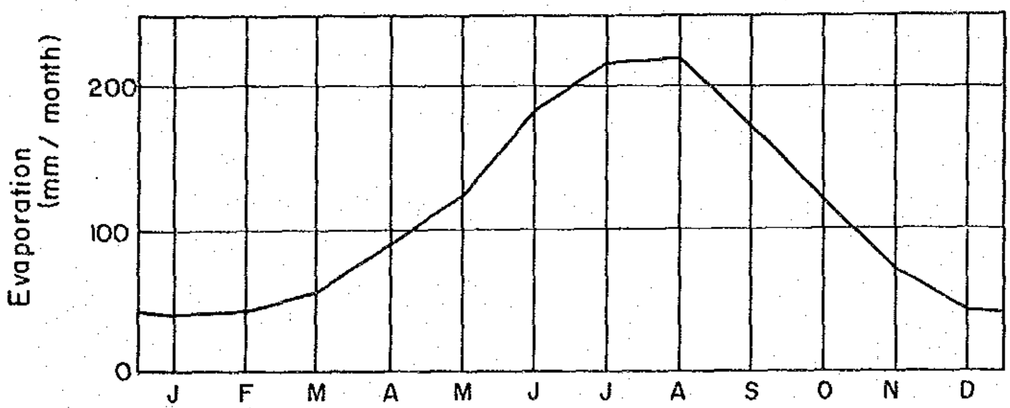
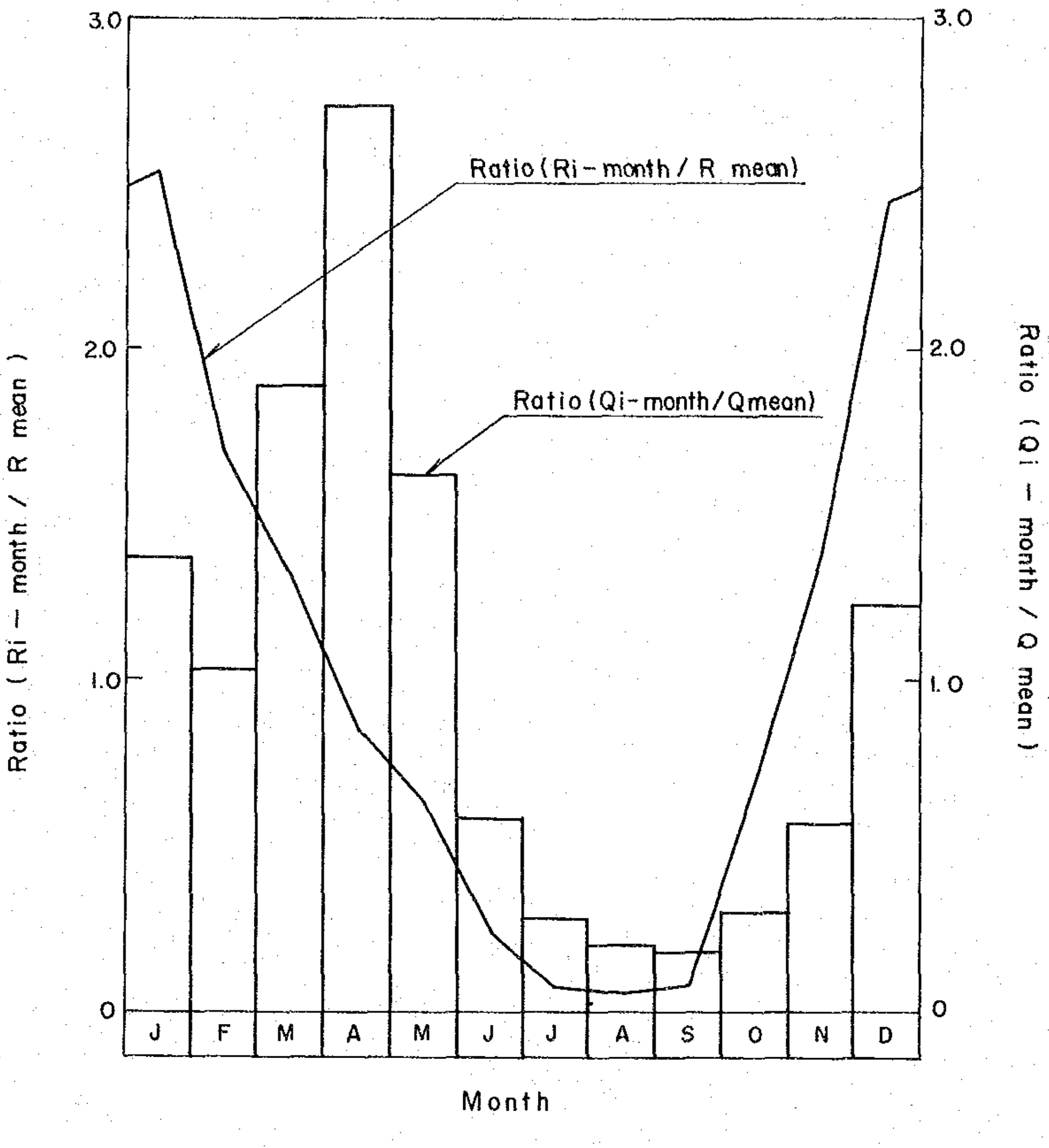
FIG. NO.




Estimated Mean Evaporation from Reservoir at Ermenek

| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C5<br>Estimated Mean Evaporation from Reservoir at Ermenek | |

FIG. NO.

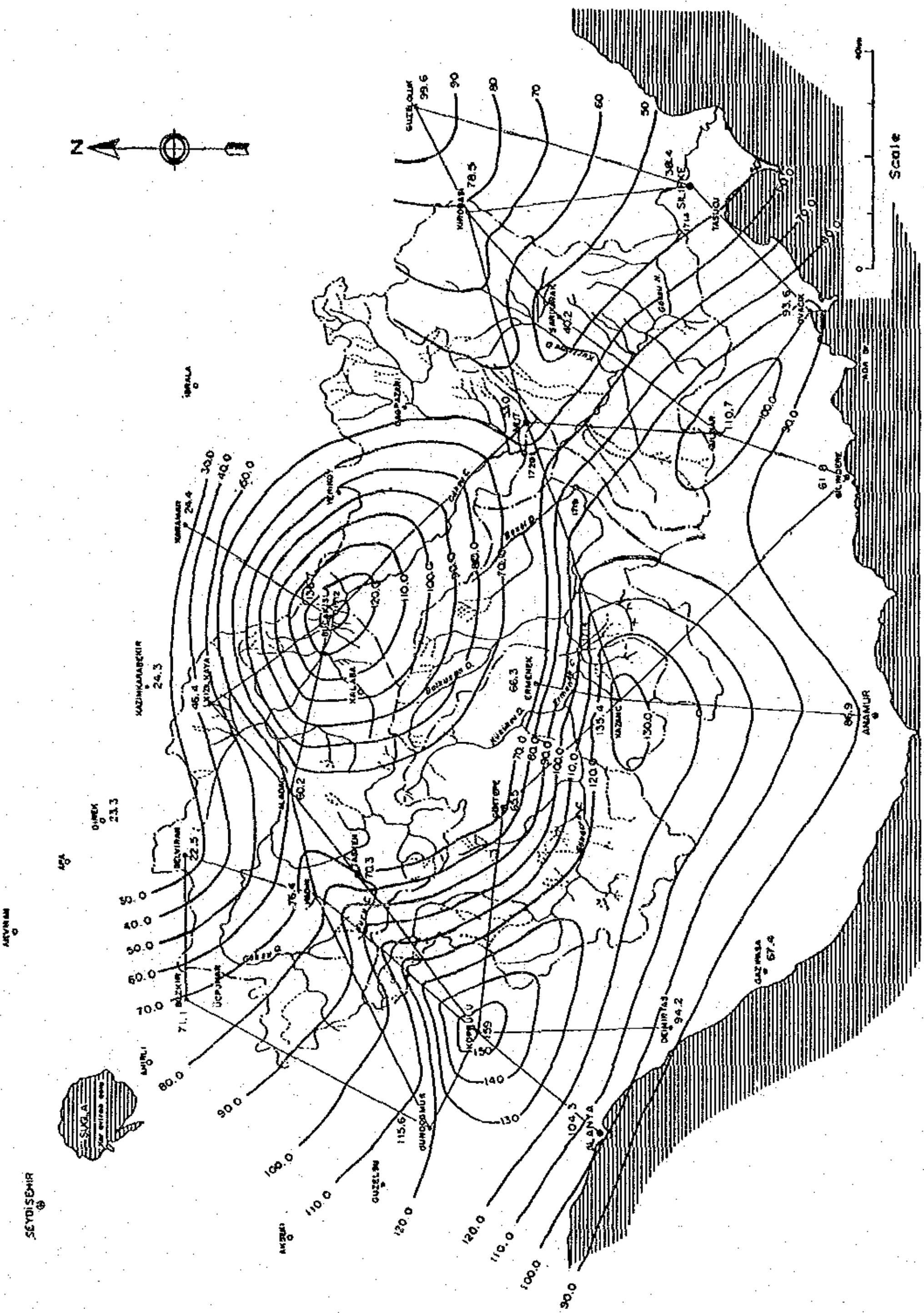



N. B. Ri – month = average value of monthly mean rainfalls of Goktepe, Ermenek and Kozanci (1965～1987)

Qi – month = monthly mean runoff at 17～14 (1965～1987)

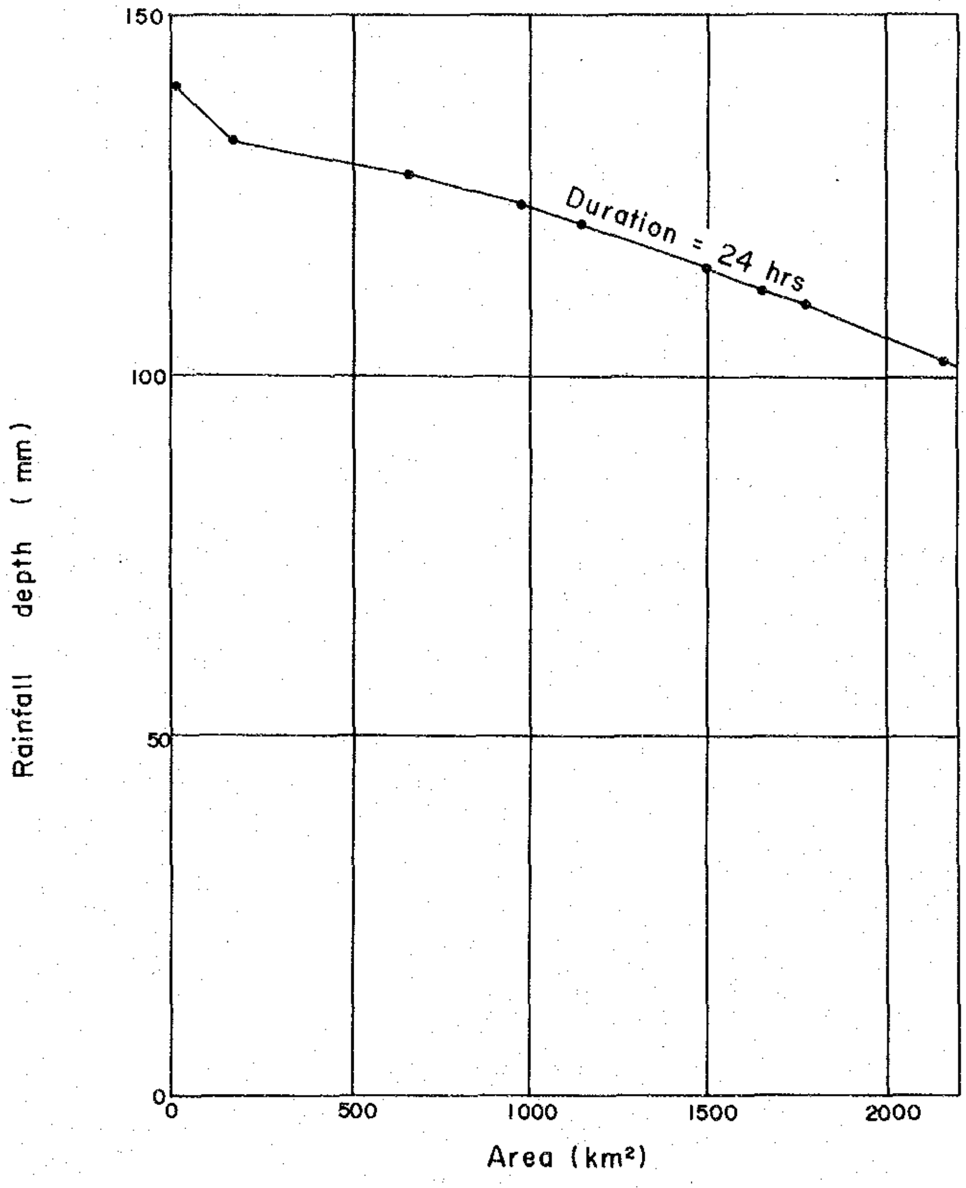
| | | |
|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C6<br>Monthly Precipitation and Runoff Patterns of the Ermenek Basin |

FIG. NO.



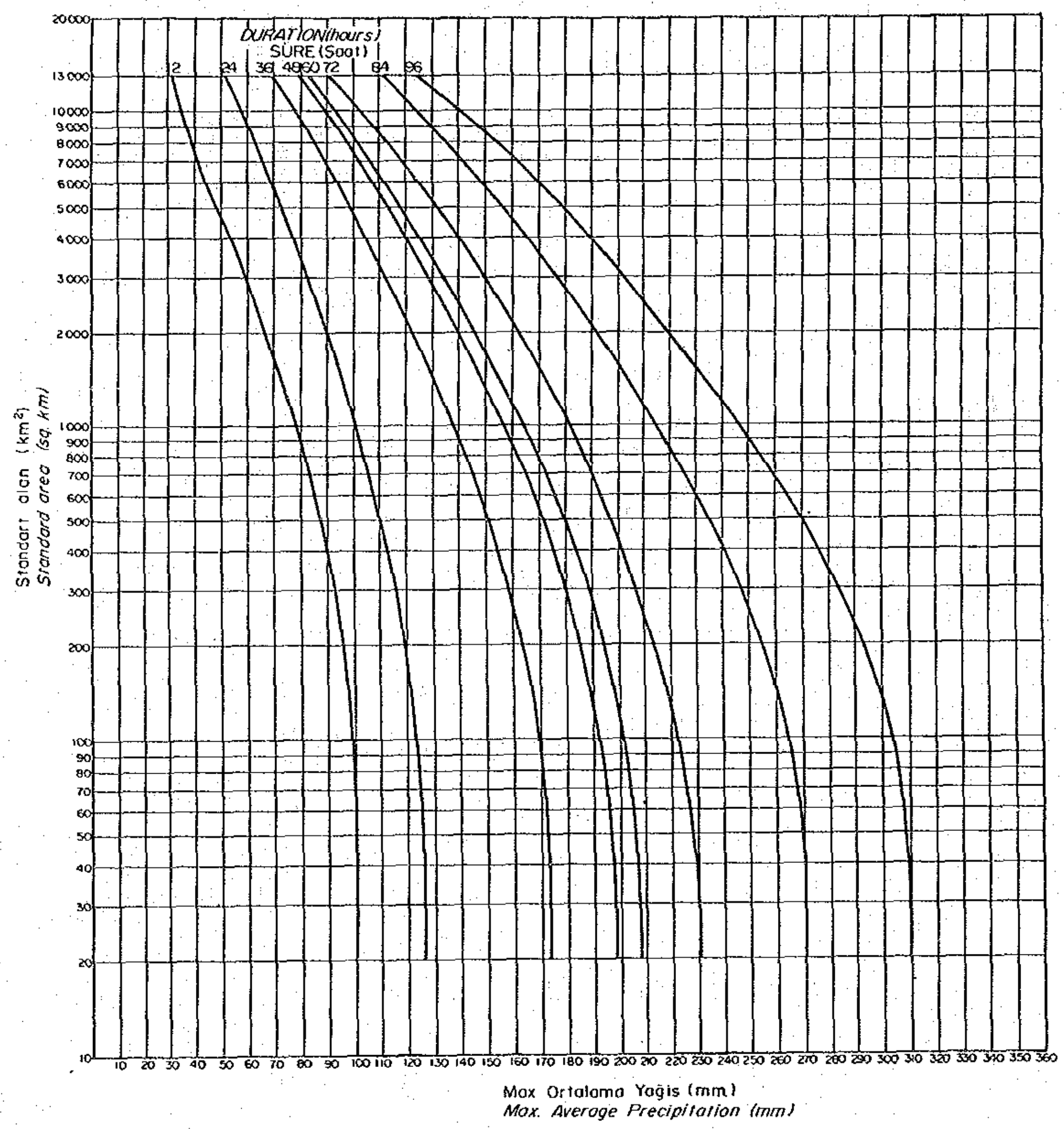
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C7<br>24-hr Isohyet During the Storm in 1975 |
|---|---|---|

FIG. NO.

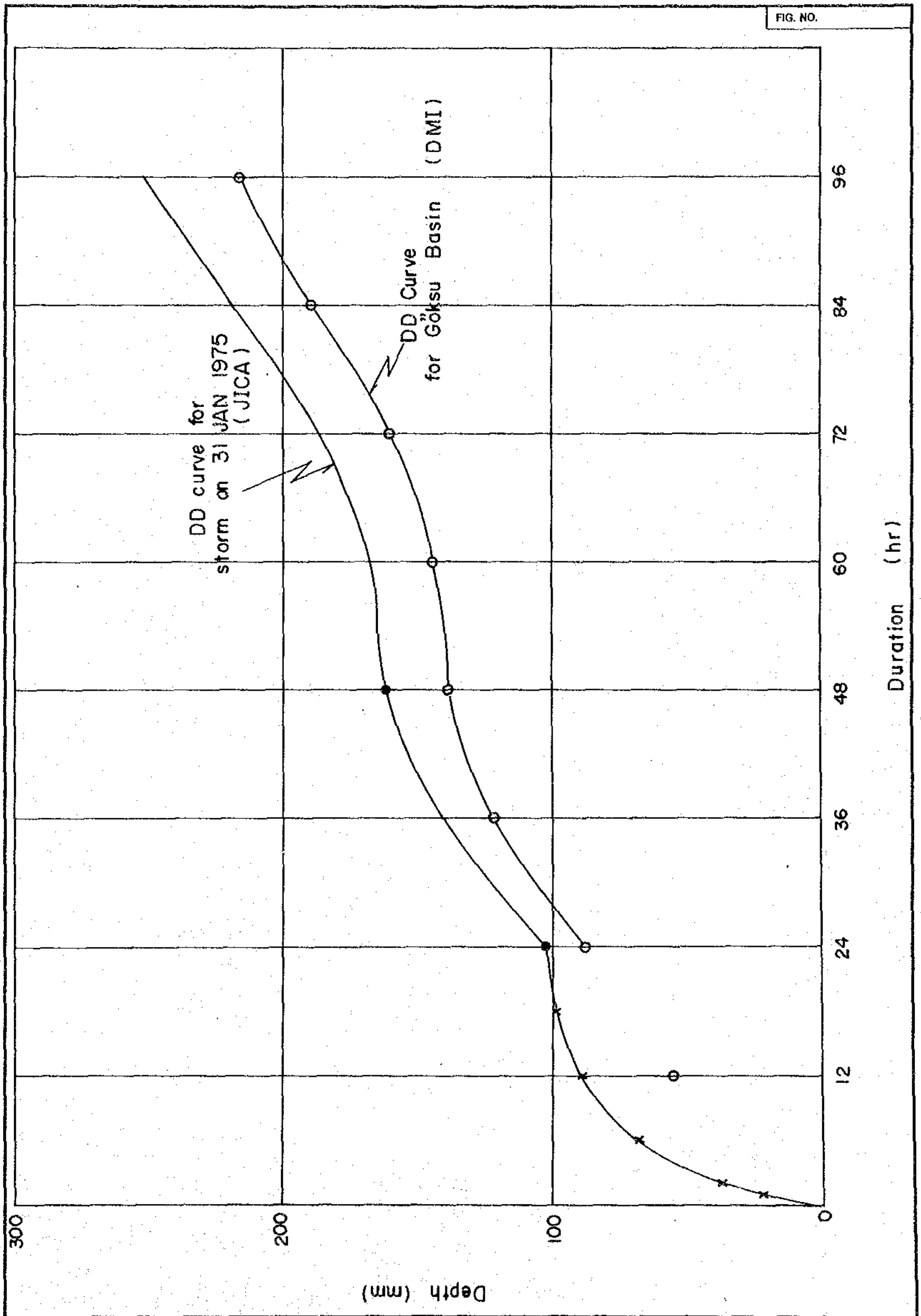



| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C8<br>Depth-Area Curve<br>of the Storm in 1975 |

FIG. NO.

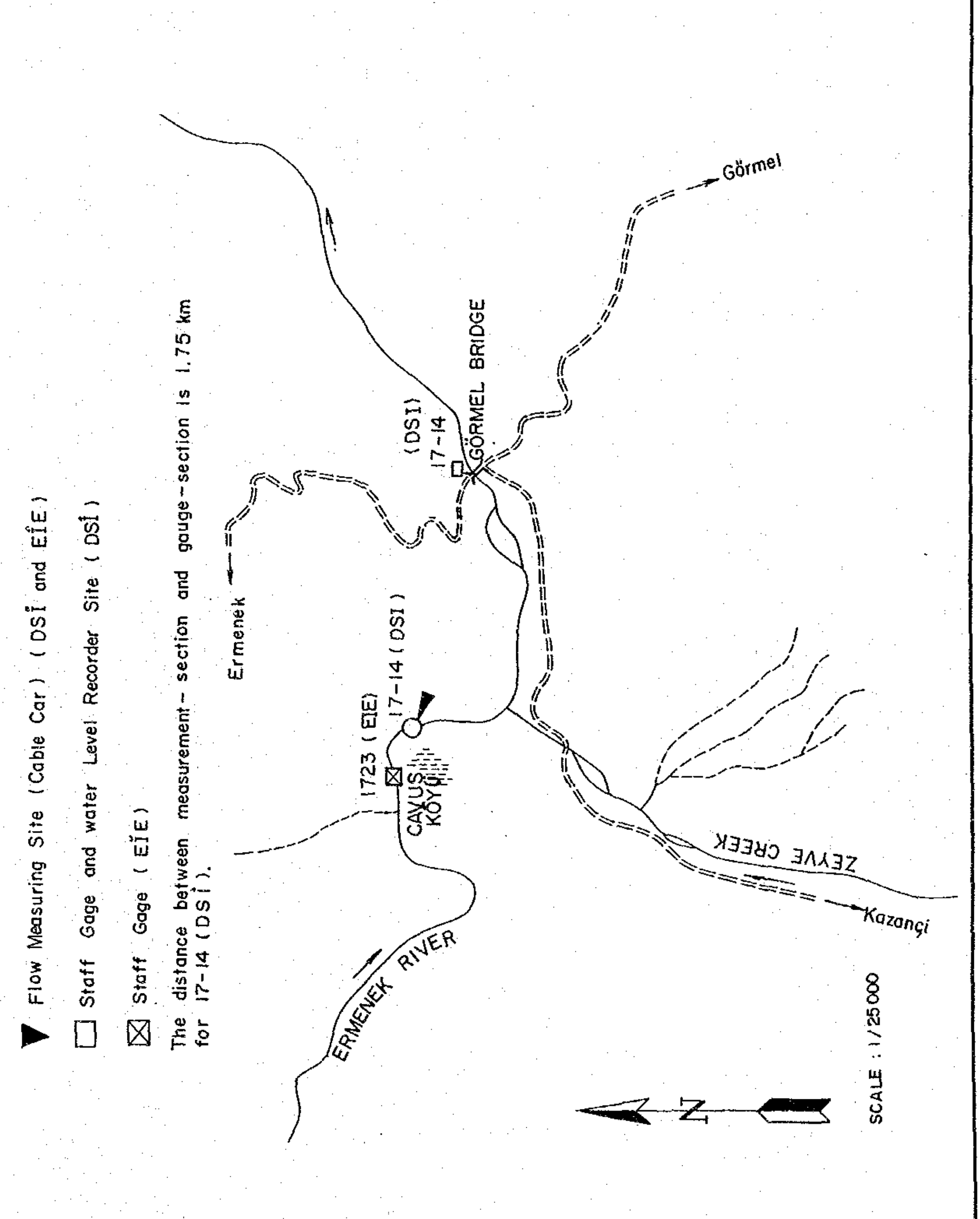



| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C9<br>Rainfall Depth-Area Duration Curves for Göksu River Basin by DMİ |



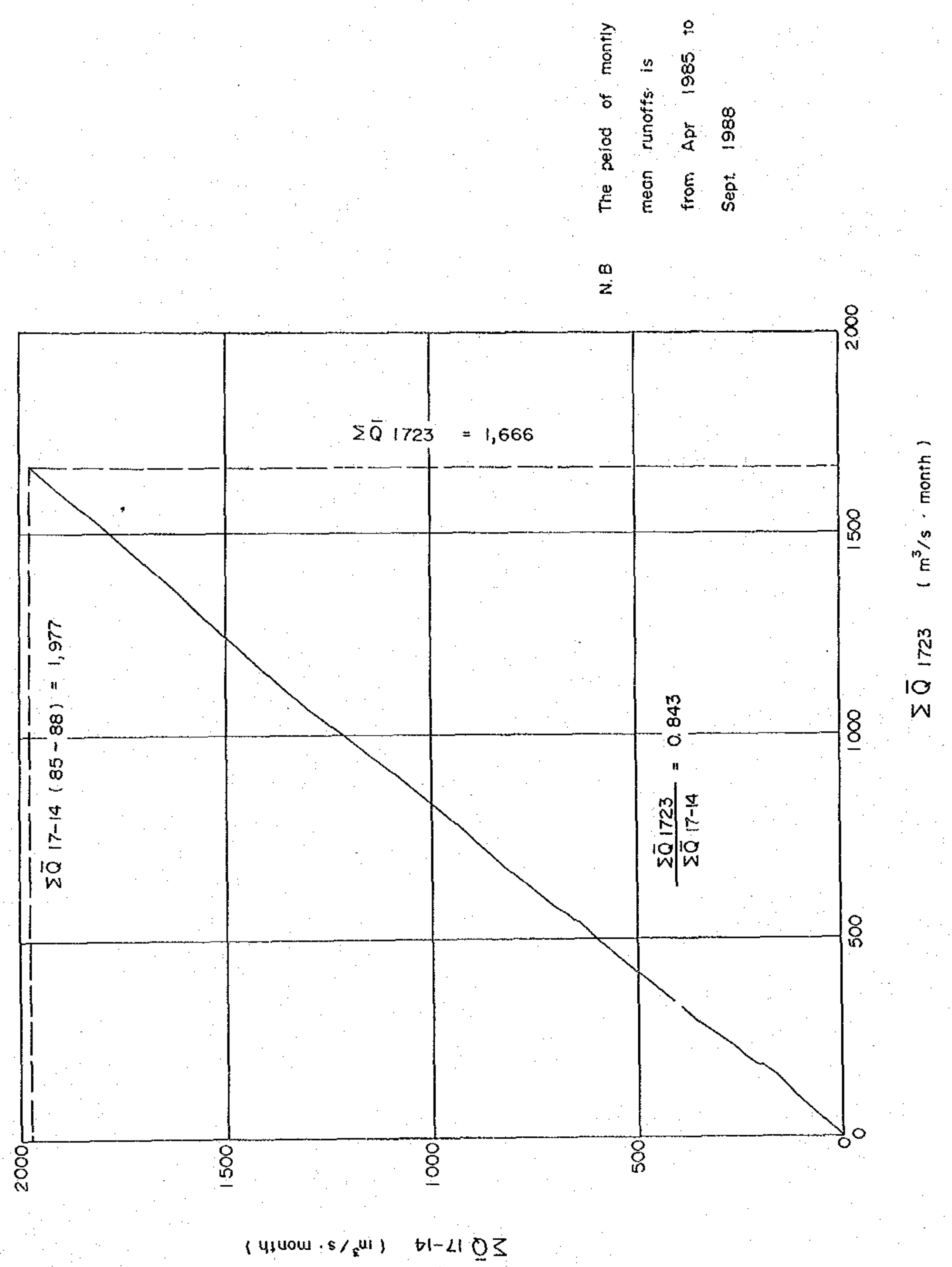

| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C10<br>Depth-Duration Curve<br>of the Storm in 1975 |

FIG. NO.



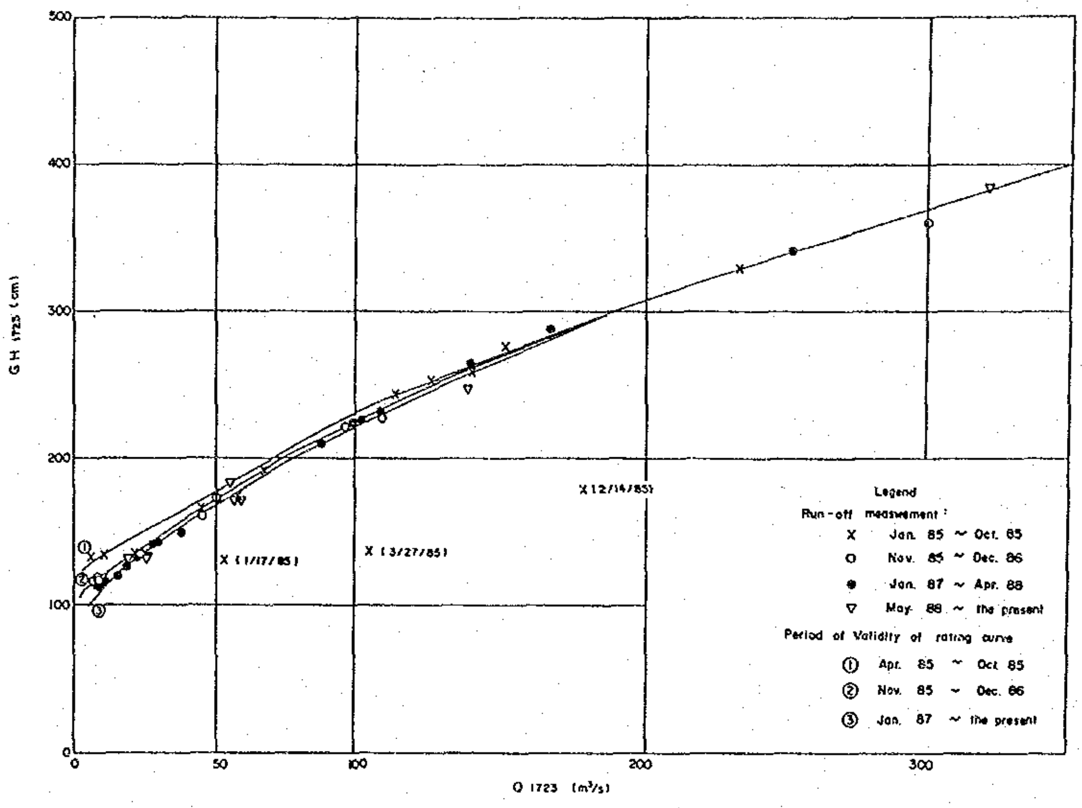

| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C11<br>Location Map of Steam Gauging Stations around the Dam Site |

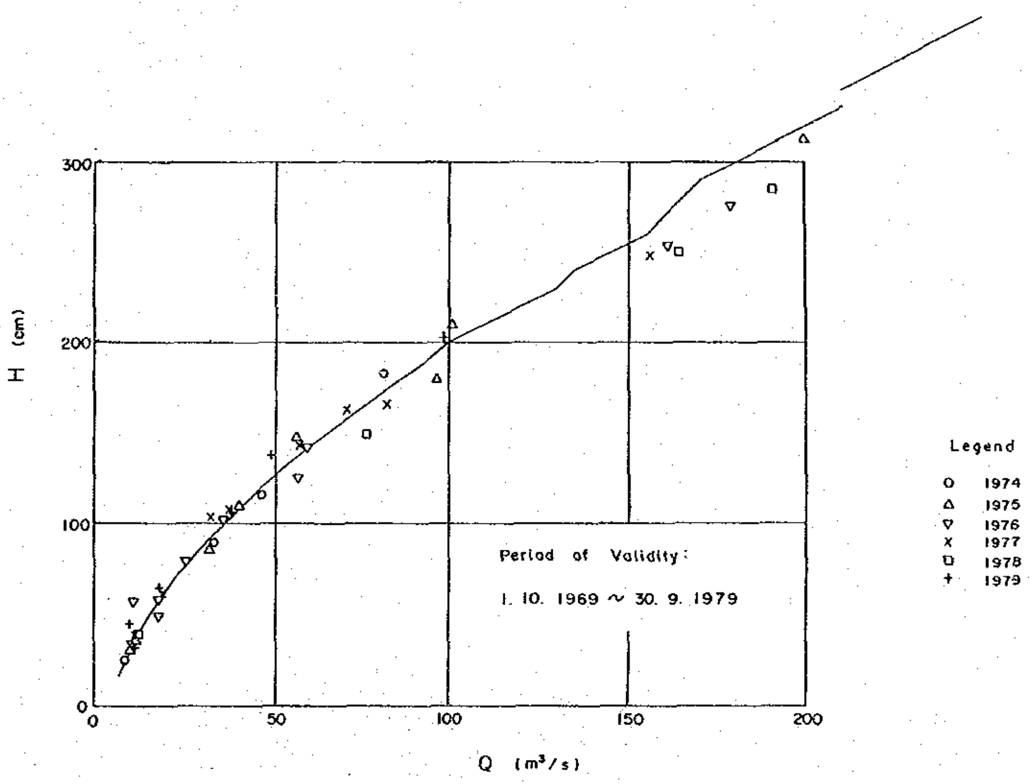
FIG. NO.




| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C12<br>Double Mass Curve of Runoffs at Stations 1723 and 17-14 |
|---|---|---|

FIG. NO.

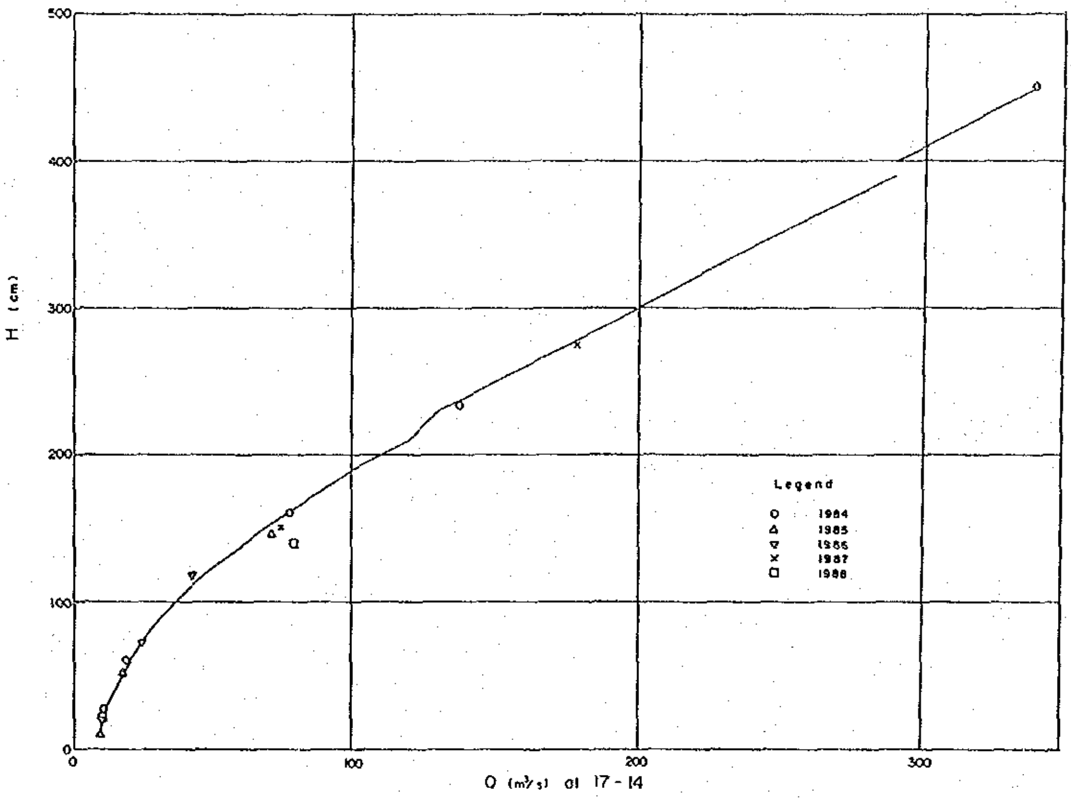


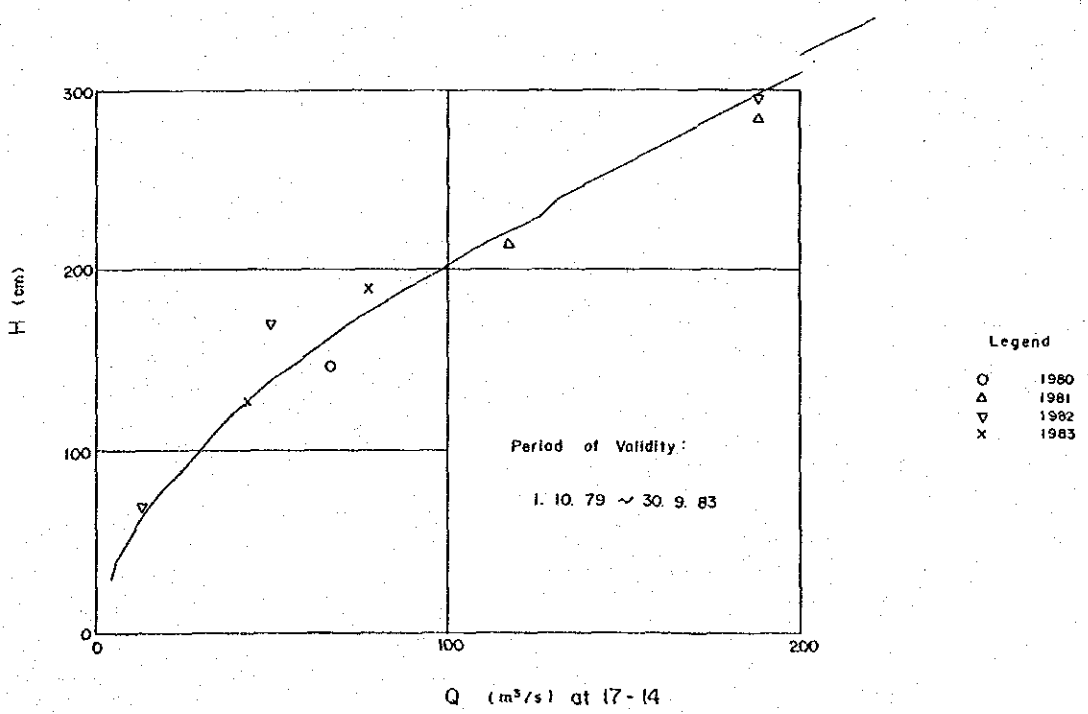




at 17-14

| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C13 (1/2)<br>Flow Rating Curves<br>of Stations 1723 and 17-14 |

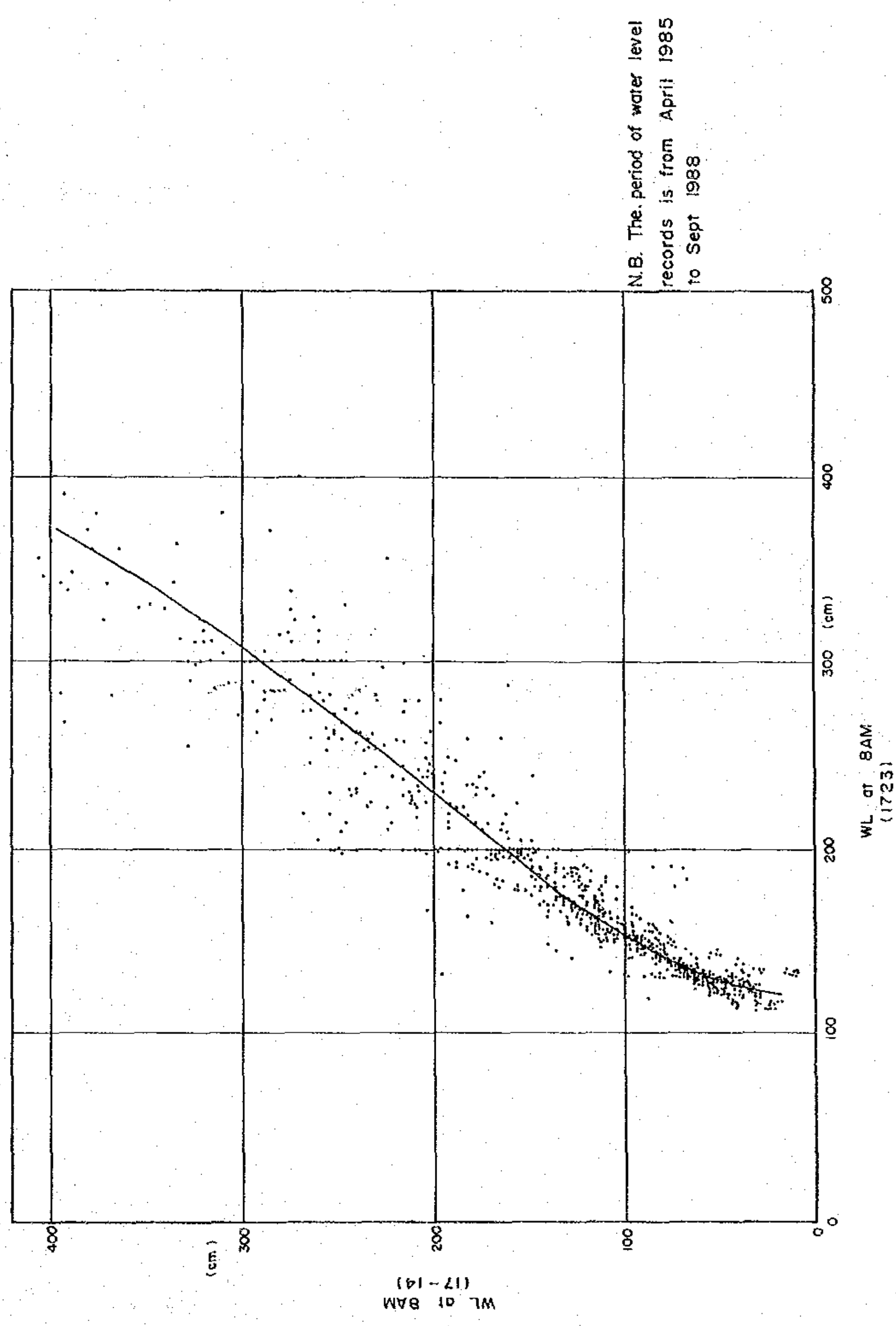
FIG. NO.




| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C13 (2/2)<br>Flow Rating Curves<br>of Stations 1723 and 17-14 |

FIG. NO.




THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜGÜ

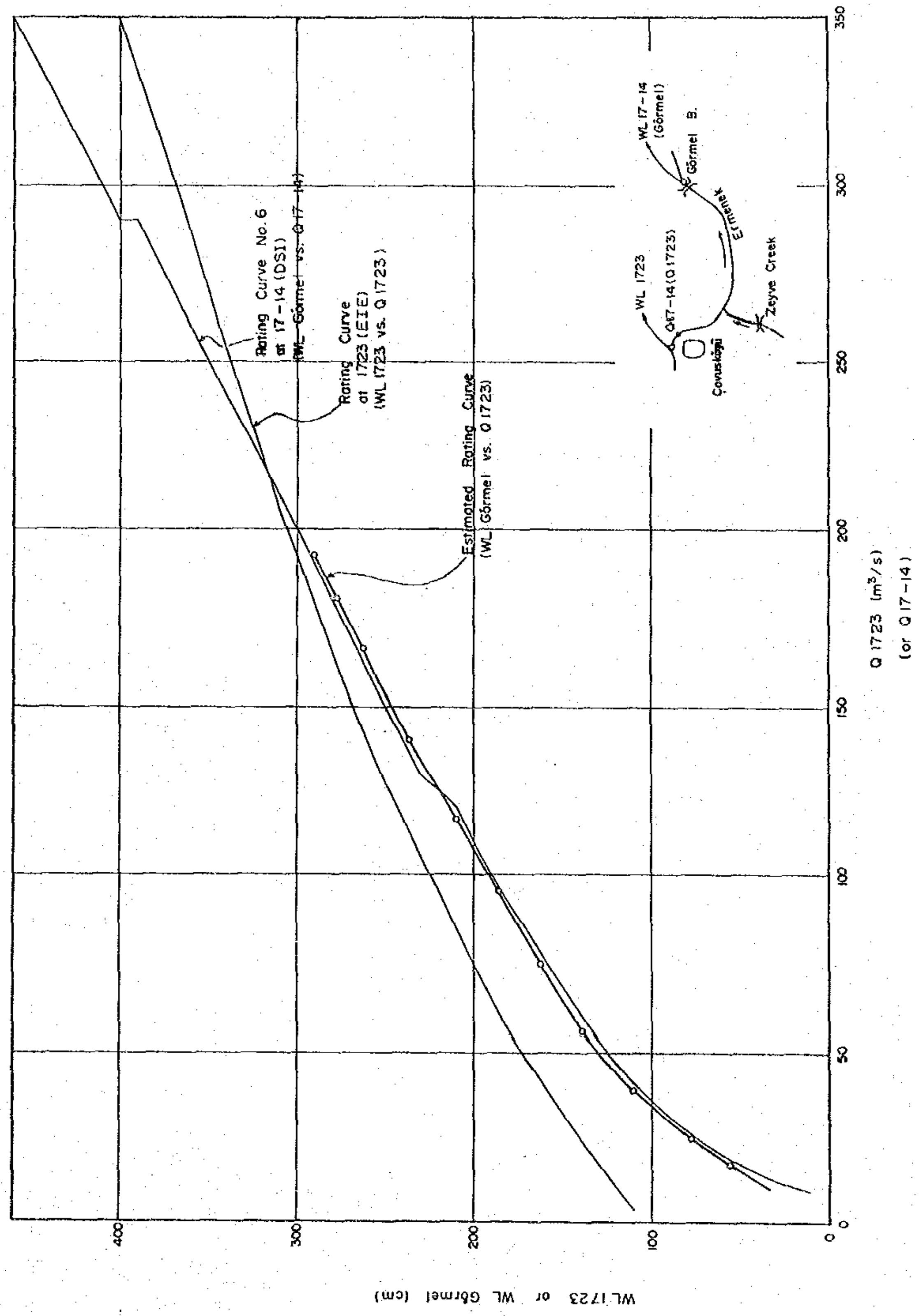
ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

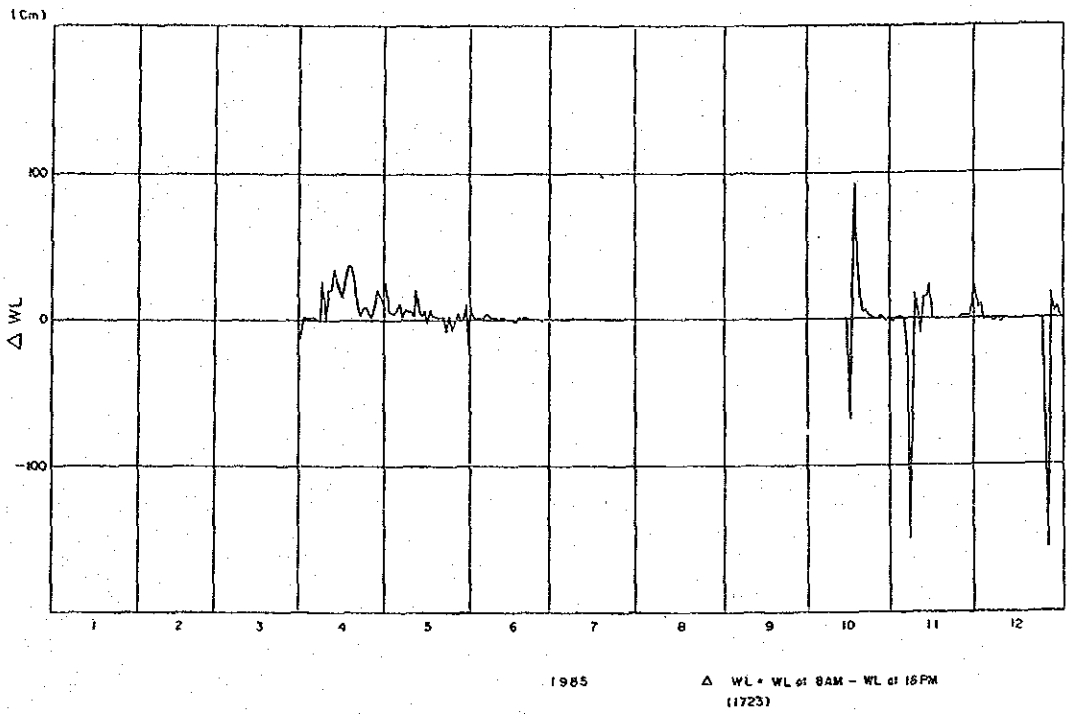
Fig. C14
Relationship of Gauge Heights between Stations 17-14 and 1723

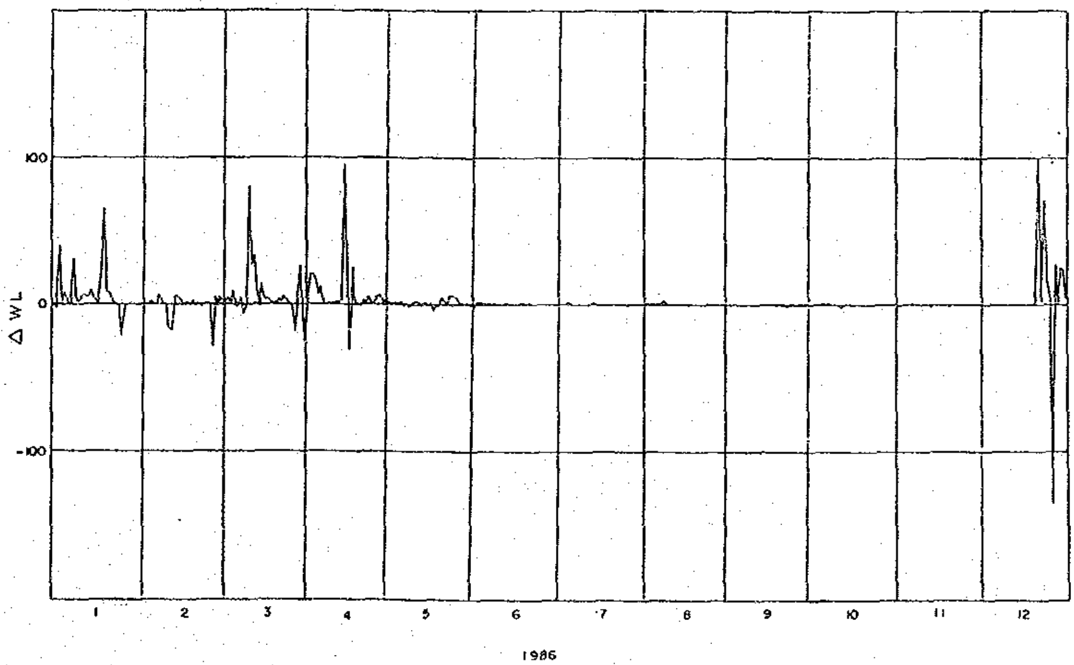
FIG. NO.




| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C15<br>Comparison of Rating Curves between Stations 17-14 and 1723 |
|---|---|---|

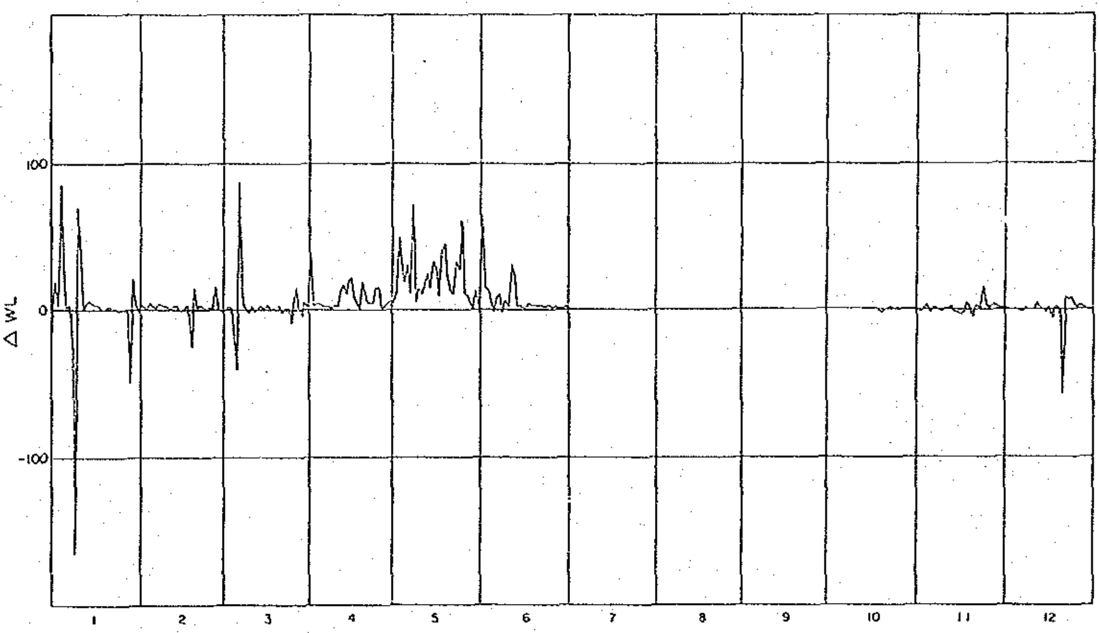
FIG. NO.

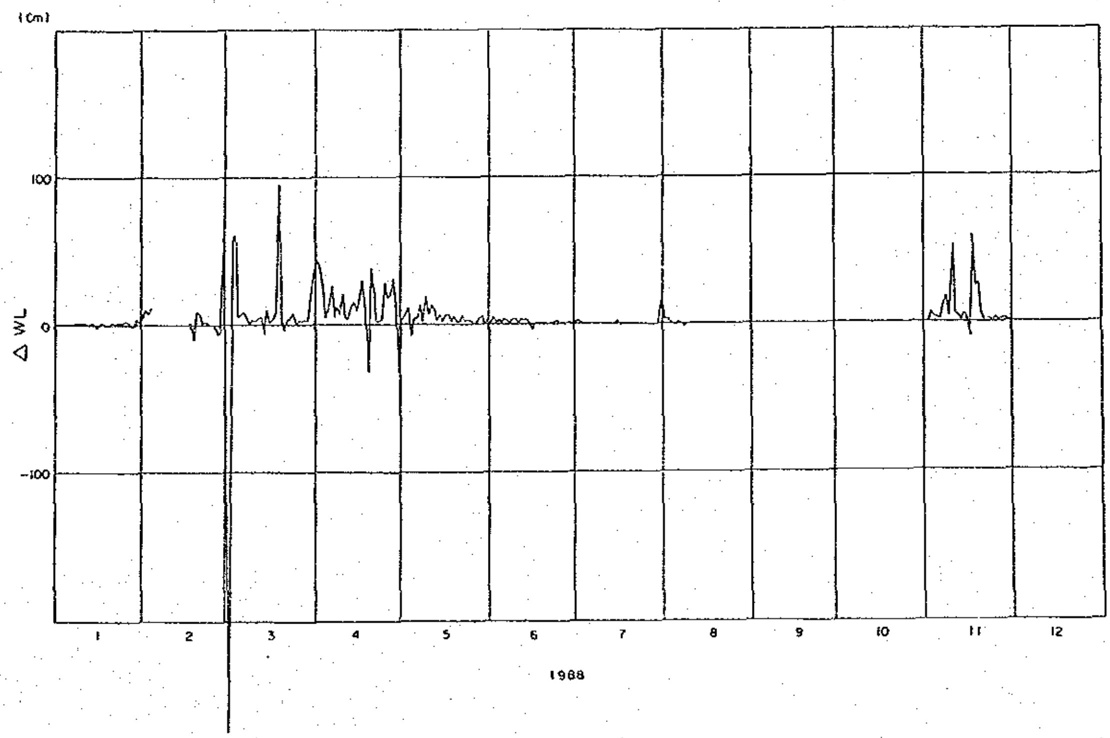



| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C16 (1/2)<br>Water Level Changes at Station 1723 between 8:00 AM and 4:00 PM |

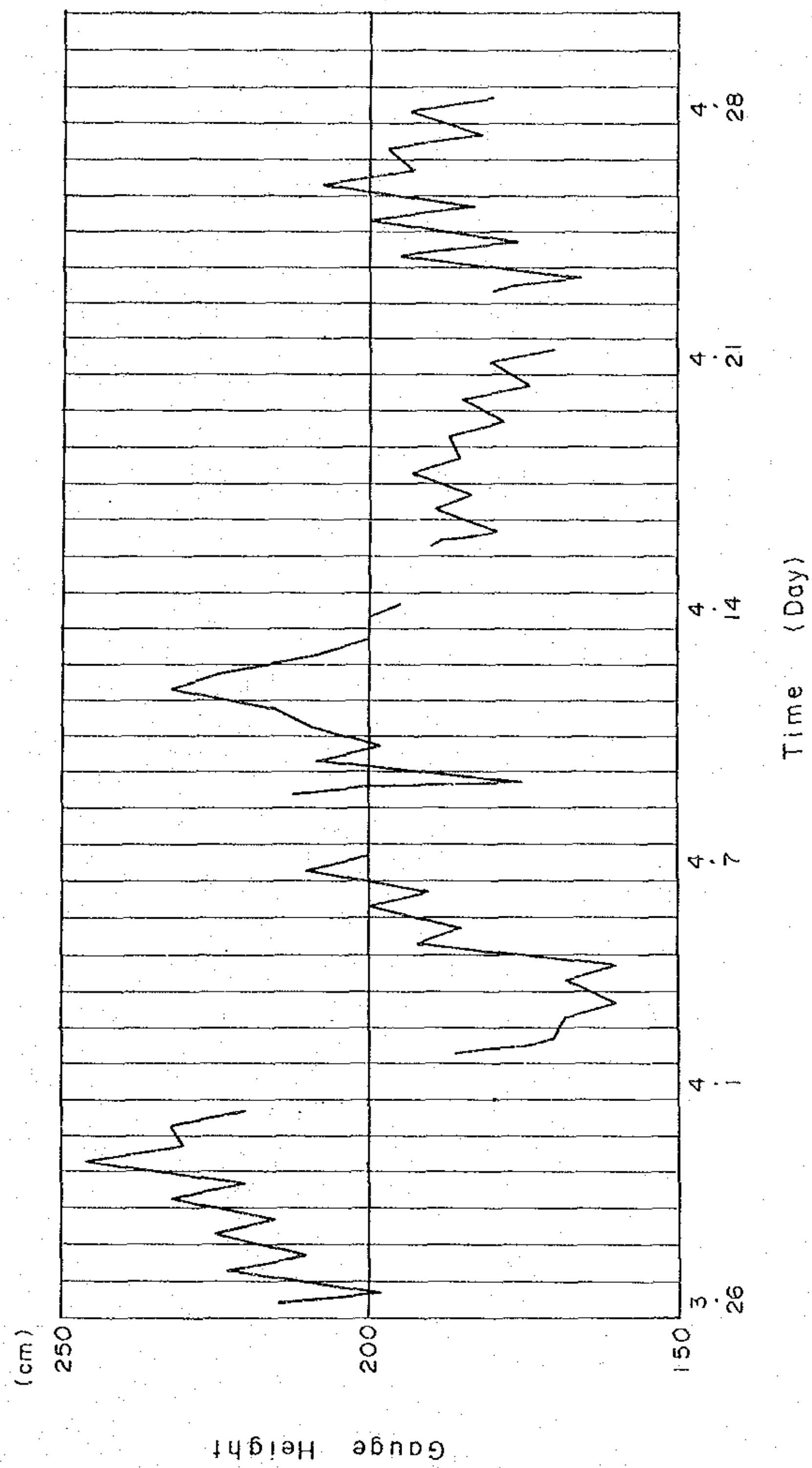
FIG. NO.



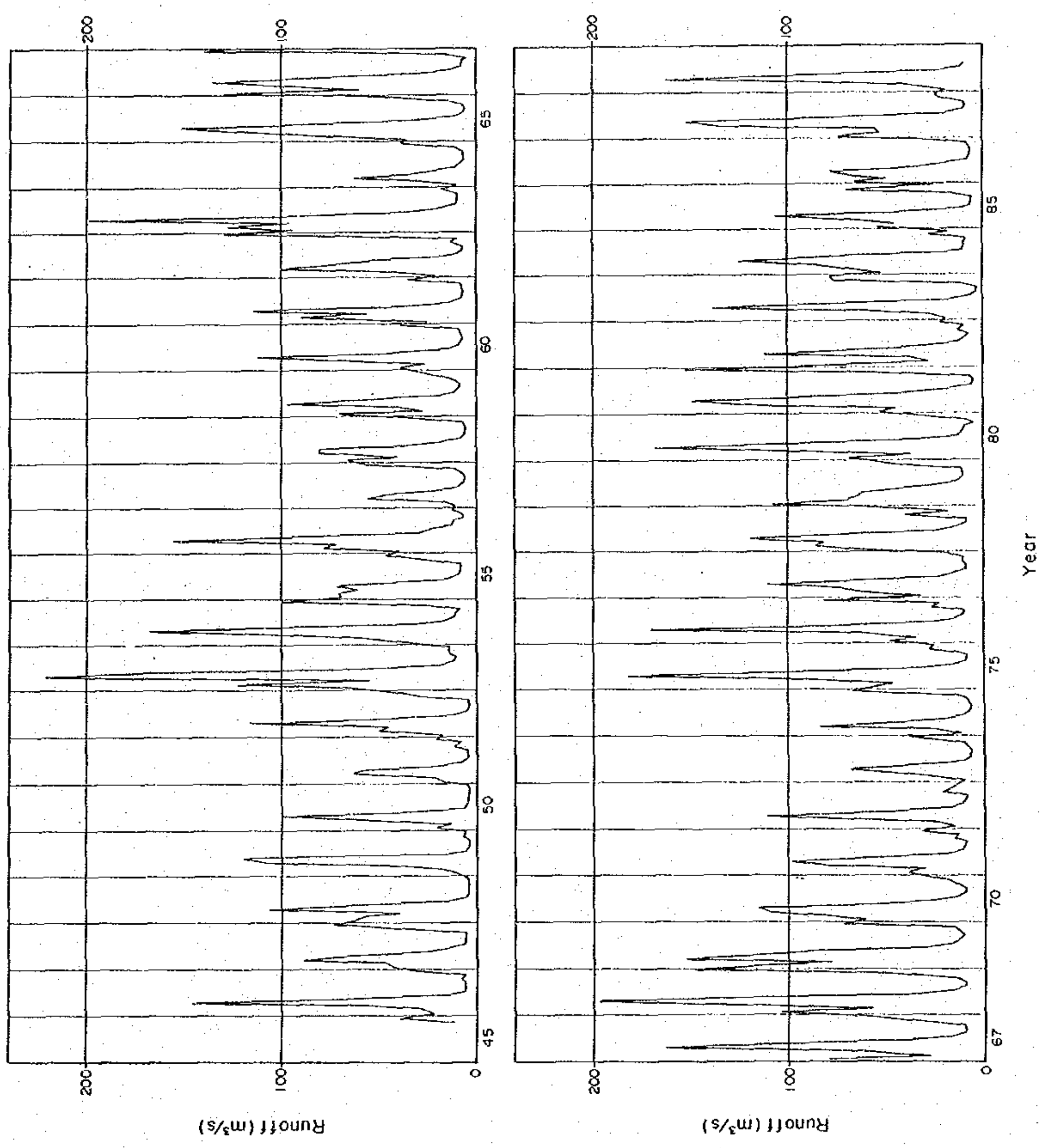

| | | |
|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C16 (2/2)<br>Water Level Changes at Station 1723 between 8:00 AM and 4:00 PM |

FIG. NO.




| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C17<br>Water Level Changes near Station 17-14 between 7:00 AM and 5:00 PM |

FIG. NO.



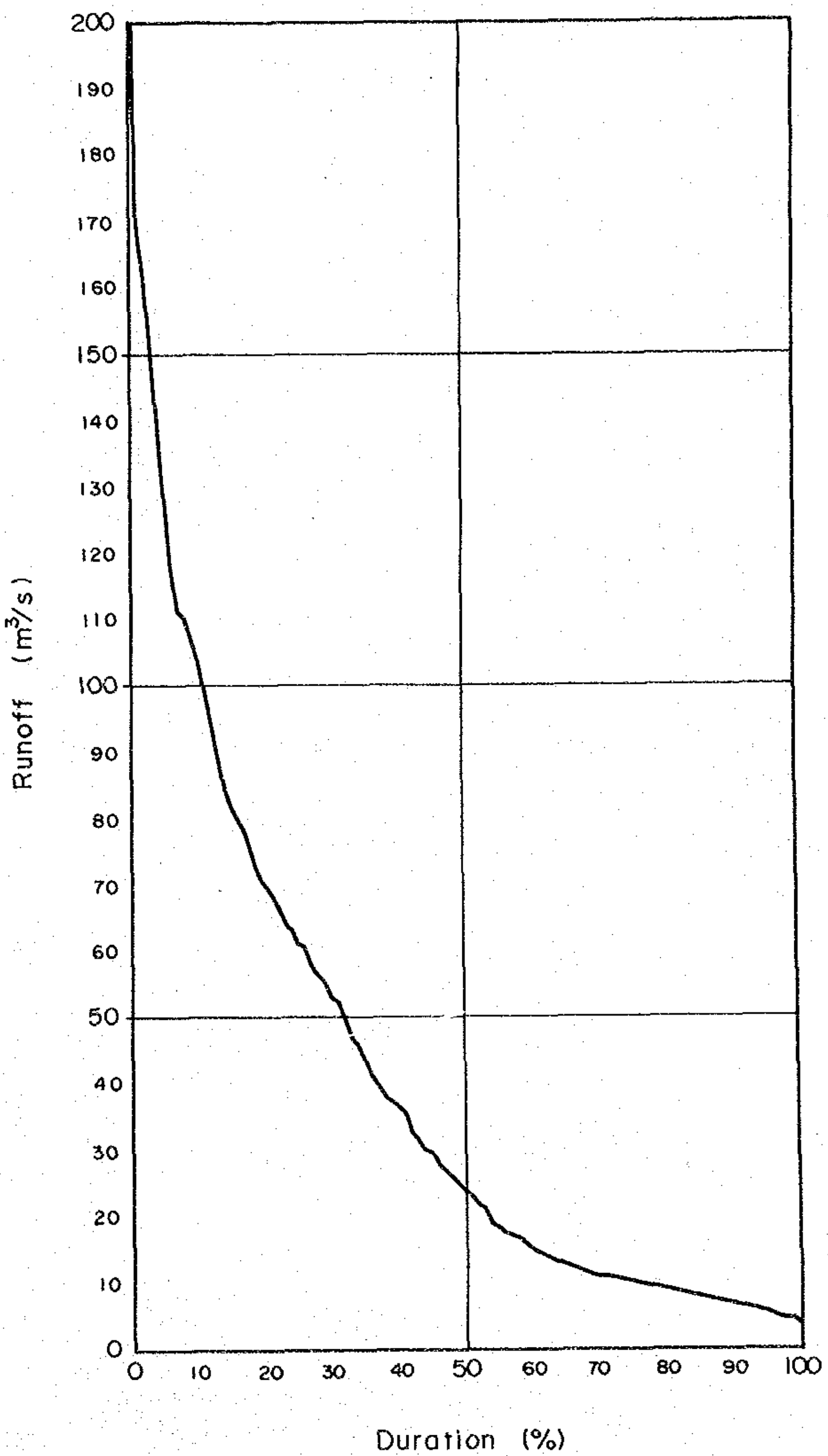

THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜĞÜ

ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

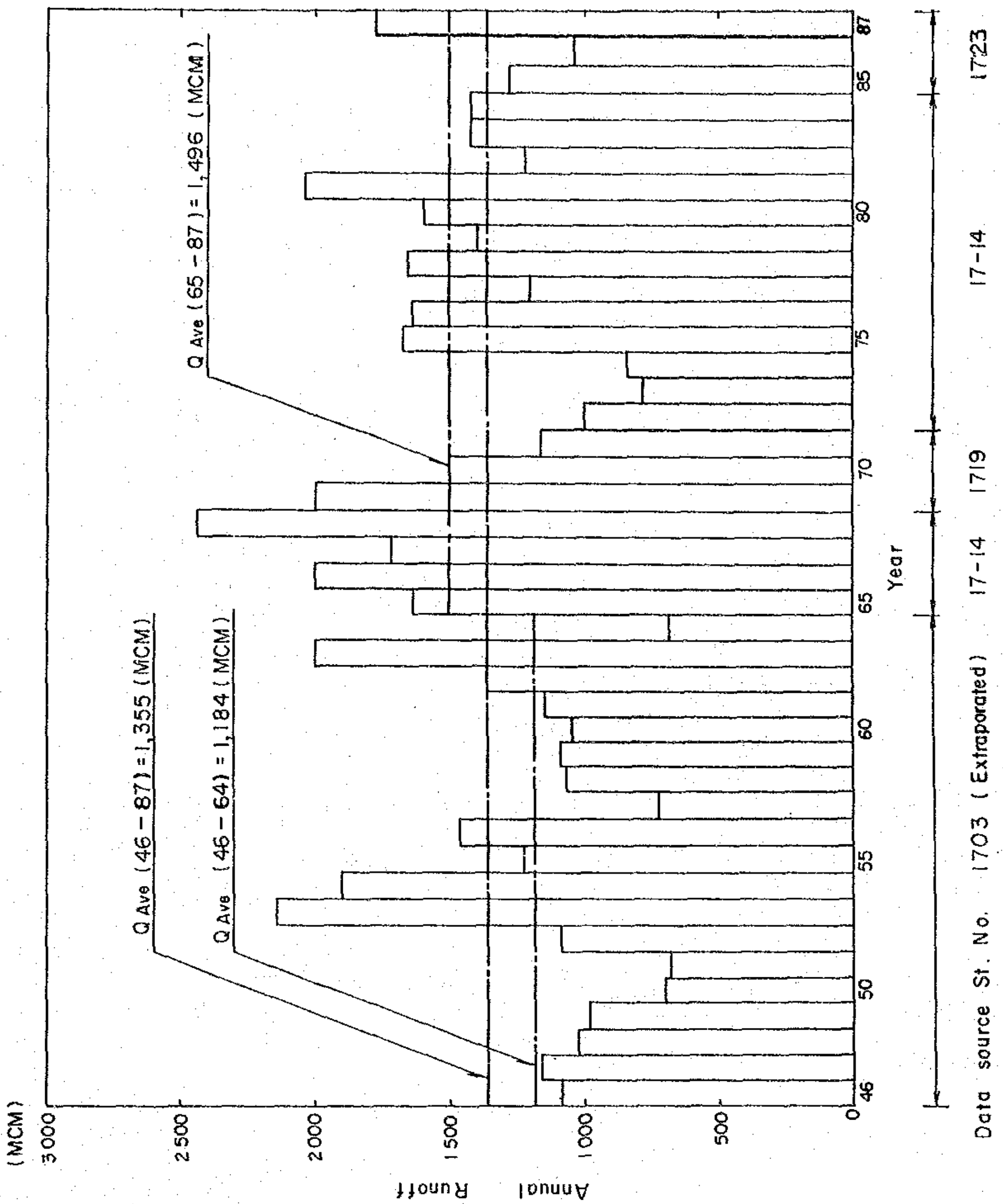
TITLE

Fig. C18
Historical changes of Estimated Monthly Runoff at 17-14



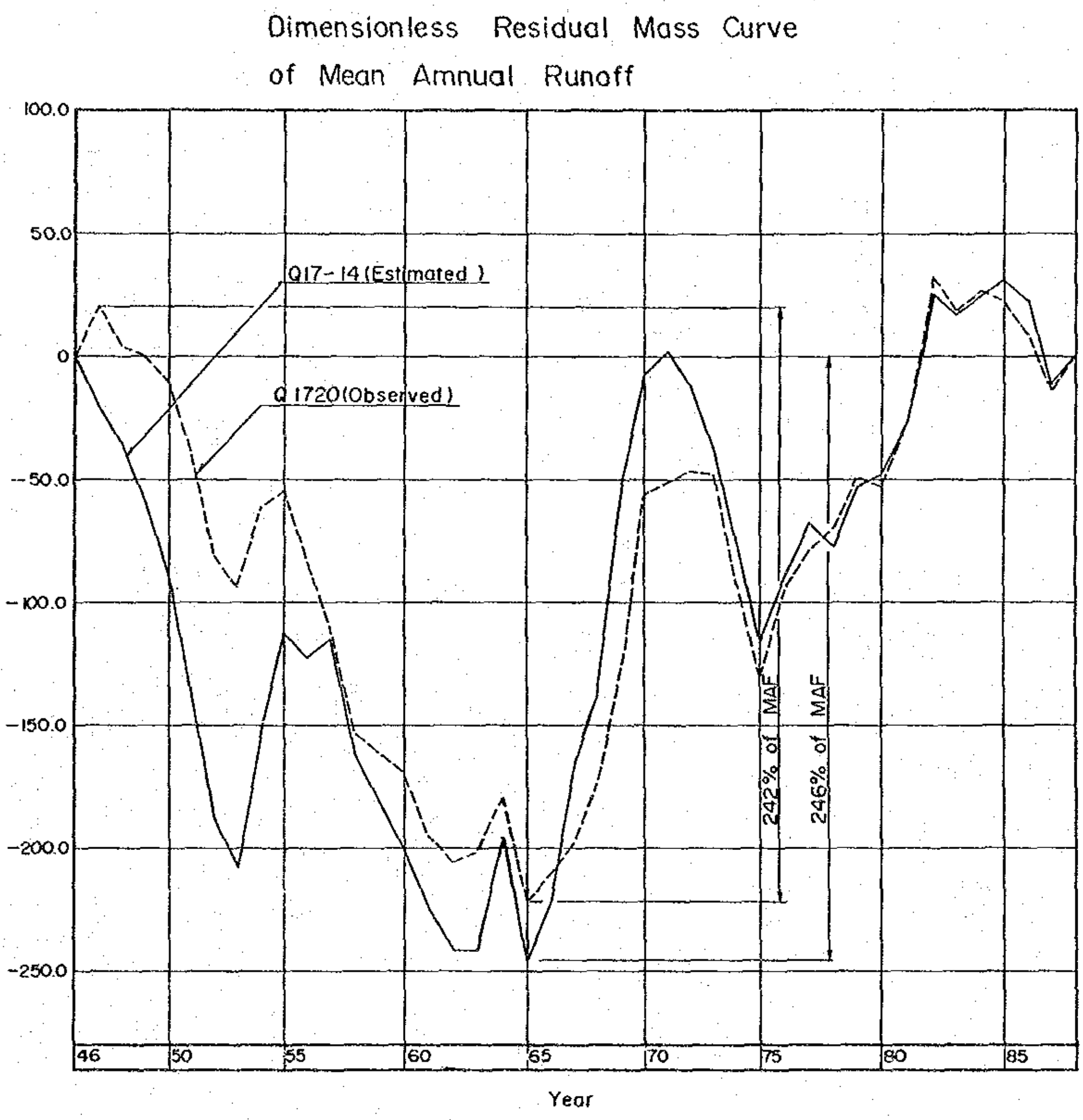

| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C19<br>Duration Curve of Estimated Monthly Runoff at 17-14 |

FIG. NO.




| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C20<br>Historical Changes of Estimated Annual Inflow Series |

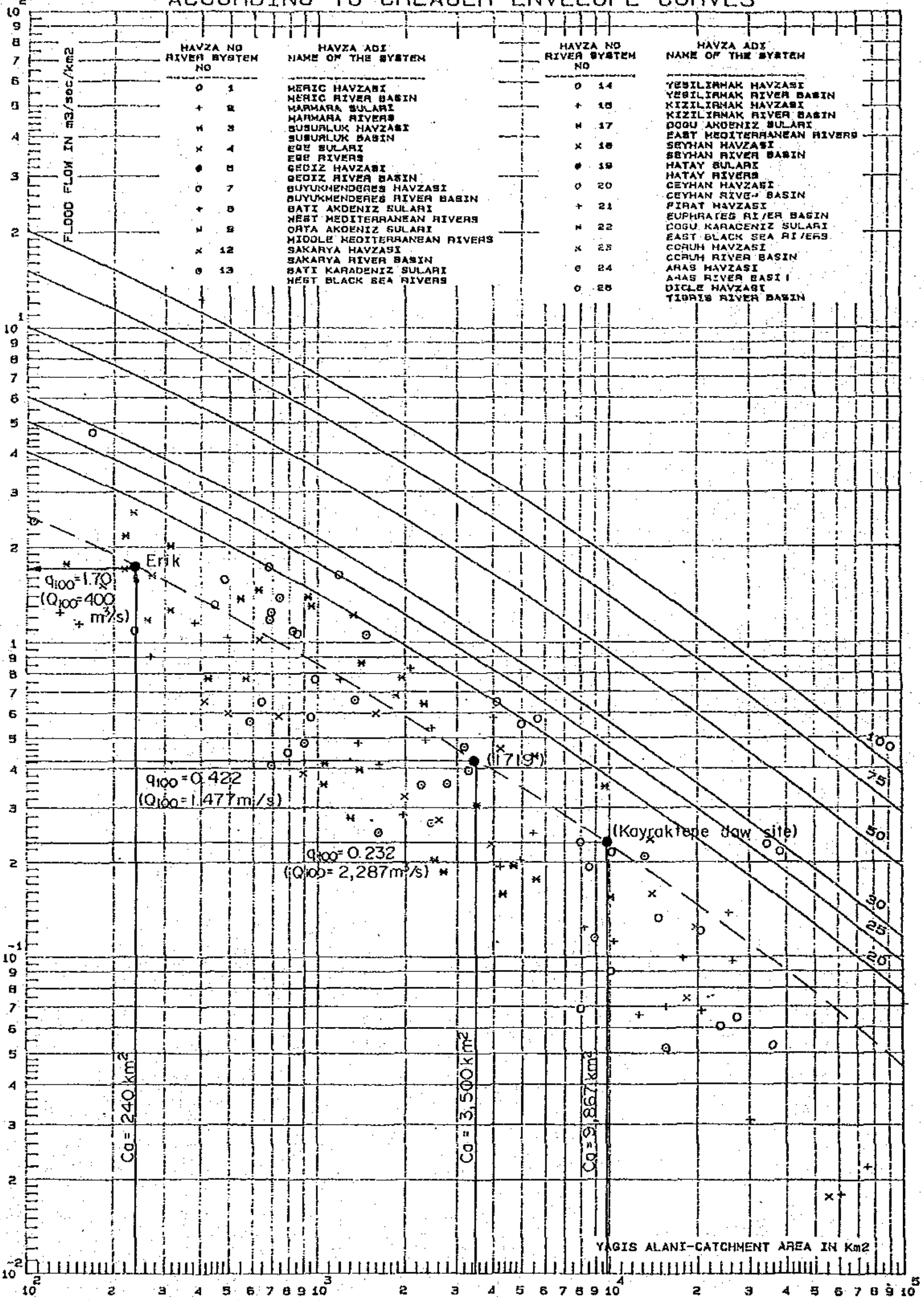
FIG. NO.




| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C21<br>Comparison of Dimensionless Mass Curves between Stations 17-14 and 1720 |

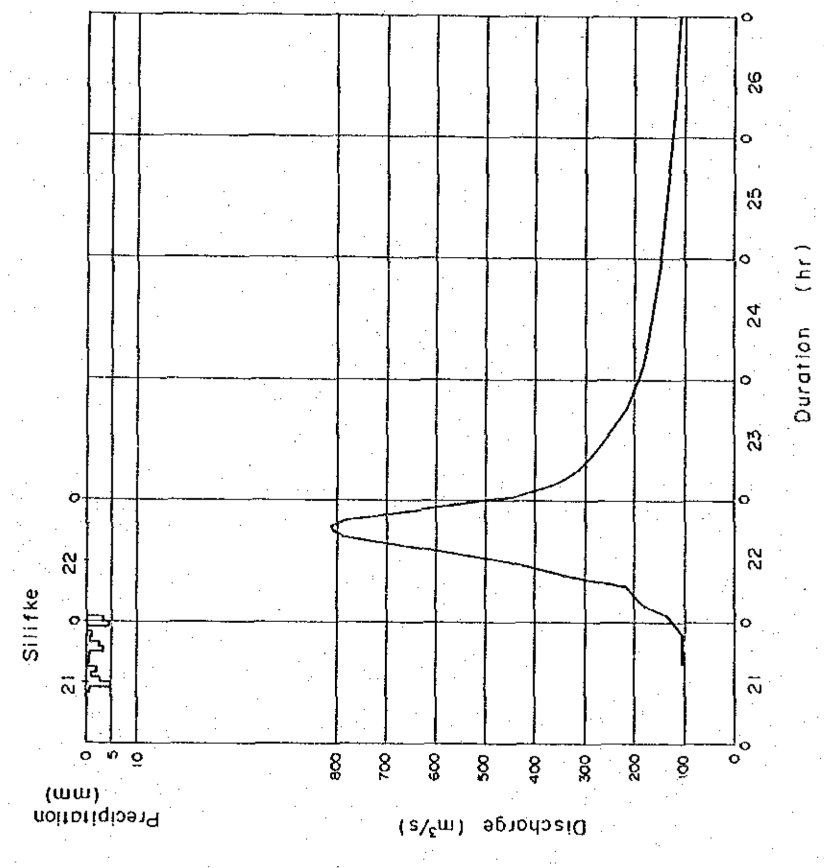
FIG. NO.

RECORDED MAXIMUM FLOOD FLOWS
AT SOME HYDROMETRIC STATIONS IN TURKEY
ACCORDING TO CREAGER ENVELOPE CURVES




| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE | Fig. C22<br>Creager Envelop Curve for Estimation of 100-year Flood at Erik Intate |

FIG. NO.




THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜĞÜ

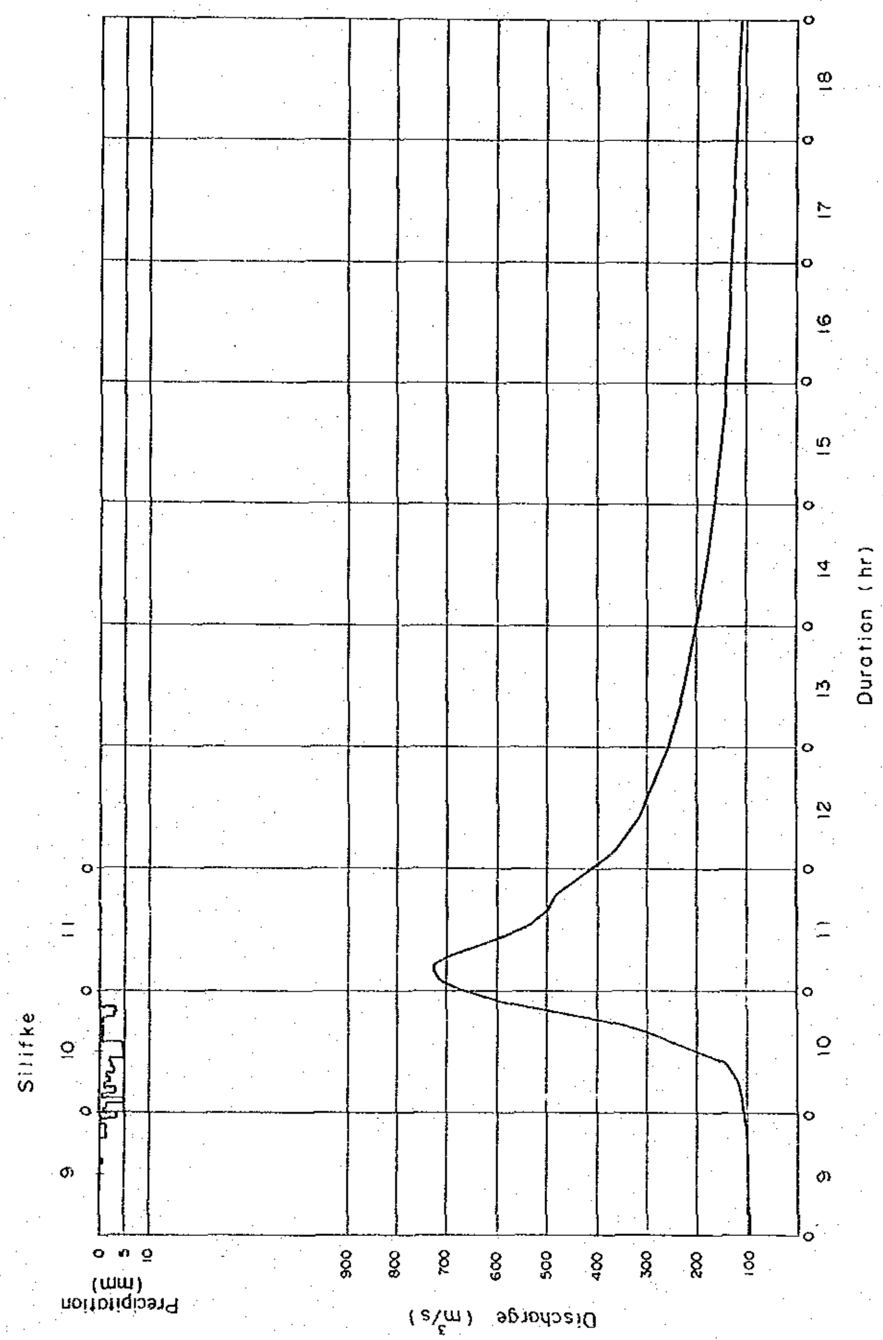
ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

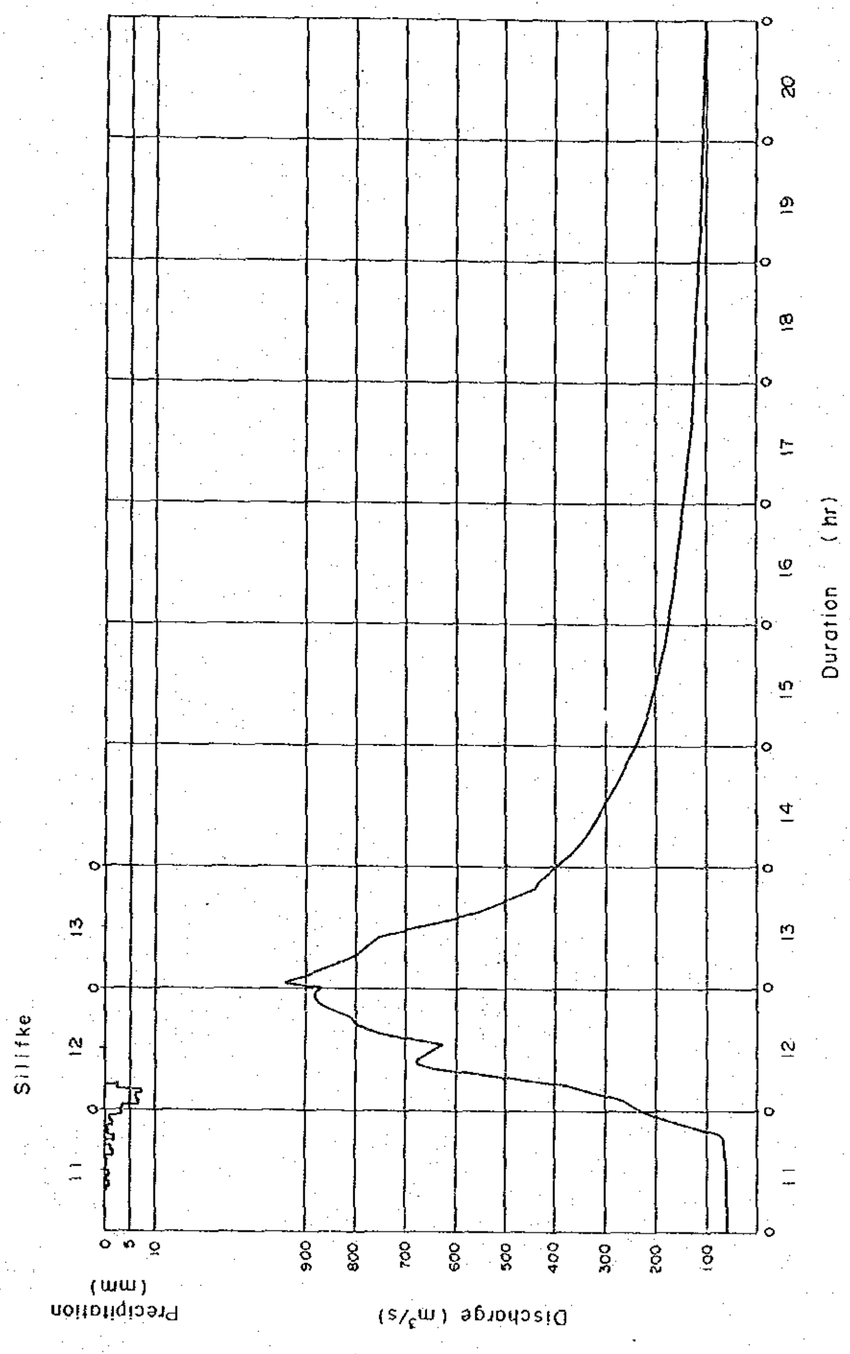
Fig. C23
Flood Hydrograph of Göksu River at 1714 in De. 1967

FIG. NO.



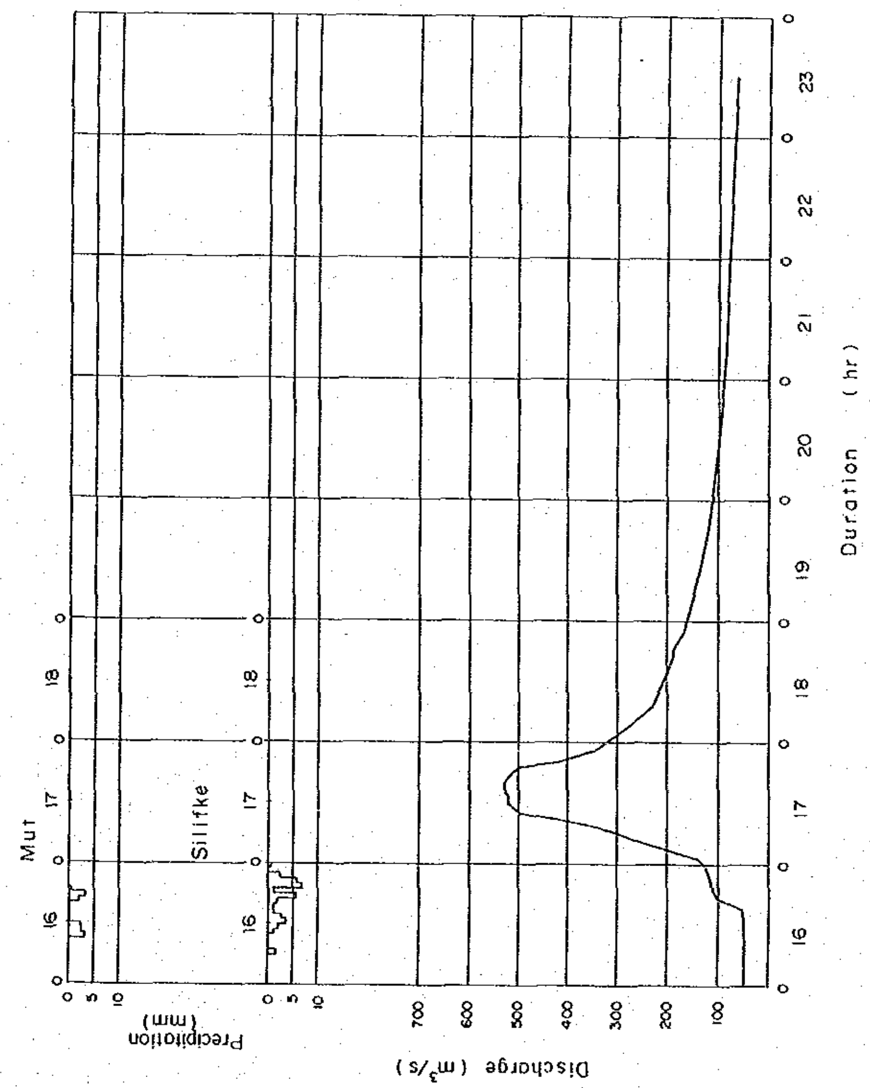

| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C24<br>Flood Hydrograph of Göksu River at 1714 in Jan. 1971 |
|---|---|---|

FIG. NO.



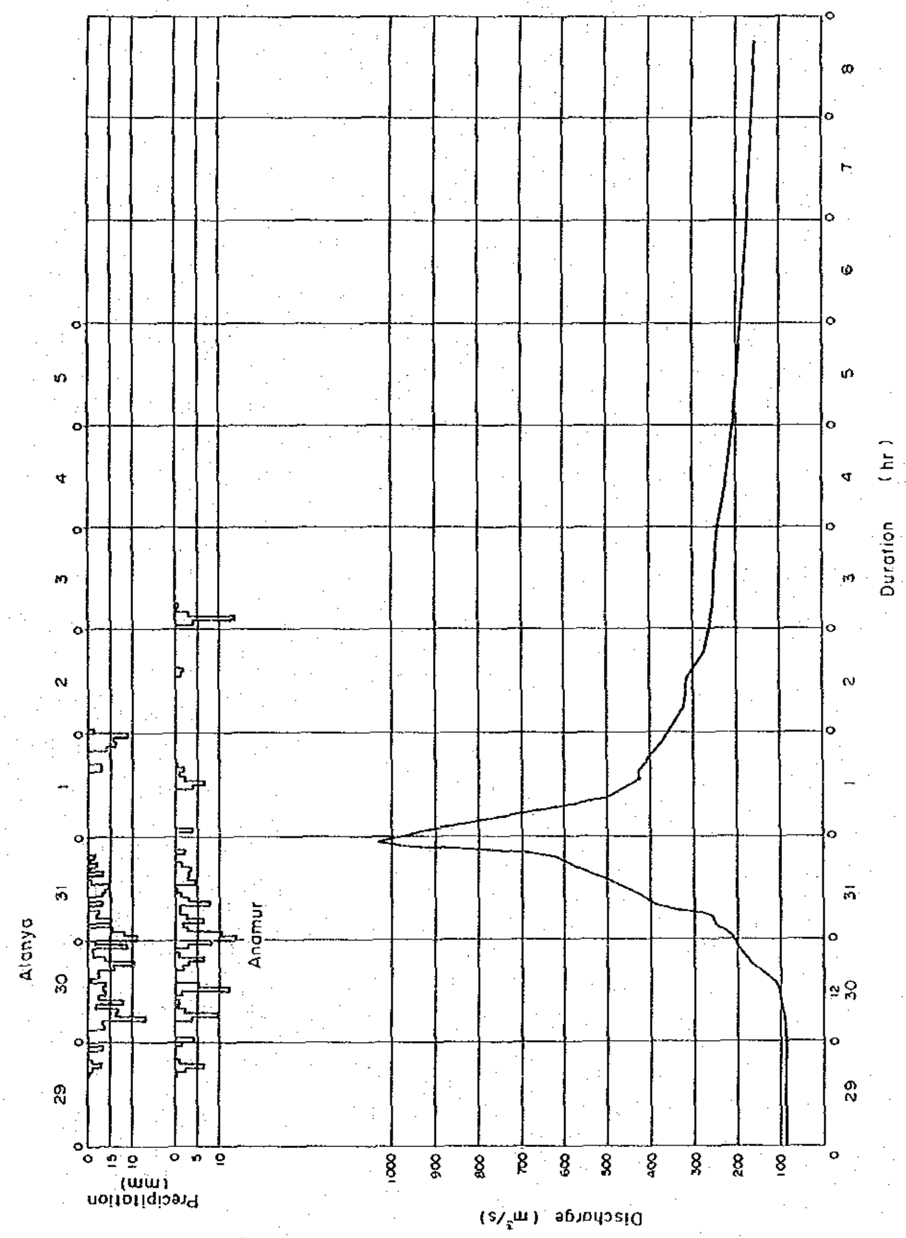

| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE | Fig. C25<br>Flood Hydrograph of Göksu River at 1714 in Dec. 1971 |

FIG. NO.



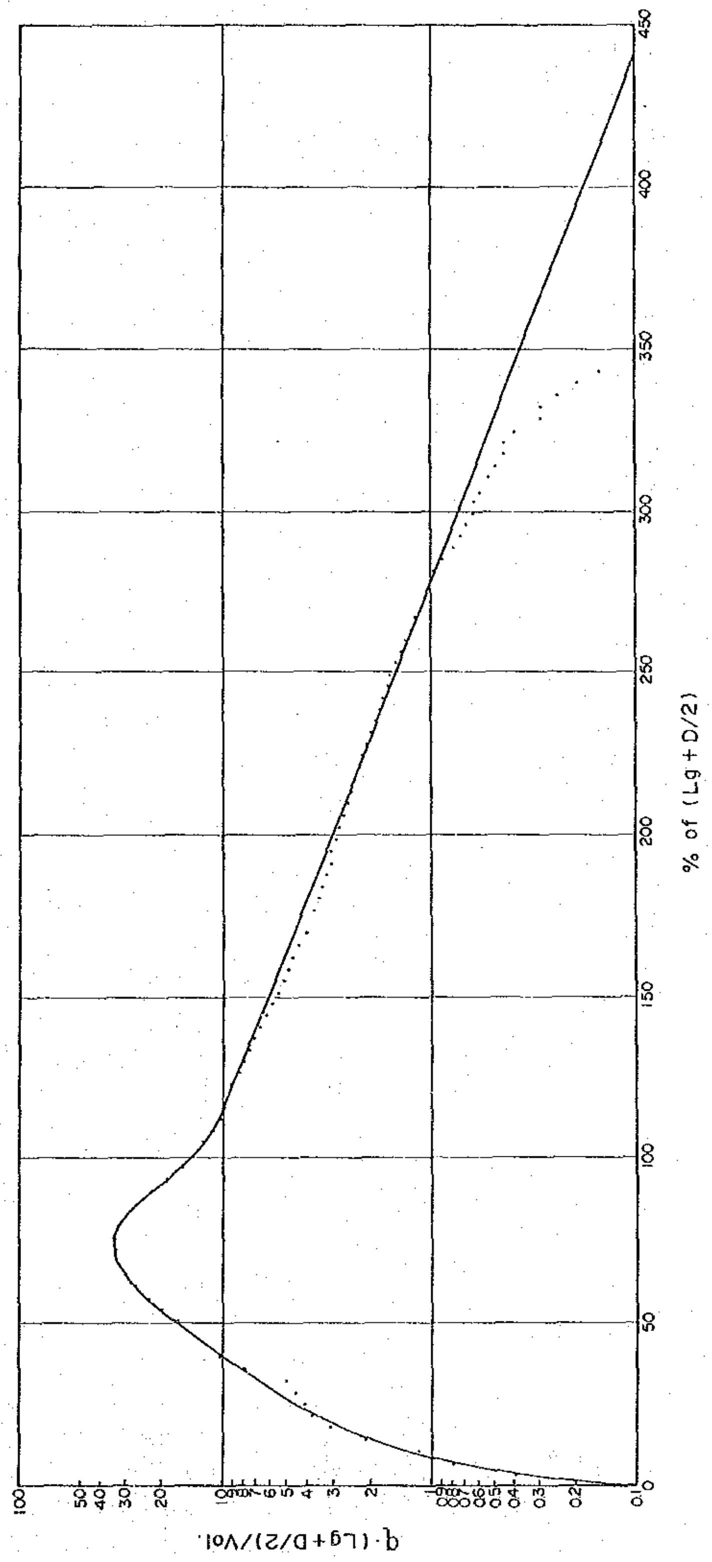

| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE | Fig C26<br>Flood Hydrograph of Göksu River at 1714 in Dec. 1973 |

FIG. NO.




| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C27<br>Flood Hydrograph of Göksu River at 1714 in Feb. 1975 |

FIG. NO.



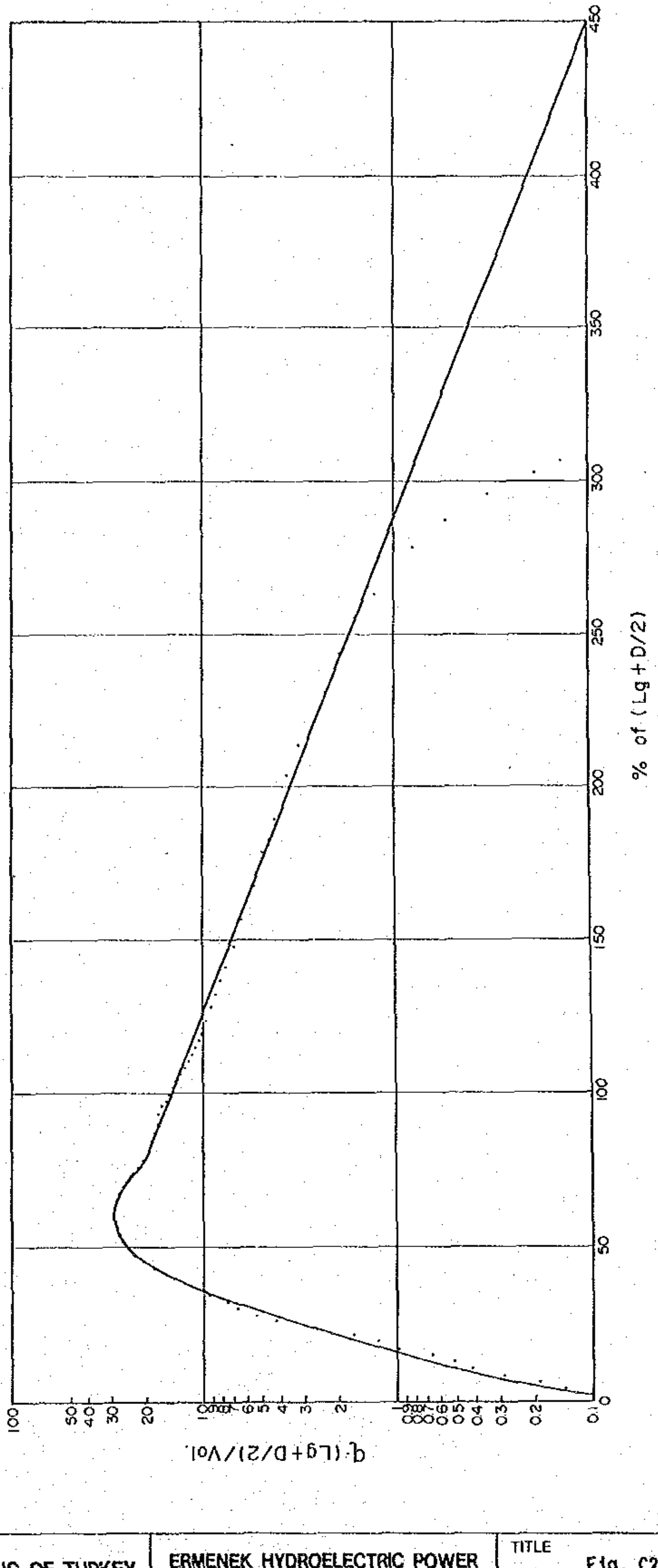

THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜĞÜ

ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

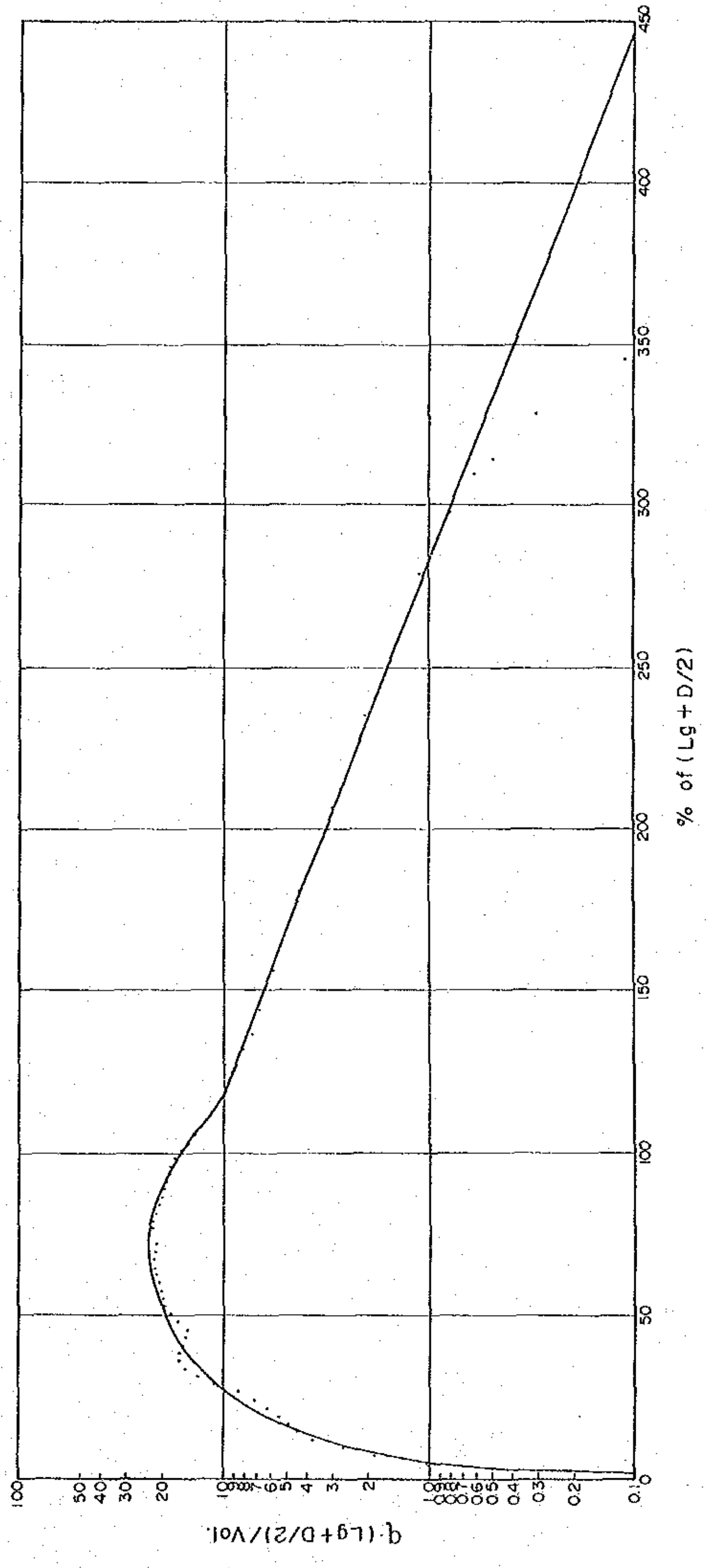
TITLE
Fig. C28
Dimensionless Graph for Göksu River at 1714 in Dec. 1967

FIG. NO.




| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE | Fig. C29<br>Dimensionless Graph for Göksu River at 1714 in Jan. 1971 |

FIG. NO.




THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜĞÜ

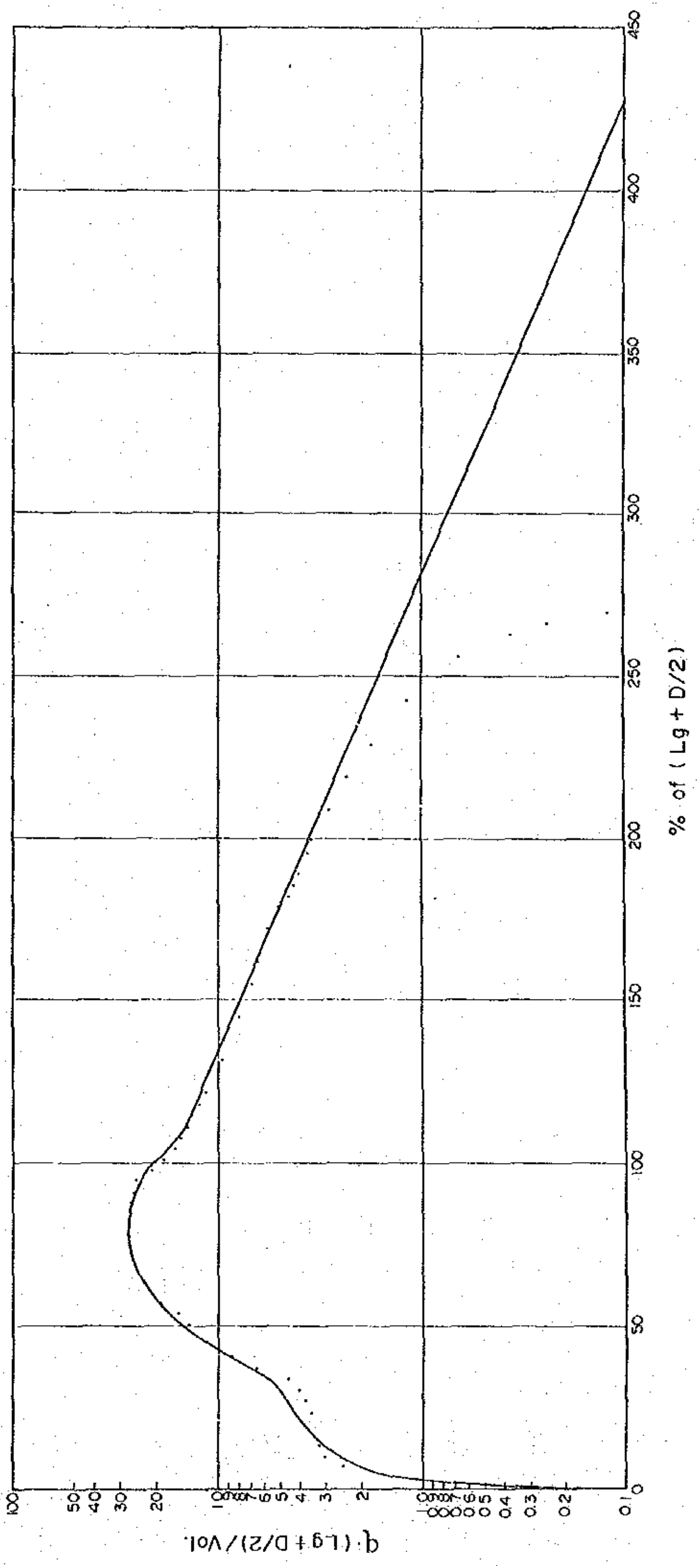
ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

Fig. C30
Dimensionless Graph for Göksu River at 1714 in Dec. 1971

FIG. NO.




THE REPUBLIC OF TURKEY
ELEKTRİK İŞLERİ ETÜD İDARESİ
GENEL MÜDÜRLÜGÜ

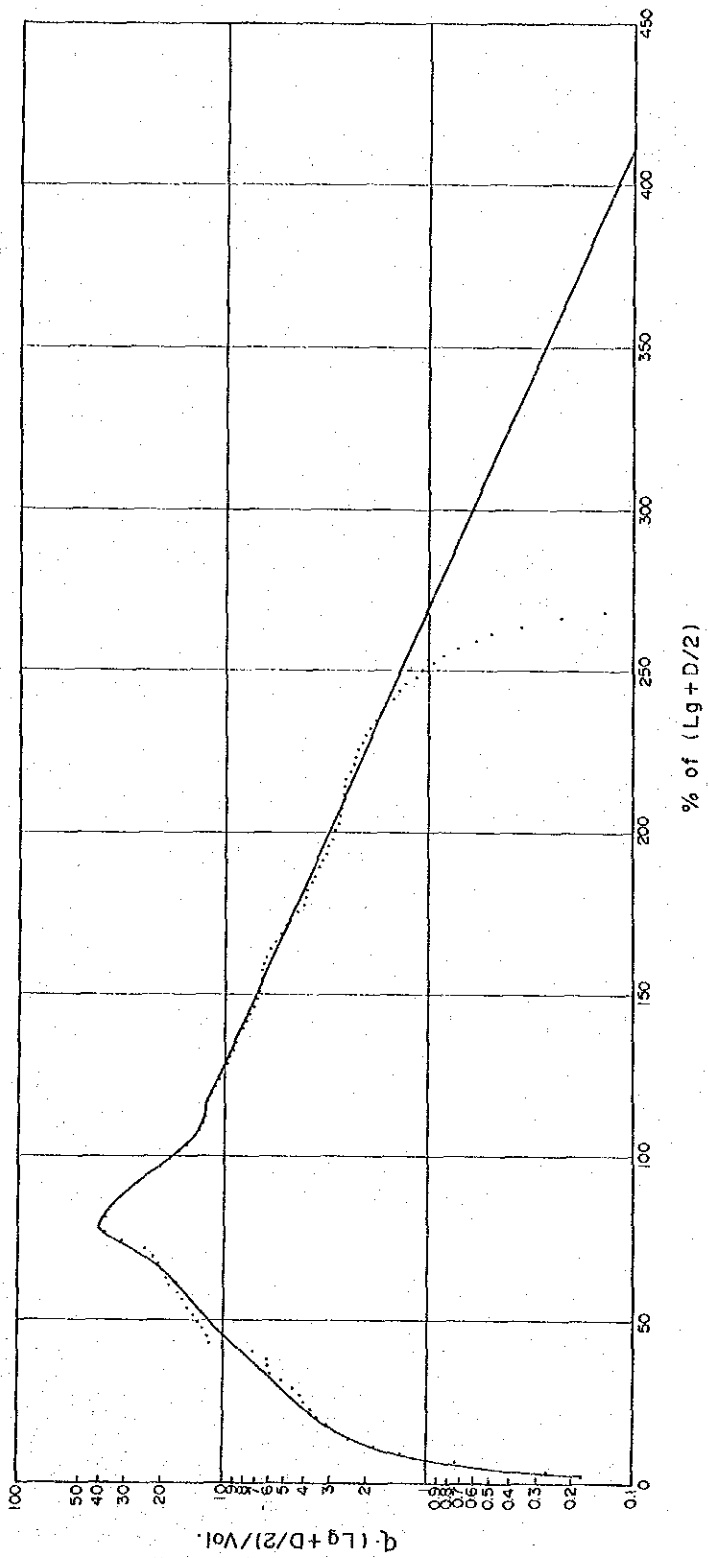
ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

TITLE

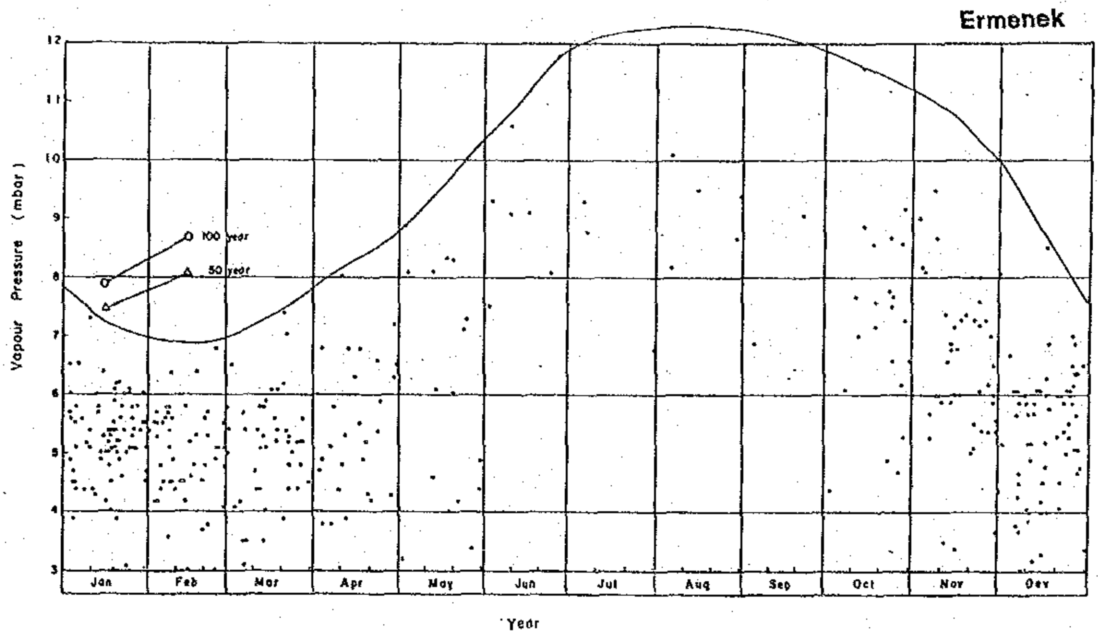
Fig. C31
Dimensionless Graph for
Göksu River at 1714
in Dec. 1973

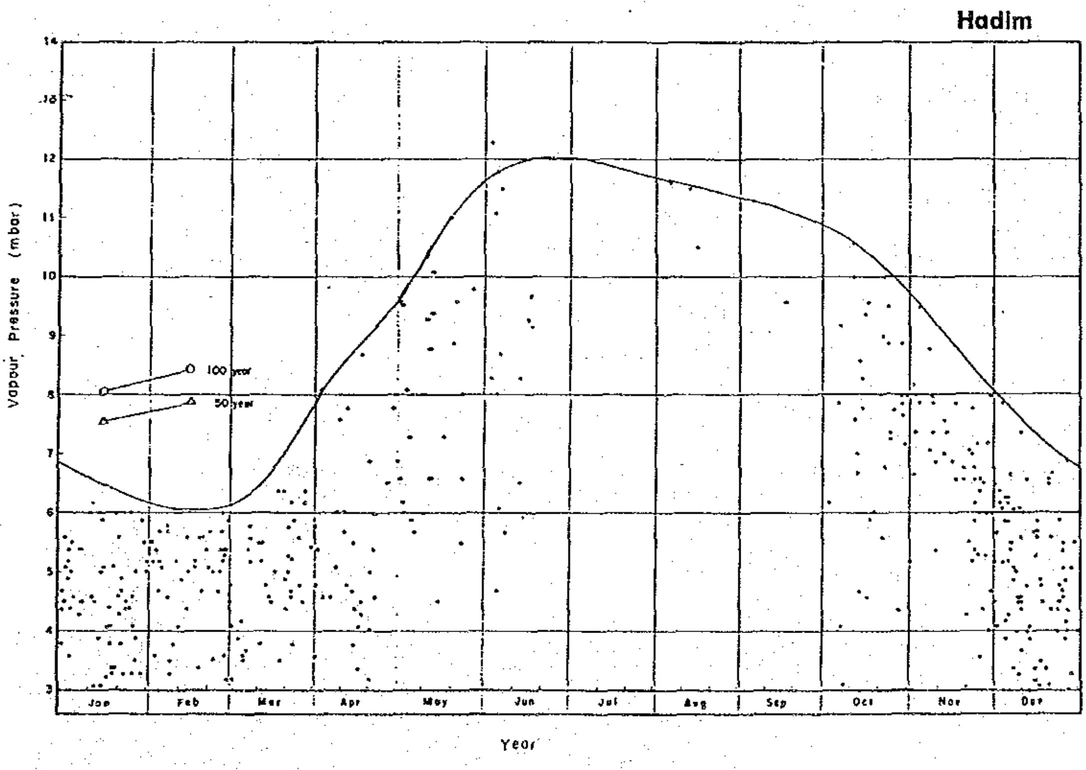
FIG. NO.




| | | |
|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C32<br>Dimensionless Graph for Göksu River at 1714 in Feb . 1975 |

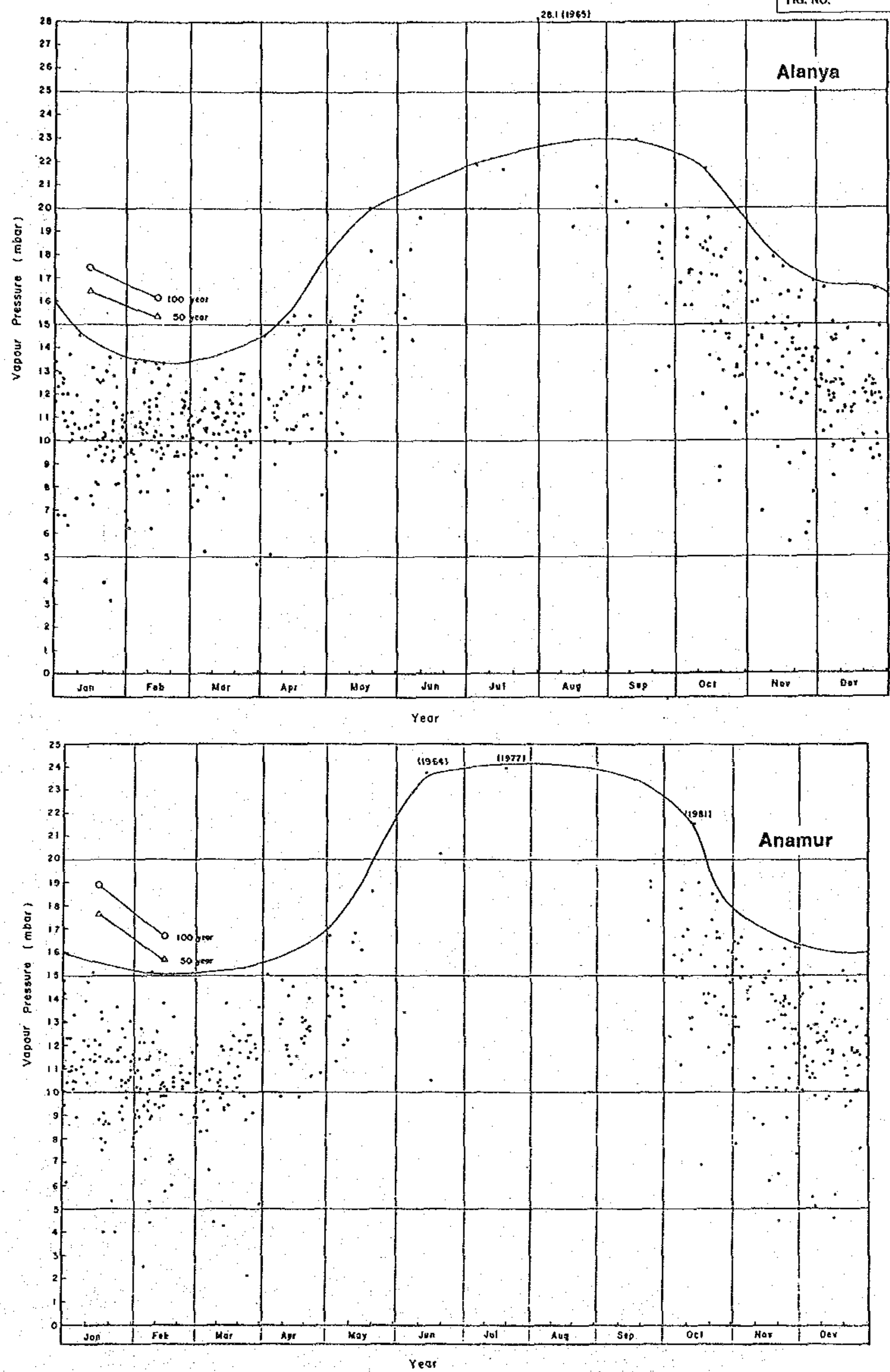
FIG. NO.



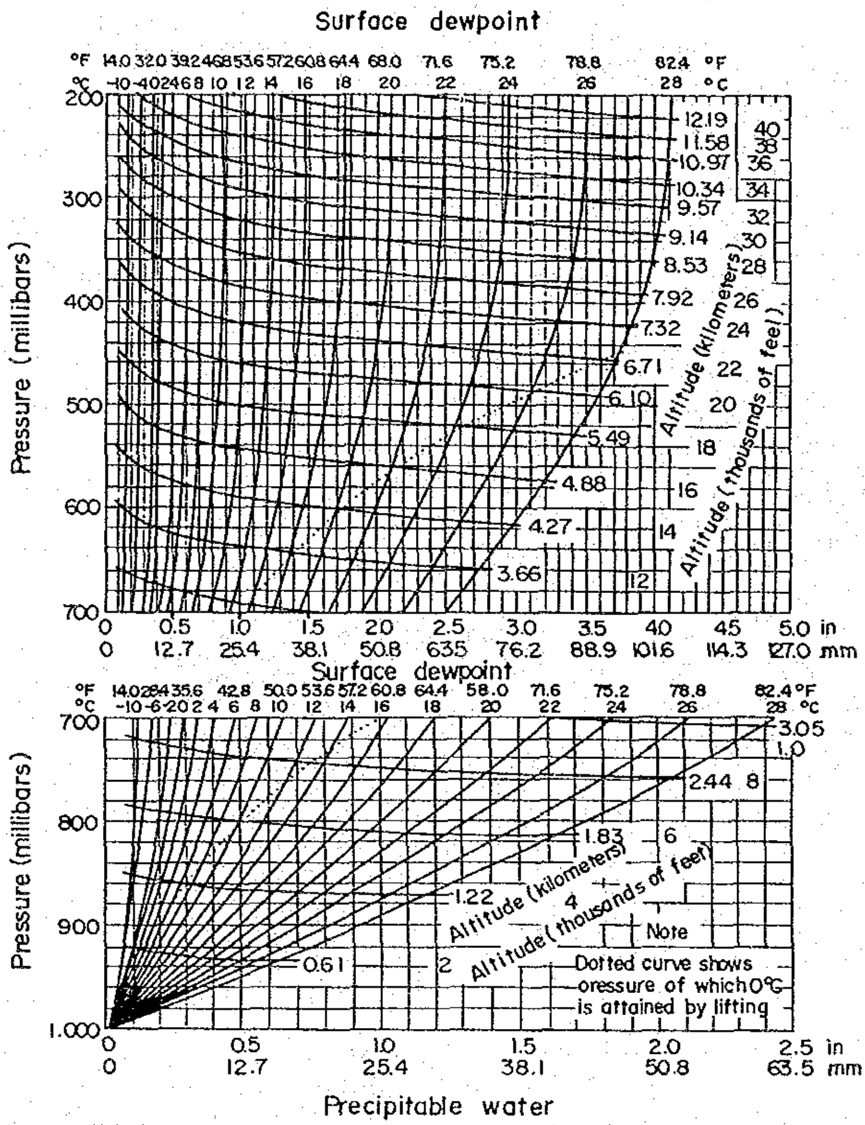




| | | |
|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C33 (1/2)<br>Seasonal Patterns of Maximum Vapor Pressure |

FIG. NO.

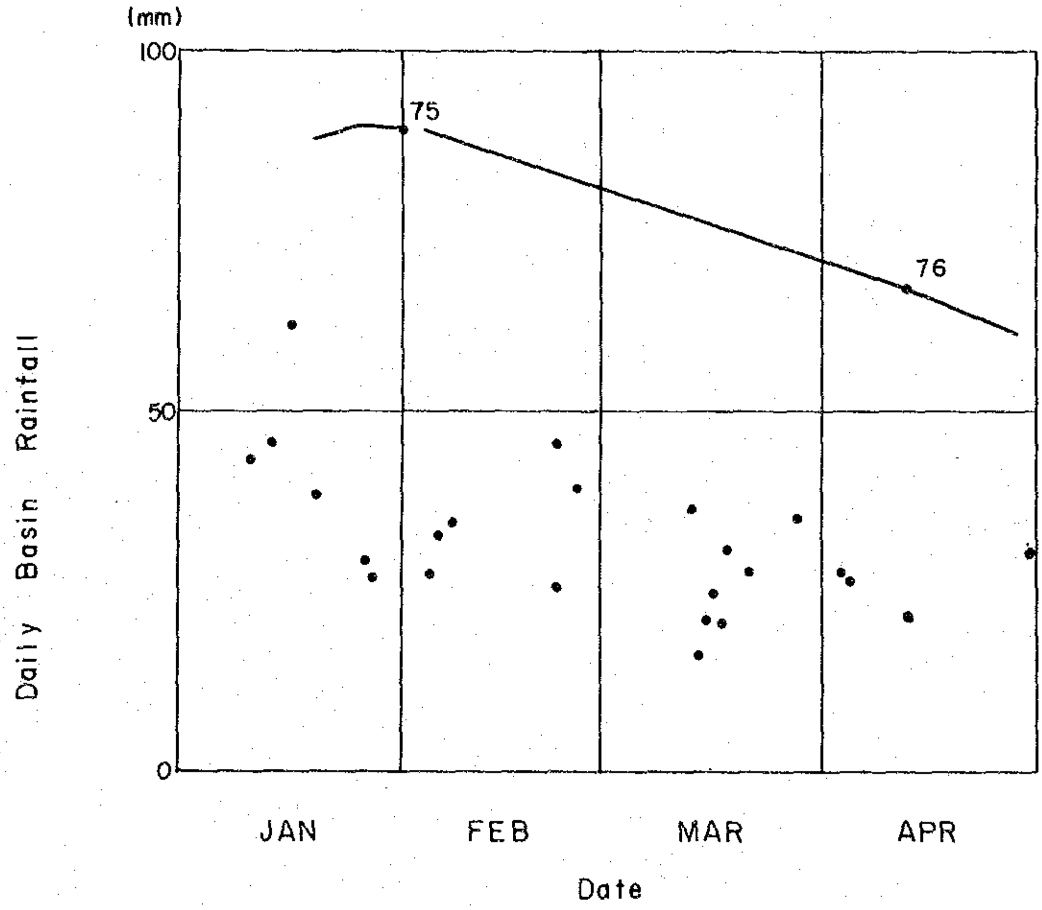



| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C33 (2/2)<br>Seasonal Patterns of Maximum Vapor Pressure |




Depths of precipitable water in a column of air of any height above the 1000-millibar level as a function of the 1000-millibar dewpoint, assuming saturation and pseuda-adiabatic lapse rate.
(U.S National Weather Service.)

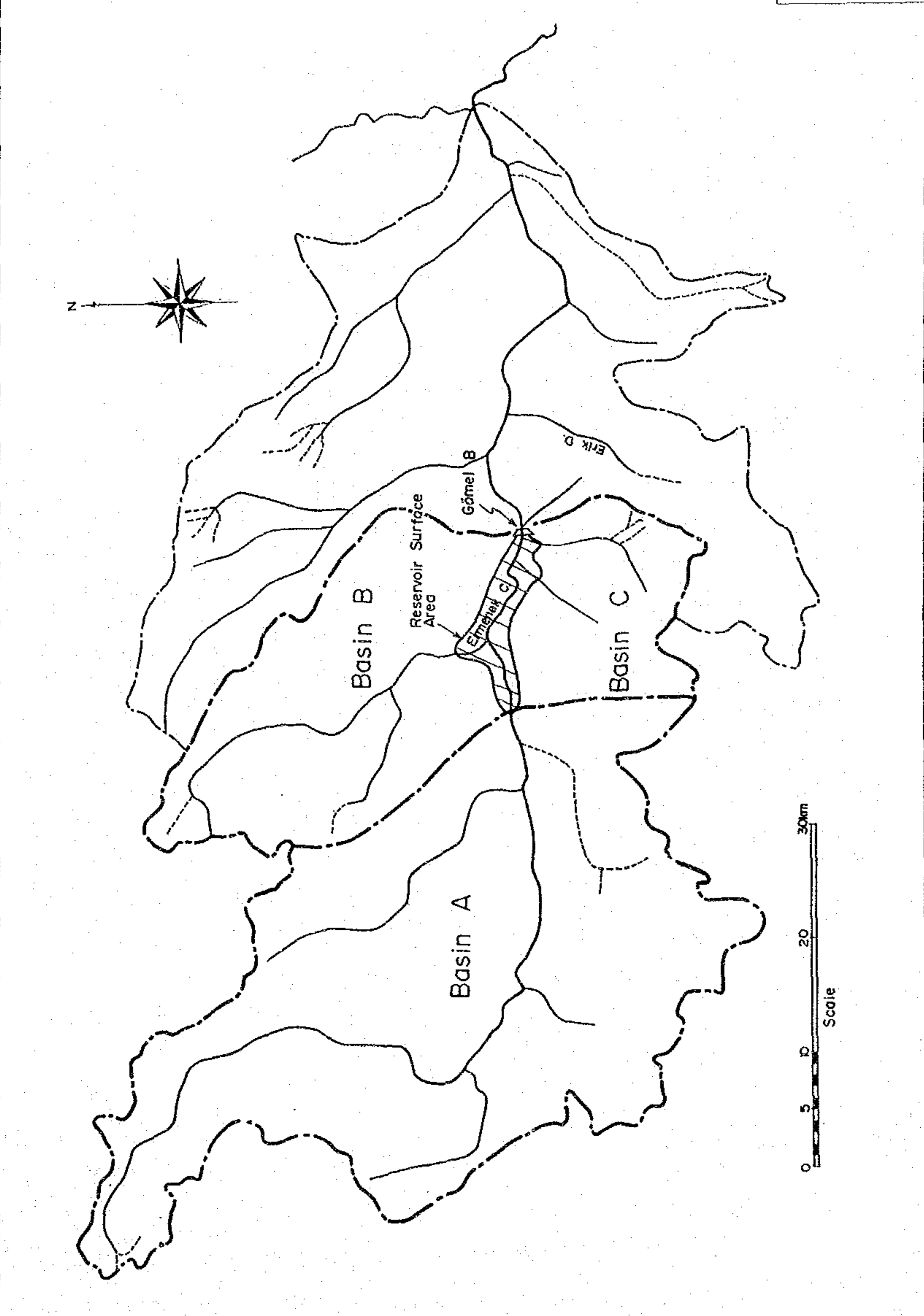
| | | | |
|---|---|---|---|
| THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C34<br>Depth of Precipitable Water in a Column of Air |




| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C35<br>Seasonal Pattern of Maximum Daily Precipitation in the Ermenek River Basin |

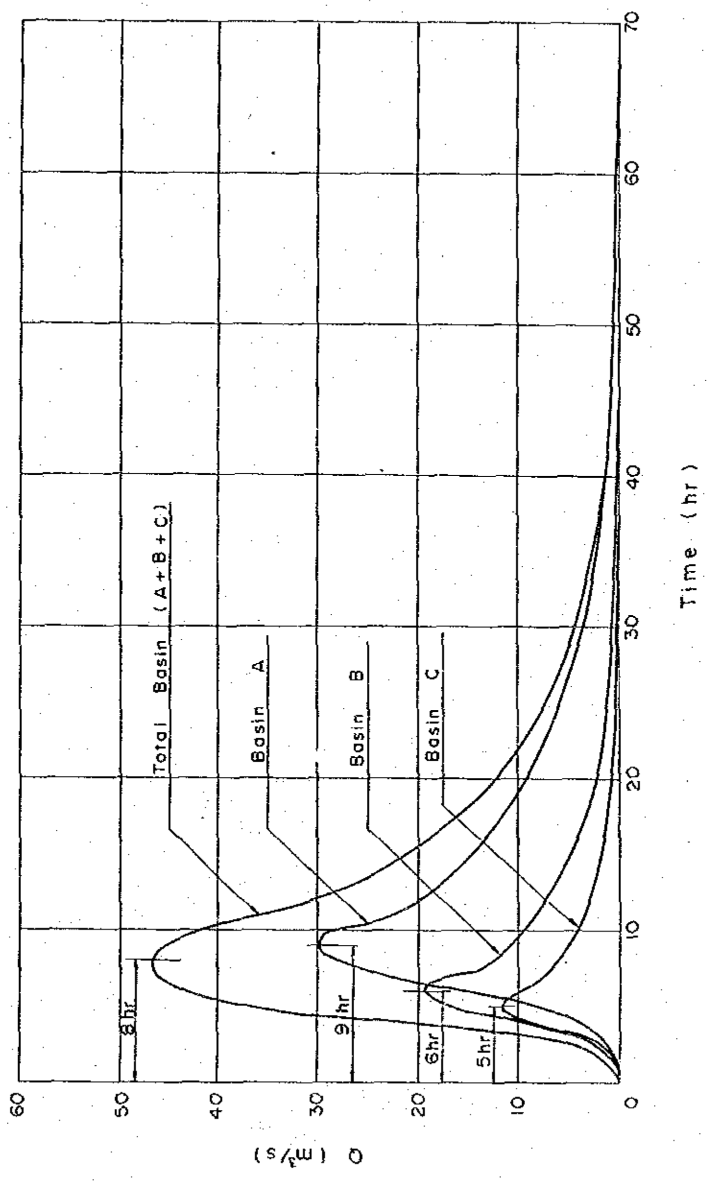
FIG. NO.

FIG. NO.



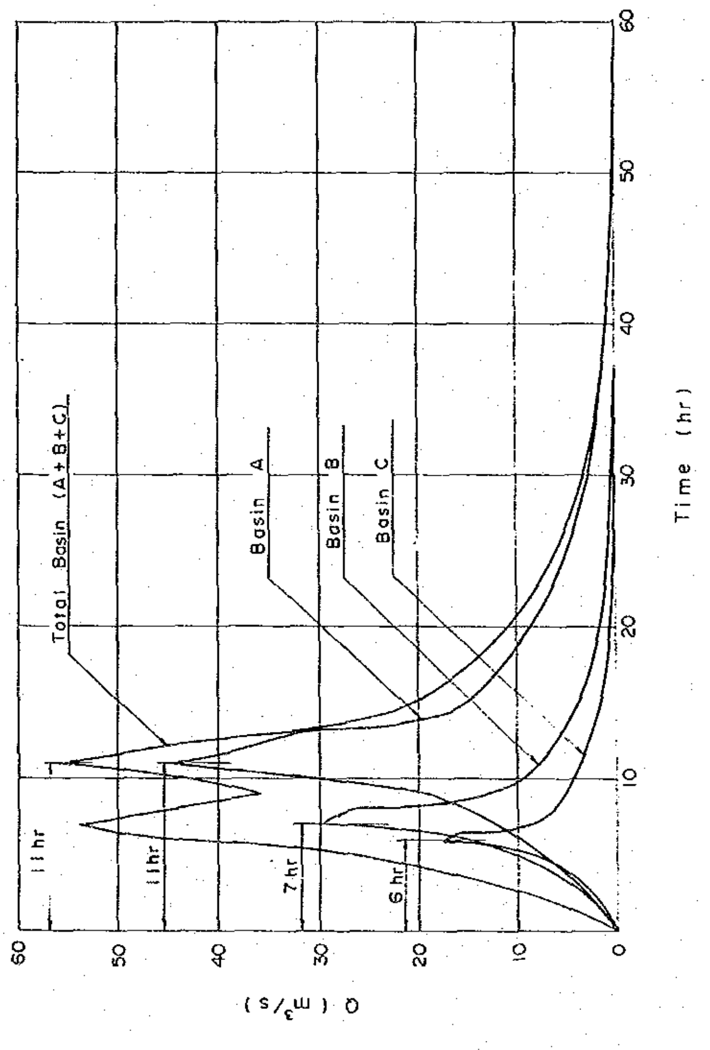

| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C36<br>3 Sub-basins in the Ermenek River Basin for Unitgraph Estimation |

FIG. NO.



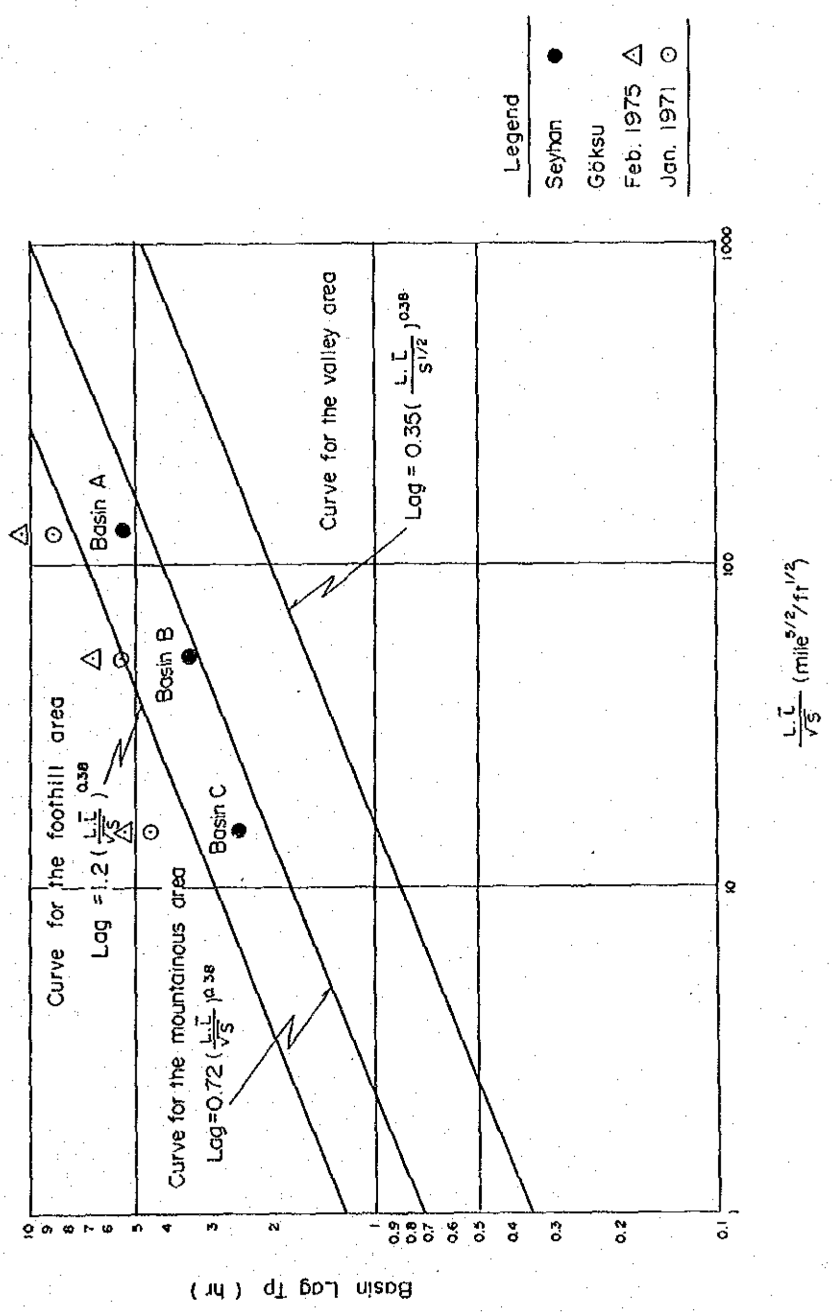

| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C37<br>Estimated Unitgraph of the Ermenek River by Jan. 1971 Flood |

FIG. NO.



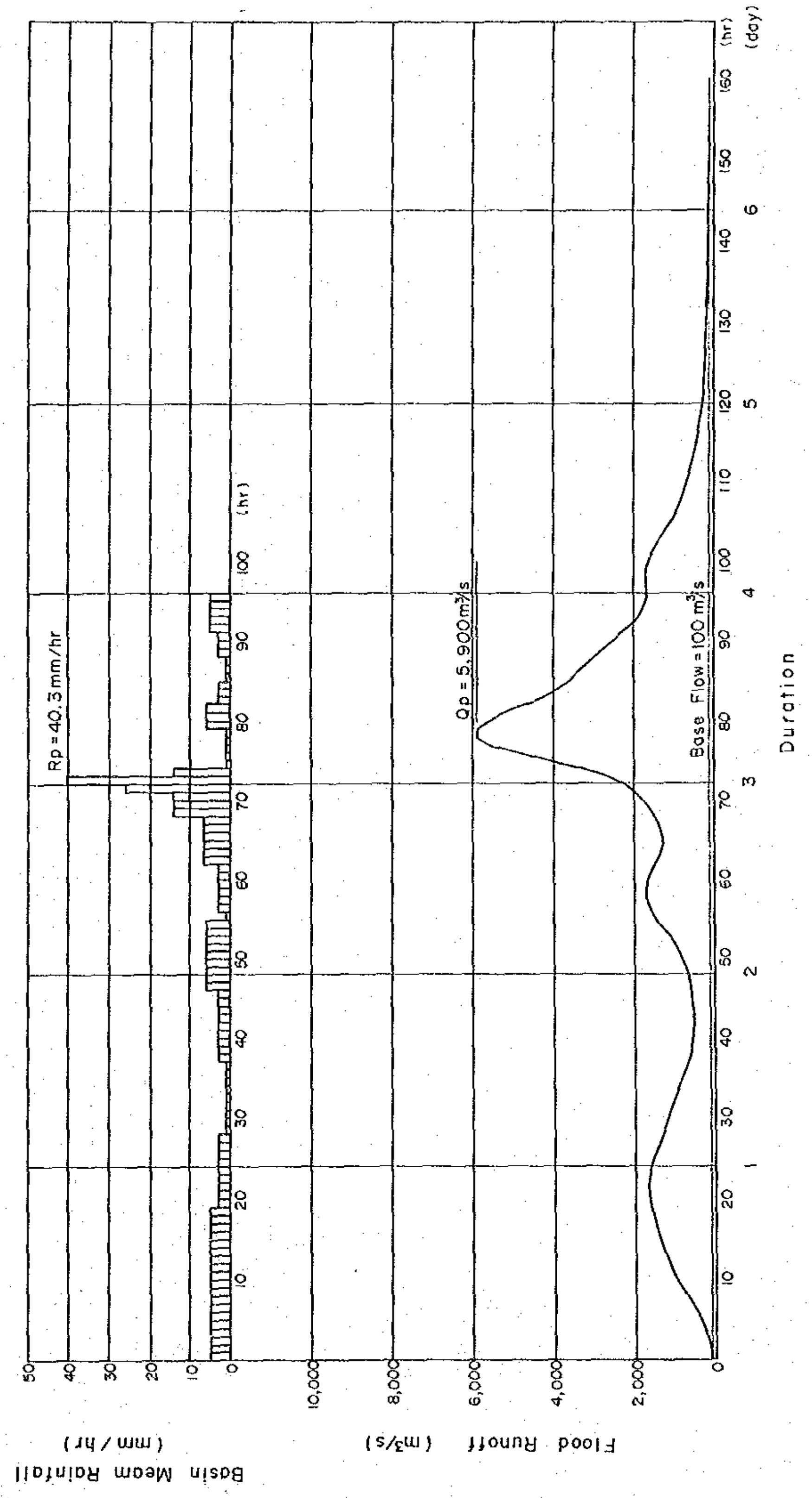

| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C38<br>Estimated Unitgraphs of the Ermenek River by Feb. 1975 Flood |

FIG. NO.




| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C39<br>Estimated Lag Times of the Ermenek Basin |

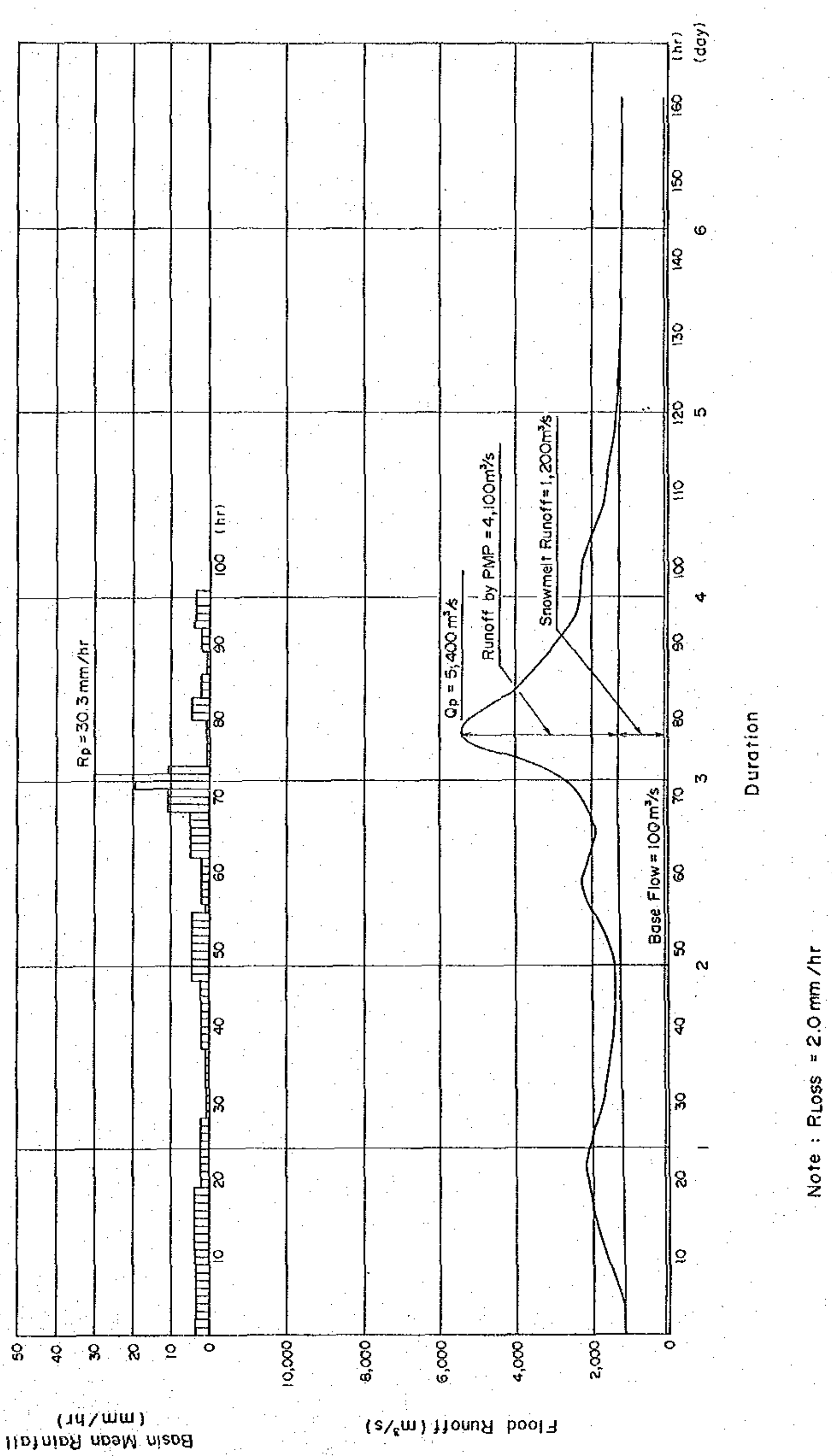
FIG. NO.



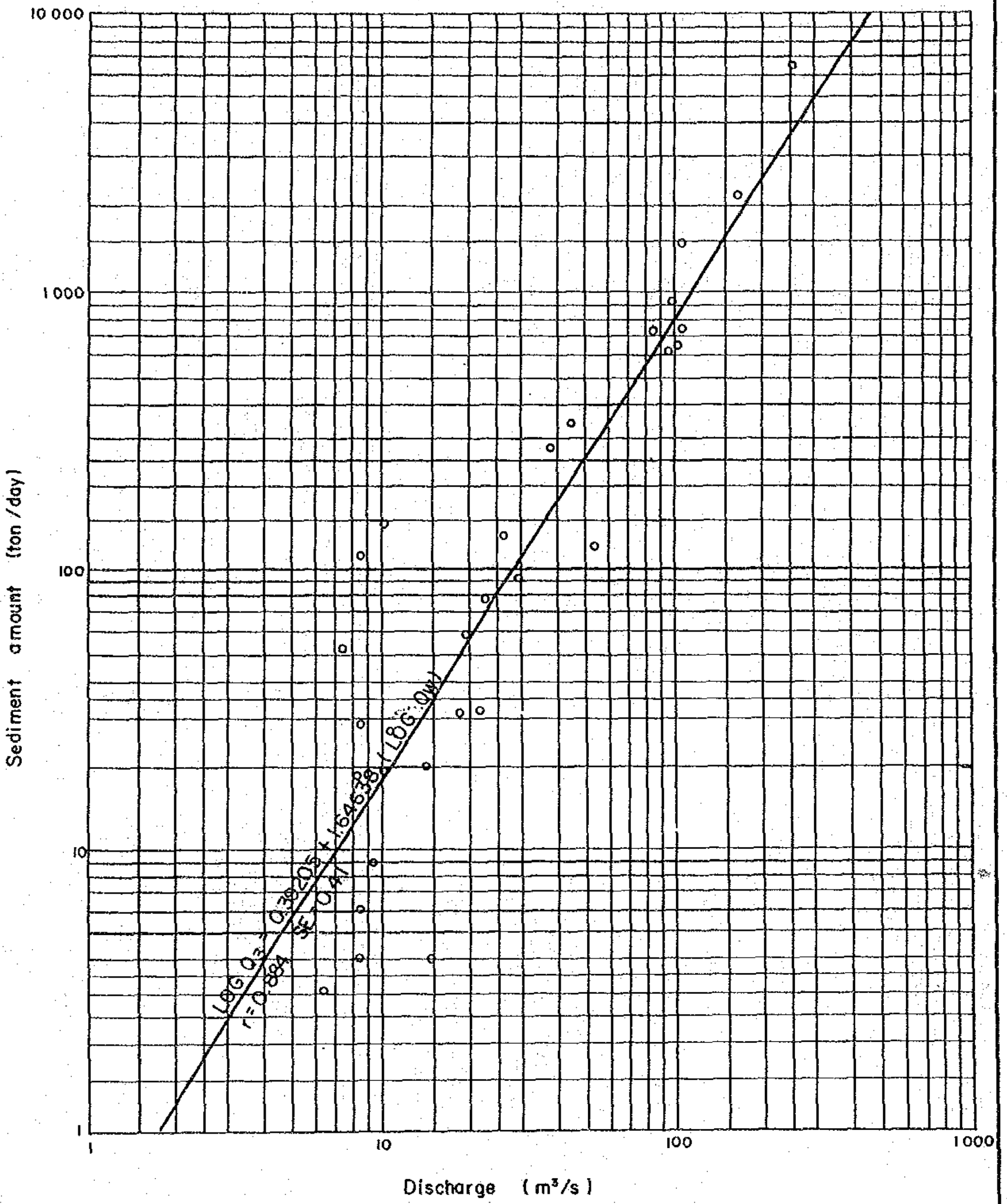

Note Rcoss = 2.0 mm/hr

| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C40<br>Probable Maximum Flood for January |

FIG. NO.




| | | | |
|---|---|---|---|
| | **THE REPUBLIC OF TURKEY**<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜGÜ | **ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT**<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C41<br>Probable Maximum Flood for April |




| | | | |
|---|---|---|---|
| | THE REPUBLIC OF TURKEY<br>ELEKTRİK İŞLERİ ETÜD İDARESİ<br>GENEL MÜDÜRLÜĞÜ | ERMENEK HYDROELECTRIC POWER DEVELOPMENT PROJECT<br>JAPAN INTERNATIONAL COOPERATION AGENCY | TITLE<br>Fig. C42<br>Rating Curve of<br>Suspended Sediment Load |

# ANNEX-D OPTIMIZATION STUDY

# ANNEX-D OPTIMIZATION STUDY

## TABLE OF CONTENTS

## LIST OF FIGURES

# TEXT

# CHAPTER 1. INTRODUCTION

This ANNEX-D presents detailed results of the optimization study on the development scale of the Ermenek Project.

The study was carried out in 3 steps and was reported as shown below:

(1) Progress Report 1: July 1989

The existing plans and studies were reviewed and preliminary studies were carried out on the alternative dam sites, type of development, and a prospective dam height.

A prospective dam height was initially obtained to be 200 m, or 695 m in HWL. However, this HWL was revised in October 1989 to be in the range of 645 to 695 m, and most probably to be around 670 m as a result of taking account of the adverse effect of the initial filling of large scale reservoir.

(2) Interim Report: March 1990

The optimum development scale was studied and HWL of 670 m was selected within the expected range.

(3) Draft Final Report: September 1990

The above development scale was reviewed based on the feasibility grade design of principal structures, and the optimum HWL is finally obtained to be 675 m.

# CHAPTER 2. METHODS

## 2.1 Background for the Plan Formulation

Of the electric energy produced in 1988, the hydropower shared 60 per cent, followed by the lignite-fired thermal power of 25 per cent. Lignite-fired thermal plants had been intensively developed through 1980s, but its energy share has started to decrease after reaching a peak of 47 per cent in 1986. The exploitable reserve of lignite in Turkey is limited, and development of imported-coal thermal plants is being planned to meet the growing demand.

Turkey has been suffering from air pollution problem especially in the winter season, when most of households use lignite for house heating. The Government is making every effort to alleviate the pollution issues by such means as shifting of lignite to imported coal or imported natural gas, and further implementation of hydropower projects.

Of the economic hydropower potential of 121 billion kWh indigenous to Turkey, 20 per cent have been developed, 13 per cent are under construction. A continuous and steady development of the rest of 67 per cent is of vital importance to support the growth of economic activities in Turkey.

Such being the background, the Project was put into the stage of feasibility study as it is blessed with the following natural and socio-economic conditions:

(1) The Ermenek River has a relatively stable river flow fed by many springs located in the basin. These springs are recharged by snow-melt mainly on the southern boundary of Ermenek River Basin.

(2) The Ermenek River has an average slope of 1/72 in the downstream reaches from the prospective dam site. On the other hand, there is a wide river valley spreading towards upstream from the dam site. These topographic features are favorable for a dam and waterway type power development.

(3) The Project area is located close to regional demand centers and has, therefore, operational advantages such as less transmission losses in peak power supply.

(4) The Gezende Hydroelectric Power Project is under construction in the downstream reaches of the proposed dam site. Because of its limited capacity of the reservoir, it would function like a run-of-river type power plant. The firm energy would be greatly enhanced upon implementation of the Project. This firming-up effect would also be expected on the Kayraktepe Hydroelectric Power Project planned on the main stream of Göksu River.

(5) The reservoir area is sparsely populated because most of houses in the area are located around springs, which are situated at a higher elevations than the reservoir level conceived. Adverse effects of the Project on the people and farmland in the reservoir area will be small.

## 2.2 Methodology

The optimization study of a development plan for the Project has been performed in 2 stages: Preliminary Study as shown in Fig. D1, and Plan Formulation Study as shown in Fig. D2.

### 2.2.1 Preliminary study

The main objectives of the Preliminary Investigation Stage were: (1) to select the most promising development plan on the basis of the many alternative plans conceived by EİE including some additional ideas proposed by the JICA Study Team; (2) finally to identify the sites for the Additional Detailed Investigations.

The alternative studies performed at this stage were as listed below (see Fig. D1):

(1) Comparative study of alternative dam sites between I-B and I-C, assuming that the water leakage problem of the I-C dam site can be technically managed

(2) Preliminary economic examination of the newly proposed Erik Diversion Scheme

(3) Preliminary economic examination of the newly proposed underground power house combined with an underground pressure shaft and a tailrace tunnel, assuming that the power house can be situated in a limestone block

(4) Economic examination of the 2-step development plans:
(A) the Ermenek Dam plus a downstream dam at II-A site;
(B) the Ermenek Dam plus a downstream dam at II-B site;
(C) the Ermenek Dam plus an upstream dam at Nadire

Based on the results of Additional Detailed Investigations, it is judged that the limestone at I-C dam site is less karstic in its lower elevation range below the groundwater level, and that the water leakage through the limestone could be stopped by grout curtain. Accordingly, the second best alternative of the rockfill dam at I-B site was ruled out at this stage of the study.

Also the bottom elevation of the limestone block, wherein the underground power house is planned, has been sounded through the drilling of hole Nos. SK-102, 106 and 108. It is judged that the underground power house could be placed within the limestone block.

Accordingly, the tentative conclusion of the preliminary studies has been confirmed. The most prospective development plan of the Project is the 1-step development with an arch dam at the I-C site, an underground power house, and the Erik Diversion Scheme.

### 2.2.2 Optimization study

An optimization study has been carried out based on the basic development plan obtained through the preliminary study. The study procedure is as described below (Fig. D2):

(1) Assumptions and preparatory studies: (A) planning of initial filling of the Ermenek Reservoir; (B) assessment of risks accompanying to unidentified water leakage paths in the limestone block of dam site; (C) assessment of power outputs and benefits; (D) construction cost estimate and planning.

(2) Studies of the more detailed development plan of the Project: (A) the optimum dam axis in the I-C site; (B) Erik Diversion Scheme; (C) underground power house; (D) addition of Erik Power Station.

(3) Optimization of the principal Project components such as type and capacity of spillway, drawdown of reservoir, diameter of power waterway, route of headrace tunnel, location of tailrace outlet, discharge capacity and type of the Erik Diversion Scheme.

(4) Selection of an optimum development scale

The development scale was represented in this study by HWL of the Ermenek Reservoir. HWL was selected in place of dam height or dam crest elevation as HWL can be an independent parameter both from the extra dam height above HWL and from depth of the foundation excavation. The firm and secondary energies are dependent on HWL. The installed capacity is, in this study, dependent on the firm energy being determined with a capacity factor of 33 per cent for firm energy. Thus, the development scale can be represented by a single parameter of HWL.

### 2.2.3 Calculation procedure of net benefit

The calculation procedure of net benefits is shown in Fig. D3. The procedure consisted of the following 5 groups:

(1) Power study to assess the capacity factor adequate to the Ermenek Power Station as well as to assess the power values

(2) Reservoir operation study combined with a hydraulic design of waterway and determination of a installed capacity for each alternative

(3) Optimization study and preliminary design of the Project components such as spillway, waterway, etc. These are the main objectives of the Preliminary Study and Plan Formulation Study.

(4) Feasibility grade design of an arch dam, grout curtain works, and other principal structures

(5) Study of construction plan and unit construction costs

### 2.3 Assessment of Power Outputs

#### 2.3.1 Capacity factor

The capacity factor of the Ermenek Power Station was selected at 33 per cent for the annual firm energy (see Sub-section 3.5.2 of Volume 2, Main Report).

#### 2.3.2 Reservoir operation study

Reservoir operation studies were carried out for each alternative to assess the energy output. The operation study was made first to assess the firm energy output assuming diameters of power waterway, installed capacity of generating equipment, rated water level, etc. With this firm energy and capacity factor of 33 per cent, a new installed capacity was obtained. Then the maximum turbine discharge was obtained by iterative calculation so that the maximum power becomes equal to the installed capacity taking into consideration the loss of head. At this time, diameter of power waterway were obtained in accordance with the empirical formula (see Sections 5.3 and 5.5).

After determining the diameters of power waterway, maximum turbine discharge, etc., the operation study was made again to confirm the firm energy. When significant difference is found in the firm energies obtained by 2 studies, the procedure above is repeated.

Energy outputs of power stations located serially on the Ermenek River such as Nadire, Ermenek, Lower Ermenek at II-A or II-B site, Gezende Power Station were estimated through simulation of the combined operation of serial reservoirs.

The simulation study was carried out using the monthly inflow series in accordance with the following simple operation rule:

(1) Individual operation based on the inflow and water storage of the own reservoir, without considering water storage of the upstream or downstream reservoir

(2) Constant power generation at the firm energy when the reservoir level is between HWL and LWL

(3) Level operation at HWL by secondary energy generation within the capacity in addition to the firm energy when the reservoir water level rises to reach HWL

(4) Level operation at LWL by energy generation with available inflow when the reservoir water level lowers to reach LWL

In the simulation, a monthly inflow series of the 42 years from 1946 to 1987 was used (see Sub-section 5.5.3 of Main Report). However, the series was rearranged into the following order so as to recover HWL at the end of the simulation period:

1982 to 1987 followed by 1946 to 1981

The firm energy is herein defined as the 100 per cent dependable annual energy, which can be secured throughout the above 42 years' period of the inflow series.

The following assumptions were made on the probable water losses from the reservoir, and were applied to the operation study:

(1) The reservoir evaporation loss rate was assumed at 70 per cent of the mean monthly pan evaporation records, or at a rate of 1,380 mm/yr. A monthly variation pattern was considered but yearly change was neglected

as there were no such data available.

(2) The risk of water leakage through dam foundation and reservoir would become high, probably in proportion to the hydraulic head and the necessary area of grout curtain. The higher dam would have the higher risk of water leakage. In order to take into account this leakage risk in the determination of the Project scale, certain leakage loss was deducted from the inflow before using it in the operation study. The loss was assumed as described below:

Representative dams constructed on limestone foundations in the world have more or less water leakage through foundations as summarized below:

Water Leakage Through Limestone Foundations

| Dam Name | Dam Height (m) | Initial Stage H. Head (m) | Initial Stage Leakage (cc/s/m2) | Final Stage H. Head (m) | Final Stage Leakage (cc/s/m2) |
|---|---|---|---|---|---|
| Chicoasen | 262 | 252 | 0.8 | no supplemental curtain | |
| Keban | 211 | 202 | 130 | 204 | 15 |
| Oymapinar | 185 | 183 | 1 | no supplemental curtain | |
| Kremasta | 160 | 145 | 50 | | |
| Cannelles | 151 | 85 | 200 | 138 | 0.4 |
| Pueblo Viejo | 140 | 135 | 0.04 | 135 | 0.03 |
| Dokan | 120 | 80 | 14 | 118 | 0 |

Source: Dam foundations on karstic formations, 1985, Commission Internationale Des Grandes Barrages

With reference to the leakage records above, the basic rate of leakage was assumed at 1.0 $cm^3/sec/m^2$ under the hydraulic head of 100 m. The water leakage for various alternative dam heights was assumed as shown below:

Assumed Water Leakage from Limestone Foundation

| HWL (m) | H. Head (m) | Curtain Area (1000 $m^2$) | Unit Leakage ($cm^3/s$) | Total Leakage ($m^3/s$) |
|---|---|---|---|---|
| 650 | 145 | 511 | 1.45 | 0.69 |
| 660 | 155 | 542 | 1.55 | 0.84 |
| 670 | 165 | 574 | 1.65 | 0.95 |
| 680 | 175 | 619 | 1.75 | 1.08 |
| 690 | 185 | 664 | 1.85 | 1.23 |

### 2.3.3 Initial Filling Plan of Reservoir

The optimum dam height will be affected by the initial reservoir filling, because the Ermenek Reservoir will have a large reservoir capacity of 3,529 MCM in gross in the case of the HWL of 675 m. This size of reservoir would be fully filled in about 2.4 years with the mean annual inflow of 1,466 MCM or 46.5 $m^3/s$ including the Erik flow.

During the above initial filling period, the output of Gezende Power Station will inevitably be decreased or stopped. The Ermenek Power Station can start its full power generation only after completion of the initial filling.

In order to take into consideration the above adverse effect, initial reservoir filling plans were prepared for alternative HWLs to be compared.

The plans were prepared with the following criteria (refer to Fig. D11):

(1) The initial filling plans will be prepared assuming that the average annual inflow will continue throughout the filling period.

(2) The full power generation will be started when the reservoir will be filled for half the effective reservoir capacity. The corresponding water level to this storage is hereinafter referred to as the Medium Water Level (MWL). This MWL was selected in conjunction with the above assumption (1).

(3) In order to start power generation at the Ermenek Power Station as early as possible, the first stage filling to fill the reservoir up to LWL will be started when construction works of the following structures are completed:

- dam and curtain grout up to LWL
- power intake
- bottom outlets and tunnel spillways

(4) Throughout the first stage filling period, necessary discharge will be released from the Ermenek Dam to secure the firm outputs of Gezende Power Station.

(5) If the reservoir water level reaches LWL before completion of the dam construction works, the reservoir water level will be controlled to keep an adequate level by operating the bottom outlets and or tunnel spillway. This operation is herein called as the second stage filling. In the case of a development scale lower than 670 m in HWL, the initial filling would be completed during this second stage filling, to fill up to MWL.

(6) When the construction works of dam and curtain grout are completed, the third stage filling will be started, by generating power at 50 per cent of the firm energy.

(5) The third stage filling will be continued up to when the reservoir water level reaches MWL. Thereafter, the

full operation will be started as planned.

In accordance with the above criteria, the initial filling periods were estimated for the 11 alternative HWLs. The results are shown in Fig. D10. In the case of HWL of 675 m, the reservoir filling period and the rising speed of reservoir water level would be as shown in Fig. D11.

### 2.3.4 Effects on the Gezende Power Station

The firm-up benefit of the Gezende Project was assessed at US$16.3 million as summarized below:

| No. | Items | Unit | without Ermenek | with Ermenek | Increase |
|---|---|---|---|---|---|
| (1) | Firm energy | GWh | 118 | 526 | 408 |
| (2) | 2ndary energy | GWh | 448 | 115 | -333 |
| (3) | Annual energy | GWh | 566 | 641 | 75 |
| (4) | 90% dependable power | MW | 41 | 150 | 109 |
| (5) | Annual benefit | Mil.$ | 18.6 | 34.9 | 16.3 |

Adverse effect on the Gezende Power Station during first stage filling was estimated as summarized below:

| No. | Items | Unit | without Ermenek | with Ermenek | Increase |
|---|---|---|---|---|---|
| (1) | Firm energy | GWh | 118 | 118 | - |
| (2) | 2ndary energy | GWh | 448 | 71 | -377 |
| (3) | Annual energy | GWh | 566 | 189 | -377 |
| (4) | 90% dependable power | MW | 41 | 41 | - |
| (5) | Annual benefit | Mil.$ | 18.6 | 9.8 | -8.8 |

## 2.4 Assessment of Power Values

The alternative thermal power plant for economic evaluation of the Project is selected as an appropriate least cost thermal plant which will be required when the Project is not implemented.

Future main thermal power plants of Turkey would be a combination of imported coal-fired steam thermal and natural gas-fired combined cycle plants. When the Project is not implemented, both of these two types of thermal plant will be required. The overall energy generation costs of these two types of thermal plant are comparable though the unit construction cost is cheaper for the natural gas combined cycle and the fuel cost is cheaper for the coal thermal. In conclusion, based on a government policy for the diversification of energy sources, a combination of these two types of plant is selected as the alternative thermal plant for the Project. In view of the peak power generation which is expected to the Project, the allocation of power and energy outputs of the Project to these two alternative thermal plants was assumed as shown below:

| | Power | Energy |
|---|---|---|
| Coal thermal | 50 % | 70 % |
| Gas combined cycle | 50 % | 30 % |

The installed capacity of alternative thermal plants for calculating unit costs for construction and fuel consumption was assumed as shown below:

| | |
|---|---|
| Coal thermal | 4 x 350 MW |
| Gas combined cycle | 6 x 100 MW |

The calculation process of power values is summarized below:

(1) Basic assumptions

Power generation costs of the alternative thermal plants were estimated with basic assumptions as given below:

| | | Coal Thermal | Combined Cycle |
|---|---|---|---|
| -Net construction cost | ($/kW) | 820 1/ | 500 |
| -O & M cost | ($/kW/yr) | 2.8 | 1.5 |
| -Construction period | (yrs) | 4 | 3 |
| -Cost disbursement 2/ | (%) | 10,25,40,20,5 | 10,35,50,5 |
| -Economic life time | (yrs) | 25 | 25 |
| -Station service power | (%) | 7.5 | 2.0 |
| -Heat rate | (kcal/kWh) | 2400 | 2200 |
| -Fuel cost 3/ | (US¢/$10^6$ kcal) | 690 | 1176 |

1/: Including environmental protection equipment.

2/: The last 5% is for payment against retention to be released 1 year after the taking over.

3/: The calorific values and unit rates of fuel are as follows:

Coal : 5,800 kcal/kg and US$40/ton

Natural gas: 8,500/$m^3$ and US$$100/1000 $m^3$

(2) Calculation of basic figures

(A) Investment costs on completion

The engineering and administration costs for the thermal projects is estimated at 15 per cent of the net construction costs given above.

The amount of interest during construction is calculated assuming that an interest rate is 9.5 % per annum and that all the expenditures in one year are disbursed at the middle of each year. The calculated interest factors are 1.1665 for the coal thermal plant and 1.0975 for the gas combined cycle plant.

Then, the investment costs on completion are calculated as follows:

-Coal thermal plant:

US$ 820/kW x 1.15 x 1.1665 = US$ 1100.01/kW

-Combined cycle plant:

US$ 500/kW x 1.15 x 1.0975 = US$ 631.06/kW

(B) Annual costs of thermal plants

The capital recovery factor for the period of 25 years under a discount rate of 9.5 per cent per annum is 0.10596 and the annual costs of the thermal plants including the operation and maintenance costs are obtained as follows:

-Coal thermal plant:

US$ 1100.01/kW x 0.10596 + US$ 2.8/kW/yr
= US$ 119.36/kW/yr

-Combined cycle plant:

US$ 631.06/kW x 0.10596 + US$ 1.5/kW/yr
= US$ 68.37/kW/yr

(C) Unit fuel costs

The unit fuel costs for energy generation by the thermal power plants are obtained from their heat rates and fuel rates per heat value as follows:

-Coal thermal plant:

2,400 kcal/kWh x US¢ 690/$10^6$ kcal
= US¢ 1.656/kWh

-Combined cycle plant:
2,200 kcal/kWh x US¢ 1,176/$10^6$ kcal
= US¢ 2.587/kWh

The relative price rise of coal and natural gas to the other commodities was assumed at 1.30 for the period from 1989 to the commissioning year of the Project, based on the 1988 World Bank's estimation. The above unit costs as of 1989 were then adjusted for this relative price rise of fuels:

| | |
|---|---|
| -Coal thermal | US¢ 1.656 x 1.3<br>= US¢ 2.153/kWh |
| -Combined cycle | US¢ 2.587 x 1.3<br>= US¢ 3.363/kWh |

(D) <u>Hydro advantages</u>

The hydro advantages specific to the Project are assessed assuming the following conditions:

Unit: %

| | Ermenek Hydro | Coal Thermal | Combined Cycle |
|---|---|---|---|
| -T/L loss up to primary S/S 1/ | kW: 2<br>kWh: 1.2 | kW: 3<br>kWh: 1.8 | kW: 5<br>kWh: 3 |
| -Regular maintenance | 1 | 15 | 10 |
| -Equipment failure | 0.5 | 5 | 5 |
| -Station power | 0.3 | 7.5 | 2 |

1/: The location of the alternative coal thermal plant is assumed at somewhere along the Mediterranean coast around Antalya or Silifke and the combined cycle plant at somewhere around Istanbul.

The adjustment factors for hydro advantages are obtained as follows:

-Capacity evaluation:

| | |
|---|---|
| Ermenek/Coal thermal | 1.328 |
| Ermenek/Combined cycle | 1.209 |

-Energy evaluation:

| | |
|---|---|
| Ermenek/Coal thermal | 1.084 |
| Ermenek/Combined cycle | 1.036 |

(E) <u>Capacity and energy values for plan formulation study and economic evaluation</u>

The capacity and energy values are obtained with adjustments for the hydro advantages as shown below:

-Capacity value

| | | |
|---|---|---|
| Coal thermal | : | US$ 119.36 x 1.328<br>= US$ 158.51/kW/yr |
| Combined cycle | : | US$ 68.37 x 1.209<br>= US$ 82.66/kW/yr |
| Overall | : | 0.5(158.51 + 82.66)<br>= US$ 120.58/kW/yr |

-Energy value

| | | |
|---|---|---|
| Coal thermal | : | US¢ 2.153 x 1.084<br>= US¢ 2.334/kWh |
| Combined cycle | : | US¢ 3.363 x 1.036<br>= US¢ 3.484/kWh |
| Overall | : | 0.7x2.334 + 0.3x3.484<br>= US¢ 2.679/kWh |

-Secondary energy : US¢ 2.334/kWh

(F) Overall power values

The result of overall evaluation of the power values for the Project is summarized below:

| Particulars | Unit | Coal Thermal | Combined Cycle | Overall |
|---|---|---|---|---|
| -Plant factor | % | 46.7 | 20.0 | 33.3 |
| -Annual operation | hr | 4088 | 1752 | 2920 |
| -Energy cost | US¢/kWh | 2.334 | 3.484 | 2.679 |
| -Annual fixed cost 1/ | US$/kW | 145.63 | 75.94 | 110.78 |
| -Fixed portion of unit energy cost | US¢/kWh | 3.562 | 4.335 | 3.794 |
| -Overall energy value | US¢/kWh | 5.896 | 7.819 | 6.473 |
| -Secondary energy value | US¢/kWh | 2.334 | ----- | 2.334 |

1/: (Capacity value) x 294/320
where, 320 MW---Installed capacity
294 MW---90% dependable power

## 2.5 Preliminary Design and Work Quantity

Construction work quantities of each structure were estimated based on preliminary designs. The designs are described in Chapter 5.

## 2.6 Construction Cost Estimate

The optimization studies were worked out on the basis of the construction method, time schedule and unit construction prices, which were examined and worked out for a prospective development scale of the Project.

## 2.7 Economic Comparison

The development scale was determined by comparing the capitalized net benefit of various alternative scales based on the net benefit maximization criteria.

The net benefit was obtained for each alternative in accordance with the procedure as shown in Fig. D3. The net benefit was assessed by capitalizing the annual net benefit for the evaluation period of 60 years from the commencement of the design work, with a discount rate of 9.5 per cent.

The discount rate above was provided to the Study Team by EİE as the opportunity cost of capital (OCC) of the energy sector in Turkey.

## CHAPTER 3. SITE AND TYPE OF ERMENEK DAM

An economic comparison was made to select the site of the Ermenek Dam between the 2 conceivable sites: I-B and I-C. The comparison was made for a dam crest elevation of 600 m because of the limitation of dam height at I-B site due to the landslide debris deposits located on the left bank.

After selecting an arch dam in the I-C site, another comparison was made to select the optimum dam axis among the 3 prospective axes: I-Ca, I-Cb, I-Cc.

### 3.1 Type of Ermenek Dam

Plan and profile of the I-B dam are given in Plates A4 to A6. As shown in the plates, the crest length amounts to 1,300 m. Accordingly, rockfill type is conceived suitable to the site and only the type practically possible, except for type of core. At this level of the study, the core is assumed as a earth core type.

Plan and Profile of the I-C dam are given in Plates A35 and A37. The dam axis was assumed at I-Ca for its relatively low dam height and the shape of valley. As shown in the plate, the valley is very narrow. A concrete arch dam was considered the least cost dam type. The dam shape was preliminarily designed as a parabolic type.

Results of the comparison of above 2 dams are as summarized below:

| No. | Description | Unit | I-B Site | I-C Site |
|---|---|---|---|---|
| (1) | Dam type | | rockfill | concrete arch |
| (2) | Dam height above foundation level | m | 91 | 110 |
| (3) | High water level | m | 595 | 595 |
| (4) | Low water level | m | 550 | 550 |
| (5) | Drawdown | m | 45 | 45 |
| (6) | Excavation volume | 1000 $m^3$ | 7,100 | 150 |
| (7) | Dam volume | 1000 $m^3$ | 14,700 | 150 |
| (8) | Direct construction cost of civil works | mil.$ | | |
| | - river diversion works | | 12.1 | 2.6 |
| | - dam & spillway | | 122.9 | 31.2 |
| | - grout curtain | | 7.3 | 38.1 |
| | - power waterway | | 71.1 | 62.4 |
| | - powerhouse | | 16.5 | 16.5 |
| | - physical contingency (15 %) | | 34.4 | 22.7 |
| | Total | | 264.3 | 173.5 |

As shown in the table above, the costs of I-B scheme are higher than that of I-C scheme in its river diversion works, dam and spillway, power waterway; while it is lower in the grout curtain costs.

Total civil works costs of the I-B scheme were estimated to be higher by 90.8 million US$ than that of I-C scheme, while the power outputs of the 2 sites were the same to each other. In addition, such a high dam as exceeding 100 m could not be constructed at the I-B site.

It was concluded that the I-C site was more advantageous even for a development scale of HWL 595 m. This economic advantage would become more prominent for a larger development scale.

### 3.2 Optimum Axis in the I-C Dam Site

As described in Sub-section 5.3.3 of Main Report, it has been judged that all the 3 proposed dam axes in the I-C site have the rock foundations where a high arch type dam can be constructed, and that the grout curtain could also be technically constructed.

A preliminary design of arch dam was worked out for the following 9 alternatives of dam axis and crest elevation:

| | |
|---|---|
| For I-Ca dam axis: | 640, 660, 670 m |
| For I-Cb dam axis: | 640, 660, 670 m |
| For I-Cc dam axis: | 660, 680, 700 m |

A comparative study of the Ermenek Dam was worked out on the 3 alternative dam axes proposed in the gorge; I-Ca, I-Cb, I-Cc (Plate A34). Through the preliminary design and cost comparison, it has been revealed that the dam axis I-Cc is apparently superior to I-Ca and I-Cb axes, in terms of the construction cost, when the dam crest elevation is set at 650 m or higher.

Concrete volume curves of the arch dams are constructed for the 3 dam axes as shown in Fig. D13, with alternative dam crest elevation as the abscissa. The net dam concrete volume will be quite different by axis as compared below for the dam crest elevation of 670 m for example:

Dam Concrete Volume

| Dam Axis | $m^3$ | % to I-Cc |
|---|---|---|
| I-Ca axis | 517,000 | 206 |
| I-Cb axis | 400,000 | 159 |
| I-Cc axis | 251,000 | 100 |

Excavation volume curves for the dams are presented in Fig. D13 for the 3 dam axes. The excavation volume will also be quite different by axis as compared below for the dam crest elevation of 670 m:

Dam Excavation Volume

| Dam Axis | $m^3$ | % to I-Cc |
|---|---|---|
| I-Ca axis | 1,320,000 | 272 |
| I-Cb axis | 1,585,000 | 327 |
| I-Cc axis | 485,000 | 100 |

The dam axis I-Cc will have the least volumes both in the dam concrete and excavation. I-Ca will have the largest concrete volume, while I-Cb will have the largest excavation volume.

The dam at I-Ca has the longest crest length, resulting in the largest concrete volume (see Plates A35 and A37). The largest excavation volume at I-Cb axis is resulted from the thick weathered zone on the right bank (see Plate G6).

The construction cost of dam concrete and excavation works will be the least at the axis I-Cc as shown in Fig. D13.

The unit price of dam concrete was estimated at 130 US$/$m^3$ based on the assumed quantities of 270,000 $m^3$, and was applied to the above cost estimate. Although this unit cost will be lower for a concrete volume much larger than 270,000 $m^3$, the cost advantage of I-Cc will not change.

Furthermore, since the cost difference amounts to about US$27 million between the least cost axis I-Cc and the second axis I-Cb, the superiority of I-Cc axis will not change even including other advantageous and disadvantageous factors for I-Cc axis compared to I-Ca and I-Cb, such as (1) some decrease of the length of headrace tunnel at I-Cc axis; (2) some increase of grout curtain area for I-Cc; (3) some increase in length of access tunnel to I-Cc dam site.

The above advantage of I-Cc is clearly attributable to the narrow and steep V-shaped valley topography and the rock conditions without thick and highly weathered zone (see Plate G7).

# CHAPTER 4. TYPE OF DEVELOPMENT

## 4.1 2-step Development with a Downstream Dam

An economic comparison was made between 1-step and 2-step dam development plans for the available head of 312 m between elevations 645 m and 333 m. The elevation of 645 m was provisionally selected as a prospecting level at that stage.

The comparison was made for the following 5 alternative schemes:

| | |
|---|---|
| I-C-650: | 1 step development with a dam at the I-C site having a crest elevation of 650 m, an underground pressure shaft and power house, an Erik Diversion Scheme, and a tailrace tunnel to have a tailwater level at El. 333 m |
| II-A-400: | 2-step development with the above I-C-650 dam, a downstream dam at the II-A site having a crest elevation at 400 m, a tailwater level at 333 m (see Plate A7 for a schematic profile) |
| II-A-450: | the same with the above II-A-400 except for the crest elevation of downstream dam, set at 450 m (Plate A11) |
| II-B-400: | the same with the above II-A-400 except for the site of downstream dam, located at II-A (Plate A15) |

II-B-450: the same with the above II-B-400 except for the crest elevation of downstream dam, set at 450 m (Plate A19)

Preliminary designs of the II-A and II-B dams are shown in Plates A7 to A22.

Principal features and results of the economic comparison are summarized below for the 3 bests among the 5 alternatives:

| No. | Description | Unit | I-C-650 | II-A-400 | II-A-450 | II-B-400 | II-B-450 |
|---|---|---|---|---|---|---|---|
| (1) | Dam step | | 1 | 2 | 2 | 2 | 2 |
| (2) | Installed capacity | MW | 270 | 260 | 270 | 250 | 260 |
| | - Ermenek P.S. | MW | 270 | 200 | 160 | 200 | 160 |
| | - Second P.S. | MW | - | 60 | 110 | 50 | 100 |
| (3) | Annual energy | GWh | | | | | |
| | - firm | | 799 | 752 | 777 | 741 | 763 |
| | - secondary | | 134 | 143 | 143 | 144 | 141 |
| | total | | 933 | 895 | 920 | 885 | 904 |
| (4) | Length of headrace tunnel of Ermenek Dam | m | 9,480 | 2,315 | 1,265 | 2,315 | 1,265 |
| (5) | Height of down-stream dam | m | - | 90 | 140 | 80 | 130 |
| (6) | Embankment volume of downstream dam | MCM | - | 3.8 | 17.3 | 1.7 | 8.1 |
| (7) | Construction cost | mil.$ | 319.1 | 368.6 | 458.5 | 339.9 | 391.2 |
| | - upstream dam | | 319.1 | 252.6 | 228.5 | 252.0 | 228.5 |
| | - downstream dam | | - | 116.6 | 230.0 | 87.9 | 162.7 |
| (8) | Annual equivalent cost | mil.$ | 40.2 | 46.4 | 57.7 | 42.8 | 49.2 |
| (9) | Annual benefit including firm-up benefit of Gezende | mil.$ | 72.0 | 68.4 | 70.3 | 67.7 | 69.1 |
| (10) | Annual net benefit | mil.$ | 31.8 | 22.0 | 12.6 | 24.9 | 19.9 |

The annual equivalent cost above was obtained as construction cost x (1 + IDC) x CRF + O&M cost, where:

$$IDC = 1.095^{7yr \times 0.40} - 1$$

$$CRF = 0.0960$$

The interest during construction was calculated using a discount rate of 9.5 per cent and assuming a construction period of 7 years with a center of gravity at 40 per cent from the commissioning. The capital recovery factor was calculated assuming an assessment period of 50 years after the commissioning.

As shown in the table above, the 1-step development yields the highest annual energy among the 5 alternatives. This is because in the case of 1-step development the Erik water can be utilized for the full head of 312 m available between the HWL of the Ermenek Reservoir assumed at 645 m and the HWL of Gezende Reservoir at 333 m, while only for 112 m in the case of II-A-450 and II-B-450; and only for 62 m in the case of II-A-400 and II-B-400.

The 1-step development yields much higher net benefit compared to the 4 alternatives of 2-step development. The difference in the annual net benefit amounts to US$6.9 million between the 1-step development and the second best, II-B-400. This difference is equivalent to that in the investment cost of about US$56 million.

The 2-step development with a downstream dam was then ruled out from the further investigations and studies.

The above economic superiority will not change even for the proposed HWL of 675 m because of the following reasons:

(1) The power outputs and benefits will increase almost in the same order both in the 1-step and 2-step developments. When the crest elevation of Ermenek Dam is raised to 680 m as proposed, the annual energy will increase from 933 GWh to 1,022 GWh; the installed capacity from 270 MW to 320 MW. Almost the similar increase is expected for the same dam crest raising in the 2-step developments. Accordingly, the annual benefits of all the 5 alternatives will increase almost by the same amount compared with those for HWL of 645 m.

(2) The incremental cost accompanying to this dam crest raising is (A) incremental cost of Ermenek Dam, (B) incremental cost of generating equipment, (C) incremental cost of waterway.

(3) Of these incremental costs, (A) is common between the 1-step and 2-step developments; the difference in (B) is small as the increase in the total installed capacity is almost the same among the 5 alternatives; the main difference will arise from the incremental cost of the waterway. The incremental costs of 1-step development will be larger by about US$6 million than that of 2-step development due to the change of tunnel diameter from 5.6 m to 6.1 m. However, this difference is negligibly small compared to the economic advantage of US$56 million.

(3) Accordingly, the economic advantage of 1-step development will not change.

### 4.2 2-step Development with an Upstream Dam

The economic feasibility of the downstream and upstream Nadire Schemes was provisionally examined in conjunction with a study of the Ermenek Dam height (see Plates A23 and

A26 for schematic profiles.

A HWL of the Nadire Dam was assumed at El. 695 m in a reconnaissance study made by EİE in September 1987. However, it was assumed in this study to be El. 710 m in view of the maximum development of the head available below the confluence of the Ermenek and Günder Rivers. The potentials above the confluence were planned by EİE to be developed by 2 run-of-river type schemes: the Göktepe and the Gökdere. Because of the meandering river course of the Günder River (see Plate A38), the headrace tunnel of the Göktepe Scheme would be located on the right bank to harness the head of about 320 m by the shortest tunnel. Accordingly its power station would be located near but upstream from the confluence, of which elevation is about 710 m.

Therefore, it was necessary to set the HWL of the Nadire Scheme at about El. 710 m to develop all the head available between the confluence and the Ermenek Reservoir.

Meanwhile, the Nadire Reservoir would be located on limestone layers, which widely spread there and in the adjacent basins crossing the surface boundaries of the Ermenek Basin. Therefore, the Nadire Scheme is subject to geological conditions of these limestone.

General plan and profile of the downstream Nadire dam are shown in Plate A24 and A25, and the upstream dam in A27 and A28. The catchment area is about 1,319 km$^2$ at the dam site.

The study results are summarized below:

| No. | Description | Unit | Downstream Nadire | Upstream Nadire |
|---|---|---|---|---|
| (1) | Dam height above foundation | m | 130 | 80 |
| (2) | Dam embankment volume | MCM | 4.3 | 2.3 |
| (3) | HWL | m | 710 | 710 |
| (4) | LWL | m | 680 | 690 |
| (5) | Drawdown | m | 30 | 20 |
| (6) | Effective reservoir capacity | MCM | 84 | 28.5 |
| (7) | Mean inflow | $m^3/s$ | 32.4 | 32.4 |
| (8) | Capacity - inflow ratio | % | 8 | 3 |
| (9) | Discharge for power generation | $m^3/s.yr$ | | |
| | - firm | | 7.4 | 4.4 |
| | - secondary | | 18.0 | 20.1 |
| | total | | 25.4 | 24.5 |
| (10) | Capacity factor | % | 41.5 | 44.7 |
| (11) | Installed capacity | MW | 60 | 30 |
| (12) | Dependable peak power | MW | 22 | 7.3 |
| (13) | Annual energy | GWh | | |
| | - firm | | 62.8 | 21.0 |
| | - secondary | | 155.4 | 96.7 |
| | total | | 218.2 | 117.7 |
| (14) | Construction cost | mil.$ | 123.4 | 80.2 |
| (15) | Annual equivalent cost | mil.$ | 14.2 | 9.2 |
| (16) | Annual benefit | mil.$ | 8.0 | 3.7 |
| (17) | Annual net benefit | mil.$ | -6.2 | -5.5 |

The annual equivalent cost above was obtained in the same way as described in Section 4.1 except for the construction period assumed at 5 years.

As shown in the table above, both the upstream and downstream Nadire Schemes yield minus net benefit, that is, these schemes would not be economically viable under the power values and discount rate used. Accordingly, it was judged that the Nadire Scheme should not be combined with the Ermenek Project, and that the development scale of the Ermenek Project should be optimized by itself.

### 4.3 Relation with Development Plans of Upper Ermenek Basin

The proposed Ermenek Reservoir will extend beyond the Nadire dam site, or up to a point about 3 km downstream from the confluence of the Ermenek River and Günder River. Accordingly, for the development of hydropower potential of the Upper Ermenek River above the HWL of 675 m, a new planning study will be required.

Since no hydrological and geological surveys have been made yet, only a provisional idea conceivable for the Upper Ermenek River is shown in Plate A38 for reference.

### 4.4 Erik Diversion Scheme

The Erik River has an average runoff of 4.1 $m^3/s$ with a stable flow duration (see Fig. D14). By constructing a tunnel, most of the Erik flow can be diverted to a headrace tunnel of the Project.

An intake weir site was first conceived on the Erik River at an altitude of about 700 m, from where the Erik flow could be diverted by a tunnel of about 1,500 m long. The weir site was, however, shifted later to an upstream point near the Erik Spring because a large scale active landslide was found on the upstream reaches in the Additional Detailed Investigation Stage.

An economic comparison was made for the 2 cases; with and without the Erik Diversion Scheme under the following conditions:

| | | | |
|---|---|---|---|
| (1) | Dam site | : | I-Cc |
| (2) | Dam crest elevation | : | 680 m |
| (3) | HWL | : | 675 m |
| (4) | LWL | : | 615 m |
| (5) | Drawdown | : | 60 m |
| (6) | Tailwater level | : | 333 m |
| (7) | Erik intake weir site | : | Upstream |

Results of the economic comparison are summarized below:

| No. | Description | Unit | Without Erik | With Erik |
|---|---|---|---|---|
| (1) | Installed capacity | MW | 300 | 320 |
| (2) | 90% dependable power | MW | 290 | 294 |
| (3) | Annual energy | GWh | | |
| | - firm | | 865 | 925 |
| | - secondary | | 78 | 97 |
| | total | | 943 | 1,022 |
| (4) | Economic construction cost | mil.$ | 336.3 | 342.3 |
| (5) | Annual equivalent cost | mil.$ | 33.0 | 33.6 |
| (6) | Annual benefit excluding firm-up benefit of Gezende | mil.$ | 60.0 | 62.5 |
| (7) | Annual net benefit | mil.$ | 27.0 | 28.9 |

The annual equivalent cost above was obtained in the same way as described in Sub-section 4.1.

As shown in the table above, the Erik Diversion Scheme will increase the firm energy by 60 GWh, the secondary energy by 19 GWh, and the annual net benefit by US$1.9 million. It was concluded that the Erik Diversion Scheme should be included in the Project.

## CHAPTER 5. OPTIMIZATION OF PROJECT COMPONENTS

### 5.1 Discharge Capacity of Spillway

The dam design flood was determined with the PMF concept. The spillway discharge capacity was determined through reservoir routing calculation taking account of the storage effect of reservoir.

The larger spillway capacity reduces the flood storage capacity to be allocated above HWL, resulting in a lower dam height. In this case the spillway cost increases while the dam cost decreases; and vice versa.

The economic combination of the flood storage capacity and the spillway discharge capacity was obtained with the least cost criteria together with the criteria of minimum discharge capacity. A concept of the minimum capacity of flood discharging facilities from dam was introduced to avoid selection of an extremely small capacity. The minimum capacity of flood discharging facilities from dam was determined at 2,200 $m^3/s$, as the 1/200-year probable flood at the dam site. This flood frequency was selected in accordance with the standard for the spillway design discharge in Japan.

#### 5.1.1 Design flood

A Probable Maximum Flood (PMF) was originally derived by EİE. The PMF was reviewed and revised to two new PMFs: PMF-1 for January, PMF-2 for February (see Sub-section 5.5.5 of Main Report for details). These are characterized as shown below:

| Description | Unit | Original | PMF-1 | PMF-2 |
|---|---|---|---|---|
| 1. Peak discharge | $m^3/s$ | 3,700 | 5,900 | 5,400 |
| 2. Base flow incl. snow-melt | $m^3/s$ | 1,300 | 100 | 1,300 |

### 5.1.2 Dam design freeboard

The dam crest elevation is determined with a design freeboard to be provided above the design flood water level. The minimum freeboard was determined at 1.5 m as follows:

The maximum wave height was estimated to be about 1.6 m by the SMB method for a wind speed of 20 m/s on the 10 minutes average. This wind speed was assumed with reference to the maximum instantaneous wind speed of 23.2 m/s observed at Mut in the 21 years from 1966 to 1986. The prevailing wind direction was NW.

The Oymapinar arch dam located to the west of Ermenek Basin has a freeboard of 1.4 m above the full supply level, and 2.4 m including the parapet height.

On the basis of above figures and taking into consideration the dam type of concrete arch, the dam design freeboard was determined to be 1.5 m at the minimum.

### 5.1.3 Type of spillway

In view of the dam construction in the V-shaped valley having a narrow width of 5 to 20 m at the river water level, a combination of the bottom outlets, a tunnel spillway and a non-gate-controlled overflow spillway at the dam crest was selected.

Two bottom outlets will be provided in the dam body. The dam site is located on a limestone block. In case serious water leakage is found during the initial reservoir filling, the reservoir water level could be lowered by operating these bottom outlets. The outlet gates will have the dimension of 2.5 m wide, 4.0 m high, and the center elevation at 545 m. The discharge capacity will be 670 $m^3/s$ at LWL of 615 m, and 910 $m^3/s$ at HWL of 675 m. This capacity is provided to avoid a rise of the reservoir water level or to lower it when leakage is found.

A tunnel spillway was selected as the main discharge facility. This type has such advantages as (1) to discharge the flood water far downstream from the dam foundation, (2) to enable, together with bottom outlets, an early commencement of the initial reservoir filling, without waiting completion of the dam construction works. The capacity of tunnel spillway will be determined through the optimization study.

An overflow crest of 40 m long was selected as the secondary spillway. The overflow crest elevation is set so that the maximum operating head will be about 3 m, or the unit discharge over one meter of the overflow crest will be about 10 $m^3/s$. The overflow crest length of 40 m was determined in view of the narrow bottom of the valley. However, the discharge capacity will finally be determined through the optimization study depending on the maximum flood water level.

When the above overflow type spillway is selected as a main spillway without the tunnel spillway, a discharge capacity of about 1,300 $m^3/s$ will be necessary. To release this amount of discharge from the overflow crest of 40 m for example, the operating head of about 6.4 m will be necessary, and a unit discharge of 33 $m^3/s$ will flow down over one meter of the overflow crest. This large unit discharge may endanger a high and thin arch type dam. If the length of overflow crest is increased, this unit discharge will decrease but much of the overflowing jet will fall on the valley wall of both sides and will rush down the wall surface towards the valley bottom. Such a flow condition was judged not suitable in view of the wall slope protection. The overflow types was thus selected as the secondary spillway.

### 5.1.4 Alternative spillways

The following 4 alternatives of tunnel spillway were studied for the case of HWL at 675 m by fixing the dimension of overflow spillway and bottom outlets:

| Case No. | B (m) | H (m) | El. (m) | Nos. | Q at HWL ($m^3$/s) |
|---|---|---|---|---|---|
| Case 1 | 3.0 | 7.0 | 630.0 | 1 | 565 |
| Case 2 | 3.0 | 7.0 | 630.0 | 2 | 1,130 |
| Case 3 | 3.0 | 7.0 | 630.0 | 3 | 1,695 |
| Case 4 | 3.0 | 7.0 | 630.0 | 4 | 2,260 |

Note: B denotes gate width, H gate height, El. gate center elevation, Q discharge of only tunnel spillway.

### 5.1.5 Flood operation rule

Since the Project purpose is only the hydropower generation, the flood inflow will be released in accordance with the inflow = outflow rule in principle, or the gates will be operated to keep HWL as far as possible. Accordingly, the flood routing study of PMF was made assuming the initial reservoir water level at HWL.

However, in order to attain an easy gate operation and gradual increase of the spillway discharge, the following gate operation rule was introduced:

| Discharge Facility | Operation Starting Water Level | Full Open Water Level |
|---|---|---|
| Tunnel Spillway | 675.30 m | 675.50 m |
| Bottom Outlets | 675.50 m | 675.70 m |

In this gate operation rule, excess water of up to about 12 $m^3/s$ can be released only from the overflow crest without any gate operation.

### 5.1.6 Flood routing study

A flood routing study was carried out for the 3 PMFs and 4 alternative spillways. The results are as summarized below:

| Description | Unit | Alternative No. of Spillway | | | |
|---|---|---|---|---|---|
| | | Case 1 | Case 2 | Case 3 | Case 4 |
| 1. Original PMF | | | | | |
| - Max. rise above HWL | m | 6.28 | 3.32 | 2.07 | 1.10 |
| - Max. Q | $m^3/s$ | 2,040 | 2,600 | 2,880 | 3,270 |
| 2. PMF-1 for January | | | | | |
| - Max. rise above HWL | m | 4.05 | 3.07 | 2.43 | 1.90 |
| - Max. Q | $m^3/s$ | 1,970 | 2,540 | 2,960 | 3,420 |
| 3. PMF-2 for April | | | | | |
| - Max. rise above HWL | m | 6.04 | 3.23 | 2.42 | 1.81 |
| - Max. Q | $m^3/s$ | 1,980 | 2,580 | 2,960 | 3,400 |

The above table shows the following:

(1) The original PMF is the most critical among the 3 PMFs for the smaller spillway Cases 1 and 2. However, since its unitgraph has been judged inadequate to the Ermenek River, this result is only for reference and will not be used in the further design works of the Project.

(2) The PMF-2 is more critical than PMF-1 for the smaller spillway Cases 1 and 2, while PMF-1 becomes critical for the larger spillway Cases 3 and 4.

Accordingly, the maximum flood water level was obtained for each alternative spillway as shown below:

<u>DFWL for I-Cc axis, HWL at 675 m</u>

| Description | Unit | Alternative No. of Spillway | | | |
|---|---|---|---|---|---|
| | | Case 1 | Case 2 | Case 3 | Case 4 |
| 1. Design flood | | PMF-2 | PMF-2 | PMF-1 | PMF-1 |
| 2. Max. rise above HWL | m | 6.04 | 3.23 | 2.43 | 1.90 |
| 3. Max. outflow in total | $m^3/s$ | 1,980 | 2,580 | 2,960 | 3,420 |

The above relationship is shown in Fig. 5.5.3.

### 5.1.7 Optimum discharge capacity of spillway

Construction costs were estimated for each combination of spillway capacity and dam height. An incremental dam cost was estimated compared to the dam cost without any flood storage (design flood water level = HWL), by multiplying a unit cost of 0.49 million US$ per meter of dam height around a crest elevation of 680 m, to the flood water depth above HWL.

A total cost of the above spillway cost and incremental dam cost is shown below (Fig. D15). Case 1 is the least cost, followed by Cases 2, 3 and 4 in the order. The cost saving effect by the lower dam height is smaller than the incremental cost of the spillway.

| | | Case-1 | Case-2 | Case-3 | Case-4 |
|---|---|---|---|---|---|
| 1. Spillway discharge capacity | m3/s | 1980 | 2580 | 2960 | 3420 |
| 2. Max. flood level above HWL | m | 6.04 | 3.23 | 2.43 | 1.90 |
| 3. Dam cost for flood surcharge | m.$ | 2.96 | 1.58 | 1.19 | 0.93 |
| 4. Spillway cost | m.$ | 3.36 | 5.60 | 8.40 | 11.20 |
| 5. Total cost | m.$ | 6.32 | 7.18 | 9.59 | 12.13 |

Of the least cost Case 1, the total discharge capacity is 1,980 $m^3/s$, which is smaller than the minimum capacity of flood discharging facilities. Then the second least cost Case 2 was selected at this feasibility study stage.

For a case of HWL of 675 m, the spillway discharge capacity and the dam design flood water level were determined based on Case 2 as shown below:

- Maximum outflow (rounded) : 2,600 $m^3/s$
- Dam design flood water level (rounded) : 678.3 m
- Tunnel spillway design discharge: 1,160 $m^3/s$
- Crest spillway design discharge : 500 $m^3/s$
- Bottom outlet design discharge : 940 $m^3/s$
- Freeboard : 1.7 m
- Dam crest elevation : 680.0 m

The total discharge capacity of spillway and bottom outlets will be 2,600 $m^3$ at the design flood water level of 678.3 m, which will be larger than the 1/200-year probable flood of 2,200 $m^3/s$.

In addition to the above spillway system, an emergency spillway of overflow type will be provided on the dam crest to cope with such an incident as malfunctioning of one of 4 gates of the tunnel spillway and bottom outlets. The length of this emergency overflow crest was selected at 60 m. In this case, the total crest length including the primary overflow crest will be 100 m, which was selected in conjunction to the dam crest length of 165 m.

### 5.2 Drawdown of Reservoir

A comparative study was made for the 5 alternative drawdown of the reservoir. The study was made for the prospective HWL of 675 m. The principal features and economic indices are as summarized below:

| No. | Description | Unit | Drawdown 30m | 45m | 60m | 75m | 90m |
|---|---|---|---|---|---|---|---|
| (1) | LWL | m | 645 | 630 | 615 | 600 | 585 |
| (2) | Effective reservoir capacity | MCM | 1,329 | 1,879 | 2,339 | 2,715 | 3,009 |
| (3) | Capacity-inflow ratio | % | 91 | 128 | 160 | 185 | 205 |
| (4) | Mean reservoir water level of 42 years | m | 668 | 664 | 660 | 648 | 644 |
| (5) | Installed capacity | MW | 273 | 297 | 320 | 328 | 331 |
| (6) | 90% dependable power | MW | 260 | 276 | 294 | 274 | 272 |
| (7) | Annual energy | GWh | | | | | |
| | - firm | | 825 | 892 | 925 | 950 | 952 |
| | - secondary | | 179 | 127 | 97 | 52 | 43 |
| | total | | 1,003 | 1,018 | 1,022 | 1,002 | 995 |
| (8) | Economic construction cost | mil.$ | 322.0 | 335.8 | 342.1 | 348.6 | 351.8 |
| (9) | Annual cost | mil.$ | 40.5 | 42.3 | 43.1 | 43.9 | 44.3 |
| (10) | Annual benefit including firm-up benefit | mil.$ | 73.3 | 76.1 | 78.8 | 76.9 | 75.6 |
| (11) | Annual net benefit | mil.$ | 32.7 | 33.8 | 35.7 | 33.0 | 31.3 |

Fig. D17 shows 3 curves of the installed capacity, 90% dependable power, and the annual net benefit, with the reservoir drawdown as abscissa.

As shown in the figure and above table, the 90% dependable power has a peak at the drawdown of 60 m. This is because the mean reservoir water level lowers in accordance with the increase of drawdown. As a result, the annual energy will start decrease when the drawdown exceeds 60 m.

The reservoir drawdown was thus determined at 60 m.

### 5.3 Diameter of Headrace Tunnel

(1) Basic approach

The economic diameter was obtained with the least cost criteria, taking into consideration the power loss due to the hydraulic loss of head in the headrace tunnel as an extra cost.

(2) Construction cost

The annual equivalent of construction cost was estimated with a discount rate of 9.5 per cent, a construction period of 5 years, and an assessment period of 50 years.

(3) Annual power loss

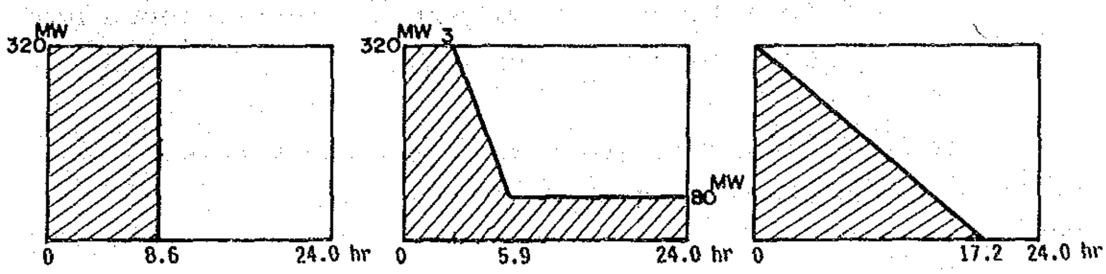
The annual power loss ($C_1$) and energy loss ($E_1$) were estimated assuming the daily operation pattern of Case 2 shown below:

$$E_1 = \text{Sum}(P_1\ dT)$$

$$= P_1 \times 8{,}760 \text{ hr} \times 0.363 \quad \text{----->} \quad E_o \quad \text{for Case 1}$$

$$= 0.42\ E_o \quad \text{for Case 2}$$

$$= 0.50\ E_o \quad \text{for Case 3}$$




Case 1 ($E_o$) Case 2 ($0.42E_o$) Case 3 ($0.50E_o$)

In general, Case 1 yields a higher energy loss, resulting in a larger tunnel diameter. Cases 2 and 3 yield lower energy losses or 42 to 50 per cent of Case 1, resulting in a smaller diameter.

The daily operation pattern of Case 1 is of the extreme peak operation, and is not practical in terms of its momentary change of load at the daily start and stop. Case 3 is another extreme and is not practical in terms of very low load at the stop. Case 2 is considered most realistic and close to the actual conditions in terms of energy loss, and was adopted in this study.

(4) <u>Work quantities per meter of headrace tunnel</u>

The work quantities of the headrace tunnel were obtained for alternative tunnel diameters with the following assumptions:

(A) Lining thickness is 0.08 times the tunnel diameter.

(B) Thickness of the extra excavation is 0.20 m.

(C) Tunnel excavation will be made by the following method:

For 50 % length: without support without NATM
For 50 % length: by NATM

(D) Consolidation grout will be made for two thirds of the tunnel length.

(E) Quantities of shotcrete, rockbolts and consolidation grout works were expressed as functions of the tunnel diameter (D) based on the quantities for D = 6.1 m as described below:

<u>Shotcrete</u> will be provided for the excavated surface of the tunnel in such tunnel sections where NATM is required. The thickness is set at 0.02 times the tunnel diameter, or D/50.

<u>Rockbolts</u> will be placed in such tunnel sections where NATM is required. The rockbolts will be placed on the arch and part of the side walls, or two thirds of the tunnel perimeter. The length of one rockbolt is set at one third of the tunnel diameter. The number of rockbolts per $m^2$ of tunnel surface is set at 0.5 nos.

<u>Consolidation grout</u> of 3 m long will be placed at 6 holes per section with section interval of 3 m.

(5) Least cost diameter

The least cost diameter for the discharge of 116.6 $m^3/s$ was obtained to be 6.3 m as of $n = 0.012$ and the operation pattern of Case 2. However, the cost differences are as small as less than US$ 0.1 million per annum for the diameter range from 6.0 m to 6.6 m.

(6) Diameter by empirical formula

Meanwhile the following empirical formula gives a diameter of 6.1 m for the discharge of 116.6 $m^3/s$:

$$D_c = 0.62\ Q^{0.48}$$

(7) Diameter of headrace tunnel

This diameter is within the above range. The annual cost of this diameter (6.1 m) is US$ 7.15 million, which is higher than that of 6.3 m (US$ 7.08 million) only by US$ 0.07 million per annum.

Accordingly, the internal diameter of headrace tunnel was determined to be 6.1 m by the above empirical formula. This diameter yields a hydraulic gradient of 1/747 at $n = 0.012$, and 1/549 for $n = 0.014$.

Diameters for the other discharges were obtained by the above formula.

## 5.4 Route of Headrace Tunnel

A comparative study was carried out on 5 alternative routes of headrace tunnel shown in Plates A30 to A32. The principal features of these routes are as shown below:

| Description | Alternative Route No. A | Ba | Bb | Bc | C |
|---|---|---|---|---|---|
| (1) Length of headrace tunnel (m) | 8,800 | 8,920 | 8,950 | 9,020 | 9,180 |
| (2) Incremental length of headrace tunnel (m) | 0 | +120 | +150 | +220 | +380 |
| (3) Length of work adits (m) | 3,360 | 2,460 | 2,320 | 2,220 | 1,940 |
| (4) Incremental length of work adits (m) | 0 | -900 | -1,040 | -1,140 | -1,420 |
| (5) (2) + (4) (m) | 0 | -780 | -890 | -920 | -1,040 |

The alternative route A has a straight alignment between the intake and surge tank sites. Because of the topography along the route, it incurs long work adits amounting to 3,360 m in total.

The economic route was obtained with the least cost criteria of a sum of the construction costs of work adits and headrace tunnel and the power loss due to the hydraulic loss of head in the headrace tunnel. The alternative routes were selected so that they pass always the mountain side of the contour line corresponding to HWL.

The locations of work adits were selected with the following criteria:

(1) The maximum length of one tunnel section should be about 4,000 m or less.

(2) The portal of work adits should be selected at those places which are located on ridges but not in the creek. The site should have a suitable topography for construction of an access road to the adit portal.

(3) The work adit route should be selected by avoiding the landslide debris deposits, which are located in the middle part of the headrace tunnel route.

(4) The total length of work adits should be as short as possible.

The annual equivalent of construction cost and the annual power loss were estimated in the similar way to that adopted for the calculation of economic diameter.

Each work adit was planned to have 2 full lanes of road to pass construction equipment of maximum width of 2.5 m. The sections of work adits are as shown in Plate No. 26. It is assumed that Adit Type I be applied to the first 100 m of each work adit, while Type II to the remaining part of the work adits. The adit Type I will be fully concrete-lined with steel support. Type II will not be concrete-lined, but it was assumed that its 50 per cent in length be excavated by NATM.

Under the above conditions of power loss and construction costs, the annual costs were obtained as shown in Fig. D18. The figure shows that the Route Bb is the least cost among the 5 alternatives.

### 5.5 Diameter of Pressure Shaft

The economic diameter of the pressure shaft was obtained in the similar way to that for the headrace tunnel. A clearance between the steel liner and the excavated surface of the shaft was set at 0.30 m, assuming that the welding works of the liner be made only from the inside.

The economic diameter of steel liner was studied for the 2 cases: single lane of the shaft; two lanes.

### 5.5.1 Single lane shaft

The annual equivalent construction cost was estimated with a discount rate of 9.5 per cent, a construction period of 3 years, and an assessment period of 50 years.

The annual power loss was estimated by the same equation as for the economic diameter of the headrace tunnel. The daily operation pattern of Case 2 was adopted in accordance with the same consideration to the economic diameter of headrace tunnel.

The work quantities of the pressure shaft were obtained for alternative diameters with the following assumptions:

(1) Working clearance between the steel liner and the tunnel surface is 0.30 m.

(2) Thickness of the extra excavation is 0.10 m.

(3) Tunnel excavation will be made by the following method:

For 50 % length: without support without NATM
For 50 % length: by NATM

(4) Consolidation grout will be made for two thirds of the tunnel length.

(5) Quantities of shotcrete, rockbolts and consolidation grout works were expressed as functions of the excavation diameter (D) based on the quantities for excavation diameter of D = 6.0 m as described below:

Shotcrete will be provided for the excavated surface of the tunnel in such sections where NATM is required. The thickness is set at 0.02 times the excavation diameter, or D/50.

Rockbolts will be placed in such sections where NATM is required. The rockbolts will be placed on two thirds of the excavation perimeter. The length of one rockbolt is set at 2.0 m. The number of rockbolts per section is set at 7 nos, with section intervals of 1.0 m.

Consolidation grout of 2 m long will be placed at 6 holes per section with section interval of 3 m.

The least cost diameter of steel liner for the discharge of 116.6 $m^3/s$ was obtained to be 4.2 m. However, the differences of annual cost are as small as within US$ 0.1 million for the diameter range from 4.1 m to 4.9 m.

Meanwhile the following empirical formula gives an average diameter of 4.9 m for the discharge of 118 $m^3/s$:

$$D_s = 1.12\ H^{-0.12}\ Q^{0.45}$$

This diameter is within the above range. Accordingly, the diameter of single lane steel liner was determined to be 4.9 m.

**5.5.2 Two lane shaft**

In the same way as for the single lane shaft, the least diameter for the discharge of 116.6 $m^3/s$ was obtained to be 3.3 m. However, differences of annual cost is again as small as within US$ 0.1 million for the diameter range from 3.1 m to 3.7 m.

The above empirical formula gives an average diameter of 3.6 m for the discharge of half the 116.6 $m^3/s$. This diameter is within the above range. Accordingly, the diameter of two lane steel liner was determined to be 3.6 m.

### 5.5.3 Selected lane number and diameter

An economic comparison was made between the single lane and two lanes as summarized below:

| Items | Unit | Single Lane | Two Lanes |
|---|---|---|---|
| 1. diameter of steel liner | m | 4.9 | 3.6 |
| 2. tunnel cost | Mil.$ | 1.43 | 1.99 |
| 3. steel liner cost | Mil.$ | 15.81 | 14.95 |
| 4. total cost | Mil.$ | 17.24 | 16.94 |
| 5. annual equivalent of construction cost | Mil.$ | 1.90 | 1.86 |
| 6. annual power loss | Mil.$ | 0.33 | 0.42 |
| 7. annual total | Mil.$ | 2.22 | 2.28 |

As shown in the table above, the single lane has the higher construction cost, lower power loss, and lower annual total cost compared to the two lanes. However, the difference is as small as US$ 0.06 per annum.

In the case of single lane, the maximum excavation diameter will be 5.9 m including working clearance of 0.3 m and extra excavation of 0.2 m. The vertical height becomes 8.3 m in an inclined shaft of 45 degree, and some difficulties are expected in construction works. In the case of two lane shaft, it becomes 6.5 m; and the shaft excavation equipment can be used for the double length compared to the single lane, resulting in a higher depreciation of the excavation equipment.

Accordingly, the two lanes were adopted in the Project, and its diameter was determined at 3.6 m.

### 5.6 Type of Penstock and Powerhouse

An economic comparison was made for aboveground and underground types of penstock and power station under the following conditions:

| | | | |
|---|---|---|---|
| (1) | Dam site | : | I-Cc |
| (2) | Dam height | : | 190 m |
| (3) | HWL | : | 675 m |
| (4) | LWL | : | 615 m |
| (5) | Drawdown | : | 60 m |
| (6) | Tailwater level | : | 333 m for the underground type with a tailrace tunnel<br>337 m for the aboveground type without tailrace tunnel |

Plan and profile of the aboveground penstock are shown in Plate A29, and of the underground type in Plate Nos. P17 to P19. An economic comparison was made as summarized below:

| No. | Description | Unit | Aboveground | Underground |
|---|---|---|---|---|
| (1) | Length of penstock | m | 1,035 | 440 |
| (2) | Installed capacity | MW | 315 | 320 |
| (3) | 90% dependable power | MW | 294 | 294 |
| (4) | Annual energy | GWh | | |
| | - firm | | 911 | 925 |
| | - secondary | | 97 | 97 |
| | total | | 1,008 | 1,022 |
| (5) | Economic construction cost | mil.$ | 341.5 | 342.3 |
| (6) | Annual equivalent | mil.$ | 42.3 | 33.6 |
| (7) | Annual benefit excluding firm-up benefit of Gezende | mil.$ | 62.1 | 62.5 |
| (8) | Annual net benefit | mil.$ | 19.8 | 20.1 |

The aboveground and underground plans are almost comparable to each other although the underground type yields a slightly higher net benefit. This is, however, mainly attributable to the additional head of 3 m available for the underground type.

The underground type has an operational advantage because of its shorter length of the pressure shaft: 1,035 m in horizontal length for the aboveground type; 440 m for the underground type. Also the underground type will not affect the natural environment on the route and will be free from slope protection works. Thus the underground pressure shaft and power station were adopted for the Project.

**5.7 Location of outlet of tailrace tunnel**

A comparative study was carried out on the 4 alternative routes and outlet locations of the tailrace tunnel as shown in Plate A33. The river profile around the outlets is also shown in Fig. D19. As shown in this figure, there are rapids of about 2 m high and the Route B is located at an immediate downstream point from this rapids.

Locations of the outlet were selected with the following criteria:

(1) The outlet should be selected at those places that are located on ridges but not in the creek.

(2) The outlet should be selected by avoiding the landslide area, which is located in the middle part of the conceivable area of the outlet.

Principal features and economic indices are summarized below for alternative routes compared to those of Route A.

| Description | Unit | Alternative No. A | B | C | D |
|---|---|---|---|---|---|
| (1) Length of tailrace tunnel (m) | | 1,200 | 1,713 | 1,838 | 2,298 |
| (2) Incremental length of tailrace tunnel | m | 0 | +513 | +638 | +1,098 |
| (3) Minimum tailwater level (m) | | 336.0 | 329.0 | 328.0 | 326.0 |
| (4) Maximum head increased | m | - | +7.0 | +8.0 | +10.0 |
| (5) (4) / (2) x 1,000 (m/km) | | - | 13.6 | 12.5 | 9.1 |
| (6) Incremental annual cost | mil.$ | - | 0.29 | 0.36 | 0.63 |
| (7) Incremental annual benefit | mil.$ | - | 0.76 | 0.77 | 0.78 |
| (8) Incremental annual net benefit | mil.$ | - | 0.35 | 0.26 | -0.08 |

As shown in the above table and in Fig. D20, the annual benefit increases sharply from the alternative Route A to Route B, but is stagnant in the far downstream points. On the other hand, annual equivalent cost and annual power loss continues increase along with the length of the tailrace tunnel. Thus the Route B becomes the least cost among the 4 alternatives. Accordingly the Route B was selected.

## 5.8 Erik Power Station

### 5.8.1 Design discharge of Erik Diversion Scheme

(1) Flow duration curve

Based on the 3 years' daily runoff data of the Erik River, a flow duration curve was developed by the serial method. The used data cover 3 hydrological years (October through September): 1965/1966, 1969/1970, 1970/1971. The 3 years' average flow was 3.82 $m^3/s$, while the long-term average was estimated at 3.5 $m^3/s$ for the 42 years from 1946 to 1987 (see Sub-section 5.5.3 of Main Report for details).

The duration curve is shown in Fig. D14, and is summarized below:

| Probability of Exceedence (%) | Flow of the 3 years ($m^3/s$) |
|---|---|
| 4 | 6.00 |
| 5 | 5.80 |
| 10 | 5.24 |
| 20 | 4.34 |
| 50 | 3.56 |
| 90 | 2.85 |
| 95 | 2.14 |
| 100 | 2.14 |

The lowest flow in 1989 was 2.16 $m^3/s$. Since most of the river flow is fed by the Erik Spring, the 100 per cent flow of 2.14 $m^3/s$ given in the above table could be deemed as a long-term firm flow of the Erik River.

(2) Maximum diversion discharge

A free flow tunnel of the minimum working dimension has a discharge capacity of 6.0 $m^3/s$ for a coefficient of roughness of 0.014 and a hydraulic gradient of 1/1,000 with clearance of 0.3 m between the water surface and the arch crown.

The above discharge capacity of 6.0 $m^3/s$ corresponds to 4 per cent on the flow duration curve. It means that there will be little overflow from the Erik diversion weir as summarized below:

| | | |
|---|---|---|
| Discharge capacity | : | 6.0 $m^3/s$ |
| Average number of spilling days | : | 11.0 days |
| Average spilling water | : | 0.06 $m^3/s.yr$ |
| Average diversion water | : | 3.44 $m^3/s.yr$ |

It is judged that the free flow tunnel of the minimum dimension is enough for the Erik Diversion Scheme as it can divert 98 per cent of the Erik flow. The design discharge of the diversion tunnel is thus determined at 6.0 $m^3/s$.

**5.8.2 Type of Erik diversion tunnel**

The proposed route of Erik diversion waterway is shown in Plate P22. There is a large scale landslide near but downstream from the Erik Spring. This occurred in 1985 and the people living there were resettled to a safe place. The control of this landslide is considered practically impossible. Accordingly, the Erik intake weir site was selected upstream from this landslide and immediately downstream from the Spring. The Erik Spring and the weir site are located in a valley created in a limestone block, and are judged free from the landslide.

As the type of the Erik waterway, a free flow tunnel type and a pressure flow tunnel type are conceivable. Of these, the free flow type was selected based on the following considerations:

(1) A pressure flow tunnel of 2.2 m in internal diameter has the same discharge capacity of 6.0 $m^3/s$ with the above free flow tunnel at the hydraulic gradient of 1/1,000 and coefficient of roughness of 0.014.

(2) The lining thickness was assumed at 0.20 m for the free flow tunnel aiming only to improve the coefficient of roughness, while it was assumed at 0.25 m for the pressure tunnel.

(3) The construction cost of free flow tunnel will be cheaper than that of the pressure tunnel by US$ 0.78 million as shown below:

(million US$)

| Work Items | Free Flow Tunnel | Pressure Tunnel |
|---|---|---|
| Tunnel excavation | 1.64 | 1.76 |
| Lining concrete | 1.29 | 1.57 |
| Consolidation grout | - | 0.38 |
| Total | 2.93 | 3.71 |

(4) In the case of the pressure flow tunnel, a surge tank will be required as the tunnel length amounts to 3,940 m. In the case of free flow tunnel, a head tank combined with an excess water spillway will be required. The spillway could be constructed on the ground surface to guide the excess water, spilt from the head tank, to the inlet shaft, which is connected to the headrace tunnel (see Plate P24).

(5) Cost of the head tank and excess water spillway would be in the similar order to that of the surge tank. Accordingly, the free flow type would be cheaper than the pressure type also in the total cost of related work items.

The Erik diversion tunnel is thus determined to be of free flow type.