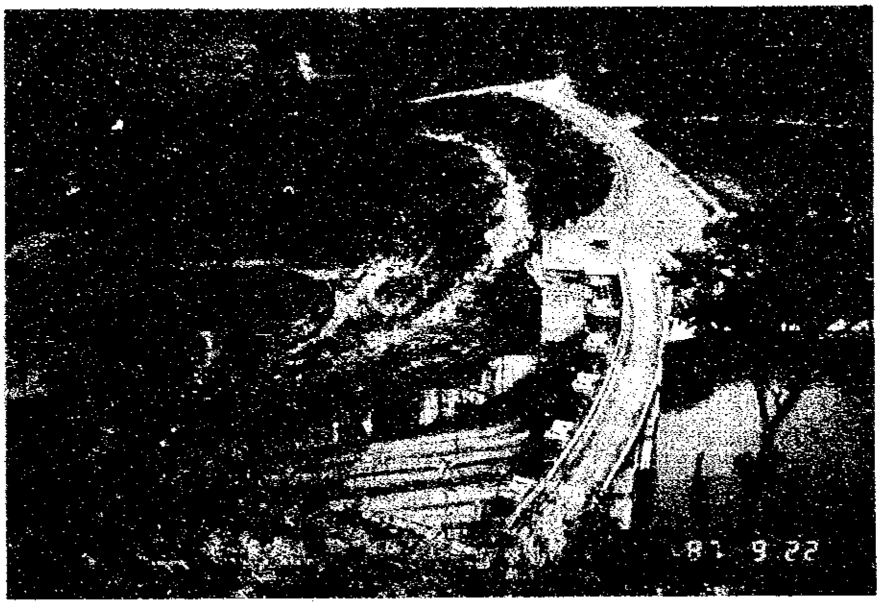
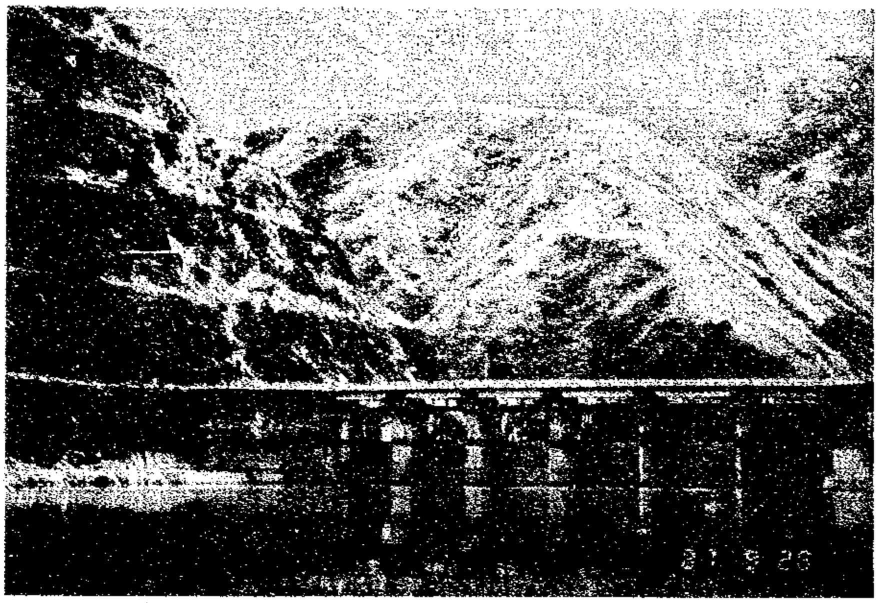
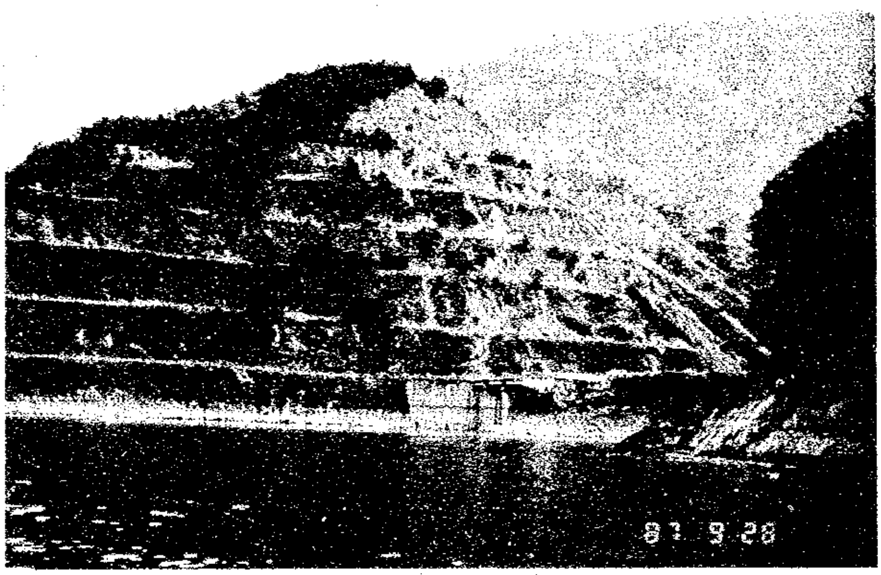



Binga Dam and Spillway

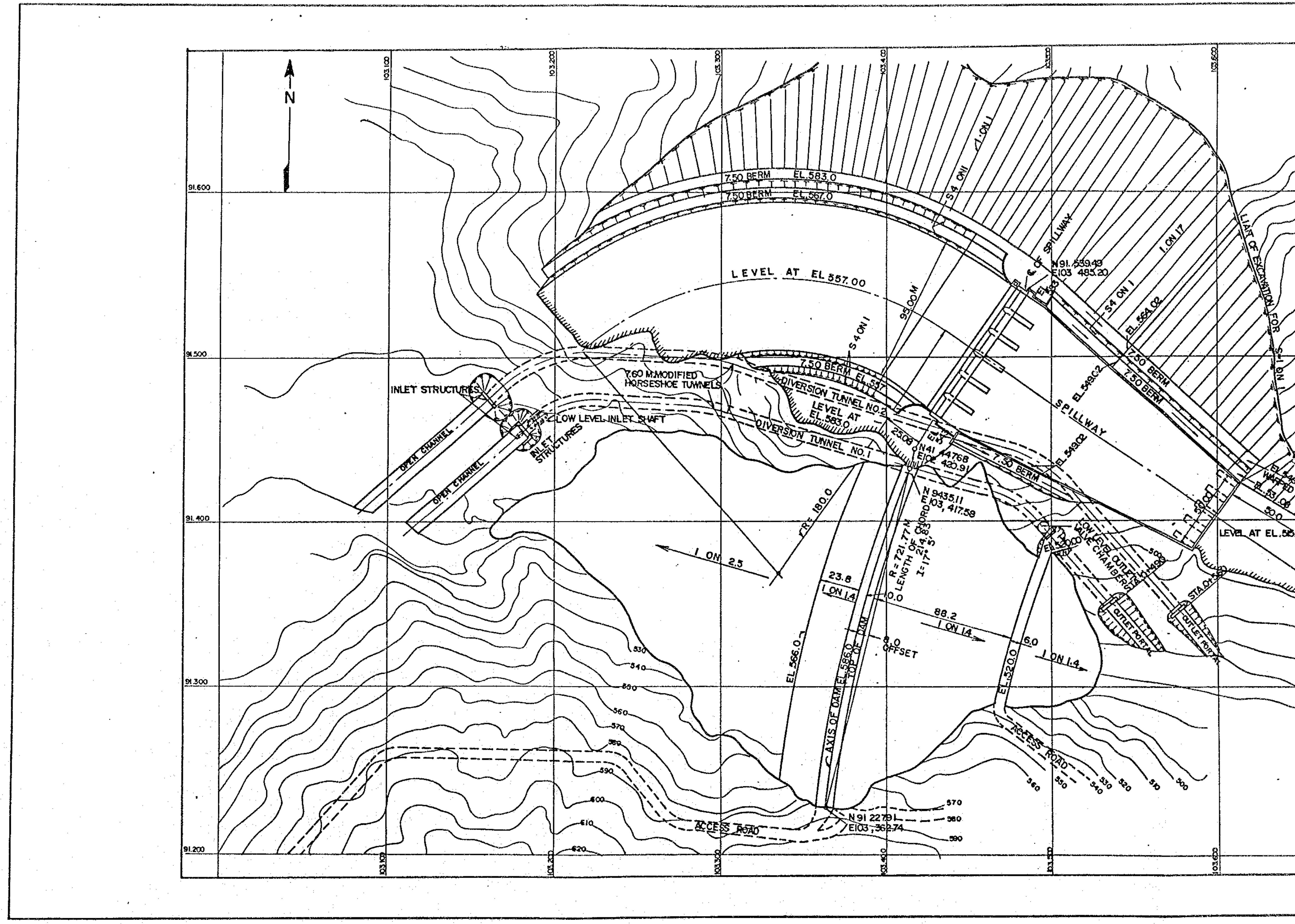
Dam Toe Protection Retaining Wall



Spillway



Left Bank Open-cut Excavation Slope



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LEVEL AT EL.557.00
7.50 BERM EL.583.0
7.50 BERM EL.567.0
SPILLWAY
INLET STRUCTURES
LOW LEVEL INLET SHAFT
7.60 M.MODIFIED HORSESHOE TUNNELS
DIVERSION TUNNEL NO.2
DIVERSION TUNNEL NO.1
OPEN CHANNEL
OPEN CHANNEL
LEVEL AT EL.583.0
1 ON 2.5
R=180.0
AXIS OF DAM EL.586.0
TOP OF DAM
EL.566.0
EL.520.0
LOW LEVEL OUTLET VALVE CHAMBER
OUTLET PORTAL
OUTLET PORTAL
ACCESS ROAD
ACCESS ROAD
N91 227.91
E103,362.74
N91 435.11
E103,417.59
N91 539.49
E103 485.20
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88.2
23.8
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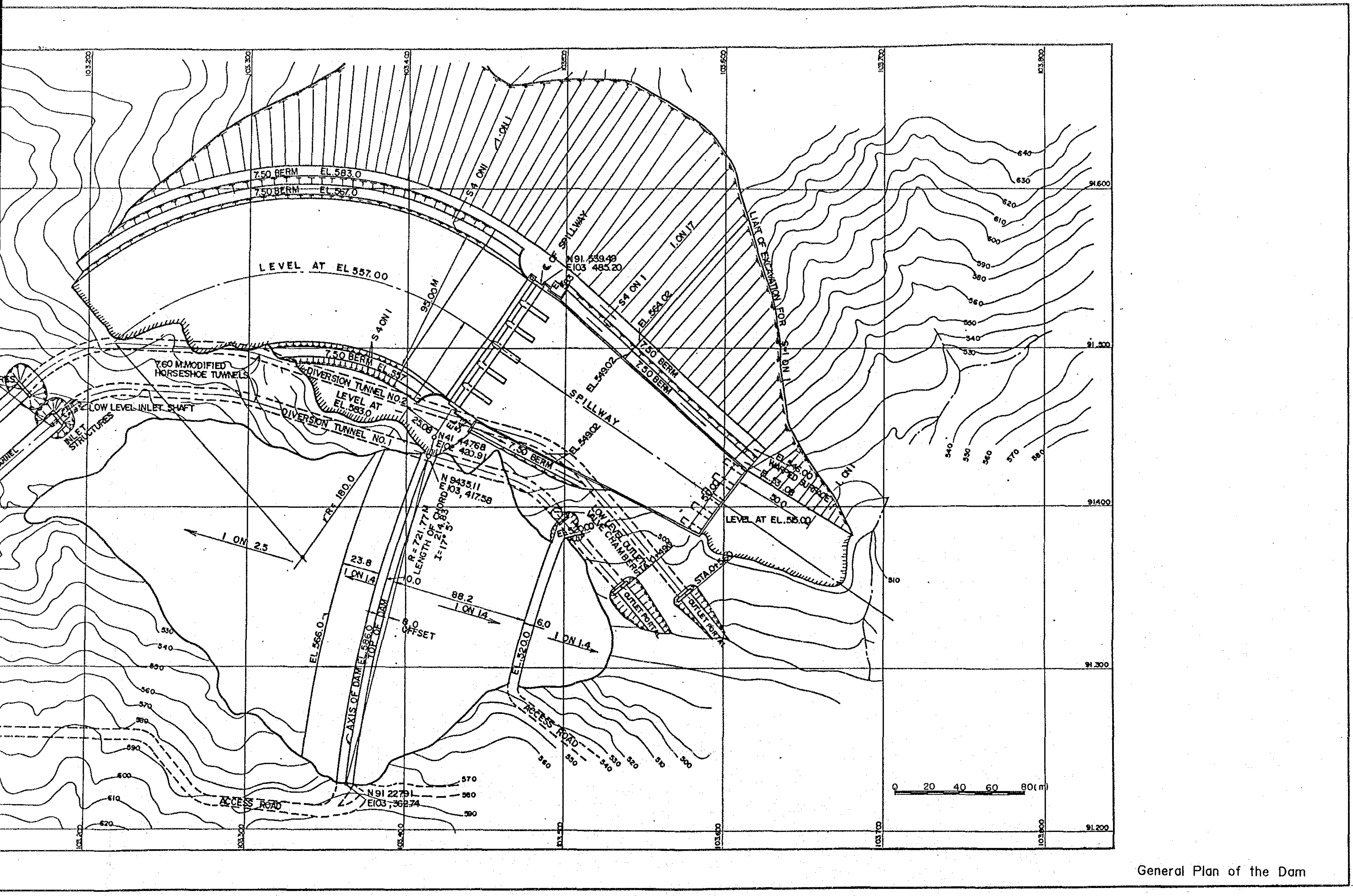

General Plan of the Dam

BINGA DAM REHABILITATION PROJECT

FINAL REPORT

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# 1. SUMMARY AND CONCLUSIONS

## 1. Summary and Conclusions

The major problems connected with the stability of the dam and proper operation of the Binga project, which were carefully studied and are presented in this Report, were with regard to all the aspects of the stability of the dam, stability of the left bank slope of the spillway open-cut excavation, and adequacy of the spillway capacity. In addition to the above, particular attention was also paid to the sedimentation in the upper reaches of the reservoir which comprises the Ambuklao powerplant tailrace, and the retaining wall at the toe of the dam badly damaged during past floods.

In order to obtain the relevant data required for the above studies, NAPOCOR carried out the following field surveys and investigations:

(1) Cross sectional survey of the dam embankment and the left bank excavated slope consisting of three traverse lines each.

(2) Planimetric survey of the plunge pool area including the retaining wall for protection of the toe of the dam.

(3) Drilling investigations at the above retaining wall consisting of three holes.

(4) Drilling investigations at the left abutment excavated slope (spillway open-cut excavation) comprising one hole.

The studies of this Report were all based on the results of the above surveys and investigations.

The following give a summary of each item of the studies:

## 1.1. Dam Stability

The Binga dam is an embankment rockfill dam with inclined core, and is 107.37 m high. The volume of the embankment is approximately 187.6 x $10^3 m^3$. The dam was completed in 1959. The typical cross section of the dam, as originally designed, is characterized by two slope gradients on the upstream face consisting of 1:1.4 between EL.586 m and EL.566 m and 1:2.5 below above elevation. A single gradient of 1:1.4 was used on the downstream face.

The survey of the cross sections of the dam performed recently revealed that a portion of the upstream slope extending from EL.557 m to EL.586 m had missed creating a new slope gradient of 1:1.30 at that location.

The stability analysis, performed as part of this Study, was done for the present section of the dam, using a methodology currently applicable to stability analysis of rockfill dams.

Properties of the dam embankment materials, such as shear strength, required for dam stability calculations were based on those available for the Ambuklao dam. The Ambuklao dam is a rockfill dam with a vertical central core constructed about during the same period (1956) and, as known, located on the same river immediately upstream of the Binga dam. Other relevant data from Ambuklao pertinent to this Study were also used. The Ambuklao dam materials data were used for this Study as the Design Report for the Binga Project was not available.

The Ambuklao data are given in the Final Report on the Ambuklao Dam Rehabilitation Project of March, 1988, by JICA. The zoning of the Binga dam, however, was based on the information provided on the original design drawings for the Binga project.

The stability analysis of the dam was carried out by the Slice Method using a circular sliding surface for both the upstream and downstream slopes of the dam. Normal reservoir operating conditions, with and without earthquake, were considered.

Based on the above analysis, it was found that the safety factor against sliding of the downstream slope of the dam was above 1.55 at the normal condition and above 1.15 at the earthquake. However, the safety factor of the upper portion of the upstream slope (above EL.557.0 meters) at the earthquake was 0.81 although it was over 1.25 at the normal condition. This was not surprising as the upstream slope above EL.557.0 m became steeper, i.e. 1 to 1.30 as compared to 1 to 1.4 of the original design.

It is therefore proposed, as part of the rehabilitation plan, to rebuild the embankment on the upstream face of the dam so that the new slope becomes 1 to 2.23.

This will raise the safety factor of the upstream slope above 1.0 even for the case when the earthquake is considered. This measure will make an effective contribution to the safety of the dam and general stability of the dam.

## 1.2. Left Bank Open-cut Excavation

The spillway open-cut excavation in the left abutment has an excavated slope about 130 meters high and about 500 meters long. Its average slope is 60°. It comprises seven berms, each 8.5 meters wide by 20 meters high. The slope consists of metamorphic and sedimentary rocks, and small dykes intruding here and there. The rocks are hard and massive but the slope surface itself is weathered and brittle.

Based on the data from the geologic investigations, and observations made in the field, the excavated slope was analyzed for stability against sliding. The analysis was performed for a surveyed cross section of the excavation.

The analysis indicated that the slope is safe against sliding as the corresponding safety factors for each section analyzed were above 2.3. It is, therefore, considered that the excavated slope in its present conditions is stable and quite safe against sliding. However, fractured zones on the slope downstream of the dam axis, around which cracks are developing, have been observed. Such zones, scoured by rain water, gradually deteriorate into a gully structure. Due to this phenomenon, some berms have collapsed over their full width and have, thus, become impassable.

In view of the above, it is considered that the rehabilitation plan for the excavated slope should not apply to the whole slope. It should principally consist of removal of the debris accumulated on the berms, and slope protection against erosion and scouring by rainfall for the vulnerable parts of the slope including the fractured zones.

## 1.3. Spillway and Plunge Pool

The spillway of the Binga dam is a chute type gated spillway which is provided with six radial (tainter) gates. The gates are 12.5 m wide by 12 m high.

An analysis was made of the probable inflow to the Binga reservoir, consisting of the discharge from the Ambuklao reservoir and the runoff from the Binga dam reservoir drainage area (excluding Ambuklao dam drainage area).

Applied as the input variable for the former was the maximum discharge through the spillway which could be produced by the gate operation procedure as studied in the Report on the Study of the Ambuklao Dam Rehabilitation Project. Applied as the input variable for the latter was the runoff determined on the basis of the maximum amount of rainfall in the Binga dam reservoir drainage area by using the runoff function method.

Based on the above analysis, it was confirmed that the Binga dam with its crest at EL.586 m has a sufficient freeboard (4.46 m) against the rise of the reservoir water level due to the probable outflow of 11,050 $m^3$/sec. This outflow corresponds to the flood with a 200-year return period multiplied by 1.20 (20 percent increase in magnitude), which is normally used in Japan for determining the design flood discharge for rockfill dams. The above freeboard includes the effect of wave due to winds and earthquakes (2.2 m).

## 1.4. Reservoir Sedimentation

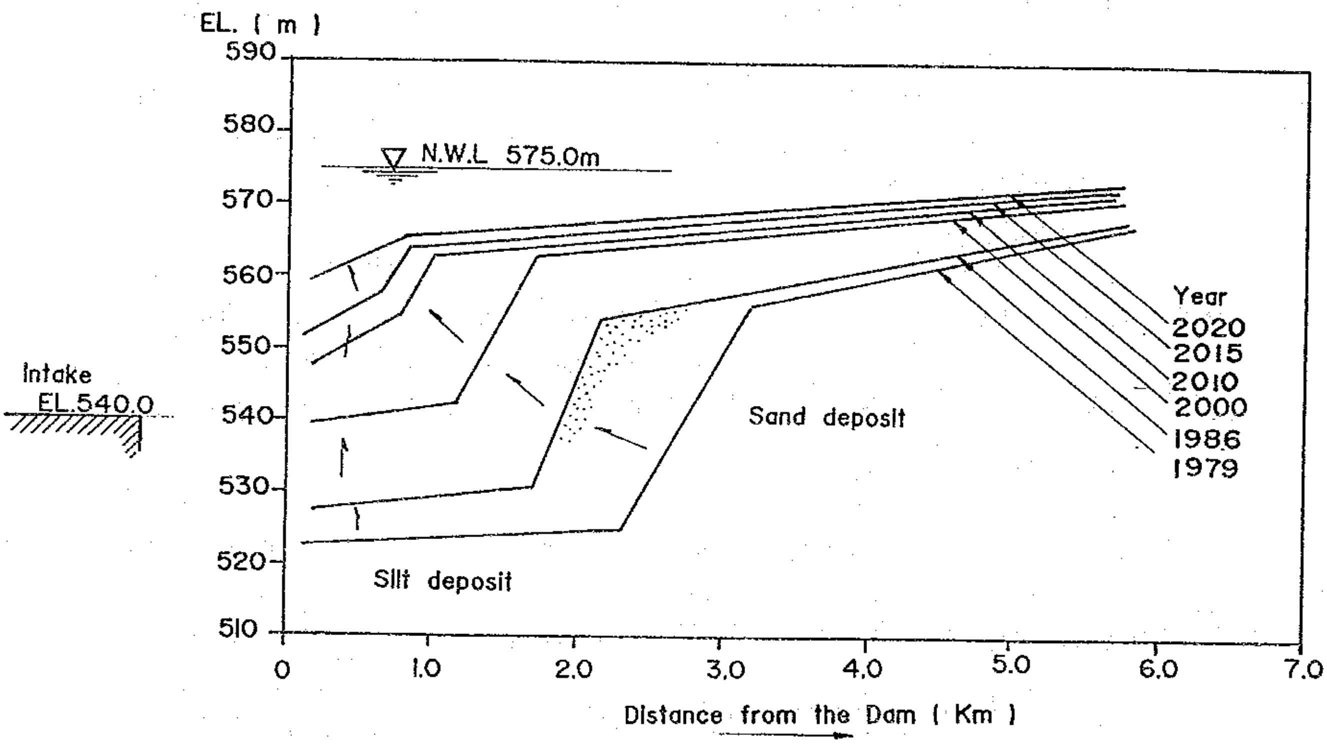
The Binga dam reservoir storage capacity has been gradually decreasing through years due to sediment inflows from the reservoir drainage area which started with reservoir impounding.

Since 1960 until 1986, the gross storage capacity of 87.4 x $10^6 m^3$ has decreased to 56.1 x $10^6 m^3$, resulting in a total decrease of 31.3 x $10^6 m^3$. During the same period, the effective storage capacity of 45.9 x $10^6 m^3$ was reduced to 36.2 x $10^6 m^3$, making it smaller by 9.7 x $10^6 m^3$.

This amount of storage reduction is equivalent to the sediment inflow of 4,900 $m^3$ per year per square kilometer (annual yield rate) of the total reservoir drainage area of 246 $km^2$. The corresponding annual sediment yield rate for the Ambuklao dam is 5,340 $m^3$/year/$km^2$ for a total reservoir drainage area of 690 $km^2$, and for the Angat dam, 5,720 $m^3$/year/$km^2$, with a drainage area of 568 $km^2$.

Section 10 of this Report deals with the anticipated progress of the sedimentation in the reservoir. The prediction of the progress was made by using a simulation model developed for this purpose for a period starting in 1986. For this study, the latest cross sections of the reservoir available from the recent depth survey were used.

As a result of the simulation study, it is concluded that the year in which the level of the silt sediments would reach the level of the power intake, EL.540 m, would be 2000 (11 years ahead), and the year in which sands and gravels start entering the power intake, would be 2015 (26 years ahead). This is briefly shown below:




Shown in the following table are the results of calculations to know a decrease in the reservoir storage capacity according to the yearly progress of sedimentation, and also a variation of the annual generated energy affected thereby. In this table, the decrease in the annual generated energy is generally to an unnoticeable level within 1.0%, and thus, can be ignored.

Progress of Sedimentation and Annual Generated Energy

| Year | Effective Storage Capacity ($10^6 m^3$) | Ratio (%) | Annual Generated Energy (GWh) | Ratio (%) |
|---|---|---|---|---|
| 1986 | 39.0 | 100 | 481 | 100 |
| 1990 | 37.0 | 95 | 481 | 100 |
| 2000 | 32.0 | 82 | 480 | 99.8 |
| 2010 | 26.5 | 68 | 478 | 99.4 |

A forecast of the rise of the level of the sediments at the outlet of the Ambuklao tailrace tunnel, located at the upstream end of the Binga reservoir, was also made on the basis of the results of the reservoir simulation model studies.

As a result, it is anticipated that the sediments at the tailrace outlet would increase by about 7 m during the period from 1986 to 2020.

The effect of the increase of the level of the sediments in this area on the Ambuklao power plant operation would be loss of power production and obstruction of the operating procedures. The operating obstructions would result from the fluctuation of the water level in the surge tank, particularly from upsurge, due to sudden load increase.

A study of the effect on the water levels in the surge tank which a sudden load increase could have for different levels of the sediments at the tailrace outlet, indicates that it is possible to limit the maximum water level rise in the surge tank to a level lower than EL.604 m (thereby preventing water inflow into the valve chamber), by increasing the turbine gate opening time to more than 90 seconds. The above study was based on the tailrace sediment levels anticipated to occur in the year 2020.

## 1.5. Proposed Rehabilitation Program

The results of the field surveys and the field investigations, and relevant studies and analyses conducted thereafter revealed that the following three rehabilitation schemes are necessary to be put into effect for the existing Binga dam:

- Dam Upstream Face Rehabilitation Scheme
- Dam Toe Protection Retaining Wall Rehabilitation Scheme
- Left Bank Excavated Slope Rehabilitation Scheme

The outline of the above each scheme is described as follows:

### 1) Dam Upstream Face Rehabilitation Scheme

In order to secure the stability of the dam at the time of earthquakes, the slope gradient of the upper part of the dam upstream face between EL.586 m (dam crest) and EL.557 m should be made gentler from the present 1:1.3 to 1:2.23 by filling rocks thereat and subsequent riprapping to prevent damages caused by the wind and wave actions.

The quarry site of rock materials is proposed at the ex-quarry site on the left bank of the dam. Rock materials are filled on the slope to a specified thickness in the manner that they are placed uniformly and compacted in turn by a bulldozer. Riprapping is performed subsequent to the filling of rock materials for the purpose of finishing the external surface of the filled materials. The volume of the filled rock materials is estimated at 50,500 $m^3$, and the entire works is scheduled for completion in six months for a dry season. During this period, the reservoir water level should be kept below EL.555 m (L.W.L.) to avoid the underwater construction work.

2) <u>Dam Toe Protection Retaining Wall Rehabilitation Scheme</u>

In replacement of the existing dam toe protection retaining wall on the right bank side of the plunge pool of the spillway which wall has been remained seriously damaged, the rock dyke is proposed at the same place aiming at more effective protection of the dam downstream toe against scouring. The proposed rock dyke is of 107 m long and 65 m wide with its slope range of 1:5.5 to 1:2.0, being formed by filling of rock materials and subsequent riprapping to finish the surface of the filled materials. The volume of the filled rock materials is estimated at 14,800 $m^3$. The same quarry site and the same method of filling and riprapping as used for the dam upstream face rehabilitation work are proposed for this work. The entire work is scheduled for completion in four months mainly for a dry season during which the dam effects little discharge.

3) <u>Left Bank Excavated Slope Rehabilitation Scheme</u>

This scheme is the slope protection work, consisting of removing debris accumulated on the berm of the slope and thereafter shotcreting for the failure portions and vulnerable parts of the excavated slope. There are fifteen portions requiring shotcreting with a total area of 13,000 $m^2$. The shotcreting is performed from the higher portion downward. The entire work is scheduled for completion in fourteen months.

The above three schemes should be put into effect and completed within a total construction period of 39 months in the sequence of the Scheme 1) completed in the first one year, Scheme 2) in the next one year and Scheme 3) in the rest of the period.

The total project cost is estimated to be US$3.70 x $10^6$ comprising the direct cost of US$2.75 x $10^6$ and the other cost of US$9.50 x $10^6$, breakdown of which is shown below:

Breakdown of Project Cost

(unit: US$x$10^3$)

| | | | |
|---|---|---|---|
| 1. | Cost of Rehabilitation Works | | |
| | 1.1. | Dam Upstream Face Rehabilitation Work | 1,310 |
| | 1.2. | Dam Toe Protection Retaining Wall Rehabilitation Work | 481 |
| | 1.3. | Left Bank Excavated Slope Rehabilitation Work | 959 |
| | | a) Removal of Debris | (61) |
| | | b) Shotcreting | (898) |
| | | Sub-total (1) | 2,750 |
| 2. | Cost of Investigation | | 170 |
| 3. | Engineering | | 165(6%) |
| 4. | NAPOCOR Administration | | 60(2%) |
| 5. | Contingency | | 555(20%) |
| | | Total (1-5) | 3,700 |

1.6. Economic Analysis

Since it is estimated to take six years to complete the Binga dam rehabilitation project, if the investigation and design be commenced from 1990, its completion is expected in 1995. The total project cost is estimated to be US$3,700 x $10^3$. During the total construction period of about four years, it is inevitably required to lower the reservoir water level below L.W.L. for a period of six months while the filling operation is proceeding on the dam upstream face. Due to the above, the power generation is reduced by 17.9 GWh for the same period. This reduced energy shall be valued in terms of the variable cost of the existing oil-fired thermal plant in the Grid, and then such valued cost shall be put together with the estimated project cost, subject to conversion of that total into the present worth as of 1995; the completion year. The present worth so obtained makes US$5,381 x $10^3$(C1).

Furthermore, since the effects of the proposed rehabilitation works are expected to last long after the completion, the above valued cost (C1) can be levelized over a period of 15 years, or the rest of the plant life, from 1996 to 2010, thereby indicating the annual levelized cost of US$876 x $10^3$(C2). On the other hand, the Binga plant value is appraised in terms of the plant capacity (MW) and the annual generated energy (GWh) with regard to the most economic alternative power source envisaged in the NAPOCOR's Power Development Program June 1988. After the comparative study, the gas turbine plant was chosen for an alternative power source to replace the Binga plant in the grid system, then the Binga plant value was appraised to be 24,145 x $10^3$ US$/year (A).

The expected annual benefit (B) stemmed from enhancement of safety factors of the dam upstream face slope (probability of the dam failure reduced from 1/79 to 1/400) is represented by $\alpha$ x A, where $\alpha$ is a function with regard to probability of the dam failure before and after the rehabilitation work (termed as P1 and P2 respectively), the rest of the dam life time (L years) and a discount rate (i). From the above equation, the expected annual benefit of 1,450 x $10^3$ US$ was obtained. This value (B) is apparently larger than the rehabilitation cost (C2) of US$876 x $10^3$, showing B/C2 = 1.66.

Consequently, the proposed rehabilitation project can be justified from the economic point of view.

Aside from the above, the rehabilitation cost accounts for 3.6% of the Binga plant value (A).

## 1.7. Dam Safety Control System

A Civil Structures Monitoring Section (tentatively named, and at the same level of responsibility as the present Mechanical and

Electrical Sections) is to be established within the Central Maintenance and Technical Services Department (CMTSD) of Northern Luzon Regional Center (NLRC). The proposed staffing should be composed of two civil engineers and three assistants. This section will be responsible for maintenance, inspection, repair works, surveillance and monitoring of the Binga Dam and its auxiliary structures. In addition, this proposed section is also responsible for maintenance and inspection of the civil structures of all the hydro power projects of NLRC, such as Ambuklao and Angat Powerstations.

It is further proposed that a Hydro-Civil Maintenance Division (tentatively named and at the same level of responsibility as the Project Design Division) within the Hydro Power Project Department, Engineering (Main), of the NAPOCOR Head Office in Manila, be established. Its staff should consist of several civil engineers with functions and assignments for the overall management of inspection, maintenance and repair works of civil structures. Coordination with the Civil Structures Monitoring Section of the CMTSD of the Regional Center, and planning and analyses of monitoring data compiled by the Technical Services Department from the civil engineering view point, will also be the responsibility of this division.

It is emphasized that one civil engineer should be permanently assigned to the Binga Powerstation for the purposes of daily dam inspection, monitoring and maintenance of civil structures.

Table 1.1 Tentative Work Schedule of Binga Dam Rehabilitation Project

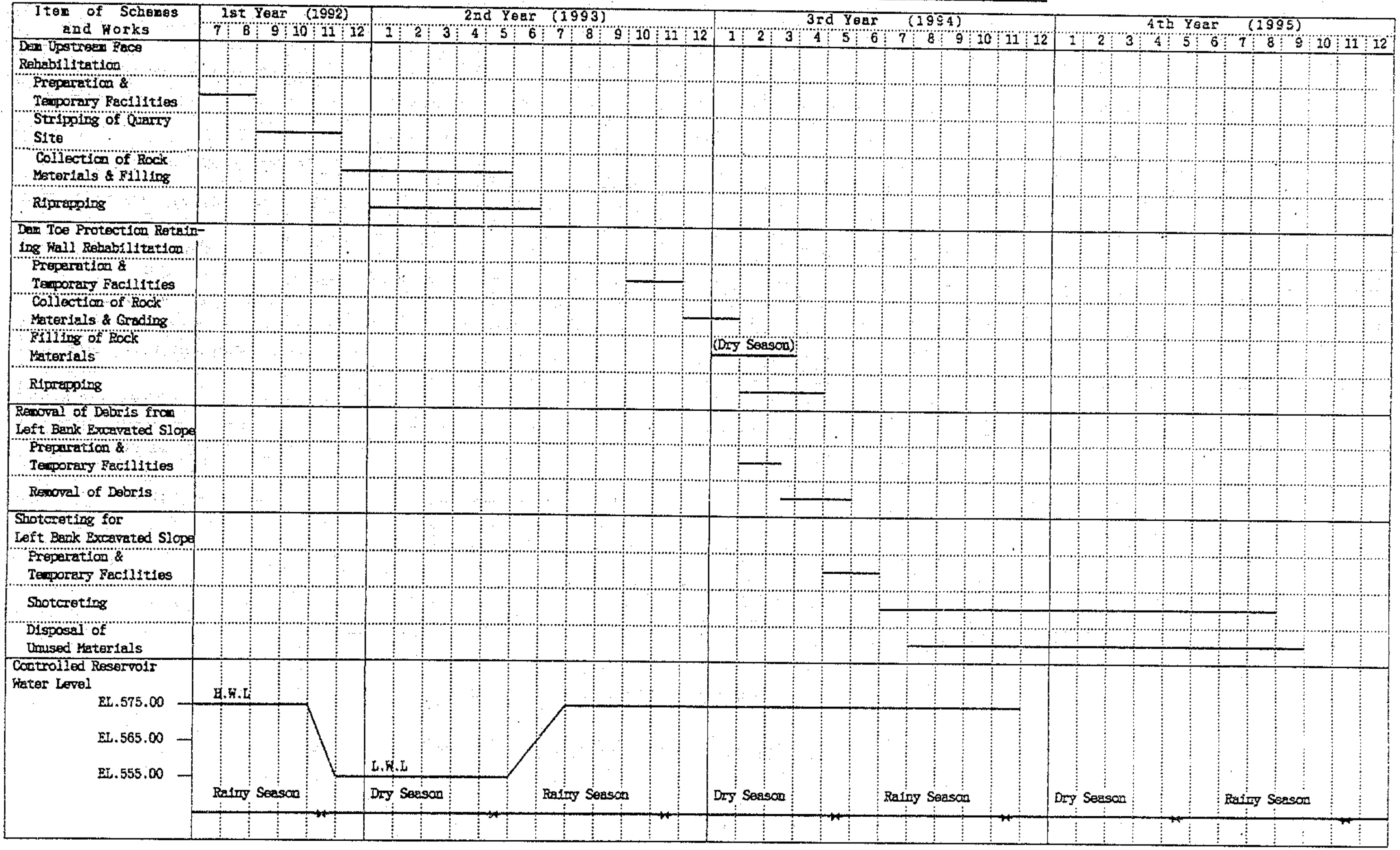


Table 1.2 Tentative Implementation Schedule of Binga Dam Rehabilitation Project

| Item | 1988 | 1989 | 1990 | 1991 | 1992 | 1993 | 1994 | 1995 |
|---|---|---|---|---|---|---|---|---|
| | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 | 2 4 6 8 10 12 |
| Feasibility study | | | | | | | | |
| Application to NEDA for Loan | | | | | | | | |
| Loan Agreement | | | | | | | | |
| Further Investigations | | | | | | | | |
| Design & Prequalification | | | | | | | | |
| Bidding & contract | | | | | | | | |
| Dam Upstream Face Rehabilitation | | | | | | | | |
| Dam Toe Protection Retaining Wall Rehabilitation | | | | | | | | |
| Left Bank Excavated Slope Rehabilitation | | | | | | | | |

# 2. OUTLINE OF THE BINGA DAM AND SCOPE OF THE STUDY

# 2. Outline of the Binga Dam and Scope of the Study

## 2.1. Outline of the Binga Dam

The Binga dam and its associated facilities are lying about 31 km east of Baguio City located some 180 km north-north west of Manila City in Luzon island, the Republic of the Philippines. The Binga dam is on the Agno River where another dam, the Ambuklao dam was constructed just upstream of the end of the Binga reservoir.

The Binga dam is a rockfill dam with an inclined core, featured by its height of 107.34 m, crest length of 215 m, the gross reservoir storage capacity of $87.4 \times 10^6 m^3$, the effective storage capacity of $48.2 \times 10^6 m^3$ and the total drainage area of 936 $km^2$.

The Binga hydro power plant is being operated at the maximum capacity of 100 MW and its generated power together with the Ambuklao Power Plant having a capacity of 75 MW is transmitted to Manila on the 230 kV transmission line facilities to cope up with the peak load of the Luzon Grid. Thus, the Binga power plant is considered as one of the principal power sources supplying the peak load of the Grid.

Subsequent to commissioning of the Ambuklao plant in 1956, the Binga plant was put into commercial operation in May, 1960.

Main features of the Binga plant are tabulated in the following Table 2.1.

Table 2.1

Main Features of the Binga Power Plant

| | | |
|---|---|---|
| Location | : | Barrio Binga, Municipality of Itogan, Benguet Province |
| Name of the River | : | Agno River |

| | | |
|---|---|---|
| Date of Operation | : | May, 1960 |
| Installed Capacity | : | 100 MW |
| Energy Capability | : | 436 GWh (Mean value during 1977 to 1986) |
| Drainage Area at Dam Site | : | 936 $km^2$ |
| Reservoir | : | Gross Storage Capacity ; $87.4 \times 10^6 m^3$ |
| | | Effective Storage Capacity ; $48.2 \times 10^6 m^3$ |
| | | Maximum Reservoir Level ; EL.579.5 m |
| | | Normal Maximum Reservoir Level ; EL.575.0 m |
| | | Minimum Reservoir Level ; EL.555.0 m |
| Dam | : | Type of Dam ; Rockfill dam with an inclined center core |
| | | Height ; 107.37 m |
| | | Crest Elevation ; EL.586.0 m |
| | | Crest Length ; 215.0 m |
| | | Crest Width ; 10.0 m |
| | | Base Width at Maximum Section ; 400 m |
| | | Volume of Embankment; $1.876 \times 10^6 m^3$ |
| Spillway | : | Type ; Chute type with gates |
| | | Length ; 94.5 m |
| | | Elevation of Crest of Overflow; EL.563.0 m |
| | | Gates ; 6 tainter gates (W12.5 m x H12.0 m) |
| | | Elevation of Top Gates; EL.575.0 m |
| | | Design Flood ; 5,200 $m^3$/sec. |
| Intake | : | Elevation of Invert of Intake Tower at Inlet ; EL.540 m |

| | | |
|---|---|---|
| Headrace Tunnel | Inside Diameter | 5.6 m |
| | Length | 760 m |
| | Number | 1 |
| | Elevation of Invert at Inlet | EL.540 m |
| Surge Tank | Type | Differential type |
| | Height | 86.0 m |
| | Diameter | 15.0 m (EL.596 m - 547 m)<br>5.8 m (EL.547 m - 510 m) |
| Penstocks | 4 lines (Diameter 3.66 m to 2.44 m) | |
| Powerhouse | Type | Underground type |
| | Dimensions | L77.3 m x W14.3 m x H24.4 m |
| | Floor Elevation of Generator Hall | EL.423.0 m |
| | Elevation of Center Line of Turbines | EL.416.5 m |
| Tailrace Tunnel | Inside Diameter | 6.0 m |
| | Length | 2,000 m |
| | Number | 1 |
| | Elevation of Invert at Outlet | EL.410.5 m |
| Turbine | Type | Vertical shaft, Francis type |
| | Rated Capacity | 25 MW x 4 units |
| | Rated Head | 156 m |
| | Rated Revolutions | 327.3 rpm |
| | Make | Riva Milano (Italy) |
| Generators | Capacity | 27.8 MVA x 4 units |
| | Nameplate Capacity | 109,000 kW |
| | Power Factor | 0.9 |
| | Frequency | 60 Hz |
| | Make | Oerlikon (Switzerland) |

| | | |
|---|---|---|
| Designed by | : | NAPOCOR, Engineering & Development Corporation of the Philippines and Tippettes - Abbett McCarthy-Stration of New York |
| Constructed by | : | Philippine Engineers Syndicate, Inc. |

## 2.2. Background and Scope of the Study

In 29 years after the completion, the Binga dam has ever suffered great damages on the retaining wall at the toe of the dam downstream slope, erosion of concrete at the end of the spillway structure, collapses on the left bank spillway open-cut excavation slope and so forth. These damages and/or failure were considered so seriously in terms of the dam safety control that the Binga dam rehabilitation study might be proposed for the early implementation.

The preliminary investigations were initiated by NAPOCOR to look into the above problems, and thereafter NAPOCOR submitted to the Embassy of Japan in the Philippines on 3rd of April, 1985 an official request regarding the implementation of the study on the Binga dam rehabilitation plan, and improvement of the relevant safety control system. The request included the same scope of works for the other two dams, the Ambuklao and the Angat. Subsequently, the Government of the Philippines requested formally the Government of Japan for implementation of the study for the above three dams through its official letter No.602 dated April, 19, 1985. Upon receipt of such request, The Japan International Cooperation Agency (JICA), an executing agency to conduct the study, had recognized the importance of the request and dispatched the JICA mission to the Philippines in February, 1987 to confirm the background of the request and discuss the proposed scope of the study with local people concerned. As a result, JICA mission and NAPOCOR concluded the Implementing Arrangement, based on which the Binga dam rehabilitation study was to be conducted. The scope of the study agreed in relation to the Binga dam was as follows:

1) Review of the previous studies, surveys and all existing data and materials relevant to the Study.

2) Review of the present monitoring system.

3) Field inspection of the dam, spillway, plunge pool, retaining wall at the toe of the dam downstream slope and sedimentation in the reservoir and upstream reaches of the reservoir.

4) Formulation of the suitable monitoring system for the dam.

5) Programming of additional field works consisting of topographic surveys, drilling, test pitting and measurements of water leakage and deformation of the dam, preparation of specifications and supervision/guidance for the additional field works.

6) Preparation of manuals for measurement of water leakage, deformation of the dam and earthquake, and supervision/ guidance for monitoring works.

7) Stability and/or safety analyses for the dam, left bank spillway open-cut excavation slope, spillway capacity and the retaining wall at the toe of the dam downstream slope.

8) Study for reduction of sedimentation at the upstream reaches of the reservoir.

9) Formulation of the rehabilitation plan and its economic evaluation.

10) Review of the present organization chart of the dam control system and proposal for the revised chart, if required.

11) Formulation of the standards of safety control system for the dam.

# 3. FIELD SURVEYS FOR THE STUDY

## 3. Field Surveys for the Study

The field surveys for the Study was conducted by the JICA study team for one month from 17th of September, 1987 to 16th of October, 1987. Names, assignments and periods of stay in the Philippines of each expert were as follows:

| Name | Assignment | Period of Stay |
|---|---|---|
| Mr. Motoyuki Doi | Team Leader | Sept. 17, 1987 to Oct. 16, 1987 |
| Mr. Masatoki Ikeda | Civil Engineer (Design, Construction and Dam Control System) | ditto |
| Mr. Rokuro Kobayashi | Civil Engineer (Hydrology and Hydraulic) | ditto |
| Mr. Tamotsu Fujiwara | Civil Engineer (Additional Field Work and Measurement) | Oct. 1, 1987 to Dec. 28, 1987 |
| Mr. Osami Endoh | Geology | Sept. 17, 1987 to Oct. 16, 1987 |

The main items of the field surveys conducted by the JICA study team were as below:

(1) Submission to NAPOCOR of the Inception Report and explanation and discussion to the concurrence by NAPOCOR.

(2) Collection and analysis of data and information relevant to the Binga dam:

- Design drawings of civil structures
- Hydrological data
- Records on the power generation and the reservoir operation
- Previous reports related to the Binga dam

(3) Inspection of damages and/or failure appeared on the left bank spillway open-cut excavation slope and on the retaining wall at the toe of the dam downstream slope.

(4) Inspection of sedimentation at the upstream reaches of the reservoir, and measurement of grain size distribution of the riverbed sediment materials.

(5) Programming and preparation of specifications for the additional field works.

(6) Guidance and supervision for the additional investigation works.

(7) Guidance on installation of the monitoring equipment and preparation of the measurement manual.

(8) Guidance and supervision for monitoring works.

At the end of the field surveys for one month, the JICA study team prepared the "Report on Field Study for the Binga Dam Rehabilitation Project" dated October 16, 1987 and "Specification of the Additional Investigation Works for the Binga Dam Rehabilitation Project" dated October 15, 1987, and submitted them to NAPOCOR for his perusal of the survey result and further arrangements for the field investigation works.

From among the above items of the field surveys, Items (6), (7) and (8) were to be carried out by NAPOCOR under guidance and supervision of the JICA expert. Hence Mr. Fujiwara in charge thereof remained at the project site to render those services through 28th December, 1987. During his stay at the site, he prepared and submitted to NAPOCOR the "Measurement Manual for the Binga Dam Rehabilitation Project" dated December, 1987. Thereafter, the same expert visited the Project site three times in March, June and October, 1988 to collect and analyze the monitoring data as well as to render the services for guidance and supervision on the NAPOCOR's monitoring works.

# 4. ADDITIONAL INVESTIGATION WORKS

## 4. Additional Investigation Works

To study the stability of the dam embankment and the left bank spillway open-cut excavation slope, to formulate the rehabilitation plan of the retaining wall at the toe of the dam downstream slope, and to obtain the necessary data for hydraulic analysis on sedimentation in the reservoir, NAPOCOR conducted the following field investigation works in accordance with the technical specifications prepared by the JICA study team.

### (1) Surveys

- Cross-sectional survey of the dam
  3 measurement lines x 350 m    S = 1/200

- Cross-sectional survey of the left bank open-cut excavation slope
  3 measurement lines x 250 m    S = 1/200

- Cross-sectional survey of the river course downstream of the dam
  11 measurement lines    S = 1/200

- Topographic survey of the area just downstream of the spillway
  300 m x 260 m    S = 1/200

### (2) Drilling

- Around the retaining wall at the toe of the dam downstream slope
  3 holes (15.5 m, 30.7 m and 33.1 m)

- Left bank open-cut excavation slope
  1 hole x 50 m

(3) <u>Material Tests of Riverbed Sediments</u>

- Test pitting 9 spots

- Material tests
  Grain size distribution, specific gravity and unit weight

(4) <u>Concentration Tests of Suspended Sediments</u>

- Each one spot at Agno River and Adonot River

# 5. MONITORING SYSTEM

# 5. Monitoring System

## 5.1. Present Monitoring System

The regular measurements currently conducted for the Binga dam are daily observations of rainfall, evaporation, temperature, wind direction and velocity and humidity with the meteorological observation devices installed on the crest of the dam. The daily records of the above observations are compiled as a monthly "Weather Report". The reservoir water level is measured at every noon by sight on the staff gauge provided upstream of the spillway inlet. Besides, sounding measurement of the reservoir was carried out three times during a period of 1960 up to 1986, the results of which sounding are summarized in "Sedimentation Studies of Ambuklao & Binga Reservoir-1986 National Power Corporation".

Other than the above, any particular monitoring on the civil structures such as the dam and the intake is not currently conducted by NAPOCOR.

## 5.2. Installation of Monitoring Facilities

Installed, on this occasion, to monitor the behavior of the dam were devices for the dam deformation measurement and the water leakage measurement and the seismograph. The measurement devices were provided and delivered by JICA and installed by NAPOCOR work force under the supervision of the JICA study team.

### (1) Dam Deformation Measurement Devices

The observation point (a fixed point) and the observed point (a fixed point) were set on the line extended between both banks where the dam is seated on the foundation rock with the former on the right bank and the latter on the left bank. These points can command a view of the center part of the dam crest on which the measurement point (a movable point) was provided.

The measurement was performed by collimation with the theodolite installed at the observation point on the right bank which can sight the observed point on the opposite side, thereby the movable point on the dam crest could be measured against its deviation from the line connecting the above fixed points on both banks. Such measured deviation corresponds to a degree of the dam deformation in a horizontal or up-downstream direction.

The dam deformation in a vertical direction was performed by levelling to measure a variation in elevation of the movable point on the dam crest using the fixed point on the right bank as the base point.

The dam deformation measurement was, in principle, based on the fixed point on either bank and the movable point on the dam crest. However, to check whether such fixed points have moved or not, another fixed point was set 100 m upstream of the observed point on the left bank. Furthermore, another movable point on the dam crest was provided to check the measured value of the dam deformation.

(2) Water Leakage Measurement Devices

Just below the retaining wall is formed a natural pond the water level of which is 1 to 2 m higher than that of the plunge pool. The leaked water from the dam is once stored in this natural pond, and then overflows to the plunge pool accordingly.

For the leakage measurement, this natural pond was reformed to serve as a collecting basin by rearranging the surrounding irregularities with concrete, and the approach channel for discharge measurements and the right-angle triangular weir which can allow the measurement up to 2,500 lit./min. were provided therewith as required to measure the leakage from the dam. According to the original plan, the measurement devices were of self-recording system with a pressure type water-level recorder and a digital type water-level and discharge recorder. However, the location of measurement was set near the plunge pool, thus there was a fear that the measurement devices might be damaged at the flood discharge. The manual system was therefore adopted by which the overflow depth can be read directly on a scale.

(3) Seismograph

The observation house made of reinforced concrete was constructed on the left bank 80 m upstream of the spillway gate. On the foundation rock in the observation house was installed the seismograph featured by two components in the horizontal direction and one component in the vertical direction. The seismograph is operational in three steps of 1 to 100 gal, 5 to 500 gal and 10 to 1,000 gal as required according to the sensitivity.

Specifications of the above newly installed monitoring devices are shown in Table 5.1, and the location of installation is exhibited in Fig.5.1. The actual schedule of installation is illustrated in Fig.5.2.

## 5.3. Results of Measurements by Newly Installed Monitoring Devices

(1) Dam Deformation

The dam deformation measurement has been carried out from December, 1987 up to date. The records during a period of eight months from the commencement of the measurement through July, 1988 indicate that, for the reservoir water level which varied 15 m from EL.575 m to EL.560 m, an amount of the deformation measured approximately 15 mm in the dam up-down-stream direction. The measured values shown in Fig.5.3, which show a little difference with each other, demonstrated elastic behaviour of the dam corresponding to changes of the reservoir water level. The amount of the dam deformation is related, with the correlation coefficient $\tau$ of 0.77, to the reservoir water level in the following equation.

$$d = 0.9948 (H-555)$$

where,

d : an amount of the dam deformation in the downstream direction based on the reservoir water level EL.555 m.

H : actual water levels of the reservoir (EL.m)

(2) <u>Water Leakage from the Dam</u>

The water leakage from the dam has been measured from March, 1988 with the triangular weir which was provided at the end of the dam downstream slope for the purpose of monitoring the discharge including leakage from the dam. The measurement records during a period of eight months from the commencement through October, 1988 indicate that, for a period from March through May in the dry season, the amount of discharge through the weir stands at 15 ℓ/min. on the average with the reservoir water level of EL.574 m to EL.569 m, and for a period from June through August in the rainy season, it stands at 300 ℓ/min. on the average, particularly for more than 150 mm continuous rainfalls, it shows 1,000 to 2,000 ℓ/min.

The weir was designed to make use of the natural pond created at the end of the dam downstream slope, and thus run-off from the surrounding drainage area and seepage water from the adjacent mountain mass flow into this natural pond as well as leakage water from the dam embankment and its foundation.

Due to this, rainfalls are to affect largely an amount of the discharge through the weir. Consequently, it is recommended that the measurement should be continued long enough to distinguish the above elements of the discharge and get an exact amount of leakage from the dam.

Table 5-1

## Specifications of Major Devices Provided by JICA

1. Water Leakage Measurement Devices

| | | |
|---|---|---|
| (1) Triangular weir | : | Dimension; 630 m x 860 mm x 6 mm |
| (2) Stilling screen | | |
| (3) Water level sensor | : | Model; Nakaasa-W-350-00 Water pressure type |
| | | Measurement range; 0 - 100 cm |
| (4) Water level recorder | : | Model; Nakaasa-W-551-02 |

2. Optical Theodolite

| | | |
|---|---|---|
| (1) Theodolite | : | Model; Nikon-MTD-2B |
| | | Accuracy; 20 seconds |
| | | Measurement distance; 1,200 m |

(2) Function key board, batteries, tripod and other appurtenances complete

3. Seismograph

| | | |
|---|---|---|
| (1) Transducer | : | Model; Katsushima-SDA-240-3, two horizontal and one vertical components |
| (2) Seismometer | : | Model; Katsushima-WSS-1FS |
| (3) Switchbox | : | Model; Katsushima-SW-1 |
| (4) Oscillograph | : | Model; Katsushima-5M26 |

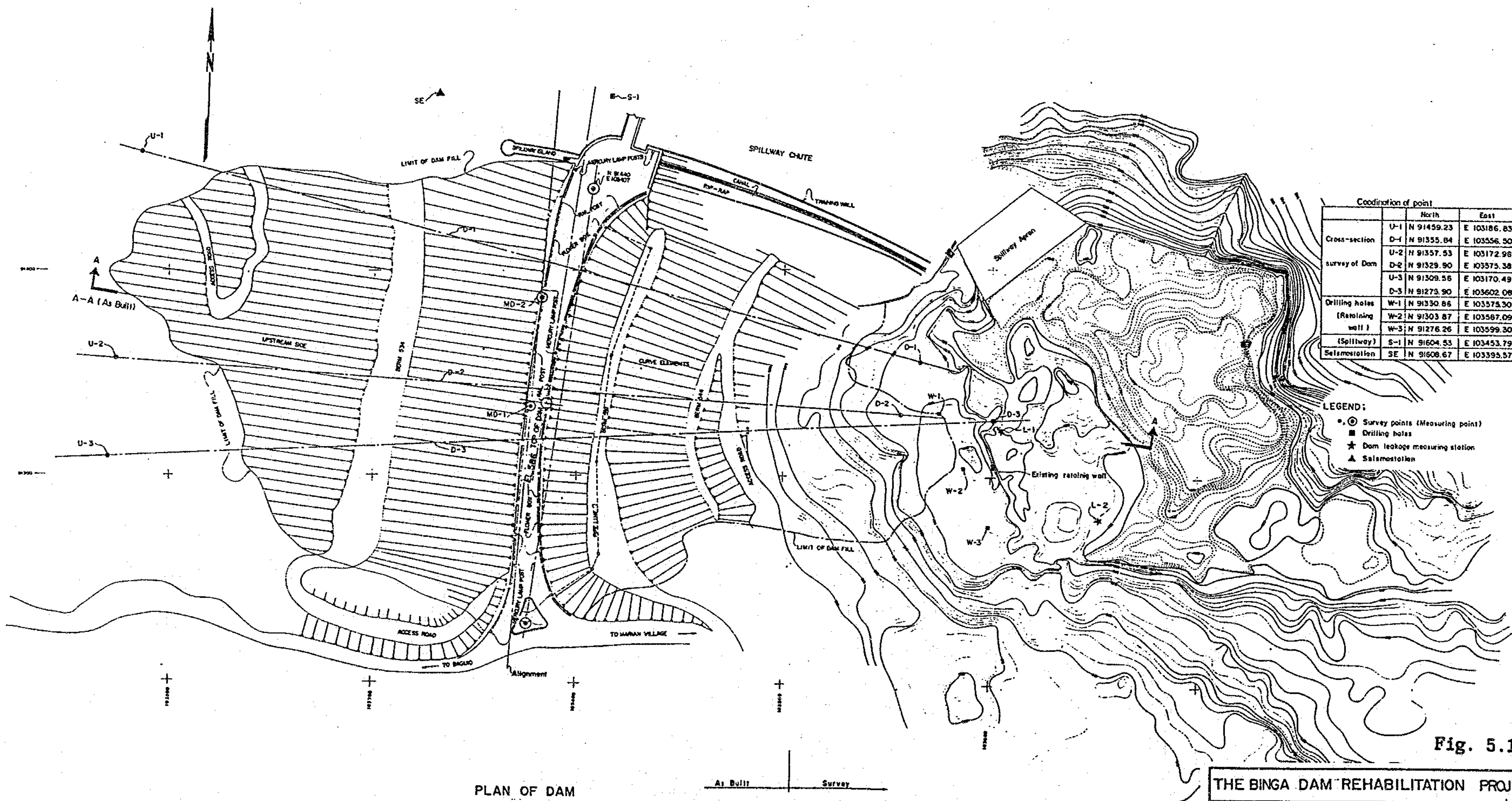
(5) Cable and other appurtenances complete

4. Water Sampler

| | | |
|---|---|---|
| Type | : | Handy sampler B type |
| Capacity | : | 1,100 cc |

5. Current Meter

| | | |
|---|---|---|
| Model | : | Nakaasa-J-061 |
| Measurement range | : | 0.3 -2 m/sec. or 4 m/sec. (switchable) |




Coodination of point

| | | North | East |
|---|---|---|---|
| Cross-section survey of Dam | U-1 | N 91459.23 | E 103186.83 |
| | D-1 | N 91355.84 | E 103556.50 |
| | U-2 | N 91357.53 | E 103172.96 |
| | D-2 | N 91329.90 | E 103575.38 |
| | U-3 | N 91309.56 | E 103170.49 |
| | D-3 | N 91275.90 | E 103602.08 |
| Drilling holes (Retaining wall) | W-1 | N 91330.86 | E 103575.30 |
| | W-2 | N 91303.87 | E 103587.09 |
| | W-3 | N 91276.26 | E 103599.30 |
| (Spillway) | S-1 | N 91604.53 | E 103453.79 |
| Seismostation | SE | N 91608.67 | E 103395.57 |

LEGEND;

- •, ◉ Survey points (Measuring point)
- ■ Drilling holes
- ★ Dam leakage measuring station
- ▲ Seismostation

Fig. 5.1

| THE BINGA DAM REHABILITATION PROJECT | |
|---|---|
| Location of Monitoring Instruments and Additional Investgation Works | |
| SCALE | DATE |
| THE JAPAN INTERNATIONAL COOPERATION AGENCY | |

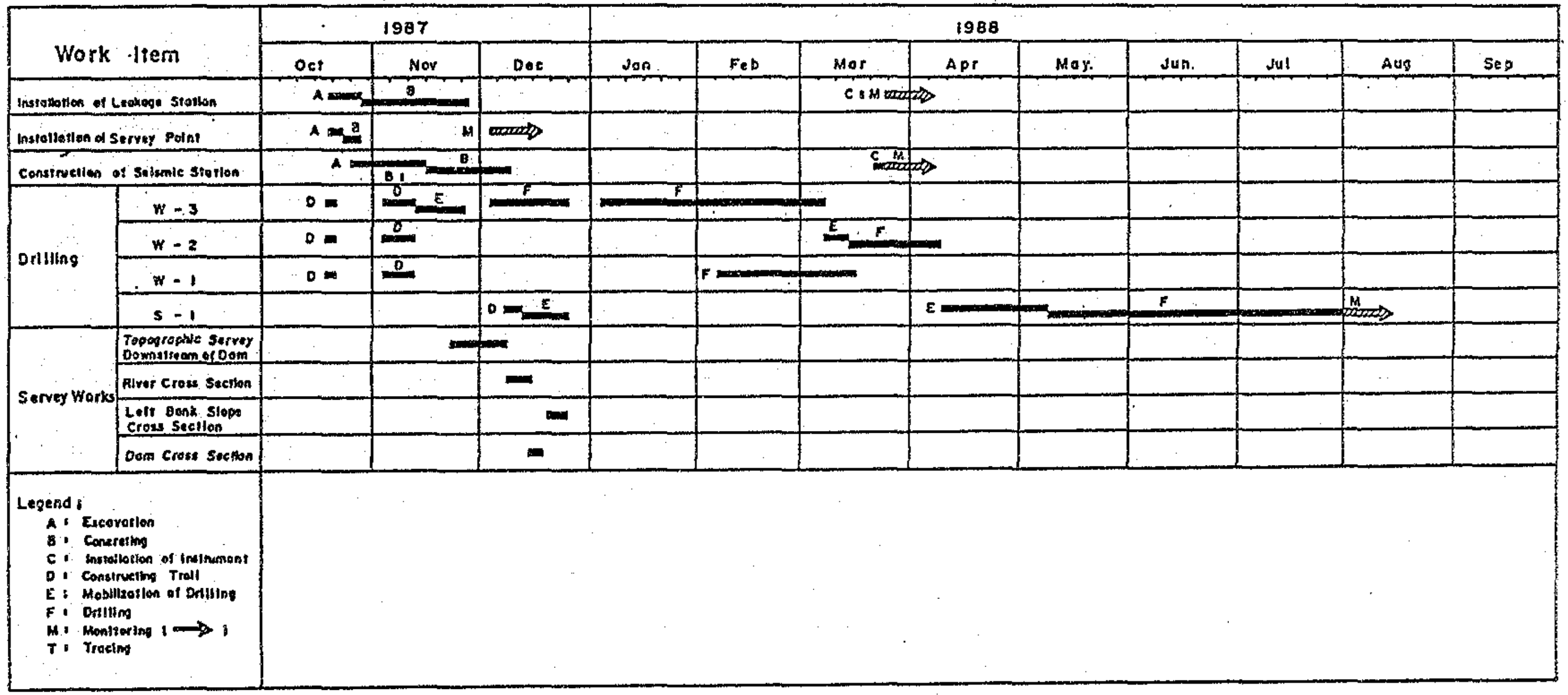



Fig. 5.2 Progress of Installation of Monitoring Instruments and Additional Investigation Works

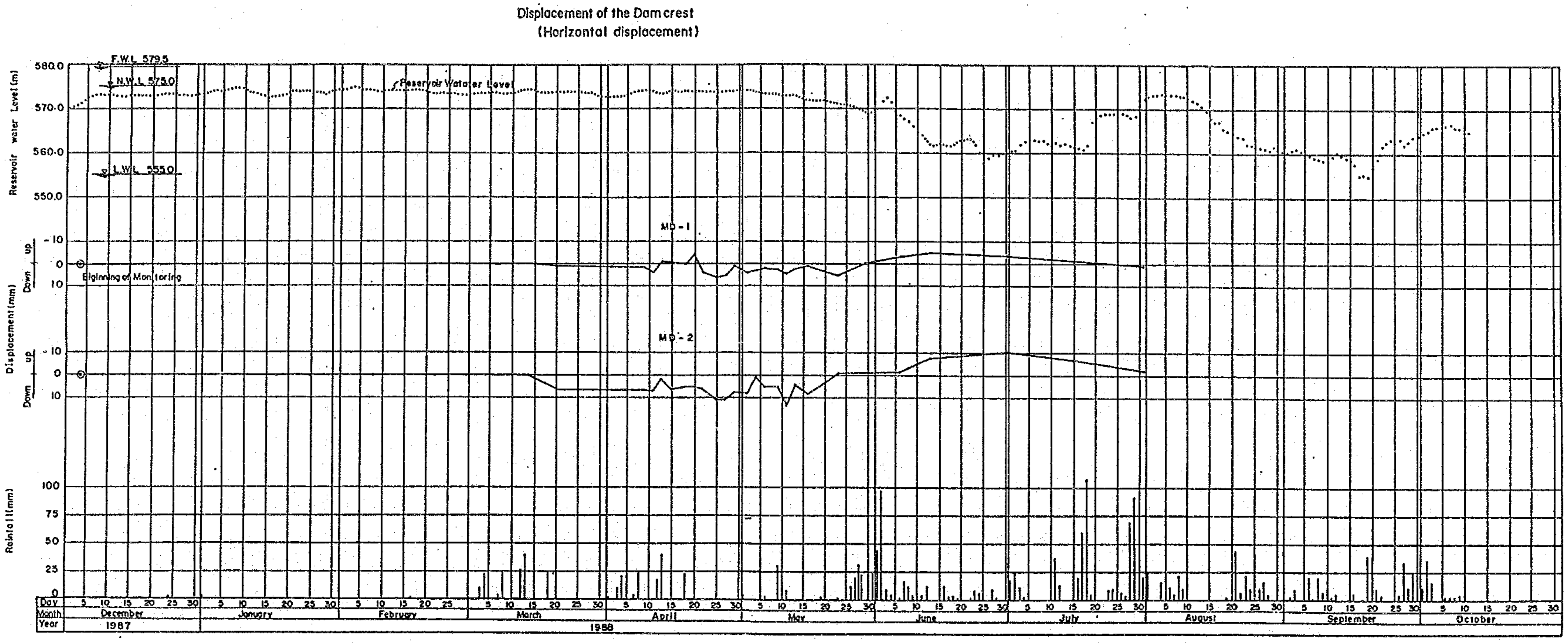



Fig. 5.3 Record of Dam Displacement

# 6. GEOLOGY

# 6. Geology

## 6.1. Background

The Agno River flows from north to south controlled by one of the branch fault zones of the main Philippines Fault. The fault zone is so large that it easily incorporates the damsite and reservoir areas. There are secondary faults within this zone trending at N10-20°E, N10-20°W and N70-80°W, which have resulted in formation of a narrow meandering valley below the damsite.

The principal rocks along the Agno River in the vicinity of the damsite are a series of metamorphic rocks and diorites.

The metamorphic rocks originated from andesitic and basaltic lava, extruded into the sea water in the Cretaceous or Paleogene age, and from sedimentary rocks, such as tuff, tuff-breccia and clayslate. These rocks were later strongly altered mechanically, mainly by the mountain-building movement, which produced on the rocks numerous fractures and joints.

Diorites intruded into the metamorphic rocks in Neogene (Miocene) age, providing some heat alteration to the metamorphic rocks. The contact between diorite and metamorphic rocks is, in general heat-fused and therefore, well-cemented, but in some portions is faulted.

## 6.2. Field Investigations

### (1) Left Abutment of the Dam

The open cut of the spillway on the left abutment has an excavated slope toward the hillside 130 meters high and 500 meters long. The average slope of the cut is 60°. It consists of seven berms, 8.5 meters wide each and 20 meters high. The slope of the open cut runs across the edge of the ridge extending in the NE-SW direction. The top of the slope reaches the upper portion of the ridge. The mountain mass at this location is relatively big with the natural slopes of the mountain on both sides of the ridge equal to about 35°.

The above excavated slope consists of metamorphics rocks which had originated from andesitic and basaltic lava, and sedimentary rocks, such as tuff and tuff-breccia, and small dykes intruding occasionally within these rocks at different locations. The rocks of the excavated slope are hard and massive without many cracks, but the weathered surface of the slope is brittle.

The strike and dip of the bedding planes of the rock on the slope are N10° E/50°W, respectively. The strike is almost perpendicular to the excavated slope.

The bedding planes of the rock are generally poorly developed and they are found to be fairly tight. The joints and cracks on the slope surface are observed to be few and to lack continuity.

The rock conditions on the part of the slope upstream of the dam axis are observed to be much better than those on the downstream part. There are practically no seams or faults upstream of the dam axis. But, there are many downstream. Around the seams and the faults of the downstream portion of the slope, many cracks have developed through time which have further deteriorated the general conditions of the rock in this area.

The above conditions in such locations and the frequent rainfalls in the region of the project have contributed to the formation of many gullies along the slope. Some of these gullies have developed, due to scouring, a width of 15 meters, and a depth of about 8 meters. Scouring along faults and seams has damaged certain berms to such an extent that passing along the berms has been made impossible.

The largest gully is the one which starts at the upper central portion of the slope, runs downwards along the seam of the slope surface, and extends all the way to the lower portion of the slope.

The debris produced by scouring, local slope failures and ravelling, are deposited on the berms. Some of these debris have acquired an enormous size. This is particularly true for the berms above which the failure of the slope was considerable.

As mentioned previously, practically all above local failures and scouring have occurred on the part of the slope downstream of the dam axis. The upstream part of the slope has remained almost intact, maintaining its original shape unchanged.

The berms on the downstream slope at EL.583 and 643 meters are used as service roads, and, therefore, they should be free from any obstacle which may hamper a safe passage.

The spillway has not been damaged by the debris and the falling rock to such an extent as to affect its functions. This was due to an ample width of more than 10 m of the lowest berm between the spillway and the slope. However, if no measures are taken against future scouring and possible collapse of the various parts of the slope and additional accumulation of debris and rocks on the berms, making them thus impassable, the above will further aggravate the maintenance and the overall stability of the slope.

If the debris which have been accumulated so far are not removed and further piling up of such debris is permitted, there is a danger that the debris might start a debris flow moving toward the spillway. This may seriously hamper the proper operation of the spillway and, eventually, even seriously damage the spillway, making it thus inoperable.

(2) Right Abutment of the Dam

The mountain mass of the right abutment of the dam is large and consists of metamorphic rocks. It had been determined during the pre-construction investigations that the right abutment, which was deeply weathered and contained numerous joints, was geologically definitely inferior to the left abutment. For this reason, it was decided at that time to locate the spillway on the left abutment. However, the investigations conducted currently reveal that the slopes adjacent to the dam right abutment and the power intake are generally stable, although small surface failures could occassionally be seen.

(3) Flip Bucket and Plunge Pool Area

The plunge pool was constructed by excavating portion of the left abutment as required for proper operation of the pool. A portion of the left abutment immediately downstream of the plunge pool still protrudes into the river channel.

The geology of the plunge pool area and the flip bucket consists mainly of metamorphic rocks. An andesite dyke about 20 meters wide has been observed at the riverbed level of the left abutment near the downstream end of the plunge pool.

The metamorphic rocks forming the slopes of the left abutment are hard and massive. The andesite dyke is extremely hard and its horizontal cooling joints are well developed. The contact between the metamorphic rocks and the andesite dyke is well-cemented, but the latter is protruding from the slope due to difference in hardness against erosion.

The current erosion of the slope is not of such magnitude as to affect the stability of the slope around the plunge pool. Based on the observations in the field, the erosion is not expected to progress rapidly and to possibly within short time cause a collapse of the slope. However, even if the above happens, no damages are expected to the adjacent concrete structures.

(4) Retaining Wall at the Toe of the Dam

The retaining wall at the toe of the dam, approximately 130 meters long, has been damaged by floods. The damages to the wall have never been repaired. The field investigations indicated that the greater part of the wall had not been founded on the bedrock, but on the riverbed deposits. Metamorphic rocks are observed under the wall near both abutments.

About 30 meters of the wall next to the spillway (left abutment) was placed on the rock. The rock at this location contains cracks, but it is adequately hard, and, therefore, it is quite suitable as a foundation for the wall. The drilling investigations carried out recently have indicated that the depth of the bedrock along the retaining wall is about 5 meters near the spillway flip bucket and about 25 meters toward the other end of the wall near the right abutment.

(5) Reservoir

Field geologic investigations were carried out as required to establish the degree of failures (slides) which had occurred on the slopes around the main Binga reservoir and its main upstream tributary, the Adonot River.

The investigation around the reservoir was undertaken by the use of rubber boat and car as a means of transportation, while on-foot reconnaissance was carried out for the tributary.

The mountain mass around the reservoir consists of metamorphic rocks which are exposed along the greater part of the shoreline. The slopes around the reservoir are relatively stable, although small amount of landslides has been observed. However, the reservoir is badly affected by sediment deposits, in particular the portion of the reservoir about 3 km upstream of the dam.

A portion of the Adonot River about 2 km upstream of its confluence was examined by foot reconnaissance. The geology along the Adonot River is characterized by metamorphic rocks and diorites, exposed here and there along the riverbed. The mountain slopes are relatively stable, but a large number of small surface slides have been observed.

## 6.3. Rock Classification and Properties

The rocks distributed in the vicinity of the Binga damsite are classified principally on the basis of the standard shown in Table 6-1, taking into account the results of the field investigations.

The correlation between the rock classification and the rock strength constant ($\tau_o$ . $\Phi$), as shown in Fig.6-1, is generally well known. For the analyses of the rocks in this Study, values of rock properties have been selected as indicated in Table 6-2, referring to the above correlation established on the basis of experience.

## 6.4. Stability Analysis on the Left Bank Spillway Open-cut Excavation Slope

The geologic data of the left bank excavated slope such as a geologic sketch and map are referred to Figs. 6.2 through 6.5

attached hereto.

In a slope consisting of hard rocks, sliding is prone to occur along geological fissures such as fractures, faults and stratified structures. On the existing spillway open-cut excavated slope are observed few continuous fractures and faults, and therefore it is considered the possibilities of sliding caused by such geological fissures are very slight. Besides, the present slope, consisting of hard rocks, gives no sign of deterioration in rock properties, thus suggesting that it would be able to keep the stable condition for long.

In case of sliding along the geological fissures, more than two sliding surfaces are usually involved, and therefore the study was conducted for possibilities on sliding which involves two sliding surfaces by the illustrating method with a stereograph. Sliding surfaces to be studied are combined for the following three cases all of which include a continuous fracture.

Case 1 : Combination of NS/50W and N35W/53N

Case 2 : Combination of NS/50W and N55E/46S

Case 3 : Combination of N10E/45W and N70E/35E

A crossing line of the above two sliding surfaces in each case is illustrated in Figs. 6.8 and 6.9. For Case 1, no sliding takes place because the crossing line inclines towards the mountain side. For Cases 2 and 3, there are some possibilities of sliding since the crossing lines incline toward the river side by 27° and 22° respectively. Shown below is a process to obtain the safety factor against sliding (Fs) with the following formula:

$$Fs = \frac{C1A1 + Rn1 \tan \phi 1 + C2A2 + Rn2 \tan \phi 2}{W \sin \varepsilon}$$

Where,

C1, C2 : Cohesion of sliding surfaces 1 and 2

$\phi 1$, $\phi 2$ : Internal friction angle of sliding surfaces 1 and 2.

A1, A2 : Area of sliding surfaces 1 and 2

Rn1, Rn2: Reaction force perpendicular to sliding surfaces 1 and 2

W : Weight of rocks overlying sliding surface

ε : Inclination of a crossing line of two sliding surfaces

The values of C1, C2 and ϕ1, ϕ2 were assumed with D class rocks as follows:

$$C1 = C2 = 20.0 \ tf/m^2$$
$$\phi 1 = \phi 2 = 30°$$

As a result of the calculations, the safety factor (Fs) indicates:

Case 2 : Fs = 51.6

Case 3 : Fs = 39.3

For all Cases 1, 2 and 3, there is no fear of sliding accordingly.

Table 6 - 1 **Examples of Quality Classifications of Rock in Dam Foundations**

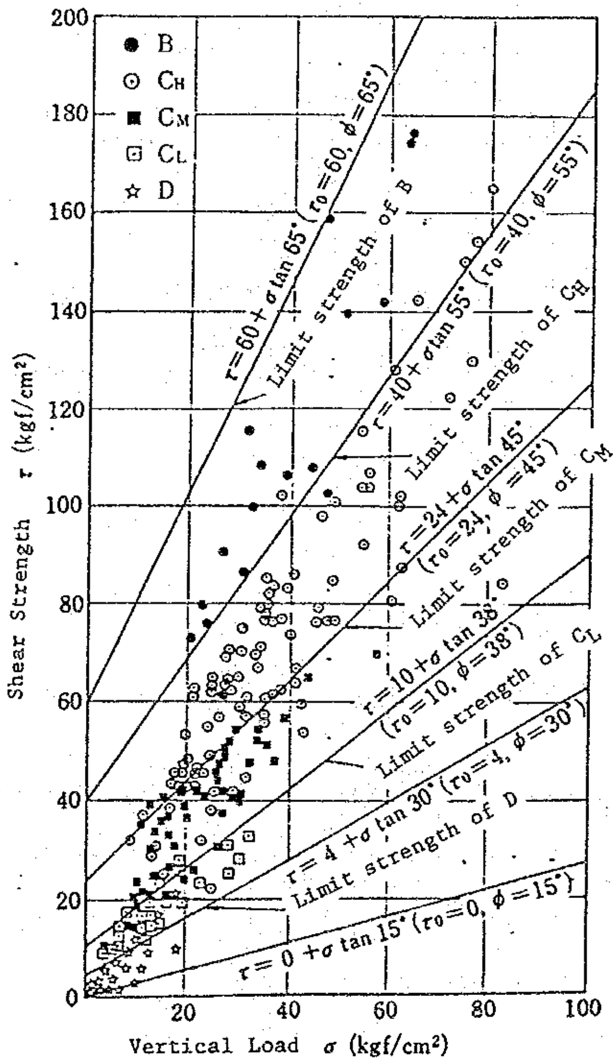
| Classification | Characteristics |
|---|---|
| A | Rock-forming minerals[1] are fresh and not weathered or altered. Joints and cracks are very closely adhered with no weathering along their planes. A clear sound is emitted when hammered. |
| B | Rock-forming minerals are weathered slightly or partially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

(1) : Except quartz

Table 6 - 2 Rock Classification and Properties

| Rock Class | Shear Strength | Unit Weight $\gamma_t$ kgf/cm³ |
|---|---|---|
| $C_M$ | $\tau = 18.0$ kgf/cm² $+ \sigma \tan 45°$ | 2.4 |
| $C_L$ | $\tau = 7.0$ kgf/cm² $+ \sigma \tan 38°$ | 2.2 |
| D | $\tau = 2.0$ kgf/cm² $+ \sigma \tan 30°$ | 2.0 |

Fig. 6 - 1  Correlation between Rock Classification and Rock Strength Test Results



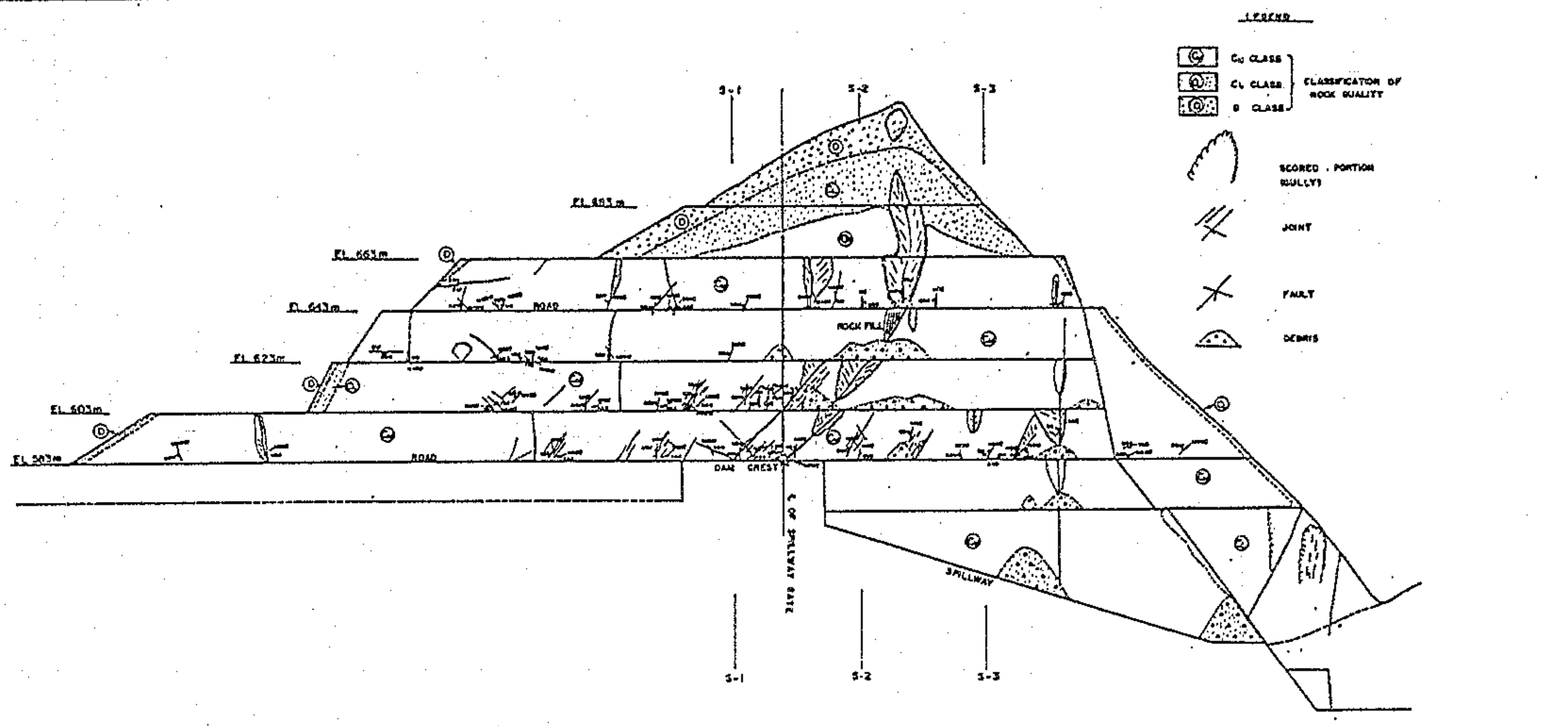



Fig.6.2

GEOLOGIC SKETCH OF EXCAVATED SLOPE AT LEFT ABUTMENT OF DAM

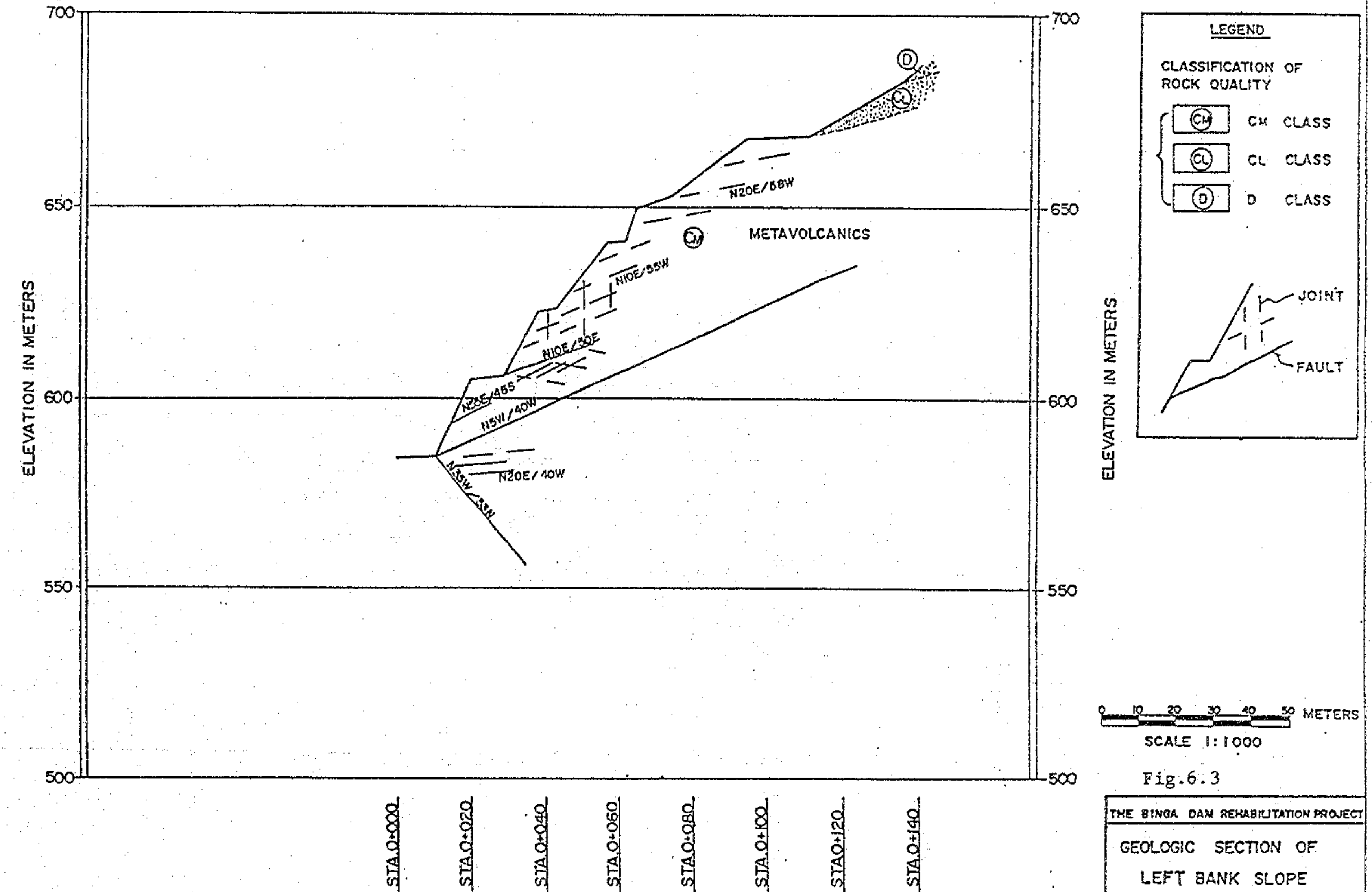



Fig.6.3

THE BINGA DAM REHABILITATION PROJECT

GEOLOGIC SECTION OF LEFT BANK SLOPE (S-1.)

SCALE 1:1000 | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

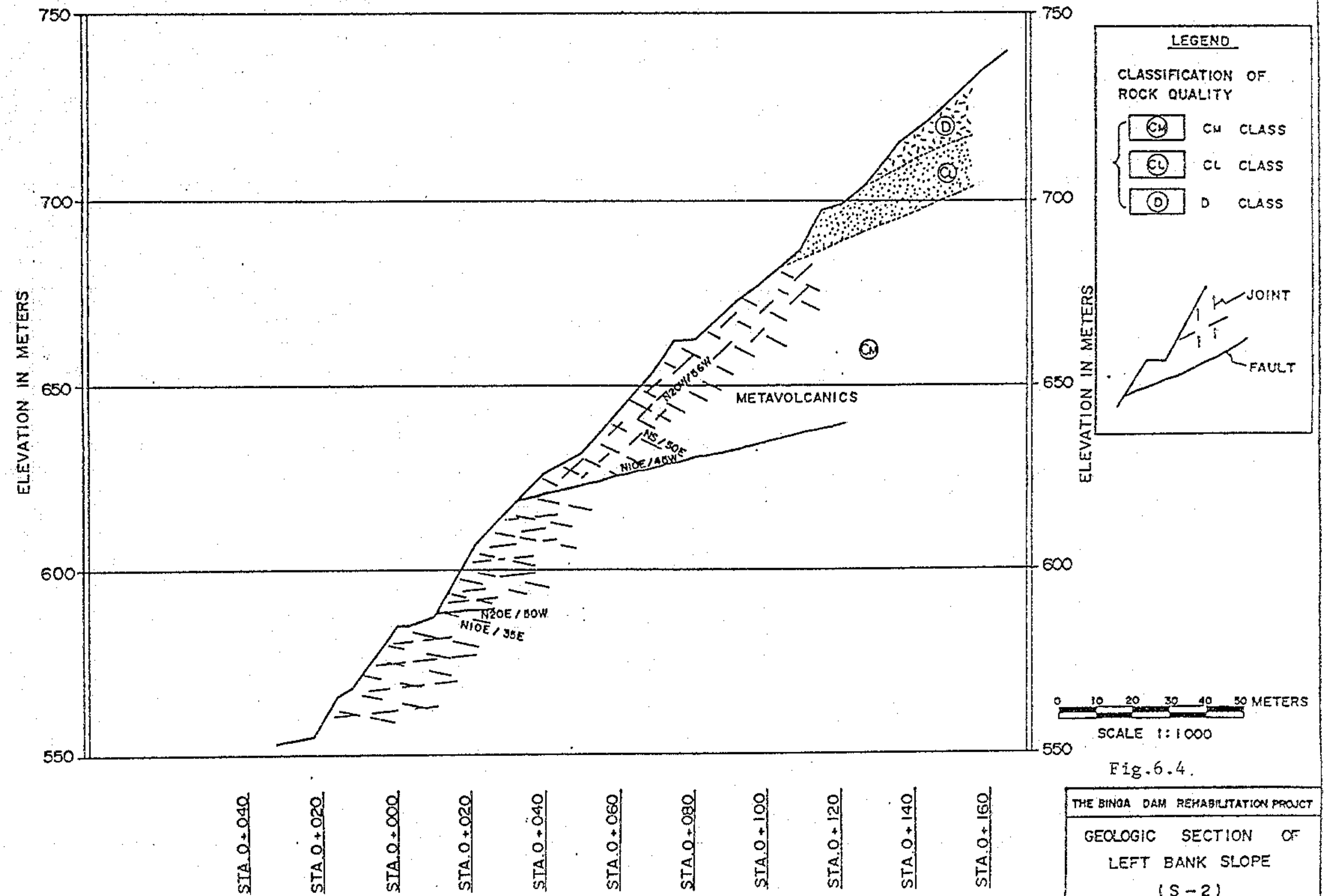



Fig.6.4.

THE BINGA DAM REHABILITATION PROJECT

GEOLOGIC SECTION OF LEFT BANK SLOPE (S-2)

SCALE 1:1000 | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

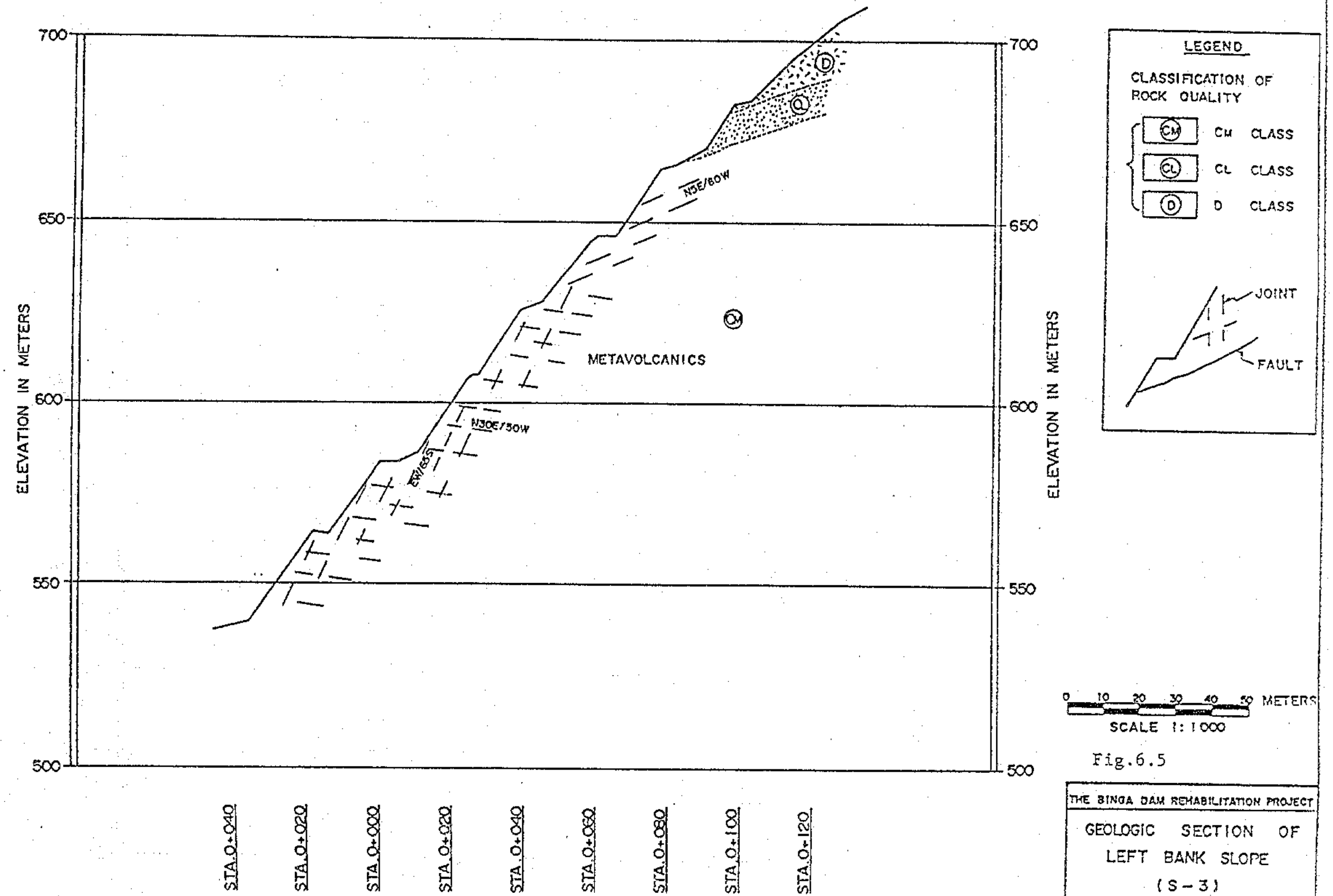



Fig.6.5

THE BINGA DAM REHABILITATION PROJECT

GEOLOGIC SECTION OF LEFT BANK SLOPE (S-3)

SCALE 1:1000 | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

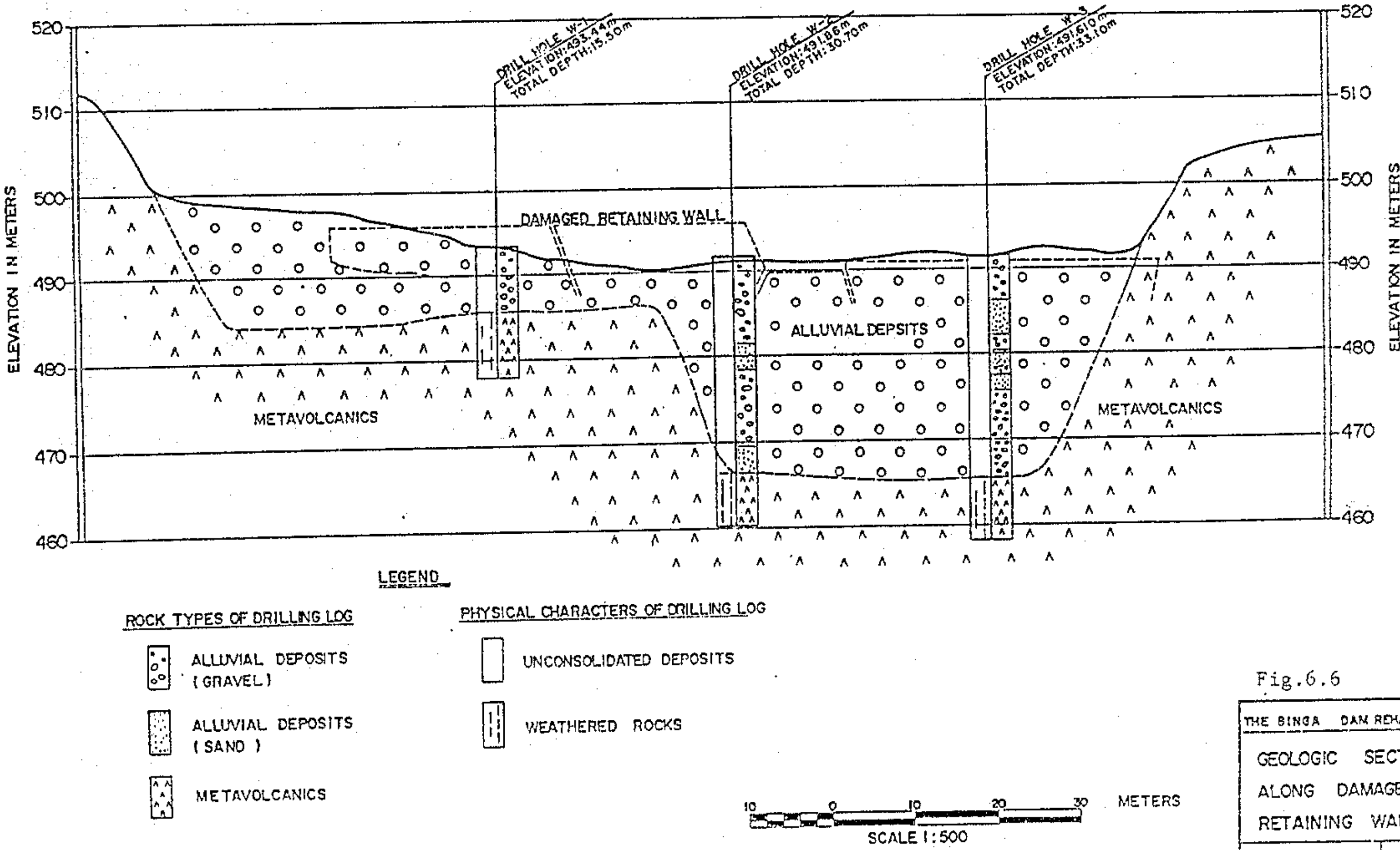



Fig.6.6

| |
|---|
| THE BINGA DAM REHABILITATON PROJECT |
| GEOLOGIC SECTION ALONG DAMAGED RETAINING WALL |
| SCALE 1:500 \| DATE |
| THE JAPAN INTERNATIONAL COPERATION AGENCY |

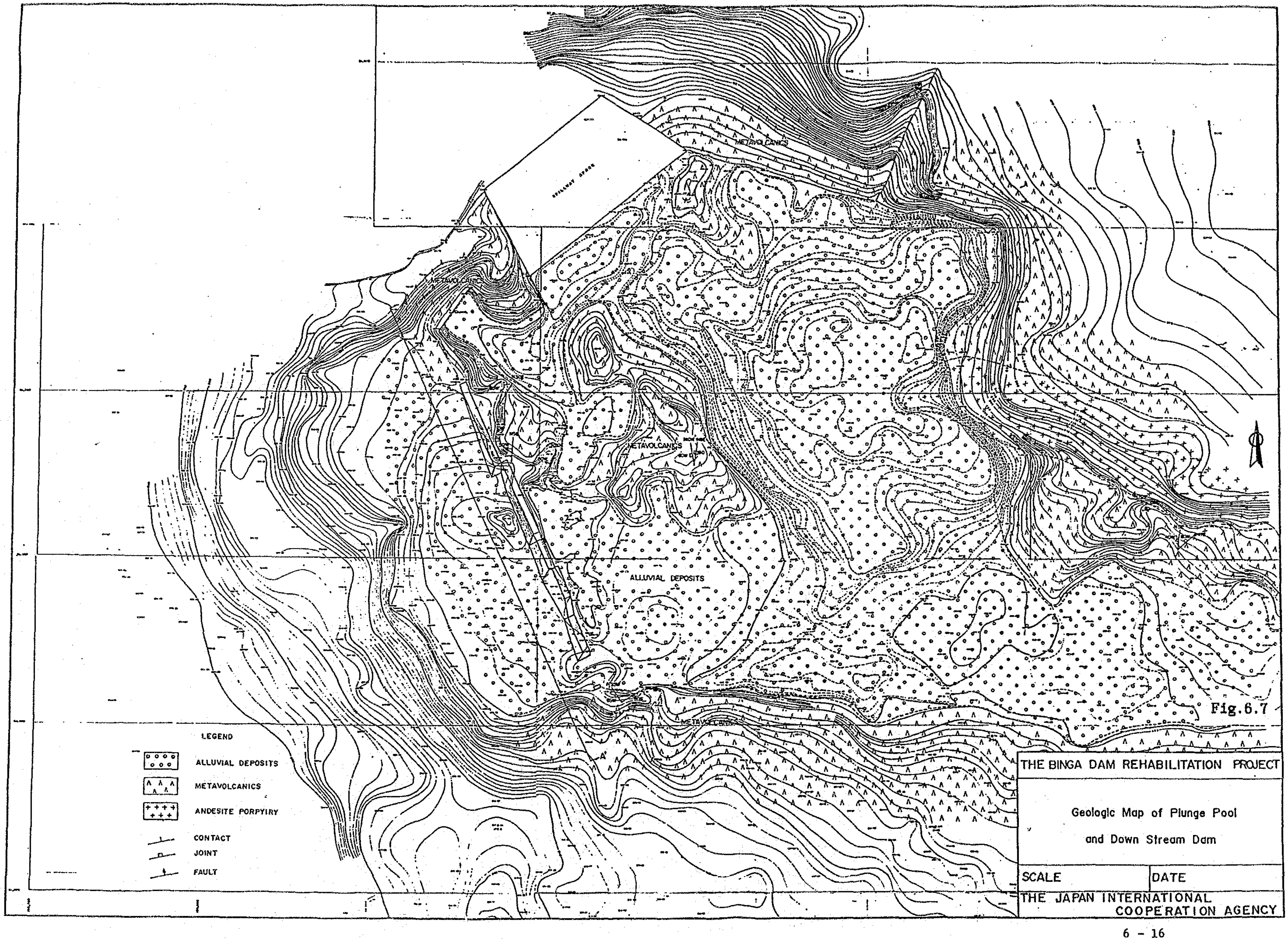

METAVOLCANICS
SPILLWAY APRON
METAVOLCANICS
METAVOLCANICS
ALLUVIAL DEPOSITS
METAVOLCANICS
LEGEND
ALLUVIAL DEPOSITS
METAVOLCANICS
ANDESITE PORPYIRY
CONTACT
JOINT
FAULT
THE BINGA DAM REHABILITATION PROJECT
Geologic Map of Plunge Pool
and Down Stream Dam
SCALE
DATE
THE JAPAN INTERNATIONAL
COOPERATION AGENCY

Fig.6.7

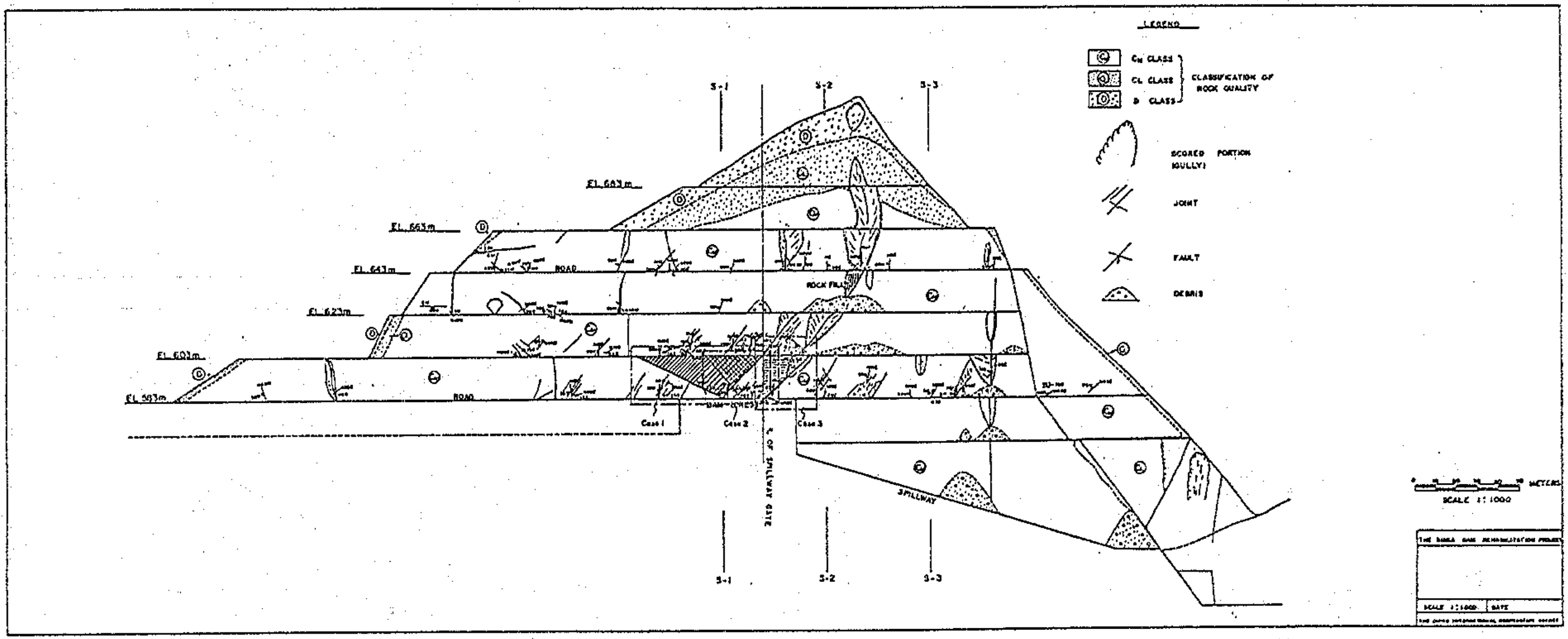
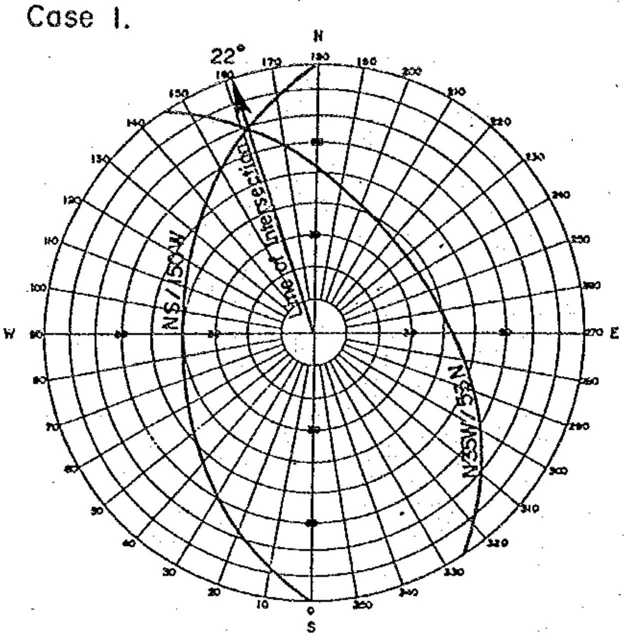


Fig. 6.8 Location Map of Sliding Planes

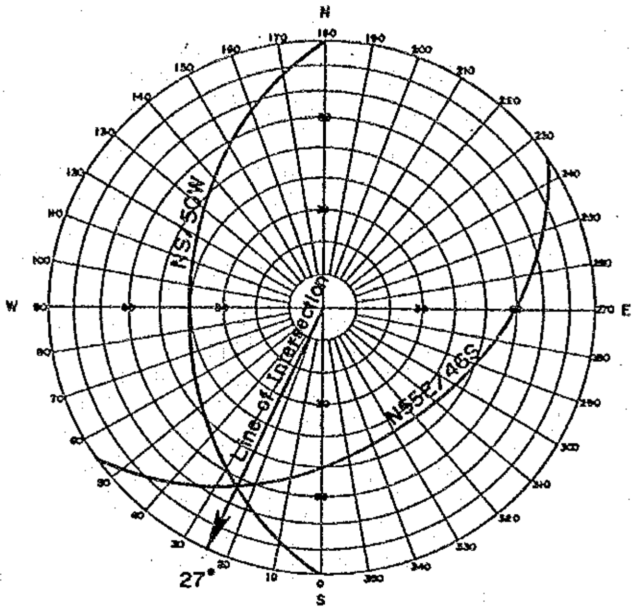
Case 1.




NS / 50W and N35W / 53N

( Lower Hemisphere )

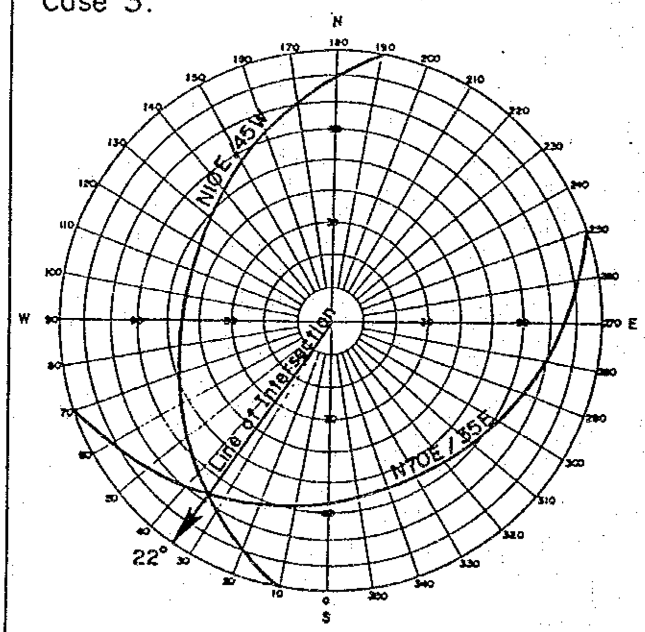
Case 2.




NS/50W and N55E/46S

( Lower Hemisphere )

Case 3.




N10E /45W and N70E / 35E

( Lower Hemisphere )

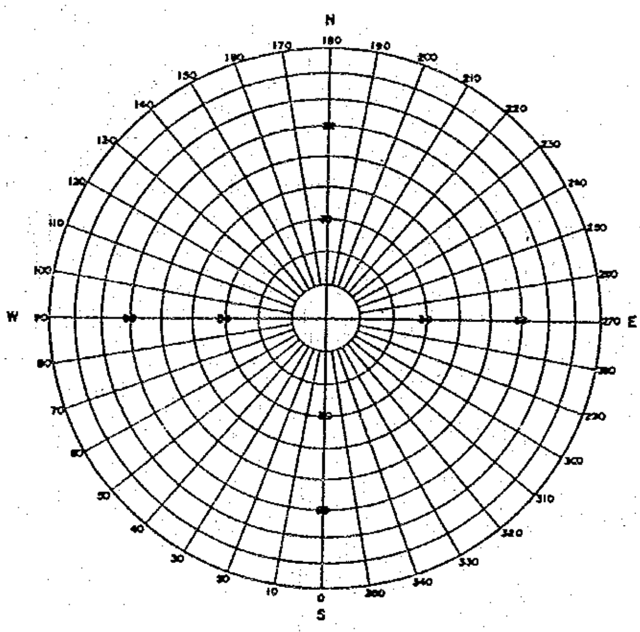



Fig. 6.9 Stereo Graph Of Sliding Planes

# 7. DAM STABILITY

# 7. Dam Stability

## 7.1. Design and Construction Records

Construction of the Binga dam commenced 32 years ago in 1956. The dam was completed in 1959. The power plant started its operation in May, 1960. According to the historical data, it appears that the first studies for the Binga Project were initiated in about 1948. The final design of the project was performed in 1956. Although adequate records on the design of the project were not available during the site investigation conducted for this study, the basic design concept could be understood from the following documents:

(1) Supplement to the Interim Geological Report on the Binga Project for the National Power Corporation, June 1957, by Irving B. Crosby.

(2) Interim Report on the Design of the Dam for the Binga Hydroelectric Project, October 1956, by EDCOP-TAMS*

* TAMS is abbreviation for Tippettes-Abbett McCarthy-Stration, a design firm from New York, United States of America.

Design and construction records from the construction period of the project could not be collected as they were not available. The only exception to the above was a design drawing showing a plan of the dam and an as-built drawing of the center portion of the dam. The main civil Contractor for the Binga project was a local firm, PESI (Philippine Engineers Syndicate, Inc.).

## 7.2. Basic Design Concept

The main characteristics of the dam and its foundations based on the information obtained from the EDCOP-TAMS Report, mentioned above, are described below. It should be noted that the final design conditions, that is, design assumptions, parameters and loadings, as actually used during construction, are not known. However, by comparison with the information from the as-built drawing, it appears that no changes to the design conditions as described in the above Report were made.

The main characteristics of the dam were thus as follows:

(1) Foundation Rock:

- The foundation rock consisted of metamorphic rocks originating from basaltic and andesitic lava.

- The rocks were hard, but cracky.

- Stripping between 4 to 8 m of depth was required.

- Permeability was moderate judging from the results of the permeability tests conducted on five bore-holes.

(2) Dam:

- The dam type was rockfill with an inclined core.

- The reason for adopting the inclined core was to facilitate the placing of the downstream rockfill zone as the first stage fill.

- The downstream rockfill zone (downstream of the core) consists of two subzones, the downstream subzone constructed of dumped rockfill, and the upstream subzone next to the core, built as a roller compacted rockfill.

- The upstream rockfill zone (upstream of the core) is a dumped rockfill. Two layers of fine filter are provided between the core and the upstream and downstream rockfill zones of the dam.

- The upstream and downstream slopes of the dam were as follows:

  Upstream face slope

  | | |
  |---|---|
  | EL.586m - EL.566m | 1 : 1.4 |
  | EL.566m - Base | 1 : 2.5 |

  Downstream face slope

  | | |
  |---|---|
  | EL.586m - Base | 1 : 1.4 |

  In addition, a berm is provided on the downstream slope at EL.520 m.

(3) Dam Embankment Materials:

- The muck produced from the spillway open-cut excavation was used for construction of the rockfill zones of the dam.

- Uniform-sized rocks with a rock size of over 30 cm were used for the dumped rockfill zones.

- Compound-sized rocks with a rock size of less than 30 cm were used for the rolled compacted rockfill.

- Terrace deposits, silt compound sand and pebbles were used as filter materials.

- Compound materials consisting of the top soils excavated from the left bank spillway and weathered rocks were used as core materials.

(4) Foundation Treatment

- A line of curtain grout holes were provided along the center of the core contact area with the foundation rock.

- A 0.5 meter thick concrete slab was provided on the top of core-foundation contact area. It extended from about one meter upstream of the grout curtain to the downstream end of the above contact area.

- The depth of the grout curtain ranged from 50 m maximum to 25 m minimum.

- A blanket grouting 5 m deep was performed on the upstream half of the core contact area.

- In weak foundation rock and zones, dental concrete was placed as required.

## 7.3. Historical Records after Construction

There are no records on the dam behavior after its completion, as no measurement devices were installed at the time of its construction. However, it is known that the dam after its completion was confronted with a number of large scale natural phenomena. The records of these natural phenomena exist and they are as follows:

| | | | | |
|---|---|---|---|---|
| (1) | Flood | : | October 17, 1967<br>Peak flood outflow | <br>2,723 $m^3$/sec. |
| (2) | Flood | : | June 30, 1976<br>Peak flood outflow | <br>2,602 $m^3$/sec. |
| (3) | Flood | : | November 5, 1980<br>Peak flood outflow | <br>2,526 $m^3$/sec. |
| (4) | Earthquake | : | April 24, 1985<br>Epicenter<br><br>Depth of seismic center<br>Magnitude | <br>About 40 km east-north from the dam.<br>33 km<br>M = 5.9 |

It was observed that during the earthquake of April 24, 1985, cracks appeared near the top of the dam on both sides of the crest road and the part of the dam toward the right abutment.

The length of the crack on the upstream side was about 69 meters. The crack on the downstream side was about 66 meters long. The width of the crack varied from a hairline to a maximum of 20 cm.

As reported, two test pits were excavated to investigate the depth of the cracks observed on the upstream side. The investigations indicated that the maximum depth was 77 cm, and the minimum, 20 cm. It is considered that these cracks were caused by a shallow and small scale surface slide(s) through the rockfill zone. The seismic coefficient at the damsite for this earthquake was estimated to be k = 0.080 g (horizontal seismic coefficient), on the basis of the Kanai Formula.

## 7.4. Current Conditions of the Dam

In order to assess the present conditions of the dam, cross-sectional surveys of the dam were carried out and monitoring devices were installed as required for the purposes of this Study.

The work connected with the above consisted of the following:

| | | | |
|---|---|---|---|
| (a) | Cross-sectional survey of the dam | : | three sections, Feb. 1988 |
| (b) | Measurement by collimation of dam deformation | : | Commencement of measurement, Dec. 4, 1987 |
| (c) | Measurement of seepage at the downstream end of the dam | : | Commencement of measurements, Mar. 25, 1988 |
| (d) | Installation of seismograph at the spillway | : | Installed on March 24, 1988, on rock foundation at EL.586 m. |

(1) Dam Cross Section

Figure 7.3 shows a comparison between the cross section of the dam as depicted on the as-built drawing and the section obtained by the recent surveys. It is found from this Figure that a section of the upstream face approximate 30 m high, extending from EL.556 to EL.586 m, was missing and that, apparently, the missing material piled up on the slope between EL.554 and EL.532 m. Silt deposits are observed below EL.532 m.

As compared with the upstream face, no major post-completion changes are found on the downstream face of the dam. However, the size of the section was slightly reduced between EL.515 and EL.500 m and somewhat enlarged between EL.500 and EL.494 m.

Slopes of the Dam Upper Portions

| | Upstream Face | Downstream Face |
|---|---|---|
| As-built | 1 : 1.35 | 1 : 1.30 |
| As measured recently | 1 : 1.30 | 1 : 1.43 |

(2) Dam Deformation

Measurement of dam deformation started on December 4, 1987. Measurement was performed at two points of the crest of the dam by collimation. Based on the measurement records taken until July 1988, the dam embankment did not show any deformation until April, 1988. During the above period, the reservoir water level was kept at the same level (N.W.L.) and there were no fluctuations. However, as the water level started decreasing at the beginning of May, 1988, deformations started being recorded. They were in the upstream direction and for the total reduction of the reservoir level of 15 meters, the recorded deformation measured from 10 to 20 mm.

(3) Seepage

Seepage measurements at the downstream end of the dam have been carried out since March 26, 1988, by installing a measurement weir with a seepage collecting channel. Negligible seepage was recorded during the dry season from April to the middle of May, 1988. During the rainy season which started June, 1988, the measured values are supposed to have been produced mostly by rainfalls.

(4) Seismograph

The setting value for operation is 3 gal, and there have been no records of tremors (activity) to the present.

(5) Comments on the Present Conditions of the Dam

On the basis of the observations to date with regard to the foundation rock, dam sectional shape, deformation of the dam, and seepage, there are no major factors which could cause anxiety about the stability of the dam.

However it is noticed that the erosion, which has been occurring on the upper portions of the upstream face of the dam, will make further progress if the slope is left as it is, and no measures are taken. In addition, the damage to the downstream protection wall and the subsequent erosion of the dam at that location which has been taking place, will continue to progress further if no appropriate measures are taken.

Based on the present data, it appears that there is no problem with the seepage through the dam or its foundations.

The deformations of the dam as observed until now are within the allowable limits and, therefore, there is no cause for any concern with regard to this matter.

### 7.5. Stability Analysis

#### 7.5.1. Outline of the Analysis

It is not possible to determine how the original stability analysis for the Binga Dam was done as the relevant data from that time are not available. However, it can be assumed that a cross section of the dam for the downstream slope would have been investigated for stability against sliding through the rockfill zone materials. It is also assumed that the stability of the upstream portion of the dam against sliding through the impervious core would have been investigated.

In the "Interim Report on the Design of the Dam for the Binga Hydroelectric Project" mentioned above, the following values are indicated as preliminary values for properties of the embankment materials.

| Item | Unit Weight ($t/m^3$) | Permeability co-efficient(cm/s) | Friction angle(°) | Cohesion ($t/m^2$) |
|---|---|---|---|---|
| Core material (surface soil) | 1.6 - 1.76 | $10^{-5}$ - $10^{-6}$ | low | - |
| Core material (surface soil, other weathered rock) | 1.84 - 2.0 | $2x10^{-5}$ - $5x10^{-6}$ | medium | - |
| Filter material (terrace deposits) | 1.92 - 2.08 | $10^{-2}$ - $10^{-3}$ | 35 | - |
| Rockfill material (rolled fill) | 2.0 - 2.08 | $10^{-1}$ - $10^{-2}$ | 40 | - |
| Rockfill material (dumped fill) | 2.0 | $10^{-1}$ | 36 | - |

The slopes adopted for the Binga dam are relatively steep, with basic gradients of 1:1.35 to 2.57 at the upstream face, and 1:1.30 to 1.37, at the downstream face (on the as-built drawing). In particular, the upper portion of the upstream face with a slope of 1 : 1.35 is considered to be too steep. This