GOVERNMENT OF MALAYSIA
ECONOMIC PLANNING UNIT OF THE PRIME MINISTER'S DEPARTMENT

# MALAYSIA

# FEASIBILITY STUDY REPORT ON THE TEKAI HYDROELECTRIC POWER DEVELOPMENT PROJECT

## Volume V Design and Construction Planning

SEPTEMBER 1983

JAPAN INTERNATIONAL COOPERATION AGENCY

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## PREFACE

In response to the request of the Government of Malaysia, the Government of Japan decided to conduct a feasibility study on the Tekai Hydro-electric Power Development Project and entrusted the study to the Japan International Cooperation Agency (JICA). The JICA sent to Malaysia a survey team headed by Mr. Keiichi Takahira from March 1, 1981 to December 15, 1982.

The team exchanged views with the officials concerned of the Government of Malaysia and conducted a field survey in the Tekai Project area, in Pahang State. After the team returned to Japan, further studies were made and the present report has been prepared.

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

I wish to express my deep appreciation to the officials concerned of the Government of Malaysia for their close cooperation extended to the team.

Tokyo, August 1983

Keisuke Arita

Keisuke Arita
President
Japan International Cooperation Agency

The feasibility study report is composed of the following volumes.

Executive Summary

| | |
|---|---|
| Volume I | Main Report |
| Volume II | Survey |
| Volume III | Hydrology |
| Volume IV | Geology |
| Volume IV | Geology Appendix |
| Volume V | Design and Construction Planning |
| Volume VI | Drawings |
| Supplementary Data | Estimated Construction Cost and Unit Price |

DESIGN AND CONSTRUCTION PLANNING

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## TEKAI PROJECT

### DETAILS AND PRINCIPAL DIMENSIONS OF THE UPPER TEKAI PROJECT

**A. RESERVOIR**

| | |
|---|---|
| . Catchment Area | 1,200 $km^2$ |
| . High Water Level | EL 157.00 |
| . Low Water Level | EL 147.00 |
| . Surcharge Water Level | EL 164.00 |
| . Gross Storage Capacity | 2,040 x $10^6$ $m^3$ |
| . Effective Storage Capacity | 680 x $10^6$ $m^3$ |
| . Surface Area at High Water Level | 76 $km^2$ |

**B. UPPER TEKAI DAM**

| | |
|---|---|
| . Type | Rockfill with imperbious core |
| . Crest Level | EL 166.20 |
| . Crest Length of Dam | 350 m |
| . Dam Height | 101 m |
| . Nominal Slopes | |
| Upstream Face | 1:1.80 |
| Downstream Face | 1:1.75 |
| . Volume of Dam Embankment | 3,125,000 $m^3$ |
| Rock Material | 2,162,000 $m^3$ |
| Filter Material | 280,000 $m^3$ |
| Core Material | 604,000 $m^3$ |
| Riprap | 79,000 $m^3$ |

**C. SPILLWAY**

| | |
|---|---|
| . Type | Uncontrolled, surface, reinforced concrete structure with crest pillars, bridge, chute and stilling basin |

. Total Length 280 m
. Width 47.5 m
. Maximum Spillway Discharge at Surcharge 1,504 $m^3/s$

D. INTAKE

| | |
|---|---|
| . Size | Width 6 m x 2, Height 32 m |
| . Number | 1 |
| . Gate | |
| Type | Roller gate |
| Size | 8 m x 8 m |
| . Trashrack | Width 6 m x 2, Height 33.65 m |

E. PENSTOCK

| | |
|---|---|
| . Type | Steel innerlined pressure tunnel |
| . Diameter | 7.3 m x 1 ~ 4.6 m x 2 |
| . Number | One and two after branch |
| . Length | 578.358 m |

F. POWERHOUSE

| | |
|---|---|
| . Type | Outdoor |
| . Size | Width 31.0 m, Length 54.8 m |
| . Elevation | |
| Ground Level | EL 81.00 |
| Turbine Floor | EL 71.80 |

G. ELECTRIC EQUIPMENTS

| | |
|---|---|
| . Turbine | |
| Type | Francis |
| Number of Units | 2 |
| Rated Capacity | 75 MW x 2 |
| Rated Net Head | 75.1 m |
| Rated Discharge | 117.5 $m^3/s$ |

- Generator

| | |
|---|---|
| Number of Units | 2 |
| Normal Rating | 88.2 MVA |
| Power Factor | 0.85 |
| Phases | 3 |
| Frequency | 50 Hz |
| Voltage | 13.2 kV |
| Operating Speed | 187.5 rpm |

- Transformer

| | |
|---|---|
| Number of Units | 2 |
| Capacity | 88.2 MVA |
| Phases | 3 |
| Voltage | 13.2/132 kV |
| Cooling | Outdoor type, Oil natural, Air forced (ONAF) |

# DETAILS AND PRINCIPAL DIMENSIONS OF THE LOWER TEKAI PROJECT

## A. RESERVOIR

| | |
|---|---|
| . Catchment Area | 1,380 $km^2$ |
| . High Water Level | EL 75.00 |
| . Low Water Level | EL 70.50 |
| . Surcharge Water Level | EL 79.00 |
| . Gross Storage Capacity | $41.5 \times 10^6$ $m^3$ |
| . Effective Storage Capacity | $21.5 \times 10^6 m^3$ |
| . Surface Area at High Water Level | 6.1 $km^2$ |

## B. LOWER TEKAI DAM

| | |
|---|---|
| . Type | Concrete gravity |
| . Crest Level | EL 81.00 |
| . Crest Length of Dam | 160 m |
| . Dam height | 38 m |
| . Nominal Slopes | |
| Upstrem Face | 1:0.1 |
| Downstream Face | 1:0.75 |
| . Volume of Dam Concrete | 56,900 $m^3$ |

## C. SPILLWAY

| | |
|---|---|
| . Type | Uncontrolled, surface along the dam body with crest pillars, bride, chute and stilling basin |
| . Total Length | 77.5 m |
| . Width | 91.5 m at chute and 50 m at stilling basin |
| . Maximum Spillway Discharge at Surcharge Water Level | 1,100 $m^3/s$ |

D. INTAKE

| | |
|---|---|
| . Size | Width 9 m x 1, Height 11 m |
| . Number | 1 |
| . Gate | |
| Type | Roller gate |
| Size | 5.5 m x 5.5 m |
| . Trashrack | Width 9 m x 1, Height 15 m |

E. PENSTOCK

| | |
|---|---|
| . Type | Steel innerlined penstock in dam and covered with concrete |
| . Diameter | 5 m ~ 2.7 m |
| . Number | 1 |
| . Length | 49.782 m |

F. POWERHOUSE

| | |
|---|---|
| . Type | Outdoor |
| . Size | Width 21.2 m, Length 28 m |
| . Elevation | |
| Ground Level | EL 65.00 |
| Turbine Floor | EL 52.00 |

G. ELECTRIC EQUIPMENT

| | |
|---|---|
| . Turbine | |
| Type | Kaplan |
| Number of Units | 1 |
| Rated Capacity | 5.8 MW |
| Rated Net Head | 17.2 m |
| Rated Discharge | 40 $m^3/s$ |
| . Generator | |
| Number of Unit | 1 |
| Normal Rating | 6.8 MVA |
| Power Factor | 0.85 |
| Phases | 3 |

| | |
|---|---|
| Frequency | 50 Hz |
| Voltage | 6.6 kV |
| Operating Speed | 250 rpm |

. Transformer

| | |
|---|---|
| Number of Units | 1 |
| Capacity | 6.8 MVA |
| Phases | 3 |
| Voltage | 11/132 kV |
| Cooling | Outdoor type, Oil natural, Air natural (ONAN) |

# 1. SELECTION OF POSSIBLE AND FORMATION OF DEBELOPMENT PLAN

# 1. Selection of Possible Site and Formulation of Development Plan

## 1.1 General

The survey team visited Malaysia for a period of approximately five months from the middle of June through the end of October, 1981 in order to undertake the preliminary field studies for ascertaining the possibility of the Tekai Hydro-electric Power Development Project. In the said study, the survey team carried out preparation of the 1/10,000 scale aerial photogrammetric maps of the whole basin and the 1/5,000 scale aerial photogrammetric maps at the dam sites, the field geological survey, collection and analysis of hydrological data, selected the dam sites, and formulated the development plan and scale of development.

The results of the comparative study of the various alternatives indicated that the series development plan is more advantageous than the one stage development plan. Two dam sites, i.e., upper dam site and lower dam site were selected based on the said conclusion. The survey team conducted detailed geological investigations including drilling and seismic prospecting, detailed hydrological analysis including measurement of river discharge, preparation of 1/1,000 scale topographical maps and reviewed the development scale and the dam height so as to determine the type of dam and design the various structures of the upper dam site and lower dam site.

### 1.1.1 Determination of Dam Sites

(1) Upper Site

The three dam site, U-1, U-2 and U-3 were selected based upon the results of the field studies (Refer to the Fig. 1-1). The preferability of the site was evaluated through comparison of the estimated construction of the main dam, provided that the height of dam, 100 m, is to be constant and of the fill type. The results are as follows:

Table 1 – 1

| Name of Dam Site | U-1 | U-2 | U-3 |
|---|---|---|---|
| Embankment ($10^6$ $m^3$) | 3.8 | 3.1 | 3.7 |
| Construction Cost of Main Dam ($10^6$ M$) | 80 | 65 | 78 |

As seen from the above table, U-2 is most advantageous, and it is proposed that U-2 be taken up as the optimum dam site.

(2) Lower Site

Two dam sites; L-1 and L-2 have been selected as a result of the field studies. The preferability of the said dam sites was judged according to the estimated construction cost of the main dam including a spillway (Refer to the Fig. 1-1).

The bases of comparison of the said construction cost are; the height of dam (= 55.00 m) and type of dam (concrete gravity type).

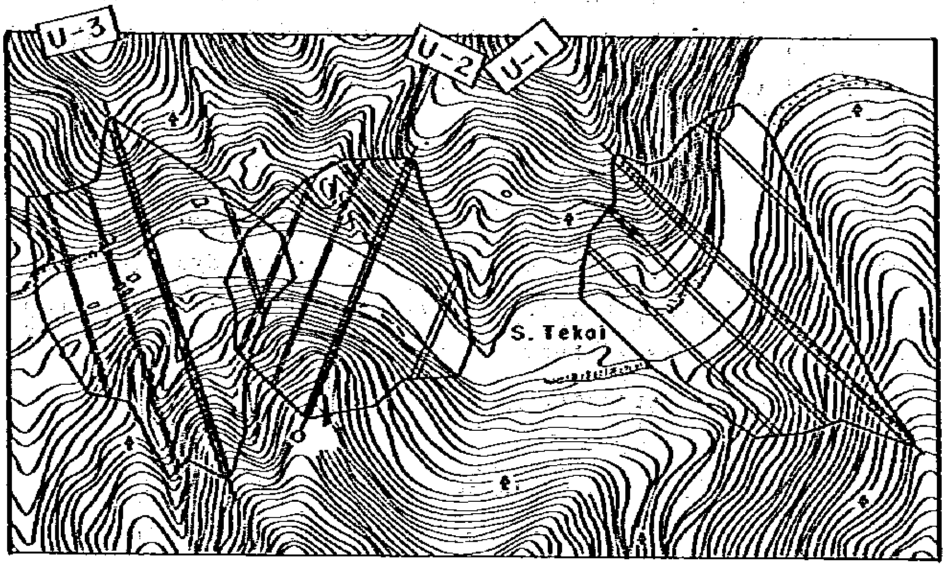
Table 1 – 2

| Name of Dam Site | L-1 | L-2 |
|---|---|---|
| Main Dam Concrete Volume including Spillway ($10^5$ $m^3$) | 1.7 | 2.1 |
| Construction Cost of Main Dam ($10^6$ M$) | 45 | 52 |

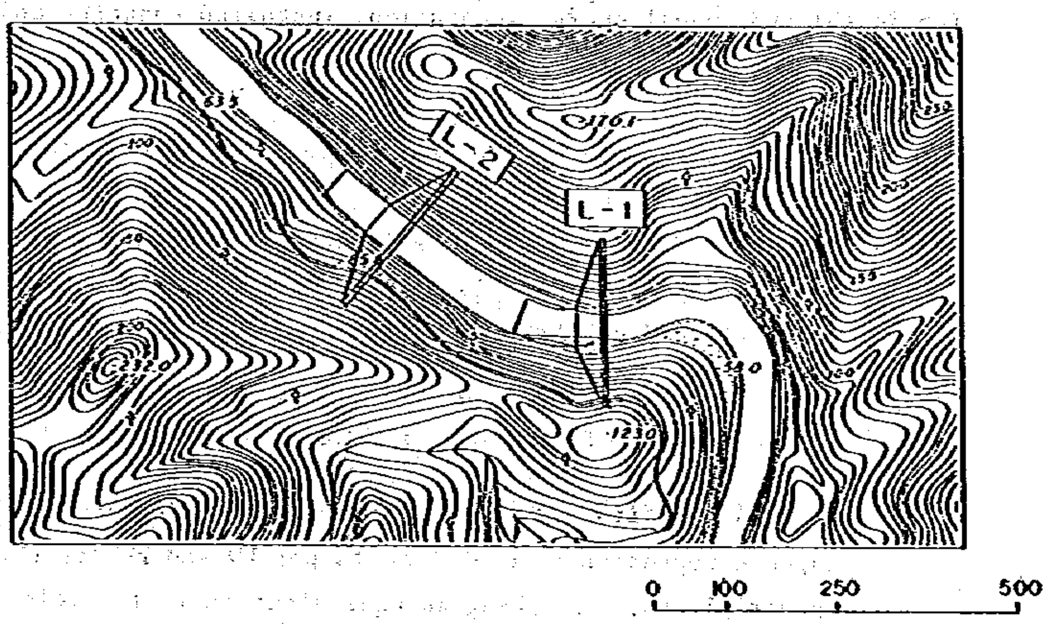
This study proves that L-1 would be the most advantageous dam site.

## Fig 1-1 Selection of Dam Sites

### (1) Upper Dam Sites




### (2) Lower Dam Sites



### 1.1.2 Formulation of Development Plan

The formulation of development plan was examined for two alternatives, iie., the independent development of the upper site and lower site and their series development.

For determining the optimum scale of the single (one dam) development scheme, studies were made on the height of dam, operation time of a power plant, maximum turbine discharge, drawdown of a reservoir (effective depth), annual energy generation, installed capacity and economic analysis in terms of benefit/cost ratio and according to a formula of benefit-cost.

The optimum scale involved in the series (two dams) development scheme has been determined on the assumption that the normal water level of the lower dam is to be near the same as the tailrace water level of the upper dam upon determination of the optimum scale of the upper dam plan which will produce a larger amount of energy. Studies were made on the running hours of a power plant, drawdown of a reservoir (effective depth) and annual energy generation, installed capacity and analysis of benefit/cost and benefit-cost.

The comparison was made based upon the following assumptions.

i) The mean discharge covering twenty years from January 1961 through December 1980 is adopted as the firm discharge at the respective dam sites.

Upper Dam Sites: $Q_f = 34.84\ m^3/sec.$
Lower Dam Sites: $Q_f = 40.07\ m^3/sec.$

ii) Annual benefits accrued from power generation were calculated upon computation of annual costs per kW and kWh of thermal power plants, by making an equivalent thermal development scheme as the alternatives.

M$ 0.19/kWh
M$ 142.7 /kW

iii) The effective head was obtained by reduction of head losses equivalent to approximately four percent of the gross head. Then, annual benefit and cost attributable to the alternatives was included in the single (one dam) development scheme corresponding to the size of dam, thereby obtaining benefit/cost and benefit-cost.

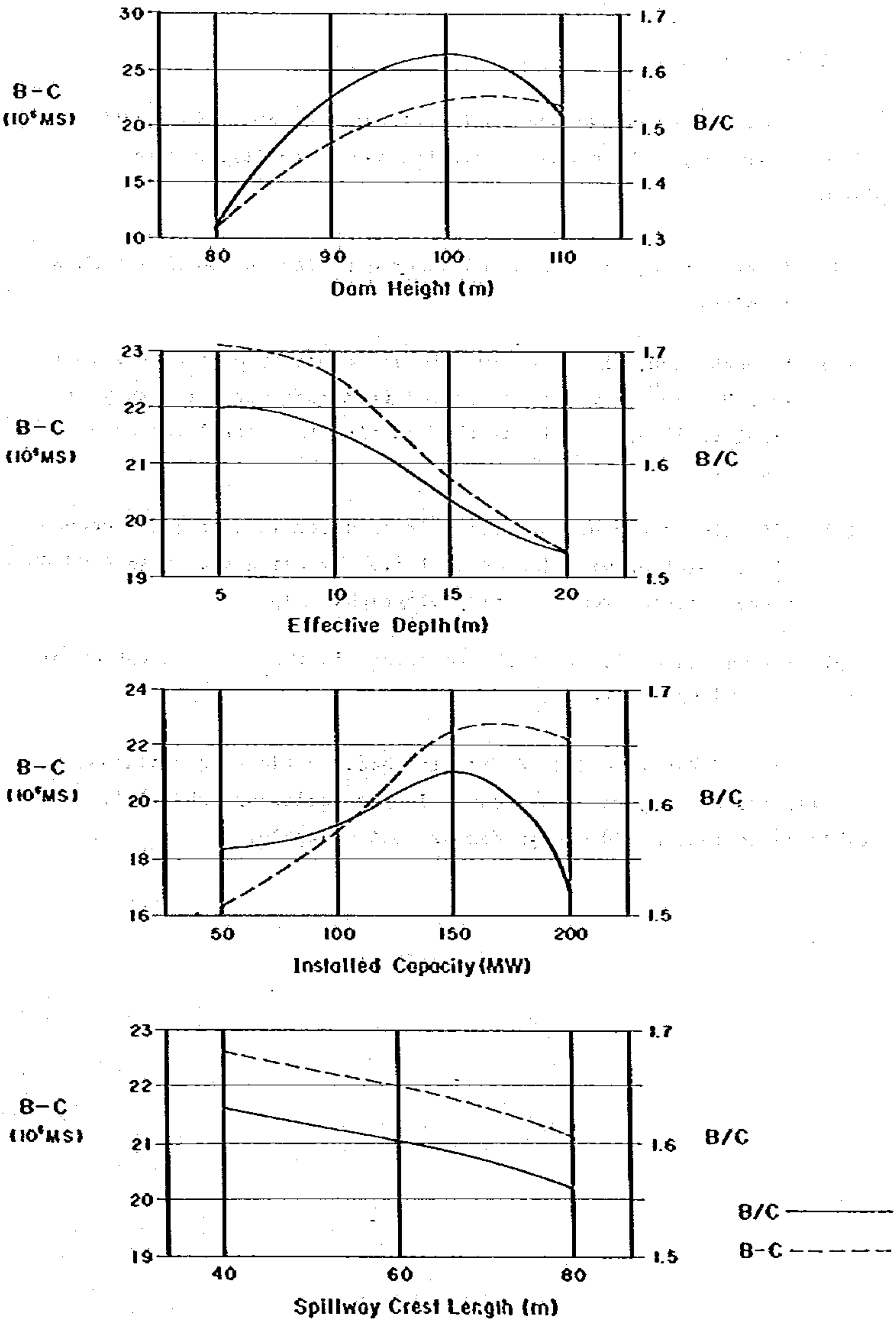
iv) Two units of electro-mechanical equipment have been proposed in due consideration of operation and maintenance of such equipment.

v) The normal water level of the lower dam will be the same as the tailrace level of the upper dam regarding the series (two dams) development scheme.

Table 1-3 Optimum Scale of Development

| Item | | Single (One Dam) Development | | Series (Two Dams) Development | |
|---|---|---|---|---|---|
| Power Site | | Upper Dam | Lower Dam | Upper Dam | Lower Dam |
| Dam Type | | Rockfill | Gravity | Rockfill | Gravity |
| Dam Height | (m) | 101 | 55 | 101 | 38 |
| Full Supply Level | (m) | EL 157.00 | EL 90.00 | EL 157.00 | EL 75.00 |
| Minimum Operating Level | (m) | EL 147.00 | El 80.00 | EL 147.00 | EL 70.50 |
| Effective Depth | (m) | 10.0 | 10.0 | 10.0 | 4.5 |
| Plant Operation Hours | (hr/day) | 3.6 | 12 | 3.6 | 24 |
| Maximum Turbine Discharge | ($m^3/s$) | 235.00 | 80.00 | 235.00 | 40.00 |
| Installed Capacity | (kW) | 150,000 | 20,000 | 155,800 | |
| Annual Energy Generation | (MWH) | 194,800 | 89,900 | 235,100 | |
| Construction Cost | ($10^6$M$) | 289 | 123 | 351 | |
| Annual Benefit | ($10^6$M$) | 58.42 | 19.94 | 66.91 | |
| Annual Cost | ($10^6$M$) | 35.83 | 14.88 | 43.81 | |
| B/C | ($10^6$M$) | 1.63 | 1.34 | 1.53 | |
| B-C | ($10^6$M$) | 22.59 | 5.06 | 23.10 | |

Fig. 1－2 B/C and B−C for Optimum Project Size (Upper Tekai)



## 1.2 Upper Tekai Power Station

### 1.2.1 Height of Dam

In study of dam height, 4 (four) cases of 80 m, 90 m, 100 m and 110 m were compared, and the following basic conditions were applied.

i) In each case, the installed power capacity is 150 MW (75 MW x 2 units).

ii) In each case, the effective storage capacity of the reservoir is 680 x $10^6$ $m^3$, which is derived from mass curve for 20 years (1961 to 1980) run-off at the dam site to control the annual run-off.

iii) Effective depth of the reservoir is determined from the economical point of view (Refer to 1.2.3) by using the storage capacity curve plotted from the aerophotographic map.

iv) Maximum water level of the reservoir is the crest elevation of ungated spillway.

B/C and B-C were judged from the abovementioned conditions, and the results of comparison are given in the following Table 1-4. The table shows that a 100 m high dam is most suitable.

Table 1-4 Comparison of Dam Height

| Dam Height (m) | 80 | 90 | 100 | 110 |
|---|---|---|---|---|
| Dam Crest Elevation (m) | 145 | 155 | 165 | 175 |
| Elevation of Dam Foundation (m) | 65 | 65 | 65 | 65 |
| High Water Level (m) | 135 | 146 | 157 | 158 |
| Low Water Level (m) | 106 | 132 | 147 | 150.5 |
| Effective Storage Capacity ($10^6 m^3$) | 680 | 680 | 680 | 680 |
| Effective Depth (m) | 29 | 14 | 10 | 7.5 |
| Normal Water Level (m) | 125.3 | 141.3 | 153.7 | 165.5 |
| Tailrace Water Level (m) | 75.6 | 75.3 | 75.0 | 74.7 |
| Maximum Turbine Discharge ($m^3/s$) | 366 | 275 | 235 | 200 |
| Installed Capacity (mW) | 150 | 150 | 150 | 150 |
| Construction Cost ($10^6$m$) | 277 | 273 | 289 | 345 |
| Annual Cost ($10^6$m$) | 34.37 | 33.91 | 35.83 | 42.33 |
| Annual Energy Output ($10^6$kWh) | 126.2 | 163.4 | 194.8 | 225.8 |
| Annual Benefit ($10^6$M$) | 45.39 | 52.46 | 58.42 | 64.31 |
| B/C | 1.32 | 1.55 | 1.63 | 1.52 |
| B - C ($10^6$M$) | 11.02 | 18.55 | 22.59 | 21.98 |

(Reference)

| | | | | |
|---|---|---|---|---|
| Water Surface Area ($km^2$) | 42.0 | 59.0 | 76.0 | 94.5 |

### 1.2.2 Type of Dam

As for the type of dam, the following dams having an EL 157.00 of high water level were compared.

i) Rockfill dam with center soil core

ii) Rockfill dam with concrete facing

iii) Rockfill dam with asphalt facing

iv) Concrete gravity dam

Comparison was made by total cost of civil works. Prior to comparison, the spillway, penstock and turbine and generator were designed to be most advantageous. The comparison is shown in Table 1-5.

As a result of this study, it is judged that a rockfill type dam is preferable.

Table 1-5 Comparison of Dam Type

(Unit: M$)

| Alternative / Item | Rockfill Dam | | | Concrete Gravity Dam | Rockfill Dam in Interium Report |
|---|---|---|---|---|---|
| | Center Core Type | Concrete Facing Type | Asphalt Facing Type | | |
| Diversion Works | 20,572,000 | 24,124,000 | 24,014,000 | 19,805,500 | 27,048,000 |
| Main Dam | 59,976,000 | 62,508,000 | 64,819,000 | 223,372,000 | 61,670,000 |
| Spillway | 21,200,000 | 21,200,000 | 21,200,000 | 8,000,000 | 27,510,000 |
| Intake | 3,512,000 | 3,512,000 | 3,512,000 | 9,034,000 | 11,277,000 |
| Pressure Tunnel | 25,654,000 | 25,654,000 | 25,654,000 | | 28,259,000 |
| Powerhouse | 20,450,000 | 20,450,000 | 20,450,000 | 20,030,000 | 22,093,000 |
| Switchyard | 1,330,000 | 1,330,000 | 1,330,000 | 1,330,000 | 1,330,000 |
| Gate & Screen | 3,500,000 | 3,500,000 | 3,500,000 | 3,927,000 | 4,918,000 |
| Total | 156,194,000 | 162,278,000 | 164,479,000 | 285,498,500 | 184,105,000 |

* The main dam was re-studied regarding the foundation treatment by geological analysis.

### 1.2.3 Effective Depth of Reservoir

In the foregoing study, it was determined that the optimum height of the dam is 100 m. Following this study, four cases, 20 m, 15 m, 10 m and 5 m of drawdown were examined. In this study the following basic conditions were applied.

i) In each case, the installed power capacity is 150 MW (75 MW x 2 units).

ii) Reservoir normal level is the center level of the effective storage capacity.

iii) There are slight differences between each case, but the tailrace water level was assumed to be as 75.0 m for all cases.

The B/C and B-C ratios were judged based on the mentioned basic conditions. The results of the study are given in the following table.

The results of the study indicate that a drawdown of 5 m is slightly more advantageous than a drawdown of 10 m. However, a larger effective storage capacity is more beneficial from the standpoint of flood control (this benefit is not considered in the above table), and also from the standpoint of effective utilization of water resources. Therefore, a drawdown of 10 m was selected.

Table 1-6 Camparison of Effective Depth

| Effective Depth (m) | 20 | 15 | 10 | 5 |
|---|---|---|---|---|
| Dam Height (m) | 100 | 100 | 100 | 100 |
| High Water Level (m) | 157 | 157 | 157 | 157 |
| Low Water Level (m) | 137 | 142 | 147 | 152 |
| Effective Storage Capacity ($10^6m^3$) | 1,210 | 960 | 680 | 360 |
| Normal Water Level (m) | 150.3 | 152.0 | 153.7 | 155.3 |
| Tailrace Water Level (m) | 75.0 | 75.0 | 75.0 | 75.0 |
| Maximum Turbine Discharge ($m^3/s$) | 241 | 236 | 235 | 226 |
| Installed Capacity (mW) | 150 | 150 | 150 | 150 |
| Construction Cost ($10^6$M$) | 300 | 292 | 289 | 287 |
| Annual Cost ($10^6$M$) | 37.06 | 36.13 | 35.83 | 35.54 |
| Annual Energy Output ($10^6$kWh) | 184.7 | 186.8 | 194.8 | 196.0 |
| Annual Benefit ($10^6$M$) | 56.50 | 56.90 | 58.42 | 58.65 |
| B/C | 1.52 | 1.57 | 1.63 | 1.65 |
| B - C ($10^6$M$) | 19.44 | 20.77 | 22.59 | 23.11 |

### 1.2.4 Installed Capacity

(1) Installed Capacity

In this study, four cases of installed capacity - 50 MW, 100 MW, 150 MW and 200 MW - were examined on the basis of a dam height of 100 m and a drawdown of 10 m which were determined in aforementioned studies. In each above case, a unit capacity of 75 MW was adopted.

The following table contains results of the study, and indicates that an installed capacity of 150 MW is the most optimum.

Table 1-7 Comparison of Installed Capacity

| Installed Capacity (mW) | 50 | 100 | 150 | 200 |
|---|---|---|---|---|
| Dam Height (m) | 100 | 100 | 100 | 100 |
| High Water Level (m) | 157 | 157 | 157 | 157 |
| Low Water Level (m) | 147 | 147 | 147 | 147 |
| Effective Depth (m) | 10 | 10 | 10 | 10 |
| Effective Storage Capacity ($10^6 m^3$) | 680 | 680 | 680 | 680 |
| Normal Water Level (m) | 153.7 | 153.7 | 153.7 | 153.7 |
| Tailrace Water Level (m) | 73.0 | 74.2 | 75.0 | 75.4 |
| Maximum Turbine Discharge ($m^3/s$) | 75 | 152 | 235 | 309 |
| Construction Cost ($10^6$M$) | 230 | 263 | 289 | 351 |
| Annual Cost ($10^6$M$) | 28.87 | 32.74 | 35.83 | 43.03 |
| Annual Energy Output ($10^6$kWh) | 200.0 | 196.9 | 194.8 | 193.7 |
| Annual Benefit ($10^6$M$) | 45.14 | 51.68 | 58.42 | 65.34 |
| B/C | 1.56 | 1.58 | 1.63 | 1.52 |
| B - C | 16.27 | 18.94 | 22.59 | 22.31 |

(Reference)

| | | | | |
|---|---|---|---|---|
| Peak Plant Operation Time (hr) | 11.1 | 5.5 | 3.6 | 2.7 |

(2) Number of Units

As for the number of units, comparative study was made on 3 units x 50 MW and 2 units x 75 MW. In the case of 3 units x 50 MW, two intakes and penstocks were planned. For 2 units x 75 MW, one intake and penstock was planned. The cost comparison is as follows:

Table 1-8 Comparison of Number of Units

| Alternative | 3 units x 50 MW | 2 units x 75 MW |
|---|---|---|
| Intake | 11,277,000 | 3,512,000 |
| Penstock | 29,569,000 | 25,654,000 |
| Powerhouse | 22,093,000 | 20,450,000 |
| Gate & Screen | 4,918,000 | 3,500,000 |
| Generating Equipment | 58,420,000 | 53,000,000 |
| Total | 126,277,000 | 106,116,000 |

The results show that 2 units of 75 MW is preferable from an economic viewpoint.

As for unit size, in 1991 when the Tekai Hydropower Station can be brought into service, the maximum demand will be approximately 4,000 MW. Therefore, the size of 75 MW would be satisfactory regarding frequency drops at the time of loss of a unit and spinning reserve on the system.

### 1.2.5 Spillway Layout

(1) Crest Length

In the foregoing studies, major features of development scale were established. Now, the last study is the spillway. Comparison was made of three cases (40 m, 60 m and 80 m) of overflow crest length. Study results are given in the following table.

As a result of the above studies, a spillway overflow crest length of 40 m was adopted.

Table 1-9 Comparison of Spillway Crest Length

| Spillway Crest Length | (m) | 40 | 60 | 80 |
|---|---|---|---|---|
| Dam Height | (m) | 100 | 100 | 100 |
| Effective Storage Capacity | ($10^6 m^3$) | 680 | 680 | 680 |
| High Water Level | (m) | 157 | 157.5 | 158 |
| Low Water Level | (m) | 147 | 147.5 | 148 |
| Effective Depth | (m) | 10 | 10 | 10 |
| Normal Water Level | (m) | 153.7 | 154.2 | 154.7 |
| Tailrace Water Level | (m) | 75 | 75 | 75 |
| Maximum Turbine Discharge | ($m^3/s$) | 235 | 233.5 | 232 |
| Installed Capacity | (mW) | 150 | 150 | 150 |
| Construction Cost | ($10^6$M$) | 289 | 297 | 306 |
| Annual Cost | ($10^6$M$) | 35.83 | 36.71 | 37.77 |
| Annual Energy Output | ($10^6$kWh) | 194.8 | 196.2 | 197.3 |
| Annual Benefit | ($10^6$M$) | 58.42 | 58.68 | 58.90 |
| B/C | | 1.63 | 1.60 | 1.56 |
| B - C | | 22.59 | 21.98 | 21.13 |

(2) Layout

As for the layout of the spillway, four cases were reviewed by giving consideration to the topographical and geological conditions. The results are as follows. (Refer to the drawings.)

Table 1-10 Comparison of Spillway Layout

(Unit: M$)

| Alternative / Item | | Case A (Interium) Length: 310 m | Case B Length: 270 m | Case C Length: 280 m | Case D Length: 280 m |
|---|---|---|---|---|---|
| Excavation | Common | 1,146,600 | 999,000 | 810,000 | 630,000 |
| | Rock | 11,720,800 | 7,682,000 | 6,256,000 | 4,899,000 |
| Concrete | | 13,471,650 | 12,778,050 | 12,602,100 | 12,153,300 |
| Backfill | | 86,000 | 51,500 | 33,100 | 33,100 |
| Reinforcing Bar | | 3,003,600 | 2,859,600 | 2,823,600 | 2,715,600 |
| Bridge | | 106,000 | 106,000 | 106,000 | 106,000 |
| Miscellaneous | | 885,350 | 733,850 | 679,200 | 663,000 |
| Total | | 30,420,000 | 25,210,000 | 23,310,000 | 21,200,000 |

The table shows that Case D is most preferable.

### 1.2.6 Optimum Development Scheme

Based upon the results of various comparison studies, the optimum development scheme is decided as the follows.

Dam height 100m is used in the comparative study of dam scale. But dam height 101m is taken by the study of the optimum development scheme.

Table 1-11 Optimum Development Scheme of Upper Tekai

| | | | |
|---|---|---|---|
| Dam | Type | | Rock-fill |
| | Crest Elevation | EL m | 166.2 |
| | Foundation Elevation | EL m | 65.2 |
| | Dam Height | m | 101 |
| Reservoir | High Water Level | m | 157 |
| | Low Water Level | m | 147 |
| | Effective Depth | m | 10 |
| | Normal Water Level | m | 153.7 |
| | Effective Storage Capacity | $10^6$ $m^3$ | 680 |
| | Gross Storage Capacity | $10^6$ $m^3$ | 2,040 |
| Tail Water Level | | m | 75 |
| Rated Net Head | | m | 75.1 |
| Maximum Turbine Discharge | | $m^3/s$ | 235 |
| Installed Capacity | | MW | 150 |
| Annual Energy Output | | $10^6$ kWh | 194.8 |
| Construction Cost | | $10^6$ M$ | 289.45 |
| Annual Cost | | $10^6$ M$ | 35.83 |
| Annual Benefit | | $10^6$ M$ | 58.42 |
| B/C | | $10^6$ M$ | 1.63 |
| B-C | | | 22.59 |

## 1.3 Lower Tekai Power Station

### 1.3.1 Height of Dam

As for the most suitable scale in the series development, the high water level of the lower dam will be the same level as the tailrace water level (EL 75.00) of the upper power station, because the head between upper and lower dam could be effectively utilized. The crest elevation was determined to be EL 81.00 to discharge the flood of 1,100 $m^3/s$ by the non-gate spillway. As a result, the dam height is 38.00 m.

### 1.3.2 Type of Dam

The following dam types were compared.

i) Concrete gravity dam with non-gate spillway

ii) Rockfill dam with center soil core

iii) Rockfill dam with concrete facing

iv) Rockfill dam with asphalt facing

Comparison was made by the total cost of civil works. Prior to comparison, the most advantageous number of turbine units was decided and one unit of turbine was adopted.

The comparison is shown in Table 1-12.

Table 1-12 Comparison of Dam Type

(Unit: M$)

| Alternative / Item | Concrete Gravity Dam | Rockfill Dam: Center core type | Rockfill Dam: Concrete facing type | Rockfill Dam: Asphalt facing type |
|---|---|---|---|---|
| Diversion Works | 5,078,000 | 7,153,000 | 7,153,000 | 7,153,000 |
| Main Dam | 16,000,000 | 10,606,000 | 15,183,000 | 15,203,000 |
| Spillway | 4,950,000 | 9,427,000 | 9,427,000 | 9,427,000 |
| Intake | 920,000 | 1,210,000 | 1,210,000 | 1,210,000 |
| Penstock | 810,000 | 3,150,000 | 3,150,000 | 3,150,000 |
| Total | 27,758,000 | 31,546,000 | 36,123,000 | 36,123,000 |

As a result of this study, it is judged that a concrete gravity dam is preferable.

### 1.3.3 Effective Depth of Reservoir

The effective depth was determined to be 4.5 m so as to ensure the available storage capacity of $21.5 \times 10^6$ $m^3$ which is possible to reregulate the peak discharge of the Upper Tekai Power Station and regulate annually the run-off from the remaining catchment area. The study was made by using the mass curve of run-off at the lower dam, of which run-off was calculated by the peak discharge of Upper Tekai and run-off from the remaining catchment area. The mass curve is shown in the Appendix.

### 1.3.4 Installed Capacity

. The installed capacity was determined taking accounts of the re-regulating function of Lower Tekai Dam which can discharge to the downstream for 24-hours by re-regulating the peak discharge of the Upper Tekai Dam. By this 24-hour discharge, it will be ensured the navigation and future water use for irrigation and other purposes in the downstream.

The annual run-off at the Lower Dam Site is approximately 40 $m^3/s$, therefore the maximum discharge, 40 $m^3/s$, of the power station was designed. This can generate maximum output of 5,800 kW.

Regarding the number of units, capacity of 5,800 kW is small and the losses during maintenance is also small, therefore one unit was adopted.

### 1.3.5 Development Scheme

The development scheme is decided as the follows.

Table 1-14 Development Scheme of Lower Tekai

| | | | |
|---|---|---|---|
| Dam | Type | | Concrete Gravity |
| | Crest Elevation | EL m | 81.00 |
| | Foundation Elevation | EL m | 43.00 |
| | Dam Height | m | 38.00 |
| Reservoir | High Water Level | EL m | 75.00 |
| | Low Water Level | EL m | 70.50 |
| | Effective Depth | m | 4.50 |
| | Normal Water Level | EL m | 73.50 |
| | Effective Storage Capacity | $10^6$ $m^3$ | 21.50 |
| | Gross Storage Capacity | $10^6$ $m^3$ | 41.50 |
| Tail Water Level | | EL m | 55.60 |
| Rated Net Head | | m | 17.20 |
| Installed Capacity | | MW | 5.8 |
| Annual Generated Energy | | $10^6$ kWh | 40.3 |
| Construction Cost | | $10^6$ M$ | 62 |
| Annual Cost | | $10^6$ M$ | 7.98 |
| Annual Benefit | | $10^6$ M$ | 8.49 |
| B - C | | $10^6$ M$ | 0.51 |
| B / C | | | 1.06 |

# 2. DESIGN OF FACILITIES AND STRUCTURES OF UPPER TEKAI POWER STATION

## 2. Design of Facilities and Structures of Upper Tekai Power Station

### 2.1 Installed Capacity and Generated Energy

#### 2.1.1 Head and Tail-Water Level

The head-water level is considered EL 153.70, by subtracting 3.3 m of the available depth (10 m) from the H.W.L. 157.00. As for the tailwater level, it is considered equal to the H.W.L. of the Lower Tekai Reservoir, i.e., EL 75.00. And water level and discharge during the operation of 150 MW (discharge of 235 $m^3/s$) were confirmed as follows.

(1) River discharge capacity

i) River slope

The river slope is calculated from the 1:1,000 scale topographic map.

River slope calculation distance L = 20,000 m

Difference of elevation (Difference of elevation between upper dam and lower dam sites) H = 72.50 - 55.00 = 17.50 m

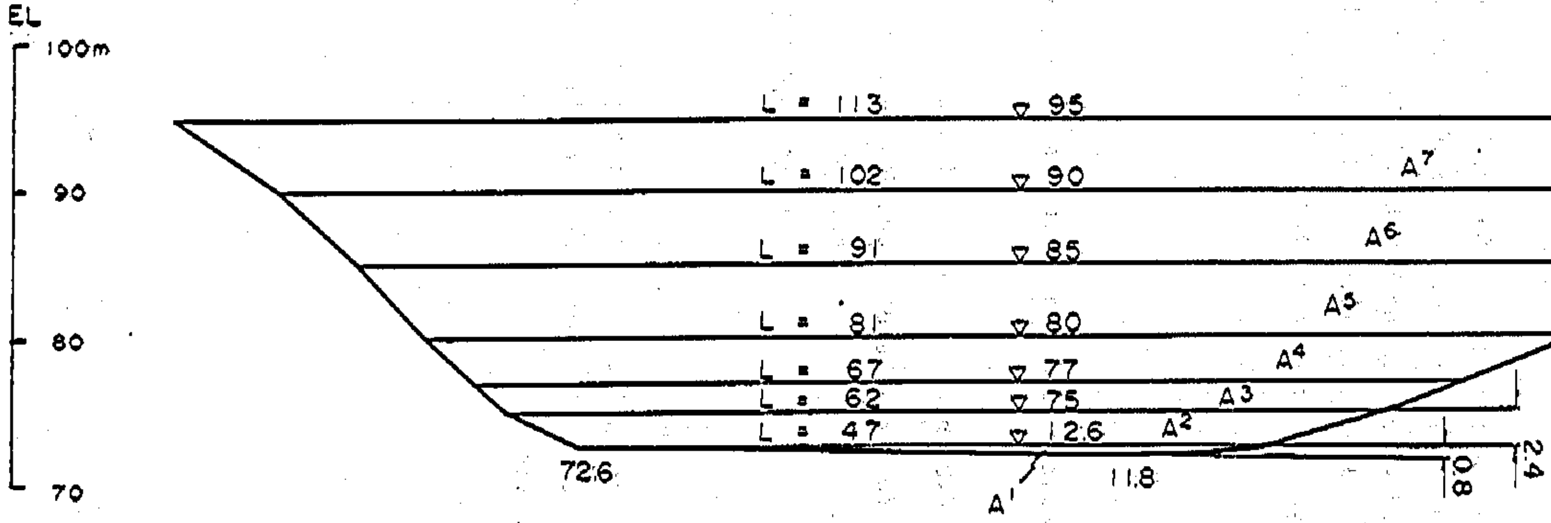
River slope I = 17.50/20,000 ≒ 1:1,000

ii) Cross section

The cross section was prepared from the 1:500 topographical map and location is approximately 640 m downstream of dam axis and at the vicinity of the outlet.

Fig. 2-1 River Cross Section at the Vicinity of the Outlet

(Scale 1:500)




$$A_1 = 49 + 0.8 \times 2 = 19 \qquad 19\ m^2$$

$$A_2 = \frac{47 + 62}{2} \times 2.4 = 131 \qquad 150$$

$$A_3 = \frac{62 + 69}{2} \times 2 = 131 \qquad 281$$

$$A_4 = \frac{69 + 81}{2} \times 3 = 225 \qquad 506$$

$$A_5 = \frac{81 + 91}{2} \times 5 = 430 \qquad 936$$

$$A_6 = \frac{91 + 102}{2} \times 5 = 483 \qquad 1\,419$$

$$A_7 = \frac{102 + 113}{2} \times 5 = 538 \qquad 1\,957$$

iii) Calculation of the river discharge

Manning's formula is used for calculation of the discharge.

$$V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$$

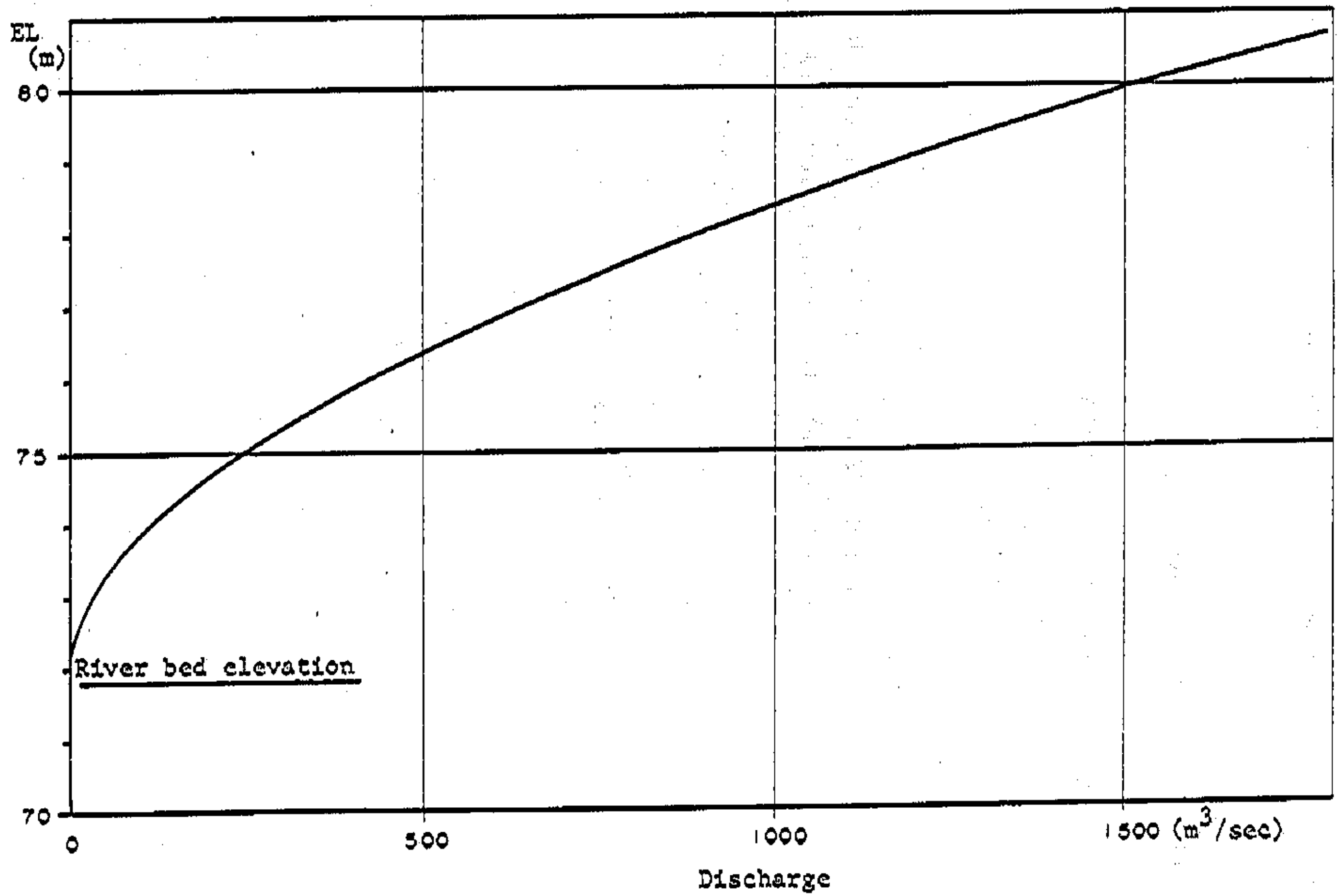
$$Q = V \cdot A$$

| | | | |
|---|---|---|---|
| where; | Q : Discharge | | ($m^3$/sec) |
| | A : Discharge cross section | | ($m^2$) |
| | I : Hydraulic gradient | = | 1/1,000 |
| | V : Velocity of flow | | (m/sec) |
| | R : Hydraulic radius | | (m) |
| | n : Coefficient of roughness | = | 0.035 |

## Calculation of Discharge

| EL | A ($m^2$) | P (m) | R (m) | $R^{2/3}$ | 1/n | $I^{1/2}$ | V (m/sec) | Q ($m^3$/sec) |
|---|---|---|---|---|---|---|---|---|
| 71.80 | | | | | | | | |
| 72.60 | 19 | 47 | 0.40 | 0.547 | 28.57 | 0.032 | 0.50 | 9.5 |
| 75.00 | 150 | 62 | 2.42 | 1.802 | 28.57 | 0.032 | 1.65 | 247.5 |
| 77.00 | 281 | 71 | 3.96 | 2.502 | 28.57 | 0.032 | 2.29 | 643.5 |
| 80.00 | 506 | 85 | 5.95 | 3.285 | 28.57 | 0.032 | 3.00 | 1,518.0 |
| 85.00 | 936 | 99 | 9.45 | 4.471 | 28.57 | 0.032 | 4.09 | 3,828.2 |
| 90.00 | 1,419 | 114 | 12.45 | 5.371 | 28.57 | 0.032 | 4.91 | 6,967.3 |

Fig. 2-2 Stage-discharge Curve



### 2.1.2 Head Loss

| | | | |
|---|---|---|---|
| Before bifurcation | $Q = 235.0\ m^3/s$ | $D = 7.3\ m$ | $L = 534.506\ m$ |
| After bifurcation | $Q = 117.5\ m^3/s$ | $D = 4.6\ m$ | $L = 43.852\ m$ |

(1) Head loss at intake

a) Head loss in inflow (ha)

$$ha = fe \cdot \frac{V_2^2}{2g}$$

where;

fe : Coefficient of loss by inflow = 0.2

$V_2$ : Mean velocity after inflow (m/s)

$$V_2 = \frac{235}{12.0 \times 23.0} = 0.851\ m/s$$

$$ha = 0.2 \times \frac{0.851^2}{2 \times 9.8} = 0.007\ m$$

b) Head loss by screen (hb)

$$hb = fr \cdot \frac{V_1^2}{2g}$$

$$fr = \beta \cdot \sin\theta \cdot (\frac{t}{b})^{4/3}$$

where; $\beta$ : Coefficient determined by the sectional form of the screen bar = 1.60

$\theta$ : Tilting angle of screen = 63°26'06"

$t$ : Thickness of screen bar = 0.016 m

$b$ : Mesh size of the screen bar = 0.15 m

$V_1$ : Mean velocity in upstream of screen (m/s) = 0.851 m/s

$$fr = 1.60 \times \sin(63.435°) \times (0.016/0.15)^{4/3} = 0.072$$

$$hb = 0.072 \times \frac{0.851^2}{2 \times 9.8} = 0.003 \text{ m}$$

c) Head loss by pier (hc)

$$hc = \left\{\frac{1}{C^2}\left(\frac{b_1}{b_2}\right)^2 - 1\right\}\frac{V_1^2}{2g}$$

$c$ : Coefficient determined by horizontal sectional form of pier
$1/C^2 = 1.181$

$b_1$ : Channel width immediately before pier = 13.0 m

$b_2$ : Width by subtracting the total pier width from the channel width = 12.0 m

$V_1$ : Mean velocity of section before inflow

$$V_1 = \frac{235}{12.0 \times 23.0} = 0.851 \text{ m/s}$$

$$hc = \{1.181 \times (13.0/12.0)^2 - 1\} \times \frac{0.851^2}{2 \times 9.8} = 0.014 \text{ m}$$

d) Head loss due to gradual contraction of section (hd)

$$hd = fgc \times \frac{V_2^2}{2g}$$

fgc : Coefficient of loss by gradual contraction

$V_2$ : Mean velocity after gradual contraction

$\theta = 60°$

$A_1 = 13.0 \times 12.0 = 156\ m^2$

$A_2 = 7.3 \times 7.3 = 53.29\ m^2$

$A_2/A_1 = 0.342$

$fgc = 0.075$

$$V_2 = \frac{235.0}{\frac{\pi \times 7.3^2}{4}} = 5.615\ m/s$$

$$hd = 0.075 \times \frac{5.615^2}{2 \times 9.8} = 0.121\ m$$

e) Head loss at intake ($h_1$)

$$h_1 = ha + hb + hc + hd$$
$$= 0.007 + 0.003 + 0.014 + 0.121$$
$$= 0.145\ m$$

(2) Head loss in penstock

a) Head loss by bifurcation ($h_a$)

$$ha = fa \cdot \frac{V_2^2}{2g}$$

fa : Bifurcation loss coefficient = 0.50

$V_2$ : Average velocity of flow in penstock before the bifurcation = 5.615 m/s

$$ha = 0.50 \times \frac{5.615^2}{2 \times 9.8} = 0.804\ m$$

b) Head loss by friction (hb)

$$hb = f \cdot L \cdot \frac{V^2}{2g}$$

f : Friction loss coefficient

$$f = \frac{124.5\ n^2}{D^{4/3}}$$

L : Length of penstock (m)

D : Diameter of penstock (m)

V : Average velocity of flow in the interior of penstock (m/s)

n : Coefficient of roughness $n = 0.012$

Head loss before the bifurcation ($h_{b1}$)

$$f = \frac{124.5 \times 0.012^2}{7.3^{4/3}} = 0.001266$$

$L = 534.506$ m

$V = 5.615$ m/s

$$h_{b1} = 0.001266 \times 534.506 \times \frac{5.615^2}{2 \times 9.8} = 1.089 \text{ m}$$

Head loss after the bifurcation ($h_{b2}$)

$$f = \frac{124.5 \times 0.012^2}{4.6^{4/3}} = 0.0023$$

$L = 43.852$ m

$$V = \frac{117.5}{\pi(4.6/2)^2} = 7.070 \text{ m/s}$$

$$h_{b2} = 0.0023 \times 43.852 \times \frac{7.070^2}{2 \times 9.8} = 0.257 \text{ m}$$

$$h_b = h_{b1} + h_{b2} = 1.089 + 0.257 = 1.346 \text{ m}$$

c) Head loss by curvature (hc)

$$hc = f_{b1} \times f_{b2} \times \frac{V^2}{2g}$$

$f_{b1}$ : Coefficient of loss determined by the ratio between the radius of curvature P and pipe diameter D (P/D)

$f_{b2}$ : Ratio of losses between each central angle of curvature θ and the central angle 90°

V : Mean velocity in pipe

| No. | ρ | D | θ | $f_{b1}$ | $f_{b2}$ | V | $\frac{V^2}{2g}$ | hd |
|---|---|---|---|---|---|---|---|---|
| 1 | 30.0 | 7.3 | 50°00'00" | 0.132 | 0.745 | 5.615 | 1.600 | 0.158 |
| 2 | 20.0 | 7.3 | 48°00'00" | 0.135 | 0.730 | 5.615 | 1.600 | 0.158 |
| 3 | 20.0 | 7.3 | 48°00'00" | 0.135 | 0.730 | 5.615 | 1.600 | 0.158 |
| 4 | 30.0 | 7.3 | 44°00'00" | 0.132 | 0.699 | 5.615 | 1.600 | 0.148 |
| 5 | 30.0 | 7.3 | 77°00'00" | 0.132 | 0.925 | 5.615 | 1.600 | 0.196 |
| 6 | 10.0 | 4.6 | 40°00'00" | 0.142 | 0.667 | 7.070 | 2.550 | 0.242 |
| Total | | | | | | | | 1.060 |

d) Head loss in penstock ($h_2$)

$$\begin{aligned} h_2 &= h_a + h_b + h_c \\ &= 0.804 + 1.346 + 1.060 \\ &= 3.150 \end{aligned}$$

(3) Head loss at outlet ($h_3$)

a) Abandonment head loss of reaction turbine (ha)

$$ha = fse \cdot \frac{V_1^2}{2g}$$

$$fse = \{1 - (\frac{A_1}{A_2})\}^2$$

fse : Coefficient of loss by quick expansion

$V_1$ : Mean velocity before quick expansion (m/s)

$$= \frac{117.5}{10.0 \times 5.0} = 2.35 \text{ m/s}$$

$A_1$,$A_2$ : Cross sectional area of flow before and after quick expansion

Since $A_1 \ll A_2$ fse = 1

$$ha = \frac{2.350^2}{2 \times 9.8} = 0.282 \text{ m}$$

(4) Total head loss

| | |
|---|---|
| Head loss at intake (m) | 0.145 |
| Head loss at penstock (m) | 3.150 |
| Head loss at outlet (m) | 0.282 |
| Other head losses (m) | 0.023 |
| Total (m) | 3.600 |

### 2.1.3 Power Generating Capacity

(1) Installed capacity

The installed capacity is calculated as follows:

$$P = 9.8 \times Q \times H \times \eta$$

where; Q : Maximum discharge (235 $m^3/s$)

H : Effective head
: (= 153.700 - 75.000 - 3.600 = 75.1 m)

$\eta$ : Comprehensive efficiency of turbine and generator
(= 0.87)

$$P = 9.8 \times 235 \times 75.1 \times 0.87 = 150.471 \text{ KW} \doteqdot 150.000 \text{ KW}$$

(2) Generated energy

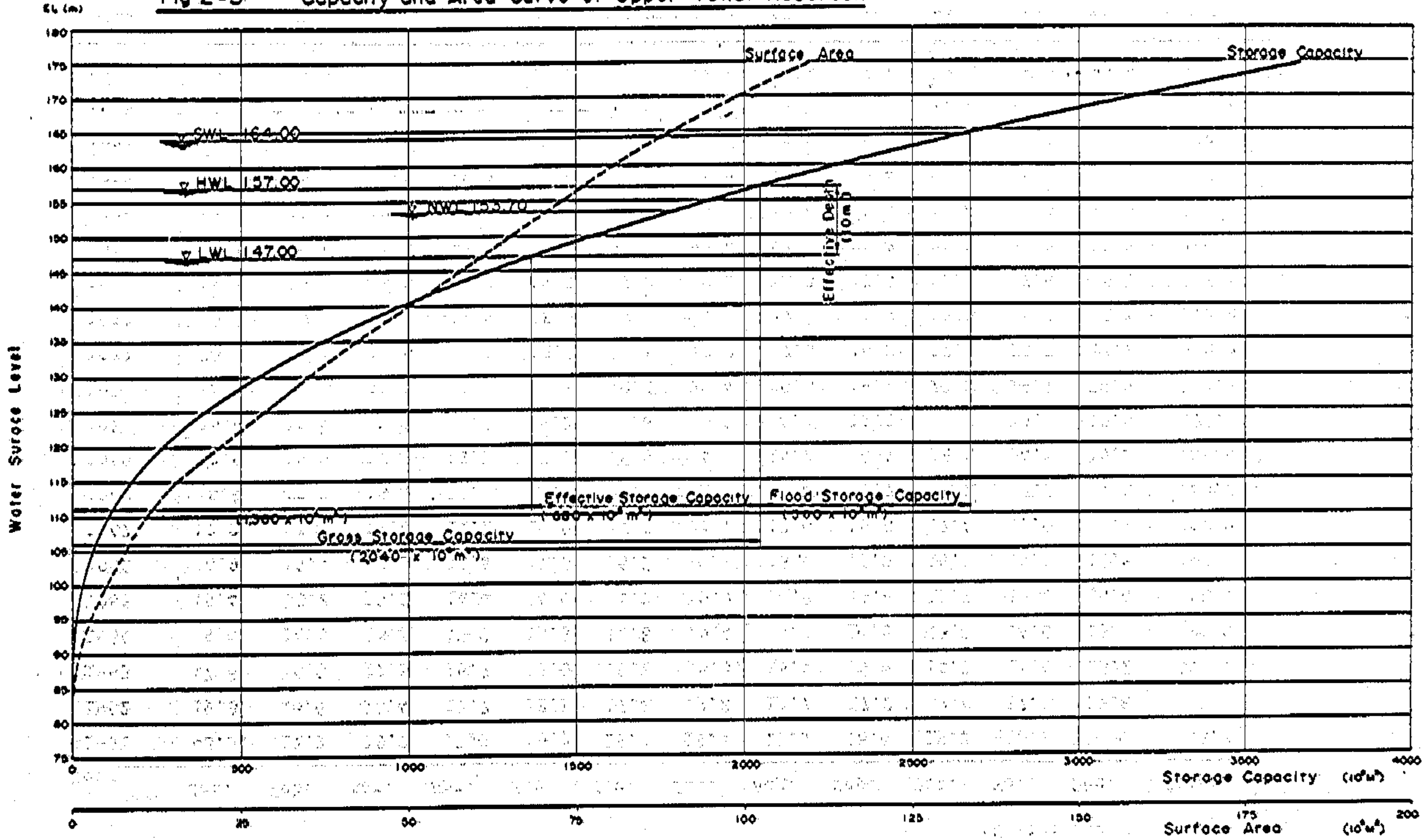
The generated energy was calculated using the daily discharge data and reservoir water level which were calculated by the mass curve from 1961 to 1980. The average annual generated energy of 20 years, is 194.8 GWH. And monthly variation is rather small, average maximum and minimum output are 17.8 GWH/month and 15.2 GWH/month.

Table 2-1 Monthly Generated Energy of the Upper Tekai Power Station

(Unit : GWH)

| Month / Year | Jan. | Feb. | Mar. | Apr. | May | Jun. | Jul. | Aug. | Sep. | Oct. | Nov. | Dec. | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1961 | 43.3 | 17.9 | 19.7 | 19.0 | 19.7 | 19.0 | 19.5 | 19.3 | 18.6 | 19.3 | 18.9 | 19.6 | 253.8 |
| 1962 | 19.8 | 16.6 | 18.4 | 17.7 | 18.1 | 17.4 | 17.8 | 17.7 | 17.2 | 17.8 | 17.1 | 17.8 | 213.4 |
| 1963 | 17.8 | 16.0 | 17.5 | 16.7 | 17.0 | 16.2 | 16.5 | 16.4 | 15.6 | 14.8 | 14.4 | 15.2 | 194.1 |
| 1964 | 15.5 | 14.5 | 15.5 | 15.0 | 15.5 | 14.8 | 15.4 | 15.3 | 14.6 | 15.0 | 14.6 | 15.3 | 180.5 |
| 1965 | 15.4 | 13.8 | 15.1 | 14.4 | 14.9 | 14.7 | 15.0 | 14.9 | 14.4 | 15.7 | 15.8 | 16.6 | 180.7 |
| 1966 | 16.9 | 15.4 | 17.0 | 16.4 | 17.0 | 16.2 | 16.6 | 16.6 | 16.0 | 16.6 | 16.1 | 16.8 | 197.6 |
| 1967 | 17.0 | 15.4 | 17.6 | 17.2 | 17.9 | 17.3 | 17.7 | 17.6 | 16.8 | 17.4 | 17.0 | 18.1 | 207.0 |
| 1968 | 18.9 | 17.9 | 18.9 | 18.1 | 18.7 | 18.0 | 18.5 | 18.4 | 17.7 | 18.2 | 17.5 | 17.9 | 218.7 |
| 1969 | 17.9 | 16.0 | 17.4 | 16.7 | 17.0 | 15.5 | 14.7 | 14.7 | 14.4 | 14.8 | 14.5 | 15.2 | 188.8 |
| 1970 | 15.4 | 13.9 | 15.2 | 14.7 | 15.2 | 14.6 | 15.0 | 14.9 | 14.3 | 15.0 | 14.8 | 15.5 | 178.5 |
| 1971 | 16.4 | 18.0 | 21.2 | 20.4 | 20.8 | 20.0 | 20.4 | 20.3 | 19.7 | 20.2 | 19.4 | 20.5 | 237.3 |
| 1972 | 20.4 | 17.5 | 18.5 | 17.7 | 18.2 | 17.5 | 17.8 | 17.6 | 16.8 | 17.3 | 16.7 | 17.7 | 213.7 |
| 1973 | 18.7 | 17.0 | 18.7 | 17.9 | 18.3 | 17.6 | 17.9 | 17.7 | 16.9 | 17.3 | 16.7 | 18.1 | 212.8 |
| 1974 | 18.7 | 16.8 | 18.5 | 17.8 | 18.5 | 17.8 | 18.4 | 18.3 | 17.6 | 18.1 | 17.6 | 18.3 | 216.4 |
| 1975 | 18.5 | 16.9 | 21.5 | 22.4 | 23.0 | 22.1 | 22.6 | 22.3 | 21.3 | 21.8 | 21.2 | 23.2 | 256.8 |
| 1976 | 14.6 | 13.1 | 13.8 | 13.3 | 13.6 | 13.1 | 13.5 | 13.4 | 12.9 | 13.4 | 13.0 | 13.5 | 161.2 |
| 1977 | 13.8 | 12.5 | 13.7 | 13.2 | 13.5 | 12.9 | 13.2 | 13.1 | 12.6 | 13.6 | 12.7 | 13.1 | 157.4 |
| 1978 | 13.0 | 11.7 | 12.8 | 12.3 | 12.6 | 12.2 | 12.6 | 12.5 | 12.0 | 12.3 | 11.0 | 11.7 | 146.7 |
| 1979 | 11.8 | 10.7 | 11.7 | 11.3 | 11.6 | 11.2 | 11.4 | 11.4 | 11.0 | 11.5 | 11.3 | 12.3 | 137.2 |
| 1980 | 12.3 | 11.5 | 12.2 | 11.8 | 12.1 | 11.7 | 12.0 | 11.9 | 11.5 | 12.0 | 11.8 | 12.3 | 143.1 |
| Total | 356.1 | 303.1 | 334.9 | 324.0 | 333.2 | 319.8 | 326.5 | 324.3 | 311.9 | 321.6 | 312.1 | 328.7 | 3.896.2 |
| Average | 17.8 | 15.2 | 16.7 | 16.2 | 16.7 | 16.0 | 16.3 | 16.2 | 15.6 | 16.1 | 15.6 | 16.4 | 194.8 |

Fig 2-3 Capacity and Area Curve of Upper Tekai Reservoir



## 2.2 Design Flood

The design flood flow is calculated by determining the 10,000-year probability rainfall from the existing rainfall data by means of Iwai's Formula and Gumbel Formula, and by conducting runoff analysis by means of the Storage Function Method from measured hydrograph. The 10,000-year probability rainfall calculated by the aforestated method is 840 mm, and accordingly the design flood flow is 7,300 $m^3/s$ (Refer to Report of Hydrology).

The upper dam has a large storage capacity and accordingly, two alternatives of the type of spillway, i.e., the type provided with gate and spillway with no gate utilizing the surcharge function of the storage reservoir, were examined. The results of the comparative examination of the aforestated alternatives indicates that the spillway without gate is preferable. The surcharge volume calculated by assuming the spillway (40 m width) without gate is shown in the following table.

Table 2-2

| | |
|---|---|
| The peak discharge of inflow | 7,300 $m^3/s$ |
| The maximum water level | EL 164.0 |
| The surcharge volumes | 585 x $10^6 m^3$ |
| The maximum outflow | 1,504 $m^3/s$ |

## 2.3 Design Sedimentation

The estimation of the sedimentation are conducted by investigating of the existing reports concerning sedimentation in the basin of Malaysia, investigation referring to the design values of existing dam projects, and investigation of measured sedimentation in the Pahang River and in the Tekai River. Furthermore, the design sedimentation is also examined by means of various methods of estimation ordinarily used in Japan for the sake of reference.

(1) Investigation of the sedimentation in the basin located of Malaysia and Indonesia

According to the "Report of Water Resources Management in the Pahang Tengara Region", the specific sedimentation in the various rivers of Malaysia and Indonesia is estimated to be as follows.

Table 2-3 Sediment Load of the Rivers in Malaysia and Indonesia

| River | Drainage Area ($km^2$) | Vegetation Cover | Sediment Load | Source |
|---|---|---|---|---|
| Perak River at Kenering | 5500 | Rainforest 86%<br>Rubber 10%<br>Padi 4% | $144m^3/km^2/yr$ | Douglas, 1970 after Shawiningan Engineering Co. |
| Gombak River at Kuala Lumpur | 140 | Forested Headwaters agriculture & mining land | $67m^3/km^2/yr$ | Douglas, 1968 |
| Kiel, Cameron Highlands | 21 | Rainforest 70%<br>Agriculture 30% | $111m^3/km^2/yr$ | Shallow, 1956 |
| Telom, Cameron | 77 | Rainforest 94%<br>Agriculture 6% | $21.1m^3/km^2/yr$ | Shallow, 1956 |
| Tjiloetoeng River, West Java | 620 | Deforested 'ladang' cultivation, pasturing | $1352m^3/km^2/yr$ | Van Dijk and Vogelzang, 1948 |

(2) Design sediment load of dam projects of Malaysia

Table 2-4 Sediment load of dam design in Malaysia

| Name of Dam | Sediment Load ($m^3/km^2/year$) | Type of Dam (dam height) | Drainage Area ($km^2$) | Name of River |
|---|---|---|---|---|
| PUAH DAM | 70 | Rock-fill (80 m) | 410 | TRENGGANU River |
| TEMBAT DAM | 17 | Concrete Gravity (24 m) | 101 | TEMBAT River |

(Note) ULU TRENGGANU Report
Upper Trengganu Development

(3) Estimation from suspended sediment

Periodic measurement of suspended sediment in the Tekai River has notbeen carried out as yet, while measurement of sedimentation in the Pahang River has been underway since 1972.

The measurement of suspended sediment in the Pahang River was carried out by the DID (Drainage and Irrigation Dept.) at the 5 points indicated below.

Table 2-5 Measured Results of Soil Sedimentation

| Station No. | Station Name | Catchment Area ($km^2$) | Data Period | Average Sediment Concentration (mg/ℓ) |
|---|---|---|---|---|
| 3423421 | Semantan | 2,920 | 72-78 | 110 |
| 3424411 | Pahang | 19,000 | 72-74 | 140 |
| 4019462 | Lipis | 1,670 | 72-76 | 130 |
| 4023412 | Pahang | 13,200 | 72-74 | 150 |
| 4223450 | Tembeling | 5,050 | 74-77 | 130 |

(Source; N.W.R.S.)

With regard to the suspended sediment qs, the estimation is conducted by assuming the maximum value 150 (mg/liter) for the sake of safety. The sediment is assumed to have the ordinary density of 1.5 $t/m^3$.

- Upper dam

The suspended sediment of the upper dam is calculated as follows, by assuming an annual average inflow of $1100 \times 10^6 m^3$ ($35\ m^3/s$).

$$qs = 150 \times 10^{-9} \times 1100 \times 10^6 \times 10^3 \div 1.5$$
$$= 110 \times 10^3\ m^3/year$$

The sedimentation Q is calculated as follows, by assuming a bed load of approximately 20% of the suspended sediment.

$$Q = 1.2 \times qs = 132 \times 10^3\ m^3/year$$

The sedimentation becomes $Qs = 110\ m^3/km^2/year$ for the basin of C.A. = 1,200 $km^2$.

$$Qs = 110\ m^3/km^2/year$$

- Lower dam

| | |
|---|---|
| Annual average inflow | $160 \times 10^6 m^3$ ($5\ m^3/s$) |
| C.A. | 180 $km^2$ |
| $Qs = 107\ m^3/km^2/year$ | |

(4) Other methods of estimation

A quantitative investigation referring to the sedimentation in dams was carried out in Japan by focusing principally on the volume of sediment accumulated in storage reservoirs located behind the fluvial dams, principally those for power generation located in various parts of Japan, and on data referring to factors of various kinds related to the said sedimentation. The purpose was to examine quantitatively how the various kinds of factors, including the jungle, influence the volume of earth flowing out of mountainous districts subjected to conditions of various kinds, such as the weather, topography, etc.

The principal factors that influence the sedimentation, taken into consideration in the aforementioned investigation, are as follows.

a) Area of the basin
b) Storage capacity of the reservoir
c) Relief energy
d) Elevation
e) Annual rainfall
f) Years of observation
g) Rate of forest area

Messrs. Kira and Nanba have proposed the following formulae to estimate sedimentation based on the aforementioned factors. These formulae are widely used in Japan.

1) $Q_S = \gamma s \cdot (C/F)$ Kira's formula

where;

$Q_S$ : Specific sedimentation ($m^3/km^2/year$)
$\gamma s$ : $0.00012.\Phi^{0.368}$
$\Phi$ : $R/(C/F)$
R : Relief energy (m)
C : Storage capacity of the reservoir ($m^3$)
F : Catchment area ($m^2$)

ii) $Q_s = 0.02743 \times \frac{R \times P}{F} - 240.9$ (Nanba's Formula)

where; $Q_s$ : Specific sedimentation ($m^3/km^2$/year)

P : Annual average rainfall (mm)

R : Relief energy (m)

F : Rate of forest area (%)

The results of the calculations carried out by using the Formulae i) and ii) are as follows.

- Kira's Formula

Table 2-6 Calculation by Kira's Formula

| Item \ Case | Upper Dam (Independent Development) | Lower Dam (Independent Development) | Lower Dam (Series Development) | Remarks |
|---|---|---|---|---|
| Catchment area F ($m^2$) | $1200 \times 10^6$ | $1380 \times 10^6$ | $180 \times 10^6$ | |
| Storage capacity C ($m^3$) | $2050 \times 10^6$ | $700 \times 10^6$ | $41.5 \times 10^6$ | |
| C/F | 1.708 | 0.507 | 0.231 | |
| Relief energy R (m) | 500 | 500 | 455 | |
| $\Phi = R/(C/F)$ | 293 | 986 | 1,970 | |
| $\gamma s = 0.00012\Phi^{0.368}$ | 0.0166 | 0.0476 | 0.0868 | |
| $Q_s = \gamma s \times \frac{C}{F}$ ($m^3/km^2$/year) | 284 | 241 | 200 | |

- Nanba's Formula

The estimated values of P, R and F of the project site are as follows.

P : 2,400 mm
R : 500 mm
F : 100%

Therefore, the specific sedimentation is

$$s = 88.3\ (m^3/km^2/year).$$

(5) Determination of the design sedimentation

The results of the foregoing items (1) to (4) are summarized in the following table.

Table 2-7 Results of Sediment Loads

| | Method | Estimated value of relative sedimentation ($m^3/km^2/year$) | Remarks |
|---|---|---|---|
| 1 | Sedimentation in Malaysia | $21.1 \leqq Q_s \leqq 144$ (82.6) | |
| 2 | Design sedimentation of dam projects in Malaysia | $17.0 \leqq Q_s \leqq 70$ (43.5) | |
| 3 | Estimation from suspended sediment | $107 \leqq Q_s \leqq 110$ (108.5) | |
| 4 | Kira's Formula | Upper Dam 284; Lower Dam $200 \leqq Q_s \leqq 241$ (200) | |
| 5 | Nanba's Formula | 88.3 | |

(Figures enclosed with parenthesis indicate average values.

The values of design sedimentation are estimated to be as follows, by taking into consideration an appropriate margin of safety.

- Upper dam 300 $m^3/km^2/year$
- Lower dam 250 $m^3/km^2/year$

## 2.4 Design Seismic Intensity

Some recent works have revealed characteristics of seismicity which are of great interest with reference to major techtonic mechanisms and patterns. It is pointed out that virtually all the epicenters are located within a degree or so of the topographic rift.

Fig. 2-4 shows the seismicity of the earth in southeast Asia over the 1961 - 1967 period. The distribution of the epicenters makes a belt along the Indonesian arch. On the other hand, there has been no earthquake over magnitude 4 on the Malay Peninsula. In fact there is a remarkable rift along the western front of the Indonesian arch on which the epicenters are scattered. The Malay Peninsula forms one part of the Sunda Shelf, which is very stable against crustal movement of the earth, and, therefore, there have been no remarkable earthquakes within it.

Fig. 2-5 is the seismological map of Indonesia for the 1904-1960 period. In particular, earthquakes with $M > 7$ are listed in Table 2-8.

The earthquake acceleration at the Tekai Dam Site is estimated by means of "Kawasumi's Map", Kanai-Seed Formula and Okamoto's Formula (based on earthquakes which occurred in the past). In any case, the earthquake acceleration has a very small value on the order of 10 gal.

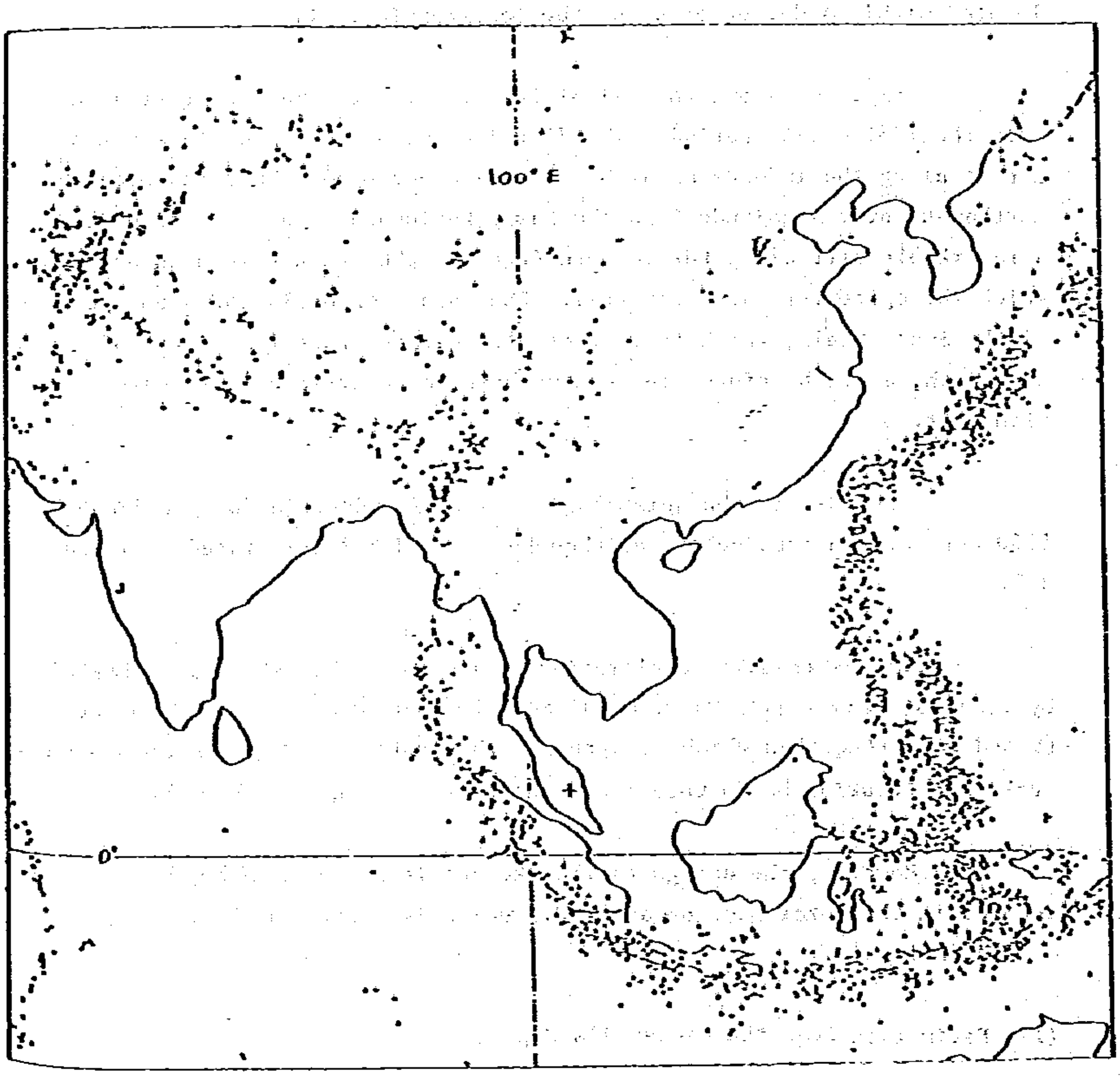
However, the design earthquake acceleration of 100 gal was adopted in the Tekai Dam, because this value is adopted for existing dams of Malaysia.

(1) Estimation from the Kawasumi's Map

Fig. 2-6 shows earthquake accelerations which are inferred from earthquakes with known magnitude, depth and location of epicenters. It is assumed that earthquake acceleration on the Malay Peninsula is less than 0.01 g ($\fallingdotseq$ 10 gal).

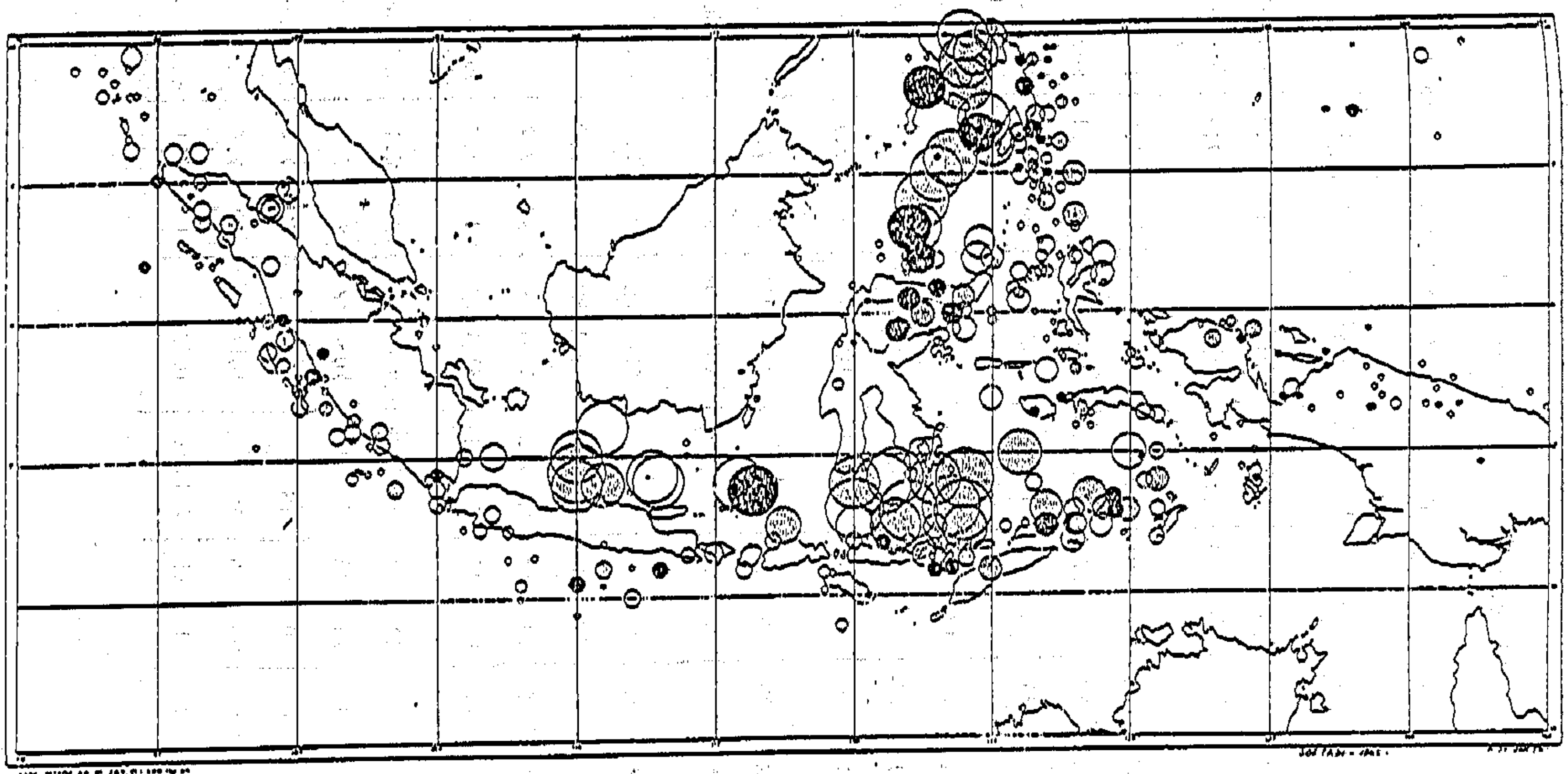
Fig 2-4 Distribution of Epicenter in Southeast Asia

( after Bulletin of the Seismological Society of America
Vol. 59, No. 1, pp. 369 380. February, 1969 )




* Muawia Barazangi & James Dorman (1969) : World Seismicity Maps Compiled from ESSA, Coast and Geodetic Survey, Epicenter Data, 1961-1967, Bulletin of the Seismological Society of America, Vol. 59, No. 1, 1969

Fig. 2-5 Seismological Map of Indonesia



* T. Boen (1971) : A Brief Outline of Seismicity and Earthquake Engineering Problems in Indonesia

Table 2-8 The Records of Seismicity in Indonesia

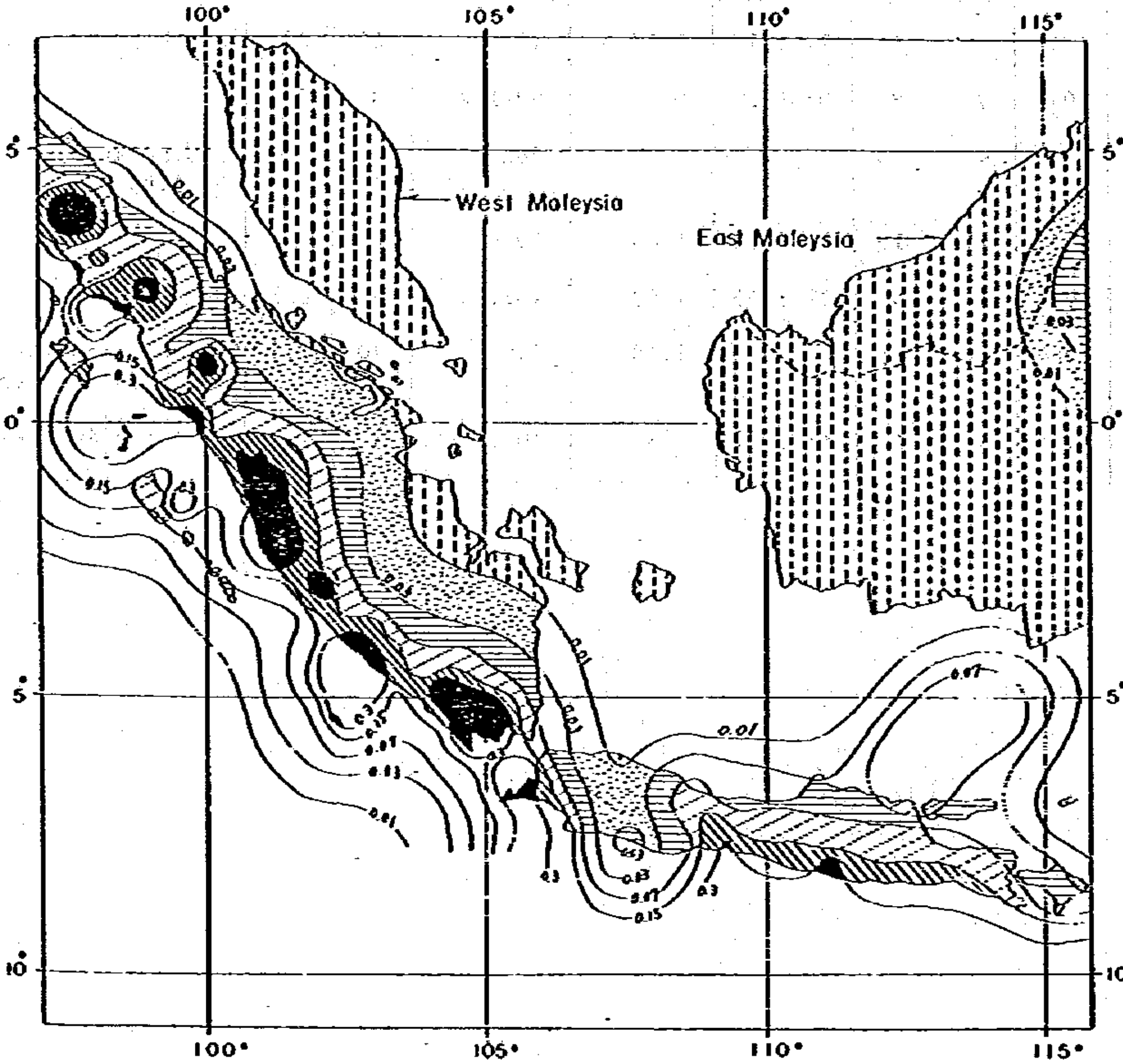
1900 ∿ 1980 M ≧ 7.0 89-1/2° ∿ 110°E 15°N ∿ 5°S

| Year | Month | Day | Time | Latitude | Longitude | Depth (km) | Magnitude | Location |
|---|---|---|---|---|---|---|---|---|
| 1906 | VI | 24 | 11 : 17 | 15N | 92E | S | 7.3 | Bay of Bengal |
| 1907 | I | 4 | 05 : 19 | 2N | 96.3E | S | 7.8 | Off west coast of northern Sumatra |
| | XI | 21 | 20 : 03 | 59N | 95.3E | | 7.4 | Northern Sumatra |
| 1913 | VIII | 13 | 04 : 25 | 5-1/2S | 105E | 75 | 7.2 | Southern Sumatra |
| 1914 | X | 11 | 16 : 17 | 12N | 94E | 80 | 7.2 | Andenman Is. region |
| 1917 | XI | 4 | 12 : 03 | 4.8N | 968E | | 7.1 | Northern Sumatra |
| 1926 | VI | 28 | | 0.5S | 100.5E | | | Sumatra |
| 1931 | II | 10 | 06 : 34 | 5-1/4S | 102-1/2E | S | 7.1 | South Sumatra |
| | IX | 25 | 05 : 59 | 5S | 102-3/4E | S | 7.4 | Southern Sumatra |
| 1933 | VI | 24 | 21 : 54 | 5-1/2S | 104-3/4E | S | 7.5 | Southern Sumatra |
| 1935 | XII | 28 | 02 : 35 | 0 | 98-1/4E | S | 7.9 | Northern Sumatra |
| 1936 | I | 2 | 22 : 34 | 0 | 99-1/2E | 60 | 7.0 | " |
| | IX | 19 | 01 : 01 | 3-3/4N | 97-1/2E | S | 7.2 | " |
| 1939 | III | 21 | 01 : 11 | 1-1/2S | 89-1/2E | S | 7.2 | South Indian Ocean |
| 1941 | VI | 26 | 11 : 52 | 12-1/2N | 92-1/2 | S | 8.1 | Andaman Sea |
| 1943 | VI | 1 | 14 : 18 | 6-1/2S | 105-1/2 | S | 7.0 | |
| | VI | 8 | 20 : 42 | 1S | 101E | 50 | 7.4 | Northern Sumatra |
| | VI | 9 | 03 : 06 | 1S | 101E | 50 | 7.6 | " |
| | XI | 26 | 21 : 25 | 2-1/2S | 100E | 130 | 7.1 | Southern Sumatra |

Table 2-8 (Cont'd)

| Year | Month | Day | Time | Latitude | Longitude | Depth | Magnitude | Location |
|---|---|---|---|---|---|---|---|---|
| 1944 | I | 5 | 21 : 12 | 3-1/2S | 102E | 60 | 70 | Southern Sumatra |
| 1946 | V | 8 | 05 : 20 | 0 | 99-1/2 | S | 7.1 | " |
| | V | 14 | | 1S | 98E | | 7.0 | Southwest of Sumatra |
| 1963 | XII | 15 | 19 : 34 | 4.8S | 108E | 650 | 7.1 | Java Sea |
| 1964 | I | 4 | 22 : 45 | 1.9S | 102.3E | 33 | 6.7 | Sumatra |
| 1969 | XI | 21 | 02 : 05 | 2.1N | 946 | 20 | 7.5 | Off W. coast of Northern Sumatra |
| 1975 | X | 1 | 13 : 29 | 49S | 102.2E | N | 7.0 | Southern Sumatra |

Fig 2-6 Isosemal Man of Southeast Asia
(after Peta Iso Seismo \Indonesia, Hiroshi Kawasumi)




Leend

| | | | |
|---|---|---|---|
| ■ | = 0.3g | ▤ | = 0.03 - 0.07g |
| ▨ | = 0.15 - 0.3g | ░ | = 0.01 - 0.03g |
| ▨ | = 0.07 - 0.15g | ▥ | 0.01g |

= Maximum acceleration g = Gravity (9.8 m/sec²)

(2) Estimation by means of formulae

The seismic motions of the two earthquakes that are presumed to have caused the biggest acceleration at the project site, among those which occurred in the past and indicated in the accompanying Seismological Map, are calculated in the followings.

a) Earthquake (1)

Magnitude: 7.9

Distance from epicenter: 600 km

b) Earthquake (2)

Magnitude. 6.8

Distance from epicenter: 375 km

The empirical formulae used to calculate the earthquake motion are as follows.

i) Kanai-Seed's Formulae

$$\log_{10} V_o = 0.61M - (1.66 + \frac{3.6}{x})\log_{10} X - (0.631 + \frac{1.83}{x})$$

$$Tp = \frac{1}{1.000}\{(0.539M - 1.621)\Delta + 50.44M - 111.16\}$$

$$\alpha = 2\pi \frac{1}{Tp} V_o$$

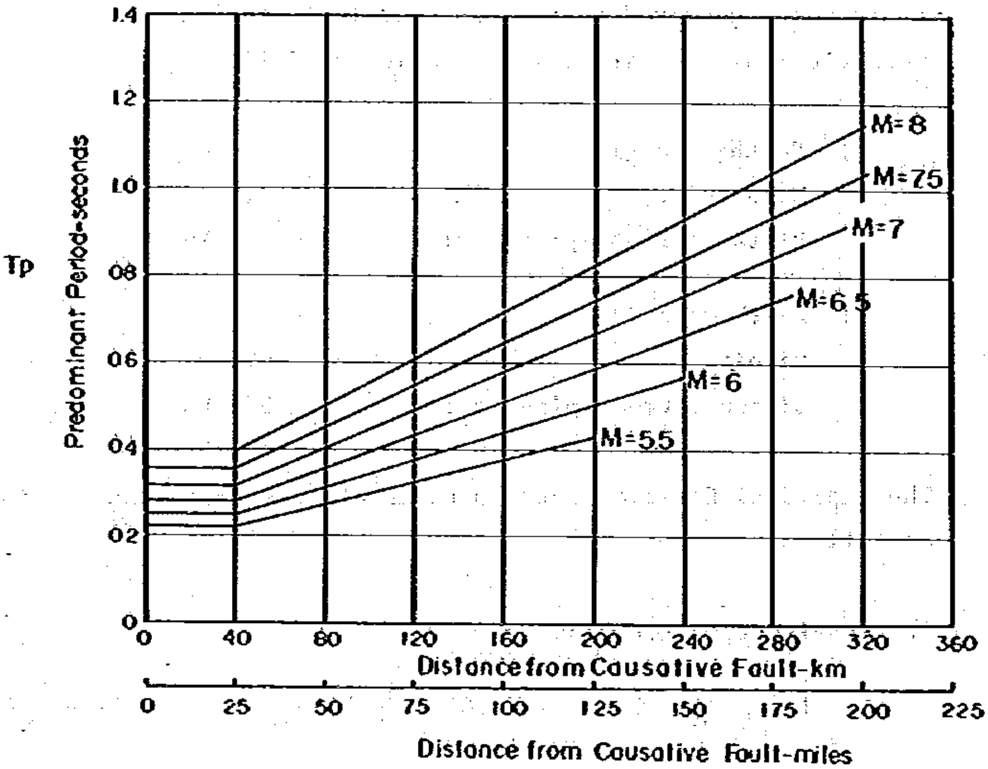
ii) Okamoto's Formula

$$\log_{10} \frac{\alpha m}{640} = \frac{(\Delta + 40)}{100} \times (-7.604 + 1.7844M - 0.1036M^2)$$

The acceleration of seismic motion calculated by means of the aforestated formulae are as follows.

| Empitical Formula / Earthquake | (1) Kanai-Seed's Formula | (2) Okamoto's Formula |
|---|---|---|
| Earthquake (1) | 1.25 gal | 0.99 gal |
| Earthquake (2) | 1.03 gal | 1.08 gal |

Fig 2-7 Periods for Maximum accelerations in Rock
(by Seed)




$$T_p = \frac{1}{1000}\{(0.539M + 1.621)\Delta + 50.44M - 111.16\}$$

$\Delta$ 40 km, $\Delta$ = 40 km

## 2.5 Dam Stability Analysis

(1) Standard section

Refer to Fig. 2-8.

(2) Design specifications

| | |
|---|---|
| Dam type: | Rock fill dam with impervious center core |
| Dam crest elevation: | EL 166.200 m |
| Dam foundation elevation: | EL 65.200 m |
| Dam crest width: | 6.000 m |
| Design flood level: | EL 164.000 m |
| Maximum water level of reservoir: | EL 157.000 m |
| Minimum water level of reservoir: | EL 147.000 m |

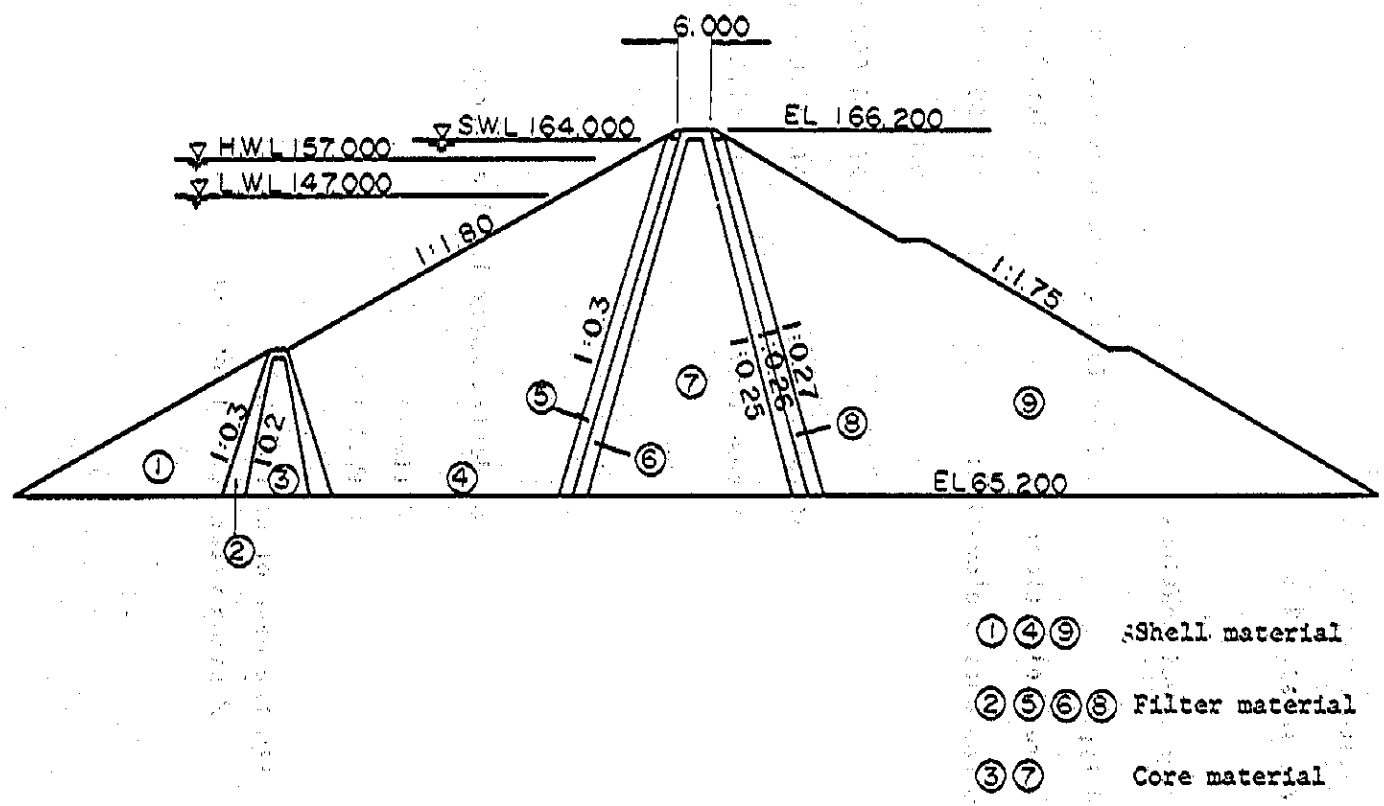
(3) Design water level

The following values of design water level are adopted.

(Under normal conditions)
Maximum water level + hw

(In the case of earthquake)
Maximum water level + hw + he

Fig. 2-8 Standard Cross Section of the Dam

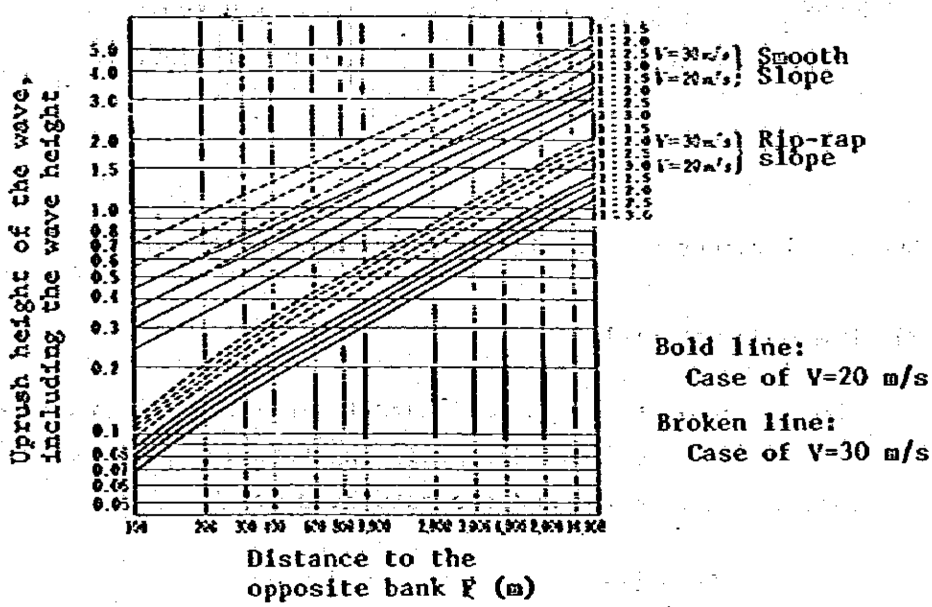


where;

hw : Height of wave caused by wind

According to the S.M.B. Method and the Saville Method, hw is 1.20 m when the upstream slope gradient is 1:1.80, the distance to the opposite bank of the storage reservoir is 4,800 m and the wind velocity is 30 m/s. (Refer to Fig. 2-9).

Fig. 2-9 Uprush Height Determined by the S.M.B. Method and Saville's Method




he : Wave height caused by earthquake

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

K : Design seismic intensity = 0.10

τ : Earthquake cycle = 1.0 second

Ho : Reservoir depth at maximum water level
= 92.000 m

$$he = \frac{1}{2} \times \frac{0.10 \times 1.0}{\pi} \times \sqrt{9.8 \times 92} = 0.478 \text{ m} \doteqdot 0.500 \text{ m}$$

Therefore, the design water level is:

| | H.W.L | S.W.L |
|---|---|---|
| Under normal conditions | EL. 157.000 + 1.200 + 0.500 = EL. 158.700 m | EL. 164.000 + 1.200 + 0.500 = EL. 165.700 m |
| In the case of earthquake | EL = 157.000 + 1.200 + 0.500 + 0.500 = EL. 159.200 m | EL. 164.000 + 1.200 + 0.500 + 0.500 = EL. 166.200 m |

(4) Numerical values used in the calculations

1) Dam body material

The values of the physical characteristics of the materials are listed in Table 2.9. The following formulae are used to calculate the wet weight and the saturated weight.

$$dw = \frac{GS + S \cdot e}{1 + e}$$

$$ds = \frac{Gs + e}{1 + e}$$

where;

dw : Wet density $(t/m^3)$

ds : Saturated density $(t/m^3)$

Gs : Specific gravity of particles

S : Degree of saturation $S = \frac{W \cdot Gs}{100e}$ (w : water content ratio)

e : Void ratio

The values of dw and ds of the shell material and filter material are estimated from our experiences accumulated in Japan. As for the core material, dw and ds are calculated from the values of Gs, e and w obtained from the tests.

Table 2-9 Physical Properties of Materials

| Properties / Material | Saturated Density ds ($t/m^3$) | Wet Density dw ($t/m^3$) | Cohesion ($t/m^2$) | Angle of internal friction $\phi$ (°) |
|---|---|---|---|---|
| Shell material | 2.10 | 1.80 | 0 (5) | 45° (40°) |
| Filter material | 2.10 | 1.80 | 0 (5) | 45° (40°) |
| Core material | 1.99 | 1.94 | 0 (9) | 14.5° (14.5°) |

Table 2-10 Test Results and Physical Properties of the Core Material

| Gs | e | W | S | dw ($t/m^3$) | ds ($t/m^3$) |
|---|---|---|---|---|---|
| 2.62 | 0.6752 | 0.214 | 0.830 | 1.90 | 1.97 |
| 2.69 | 0.7969 | 0.270 | 0.911 | 1.90 | 1.94 |
| 2.62 | 0.5394 | 0.178 | 0.865 | 2.01 | 2.05 |
| 2.67 | 0.6553 | 0.200 | 0.815 | 1.94 | 2.01 |
| | | | Average | 1.94 | 1.99 |

ii) Weight per unit volume of water 1.0 $t/m^3$

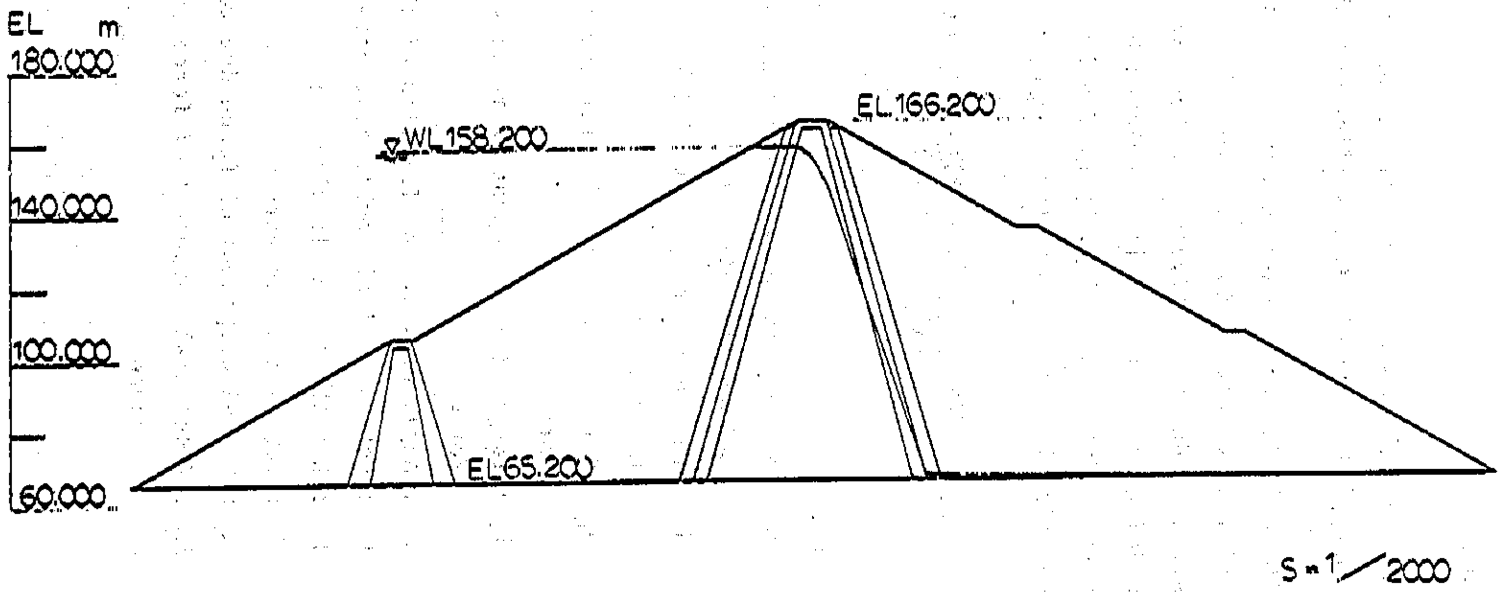
iii) Seepage line

As for the weight of the material for the sake of stability calculations, the wet weight is used for parts above the seepage line and the saturated weight is used for parts below that line. Therefore, it is necessary to identify the seepage line of the dam body. In the present calculation, the seepage line is determined by means of the method proposed by A. Casagrande.

The seepage line is shown in Fig. 2-10. The water level of the storage reservoir is assumed to be EL 158.200.

iv) Horizontal seismic intensity k = 0.1

Fig. 2-10 Seepage Line of the Dam Body



## (5) Stability calculation

### 1) Method

The stability calculation of the dam slope sliding is conducted by applying the circular sliding method.

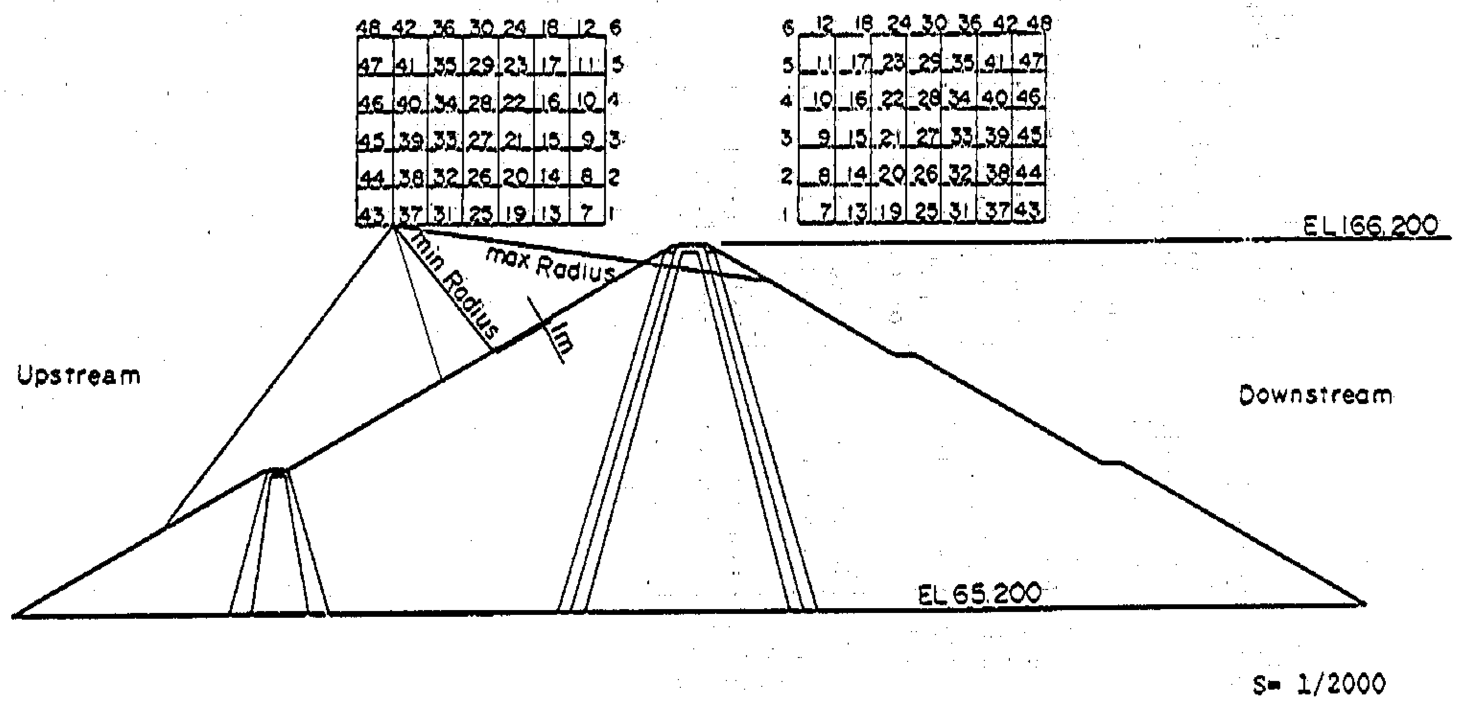
The calculation is carried out by setting the center of the critical circle at 48 points of the upstream side and down-stream side, respectively, as shown in Fig. 2-11. The slope sliding safety factor is calculated by computer using the formula indicated below, by gradually changing the radius of the circular radius from the maximum to the minimum value.

$$SF = \frac{\Sigma(N - U + P_N - N_E)\tan\phi + \Sigma C \cdot L}{\Sigma(T - P_T + T_E)}$$

where;

- $N$ : Normal component of force due to the weight of embankment material
- $T$ : Tangential component of force due to embankment material
- $N_E$ : Normal component of force due to the earthquake force
- $T_E$ : Tangential component of force due to the earthquake force
- $\phi$ : Angle of internal friction of the embankment material
- $C$ : Cohesion of the embankment material
- $L$ : Length of the circular arc
- $U$ : Pore water pressure
- $P_N$ : Normal component of force due to hydrostatic pressure
- $P_T$ : Tangential component of force due to hydrostatic pressure

Fig. 2-11 Position of the Critical Circle



ii) Examined cases

The stability calculation is carried out for the 12 cases indicated below.

| Water Level \ Properties | | Normal or Earthquake | Upstream Slope | Downstream Slope |
|---|---|---|---|---|
| In the case of maximum water level | No cohesion considered | Under normal conditions | 0 | 0 |
| | | In the case of earthquake | 0 | 0 |
| | Cohesion considered | Under normal conditions | 0 | 0 |
| | | In the case of earthquake | 0 | 0 |
| In the case of design flood | No cohesion considered | Under normal conditions | 0 | 0 |
| | Cohesion considered | Under normal conditions | 0 | 0 |

iii) Calculation results

The sliding surface configuration that guarantees the minimum safety factor obtained by the calculations and the center of the corresponding critical angle are shown in Fig. 2-12 ∿ 2-17. The minimum safety factor of each case is shown in Table 2-11.

Table 2-11 Minimum Coefficient of Safety

| | | Physical Properties | Normal or Earthquake | Upstream Slope | Downstream Slope |
|---|---|---|---|---|---|
| In the case of maximum water level | No cohesion considered | Shell and filter $C = 0\ t/m^2, \phi = 45°$ Core $C=9\ t/m^2, \phi = 14.5°$ | Under normal conditions | 1.435 | 1.761 |
| | | | In the case of earthquake | 1.219 | 1.412 |
| | Cohesion considered | Shell and filter $C=5\ t/m^2, \phi = 45°$ Core $C=9\ t/m^2, \phi = 14.5°$ | Under normal conditions | 1.664 | 2.181 |
| | | | In the case of earthquake | 1.610 | 1.621 |
| In the case of design flood | No cohesion considered | Shell and filter $C=0\ t/m^2, \phi = 45°$ Core $C=0\ t/m^2, \phi = 14.5°$ | Under normal conditions | 1.401 | 1.761 |
| | Cohesion considered | Shell and filter $C=5\ t/m^2, \phi = 40°$ Core $C=9\ t/m^2, \phi = 14.5°$ | Under normal conditions | 1.558 | 2.160 |

Fig. 2-12 Minimum Safety Factor Referring to Circular Sliding

(No cohesion, under normal conditions)

| | | | | |
|---|---|---|---|---|
| C | Shell and filter | 0 t/m$^2$ | Minimum safety factor | Upstream side: 1.435 |
| | Core | 0 t/m$^2$ | | Downstream side: 1.761 |
| $\phi$ | Shell and filter | 45° | Storage reservoir water level: | Maximum water level (El 157.000 m) |
| | Core | 14.5° | | |

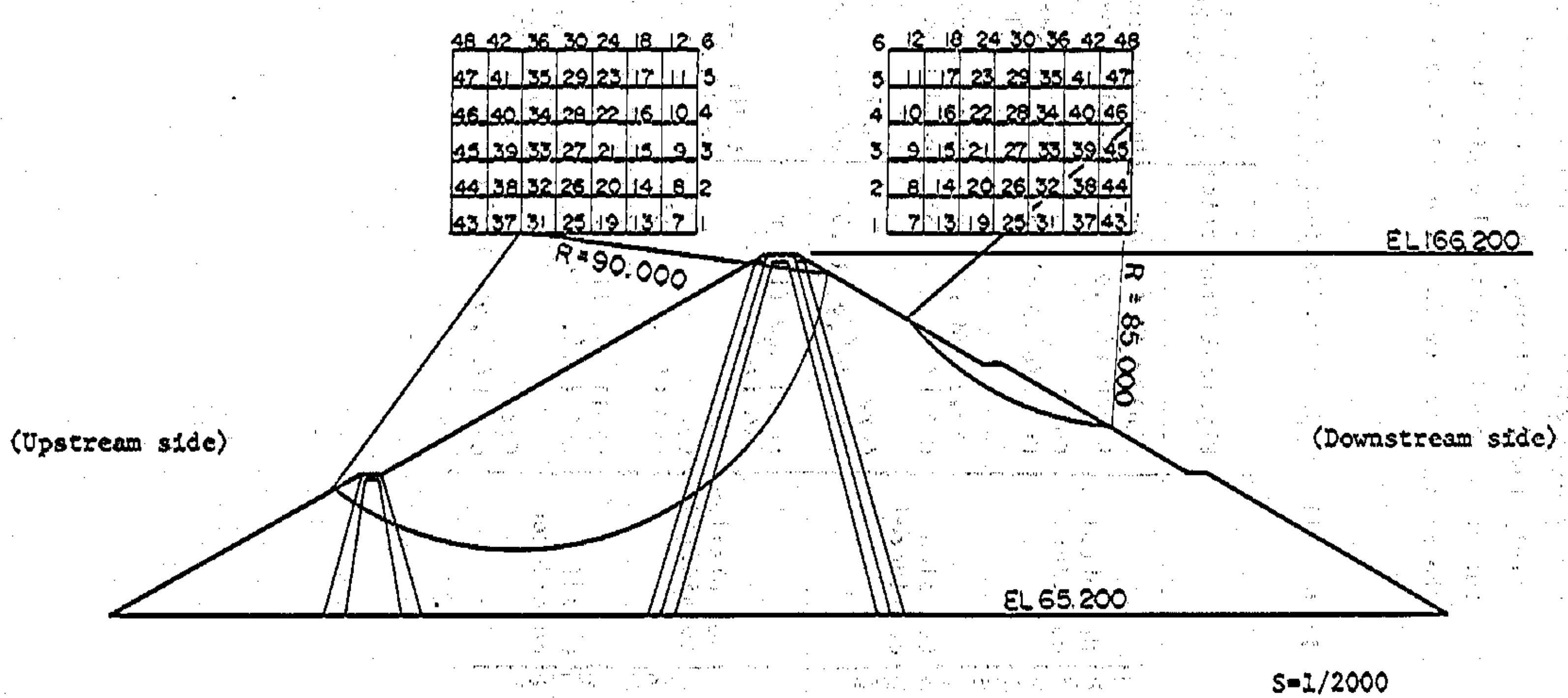
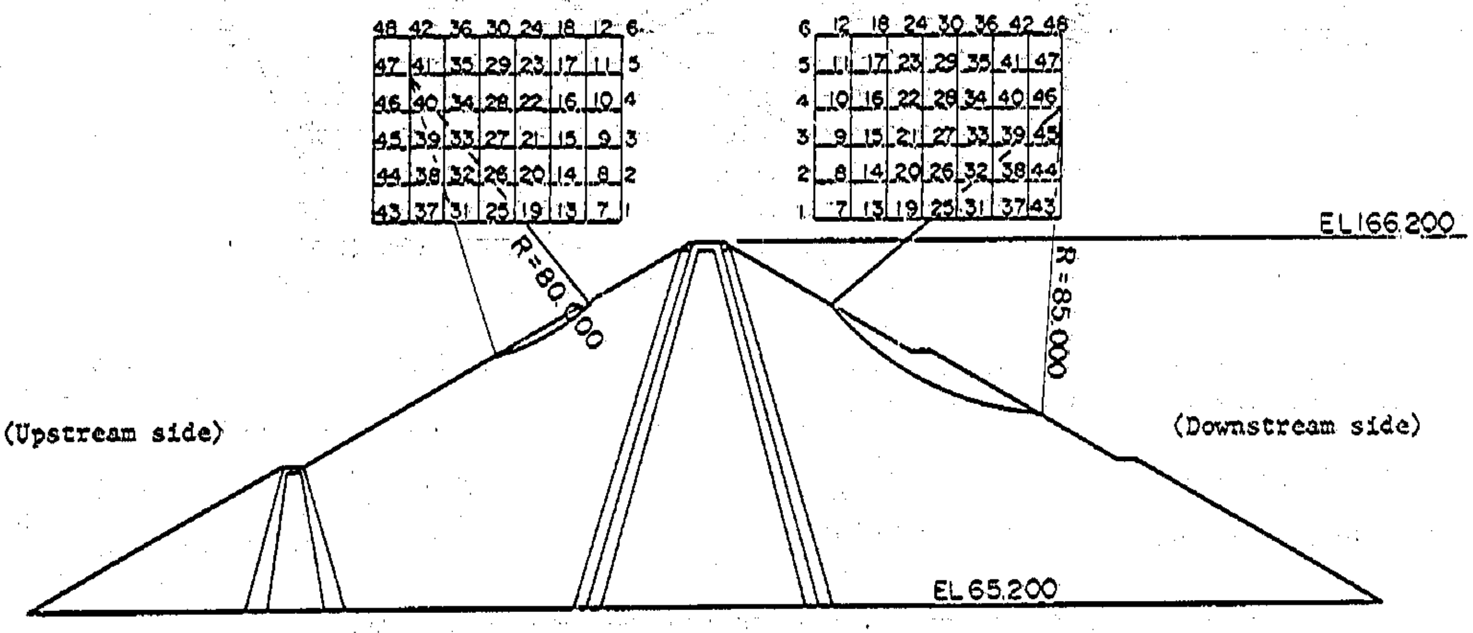


Fig. 2-13 Minimum Safety Factor Referring to Circular Sliding

(No cohesion, in the case of earthquake)

| | | | |
|---|---|---|---|
| C { Shell and filter | 0 t/m$^2$ | Minimum safety factor | { Upstream side: 1.219 |
| Core | 0 t/m$^2$ | | Downstream side: 1.412 |
| $\phi$ { Shell and filter | 45° | Storage reservoir water level: | Maximum water level (El 157.000 m) |
| Core | 14.5° | Dam body seismic intensity: | k = 0.1 |




S=1/2000

Fig. 2-14 <u>Minimum Safety Factor Referring to Circular Sliding</u>

(Taking cohesion into consideration and under normal condition)

| | | | | | |
|---|---|---|---|---|---|
| c | Shell and filter | 5 $t/m^2$ | Minimum safety factor | Upstream side: | 1.664 |
| | Core | 9 $t/m^2$ | | Downstream side: | 2.181 |
| $\phi$ | Shell and filter | 40° | Storage reservoir water level: | Maximum water level (EL 157.000 m) | |
| | Core | 14.5° | | | |

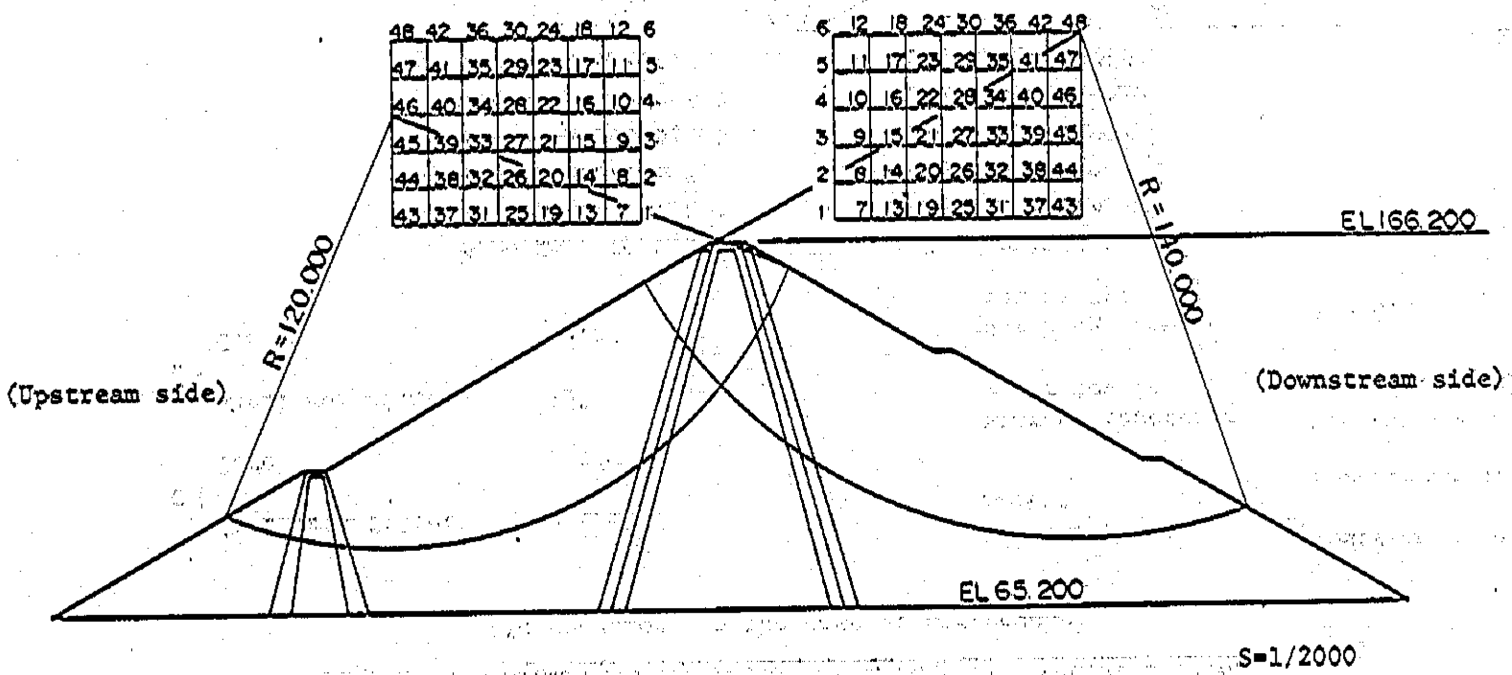


Fig. 2-15 Minimum Safety Factor Referring to Circular Sliding

(Taking cohesion into consideration and in the case of earthquake)

| | | | | |
|---|---|---|---|---|
| C | Shell and filter | 5 t/m$^2$ | Minimum safety factor | Upstream side: 1.610 |
| | Core | 9 t/m$^2$ | | Downstream side: 1.621 |
| $\phi$ | Shell and filter | 40° | Storage reservoir water level: | Maximum water level (EL 157.000 m) |
| | Core | 14.5° | Dam body seismic intensity: | k = 0.1 |

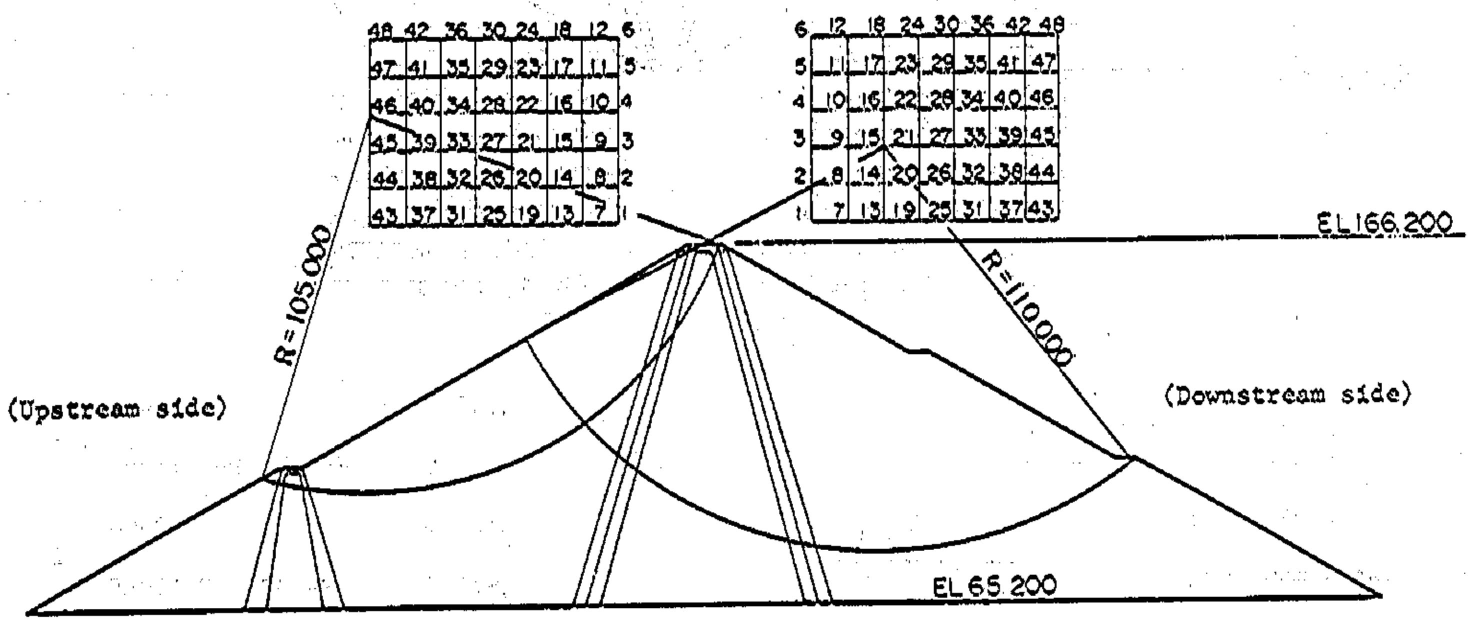


Fig. 2-16 Minimum Safety-Factor Referring to Circular Sliding

(No cohesion and under normal conditions)

| | | | | | |
|---|---|---|---|---|---|
| C | Shell and filter | 0 t/m$^2$ | Minimum safety factor | Upstream side: | 1.401 |
| | Core | 0 t/m$^2$ | | Downstream side: | 1.761 |
| $\phi$ | Shell and filter | 45° | Storage reservoir water level: | In the case of flood (EL 162.000 m) | |
| | Core | 14.5° | | | |

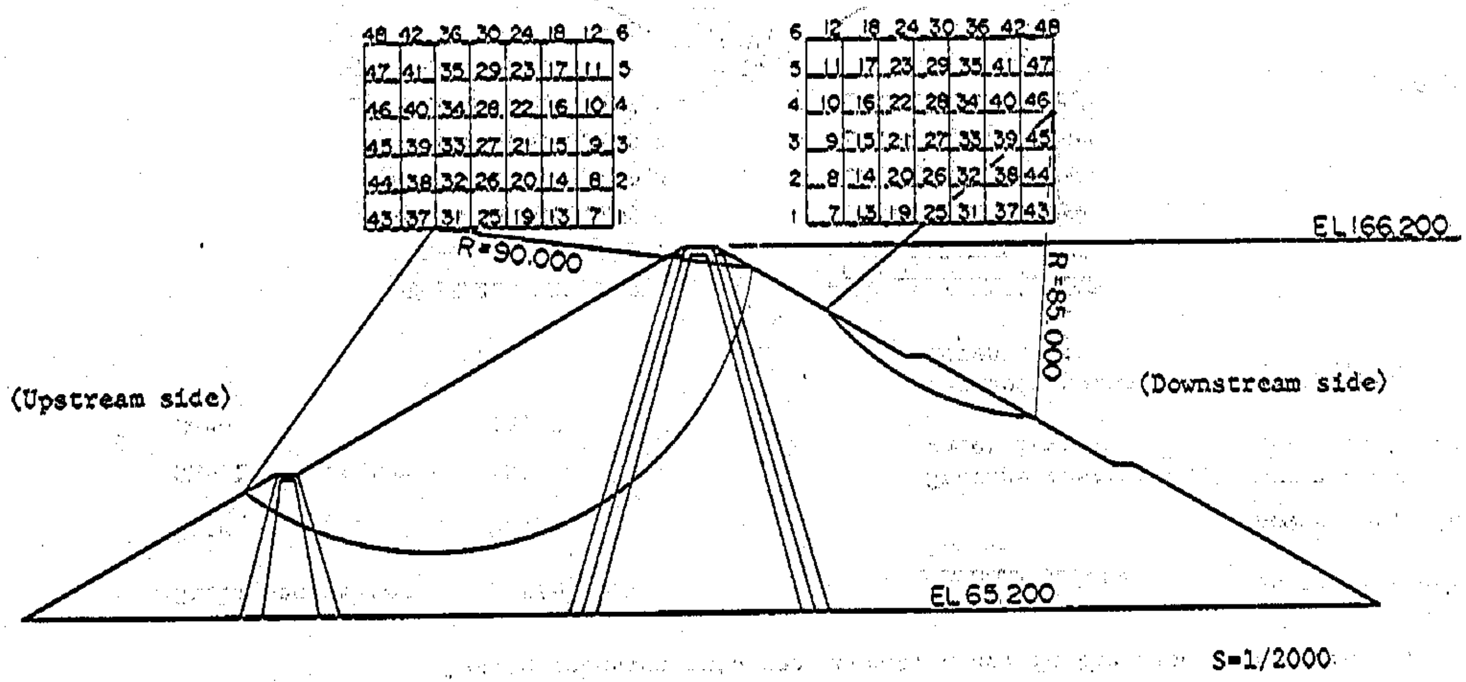
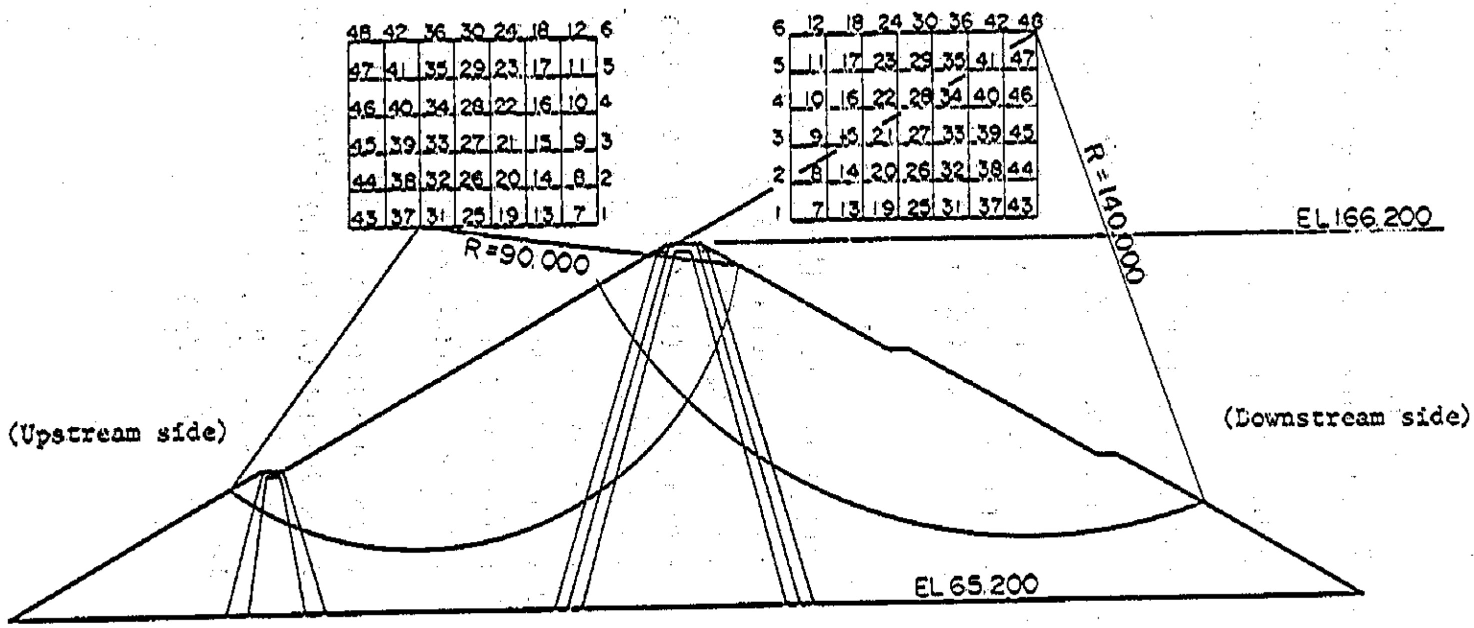


Fig. 2-17 Minimum Safety Factor Referring to Circular Sliding

(Taking cohesion into consideration and under normal conditions)

| | | | | |
|---|---|---|---|---|
| C | Shell and filter | 5 $t/m^2$ | Minimum safety factor | Upstream side: 1.558 |
| | Core | 9 $t/m^2$ | | Downstream side: 2.160 |
| $\phi$ | Shell and filter | 40° | Storage reservoir water level: | In the case of flood (EL 162.000 m) |
| | Core | 14.5° | | |




S=1/2000

### 2.6 Spillway

#### 2.6.1 Spillway Discharge

The spillway should be able to discharge the design flood flow (1,504 $m^3$/sec) at the reservoir level of EL. 164.00 m.

Discharge from the spillway is calculated by the following formula.

$$Q = C\ (B - K\ N\ H)\ H^{3/2}$$

$$Cd = 2.200 - 0.0416\ (Hd/W)^{0.9900}$$

$$C = 1.60\ \frac{1 + 2a\ (\frac{H}{Hd})}{1 + a\ \ (\frac{H}{Hd})}$$

where;

- Q : Discharge (1,504 $m^3$/s)
- B : Overflow width (40.00 m)
- H : Overflow head (m)
- Hd : Design head (164.00 - 157.00 = 7.00 m)
- W : Weir height (4.00 m)
- a : Constant
- Cd : Value of C when H = Hd
- K : Coefficient of shrinkage
- N : Number of shrinkages ( 3 × 2 = 6 )

$$\begin{aligned} Cd &= 2.200 - 0.0416\,(Hd/W)^{0.9900} \\ &= 2.200 - 0.0416(7.00/4.00)^{0.9900} = 2.128 \end{aligned}$$

$$C = 1.60\ \frac{1 + 2a\ (\frac{H}{Hd})}{1 + a\ \ (\frac{H}{Hd})}$$

$$2.126 = 1.60 \frac{1 + 2a}{1 + a}$$

$$2.128 + 2.128a = 1.60 + 3.20a$$

$$1.072a = 0.528$$

$$a = \frac{0.528}{1.072} = 0.4925$$

$$\therefore C = 1.60 \times \frac{1 + 0.9850\ (H/Hd)}{1 + 0.4925\ (H/Hd)}$$

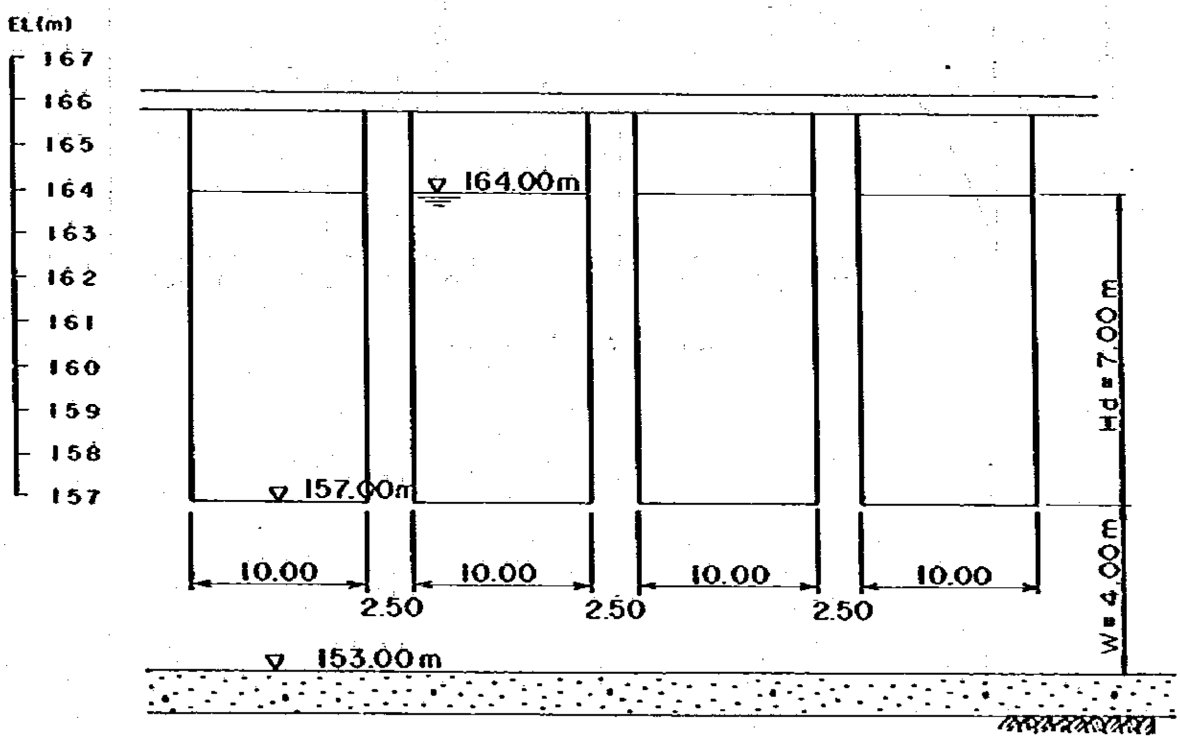


Fig. 2-18 The Spillway Rating Curve

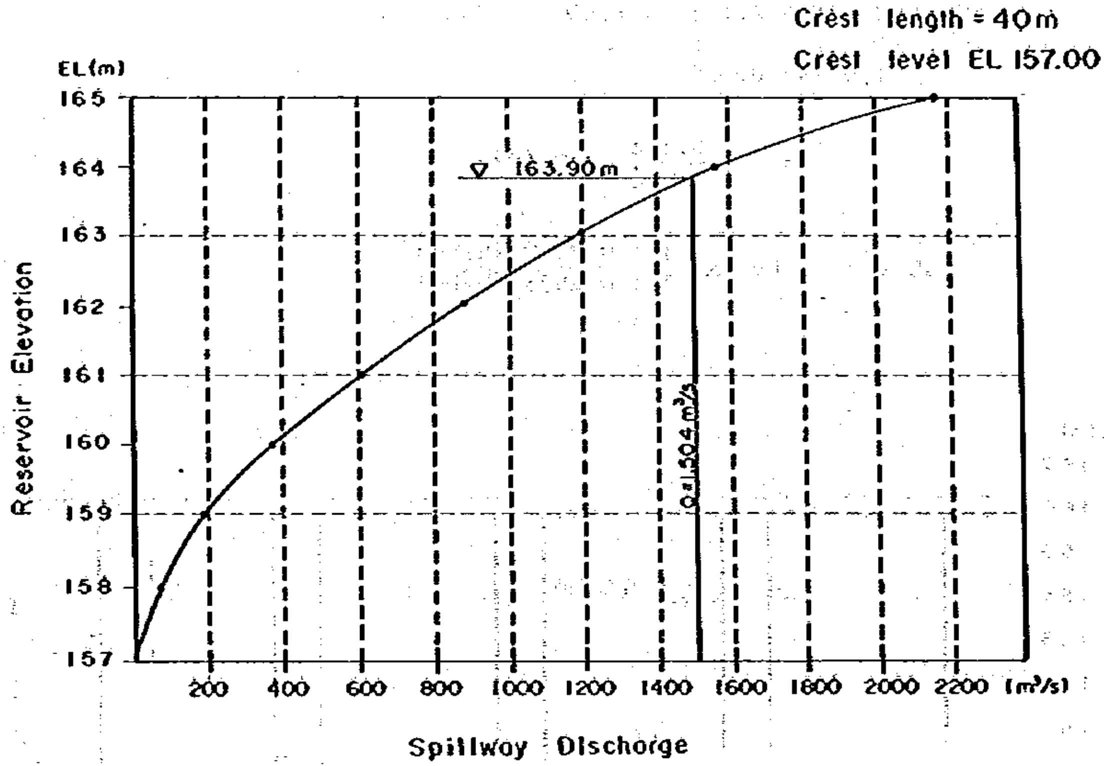
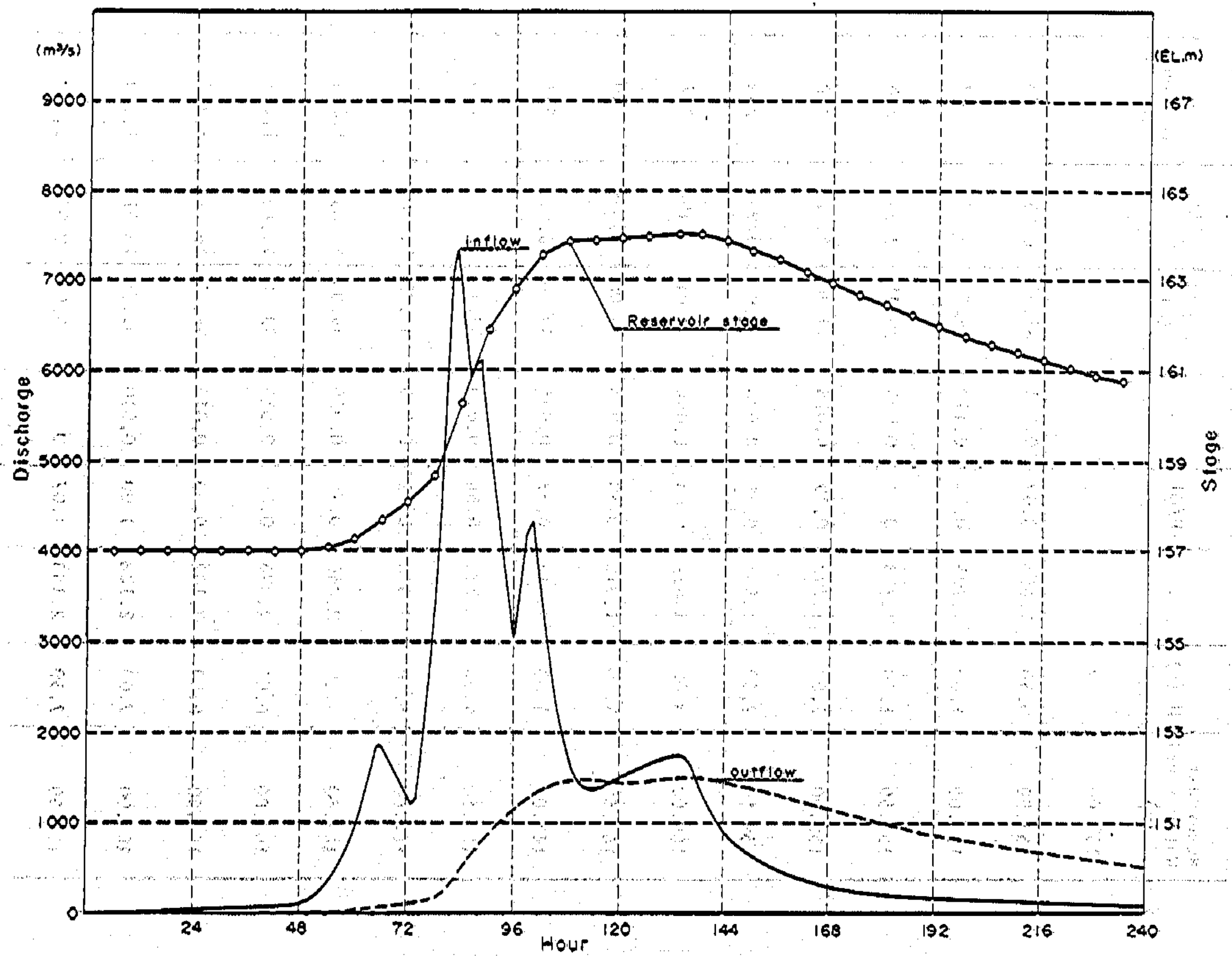


Table 2-12 Water Level and Discharge Volume

| Reservoir Water Level (m) | H (m) | C | H/Hd | K | B-KNH (m) | $H^{3/2}$ | Q ($m^3$/sec) |
|---|---|---|---|---|---|---|---|
| 157.00 | 0 | | | | | | |
| 157.50 | 0.50 | 1.654 | 0.07 | 0.125 | 39.63 | 0.354 | 23 |
| 158.00 | 1.00 | 1.705 | 0.14 | 0.110 | 39.34 | 1.000 | 67 |
| 158.50 | 1.50 | 1.753 | 0.21 | 0.095 | 39.15 | 1.837 | 126 |
| 159.00 | 2.00 | 1.797 | 0.29 | 0.075 | 39.10 | 2.828 | 199 |
| 159.50 | 2.50 | 1.839 | 0.36 | 0.070 | 38.95 | 3.953 | 283 |
| 160.00 | 3.00 | 1.879 | 0.43 | 0.055 | 39.01 | 5.196 | 381 |
| 160.50 | 3.50 | 1.916 | 0.50 | 0.045 | 39.06 | 6.548 | 490 |
| 161.00 | 4.00 | 1.951 | 0.57 | 0.035 | 39.16 | 8.000 | 611 |
| 161.50 | 4.50 | 1.985 | 0.64 | 0.030 | 39.19 | 9.546 | 743 |
| 162.00 | 5.00 | 2.016 | 0.71 | 0.025 | 39.25 | 11.180 | 885 |
| 162.50 | 5.50 | 2.046 | 0.79 | 0.020 | 39.34 | 12.899 | 1,038 |
| 163.00 | 6.00 | 2.075 | 0.86 | 0.015 | 39.46 | 14.697 | 1,203 |
| 163.50 | 6.50 | 2.102 | 0.93 | 0.013 | 39.49 | 16.572 | 1,376 |
| 164.00 | 7.00 | 2.128 | 1.00 | 0.010 | 39.58 | 18.520 | 1,560 |
| 164.50 | 7.50 | 2.153 | 1.07 | 0 | 40.00 | 20.540 | 1,769 |
| 165.00 | 8.00 | 2.176 | 1.14 | 0 | 40.00 | 22.627 | 1,969 |



Reservoir Inflow, Stage, Outflow hydrograph (Upper Dam)

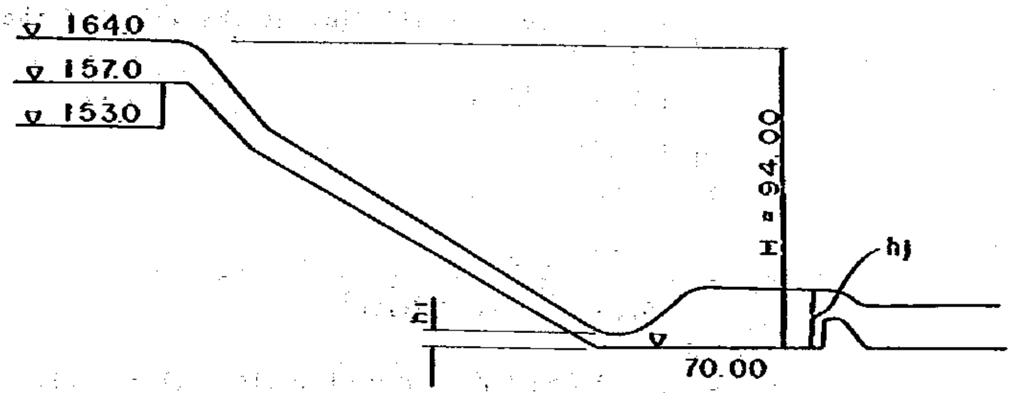
### 2.6.2 Stilling Basin

The hydraulic-jump type horizontal apron equipped with sill is adopted as the stilling basin in view of the topography of the project site.

The discharge is assumed to be $Q = 1{,}504\ m^3/sec$

Basin width $B = 35.00\ m$

(1) Hydraulic calculation of the stilling basin




1) Rapid-flow depth of the basin $h_h$

$$h_1 = \frac{Q}{0.95\ B\ \sqrt{2gH}}$$

where;

Q : Discharge to be handled by the stilling basin $= 1{,}504\ m^3/sec$

B : Basin width = 35.00 m

H : Difference of elevation between design flood level and basin surface

$= 164.00 - 70.00 = 94.00\ m$

g : Acceleration of gravity $= 9.8\ m/s^2$

$$h_1 = \frac{1,504.00}{0.95 \times 35.00 \sqrt{2 \times 9.8 \times 94.00}} = 1.054 \text{ m}$$

ii) Sequent depth of stilling basin, $h_j$

$$h_j = \frac{1}{2} h_1 \left( \sqrt{1 + 8 F_1^2} - 1 \right)$$

$$F_1 = \frac{V_1}{\sqrt{g h_1}} = \frac{Q}{B \sqrt{g h_1^3}}$$

where;

- $F_1$ : Froude's Number of rapid flow at the inlet of the basin
- $V_1$ : Velocity of rapid flow at the inlet of the basin (m/s)
- $Q$ : 1,504 $m^3$/sec
- $h_1$ : 1.054 m
- $B$ : 35.00 m

$$F_1 = \frac{1,504.00}{35.00 \sqrt{9.8 \times (1.054)^3}} \doteq 12.68$$

$$h_j = \frac{1}{2} \times 1.054 \left( \sqrt{1 + 8 \times (12.68)^2} - 1 \right) = 18.40 \text{ m}$$

The end sill is required because the water level downstream of the dam (EL 78.7 m during power generation) is beneath the sequent depth (14.20 + 70.00 = EL 84.20 m). (Refer to the stage=discharge curve of the spillway).

iii) Required sill height (d)

$$\frac{d}{h_1} = \frac{(1 + 2 F_1^2) \sqrt{1 + 8 F_1^2} - (1 + 5 F_1^2)}{1 + 4 F_1^2 - \sqrt{1 + 8 F_1^2}} - \frac{3}{2} F_1^{2/3}$$

where;

$d$ : Required sill height (m)
$h_1$ : 1.054 m
$F_1$ : 12.68 m

$$\frac{d}{h_1} = \frac{(1 + 2 \times 16.37^2)\sqrt{1 + 8 \times 12.68^2} - (1 + 5 \times 12.68^2)}{1 + 4 \times 12.68^2 - \sqrt{1 + 8 \times 12.68^2}} - \frac{3}{2} \times 12.68^{2/3}$$

$= 9.43$

$d = 9.43 \times 1.054 = 10.00$

iv) Required apron length (Ls)

$Ls = 4 \sim 5\ h_j$

$= 4.5 \times 18.40 = 82.80$

The apron is designed with Ls = 83.00 m.

## 2.7 Intake

The submergence depth is an important aspect regarding the design of the intake. It is determined by carrying out hydraulic model test.

### 2.7.1 Hydraulic Model Test

(1) Foreward

The primordial condition to be taken into consideration in designing the intake is to guarantee the required discharge. Therefore, the intake shall be designed with the required cross section. Furthermore, measures shall be taken in order to prevent suspended sediments, driftwood and undesirable materials from entering the intake.

The most important factor that determines the cross section of the intake is the velocity of flow at the screen. Cross section A is given by the following expression when the speed of flow V is assumed to be 1 m/s.

$$A = \frac{Q}{V}$$

On the other hand, the relation between the width B and the height H of the intake is normally comprehended between the following limits.

$$B/H = 1 \text{ to } 3.0$$

The velocity of flow inside the penstock connected with the intake is normally on the order of 4 to 6 m/s. Therefore, the cross section shall be shrunk to approximately 1/4 to 1/6 between the screen and the inlet of the penstock. When the cross section changes abruptly, the flow lines become discontinuous, thus causing formation of retardation areas that are not only cause vortexes, but also increase head loss at the inlet.

Therefore, the gradual shrinkage of the cross section shall be taken into consideration with sufficient care both on the ground plan and in the longitudinal section.

(2) Similitude criteria

In the experiments related to the intake, it is necessary to take into consideration not only the effect of the gravity but also the viscosity, surface tension and other physical properties of water as factors contributing to the generation of vortexes. However, the requirements of similitude referring to the gravity, viscosity, surface tension and other relevant aspects can not be satisfied at one time with a single model. Therefore, we decided to make the model as large as possible in order to negate the influences of the viscosity, surface tension, etc. of water.

In the present experiment, the model of the upper dam is constructed on a 1:36.66 scale, the model of the lower dam is constructed on a 1:30 scale and the experiment is conducted by applying the Froude's similitude criteria.

By coincide Froude's number of model with actual structure, the hydraulic model is as follows.

Froude's Number $= \dfrac{V}{\sqrt{g\,L}}$

- Upper Tekai

Scale of model : $\dfrac{1}{35.66}$

Velocity : $\dfrac{1}{\sqrt{35.66}} = \dfrac{1}{5.97}$

Discharge : $\dfrac{1}{\sqrt{35.66}} \times \dfrac{1}{35.66^2} \times 235\ m^3/s = 30.95\ \ell/s$

- Lower Tekai

Scale of model : $\dfrac{1}{30}$

Velocity : $\dfrac{1}{\sqrt{30}} = \dfrac{1}{5.48}$

Discharge : $\dfrac{1}{\sqrt{30}} \times \dfrac{1}{30^2} \times 80\ m^3/s = 16.23\ \ell/s$

Therefore, the various magnitudes of the model and the actual intake are related as follows.

| | Upper Intake | Lower Intake |
|---|---|---|
| Length : 1 | 1/35.66 | 1/30 |
| Velocity of flow : v | 1/5.97 | 1/5.48 |
| Discharge : q | 30.95 ℓ/s | 16.23 ℓ/s |

(3) Experiment equipment

The tank used to conduct the experiment of the intake model is a concrete block structure of 21 m length, 3 m width and 1.6 m height and water is supplied by means of a 150 mm caliber pump having q = 50 L/sec discharge. The upper dam intake model was constructed at the center of the tank, while the lower dam intake model was constructed at the extremity of the tank. The discharges are controlled by means of valves provided at the intake pipes of the models and are measured at the discharge measurement weir.

(4) Experiment results

1)) Upper dam intake

a) State of flow

In the upper dam intake, the pocket located behind the screen is small. Consequently the velocity of flow is rapid and the water surface is characterized by violent disturbance. This violent water surface disturbance helps to supress the generation of vortices.

When water flows into the intake with tank water at a standstill (pump stopped), there occurs formation of vortices accompanied with air. Water of the Tekai Dam is expected to attain a high degree of stillness in view of the scale of the dam. Accordingly, vortices are expected to occur more frequently than in the present model experiment.

b) Proposed improvement

The upper dam intake has a large effective depth of approximately 10 m. Accordingly, the use of beams is proposed as a means to prevent the generation of vortices in the intake. The beams have 1 m width and are evenly spaced at 1 m pitch. Part of the center pier is extended up to the upper part of the inlet and is used as an intermediate girder. Details of the aforementioned arrangement are shown in Figs. 2-20 and 2-21.

After the implementation of the aforesaid improvement in the intake model, the resistance against the flow of water is increased due to the newly installed beams. Consequently, the flow of water entering from the upper part of the intake tower inlet is reduced. The state of flow in the interior of the tower is considerably improved and the vortices disappear completely. Furthermore, no vortex was observed in front of the intake even when the rate of discharge was increased.

ii) Lower dam intake

a) State of flow

The lower dam intake is constructed with a 60° inclination at the upper part of the inlet for the purpose of preventing the occurrence of vortices. On the ground plan, the two side walls are projected normally to retain the stop logs. Water flowing into the intake from outside the side walls is conveyed to the center of the intake by centrifugal force. The space comprehended between the aforesaid flow lines and the side walls become a retardation area and a small whirling flow takes place therein.

b) Proposed improvement

A guide wall consisting of stop logs installed inside the side walls projected in the normal direction was provided in the model for the purpose of reducing the retardation area. (Fig. 2-22, 2-23).
The state of flow was considerably improved due to the insertion of the aforesaid guide wall and the undesirable whirling flow was eliminated.

(5) Consideration

Many studies concerning vortices occurring in ordinary intakes have been carried out. It is found that vortices are not generated when the ratio between the submersion depth H of the intake and the diameter D of the intake pipe is comprehended within the following limits.

$H/D > 1.5$ to $2.5$

However, the vortex generation mechanism is not simple and vortices occur even when $H/D \doteqdot 20$ if there is a plane whirling flow in the tank.

On the other hand, vortices do not occur even when the depth of the intake is small if the discharge is small, i.e., if the speed of flow at the inlet is small. The velocity of flow of water passing through the screen was approximately 0.5 to 0.6 m/s in most power station intakes constructed in the past. However, vortices are likely to occur in power stations constructed recently as the speed of flow is approximately 2 times larger, i.e., approximately 1 m/s.

Air is carried into the tunnel when vortices are generated in the intake. The problem caused by the air carried together with the vortices is negligible when the volume of air is small. However, air bubbles accumulate at the upper part of the tunnel at places where the tunnel has a variable gradient and when the tunnel is relatively long. That accumulation of air at the upper part of the tunnel reduces its cross section and obstructs the flow of water. The generation of vortices shall be reduced as much as possible as this contributes to generation of vibration in the turbine and air hammer.

Many methods to prevent vortices have been devised owing to hydraulic experiments carried out in the past. The principal methods for prevention of vortices are as follows.

| | | |
|---|---|---|
| a) | Prevention of vortices by means of floats | Fig. 2-24. |
| b) | Prevention of vortices by menas of inclined plate | Fig. 2-25. |
| c) | Prevention of vortices by means of beams | Fig. 2-26. |

There are two variations in method a), i.e, in the first variation, the water surface where vortices take place is covered with floats for the purpose of preventing suction of air; in the second viriation, several hollow pipes are interconnected with links and are floated for the purpose of preventing rotation of the water surface.

In method b), an inclined plate is installed above the intake in the form of an overhang and the angle of inclination does not exceed 60°.

The mechanism of generation of vortices commences with small whirls on the water surface of the intake which is initially at a standstill. The revolution speed of these small whirls increases gradually, aided by the flow in the vertical direction. They then grow in the form of large vortices containing an air cone at the center.

The installation of an inclined plate above the intake is the best method to prevent vortices, because it not only contributes directly to prevention of surface whirls, but also eliminates flow lines in the vertical direction required to generate the vortices.

Method c) consists of installing several concrete beams with rectangular cross sections above the inlet of the intake inlet. These beams contribute to prevent the occurrence of vortices by slowing down the velocity of flow in the vertical direction in areas where the vortices are likely to occur.

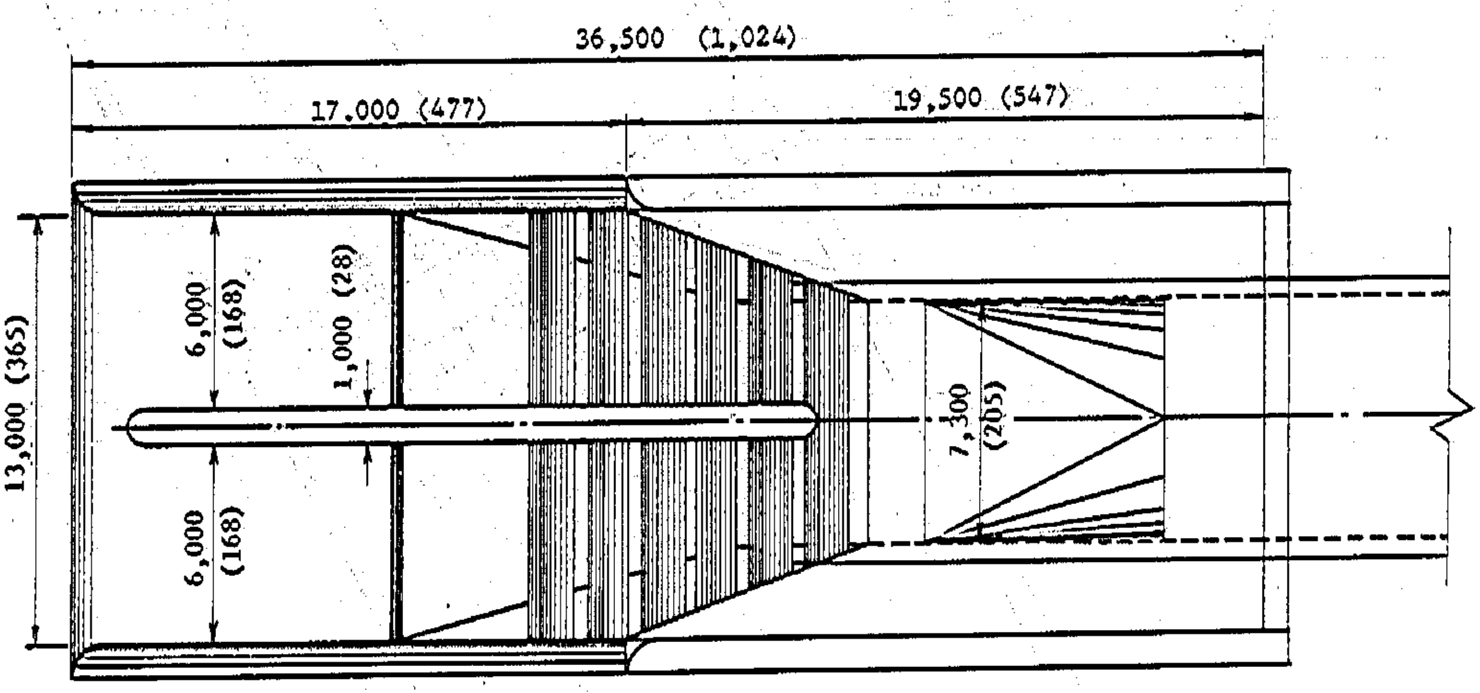
### (6) Conclusion

Measures to prevent the occurrence of vortices have been taken in the intakes of the two dams as a result of experiments with hydraulic models.

In the upper dam intake, the occurrence of vortices is prevented by installing beams of 1 m X 1 m cross section with 1 m pitch above the inlet up to the center pier. It was confirmed that these beams contribute to reduce the retardation area in the intake and make possible obtainment of the prescribed rate of discharge in a safe and reliable way.

In the lower dam intake, the occurence of vortices is prevented by installing the guide wall. And inclined upper plate was confirmed to be useful to keep smooth flow without a vortices. (Refer to Fig. 2.3.4)

Fig. 2-20 Plan of Intake Model (Upper Tekai Dam)




* Figures in ( ) show the model size.

Model =1/35.66

Fig. 2-21 Section of Intake Model (Upper Tekai Dam)

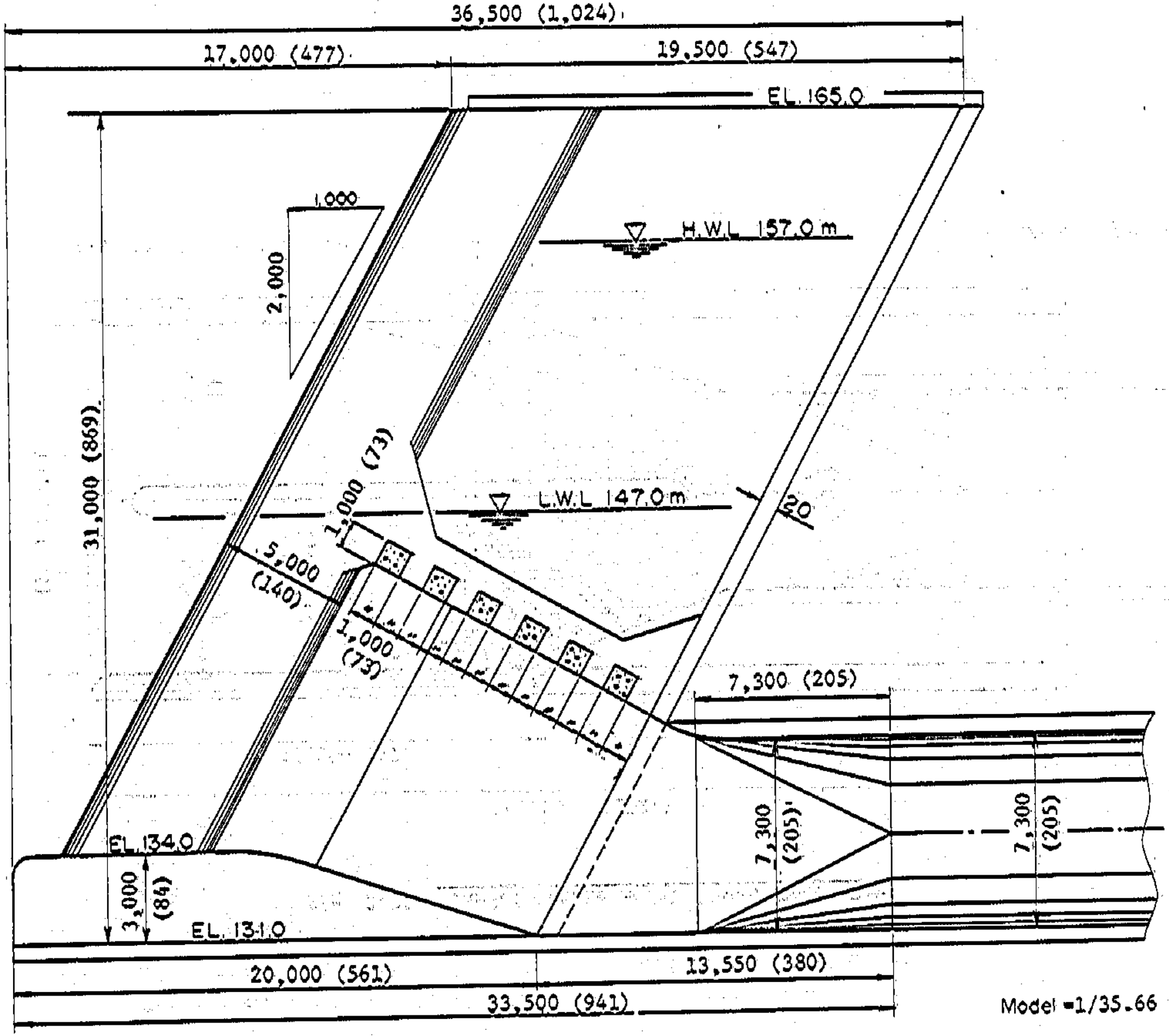
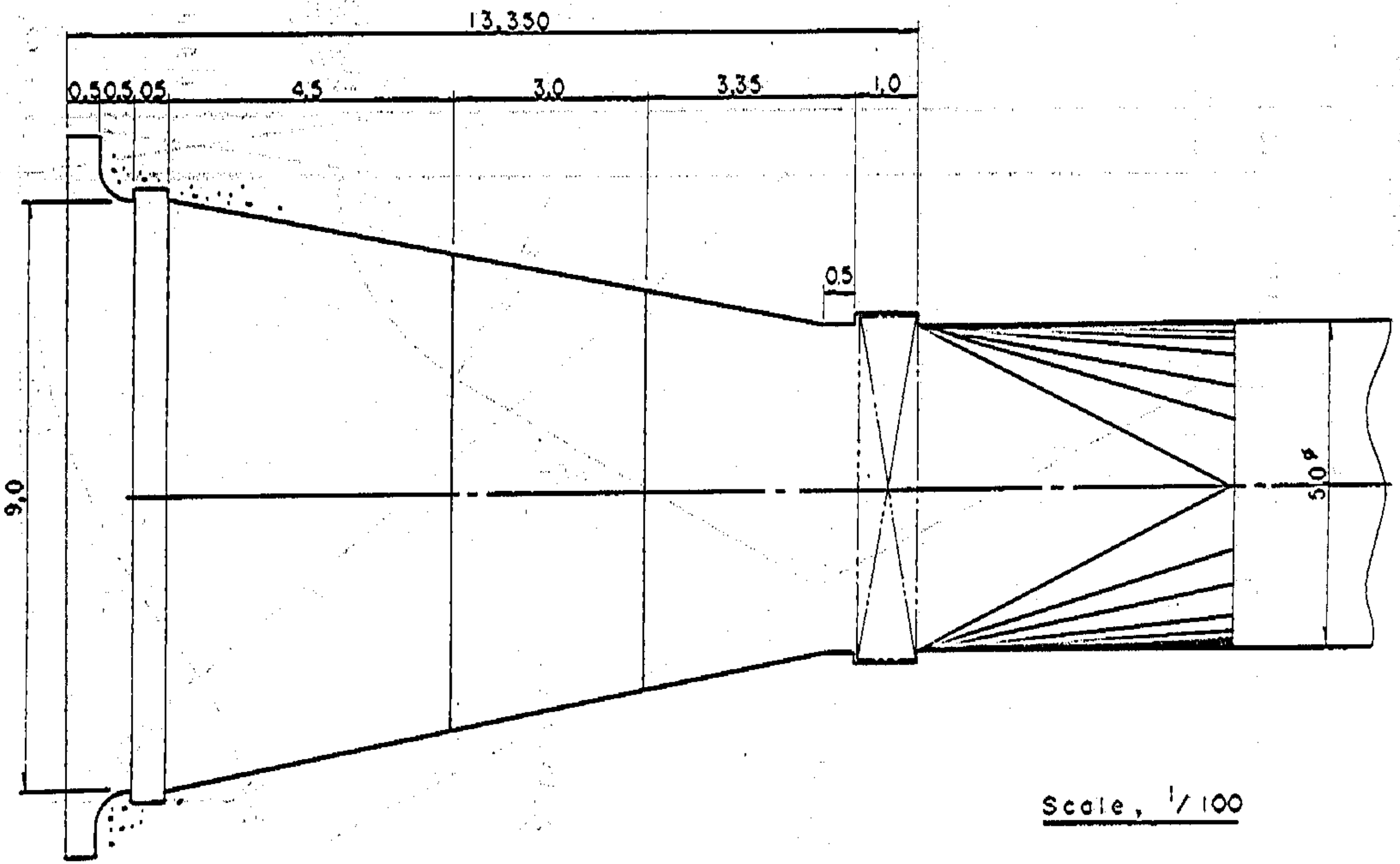


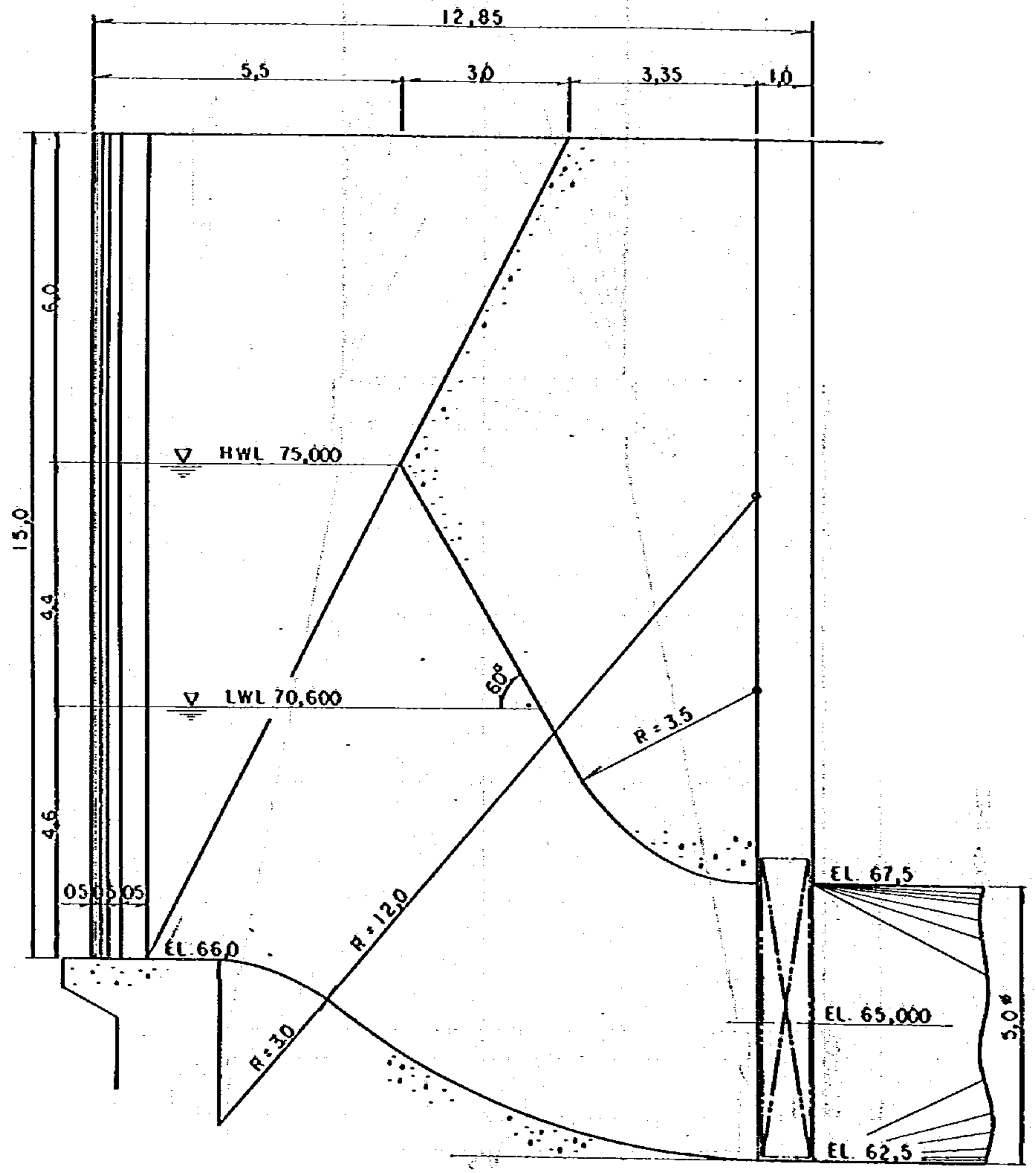
Fig. 2-22 Plan of Intake Model (Lower Tekai Dam)




Scale, 1/100

Fig. 2-23 Section of Intake Model (Lower Tekal Dam)

(model. S =1/30)




Scale, 1/100

Fig. 2-24 Prevention of Vortices by Floats

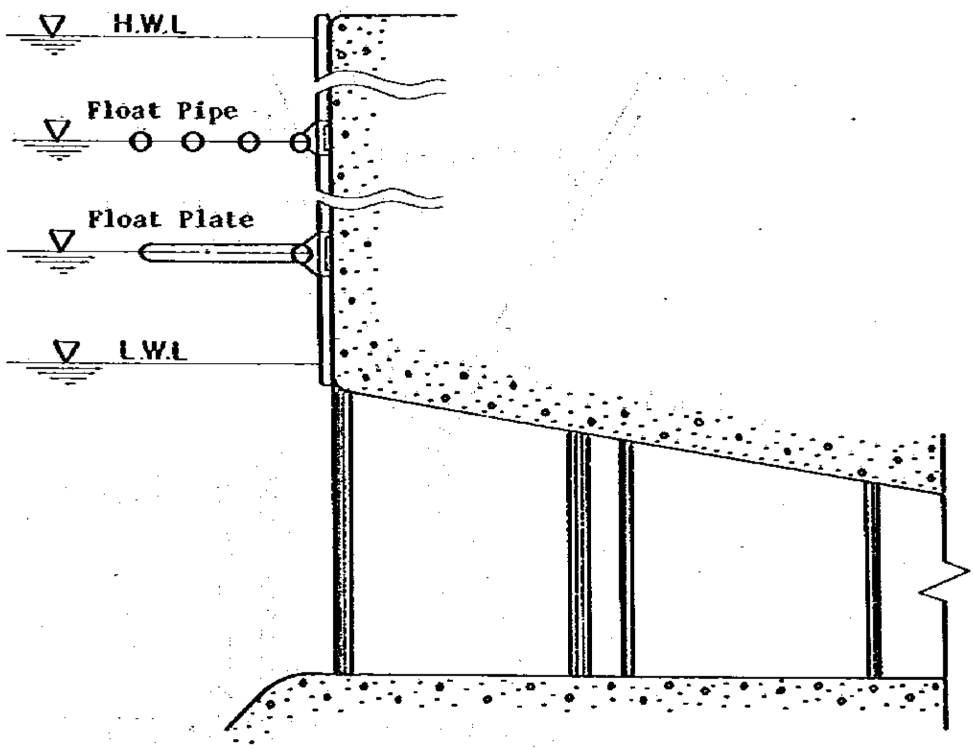


Fig. 2-25 Prevention of Vortices by Inclined Plate

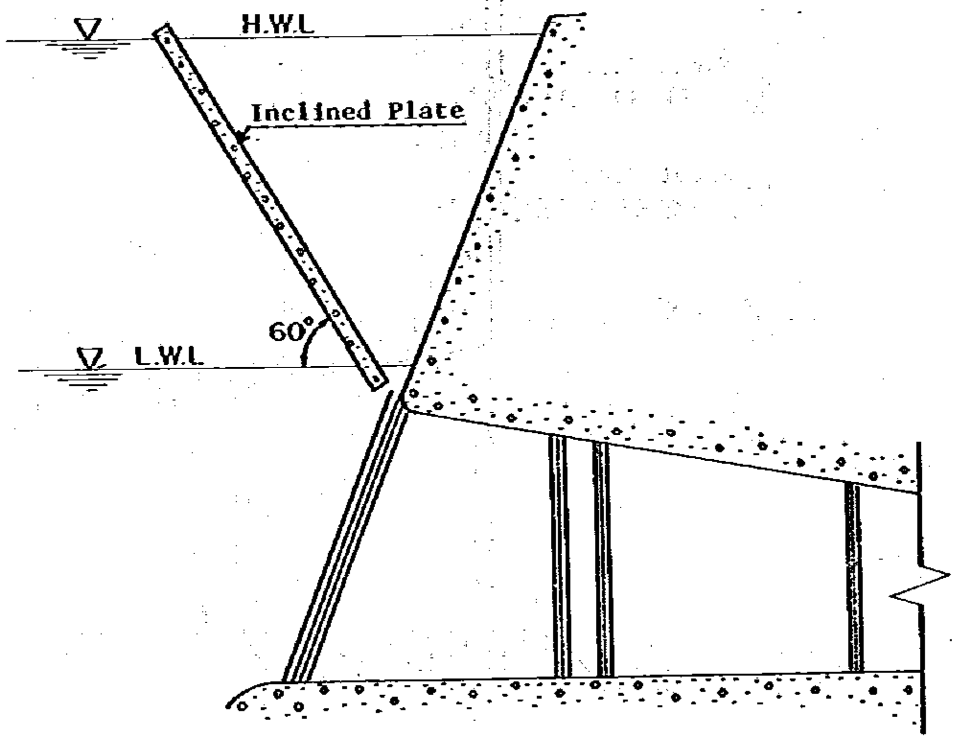
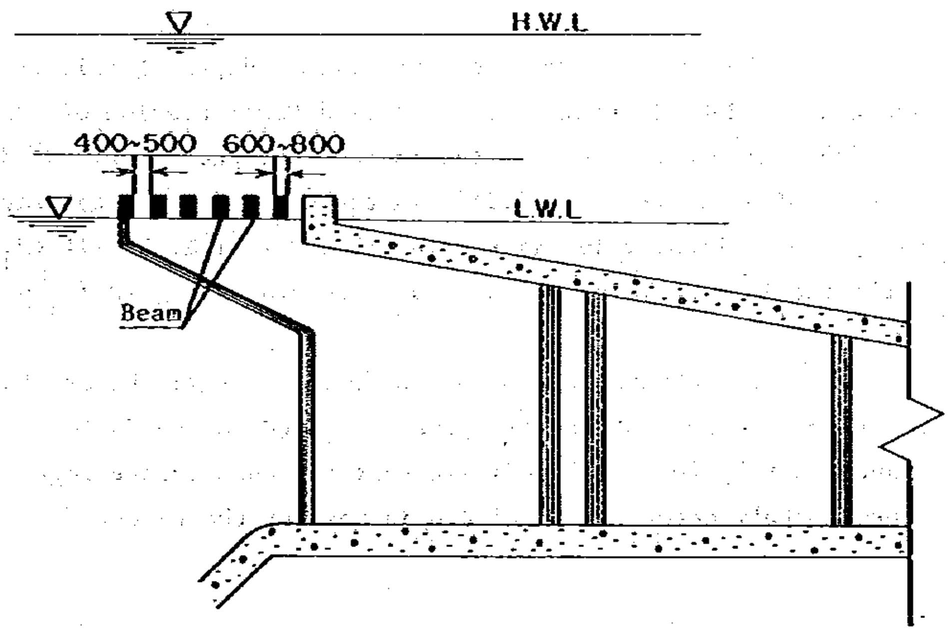


Fig. 2-26 Prevention of Vortices by Beams



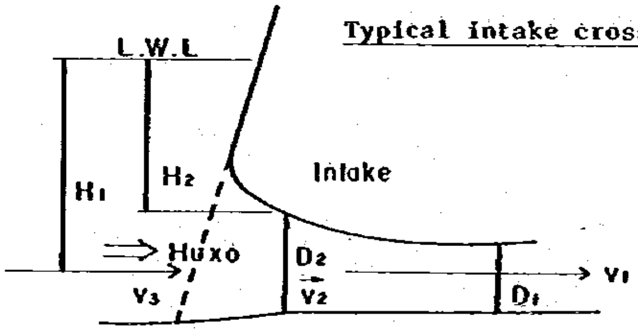
### 2.7.2 Water Depth of Existing Intake

Table 2-13 ∿ 2-18 indicate the values of intake submergence depth in existing fill type and concrete gravity type hydroelectric power stations, with exception of pumped storage power stations. The values contained in the tables are illustrated for the intake cross sections.

The smallest value of submergence depth of fill type dams among these data is that of the Esterito Power Station of Brazil ($H_2/D_2$ = 0.38), while the largest value is that for the Tedorigawa No.1 Power Station of Japan ($H_2/D_2$ = 2.3). In the case of concrete gravity type dams, the smallest value is that of Ilha Solteira, Brazil ($H_2/D_2$ = 0.66), while the largest value is that for Tagokura, Japan ($H_2/D_2$ = 2.3).

The submergence depth can be more clearly be examining further examples of dams, but the examples presented in the following tables (approximately 20 cases) indicate that the submergence depth $H_2/D_2$ is approximately between 0.5 and 2.0 in most of the cases.

Typical intake cross section




$V_1, V_2, V_3$ : Average velocity of flow

Table 2-13 Examples of Intake Design

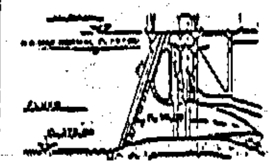
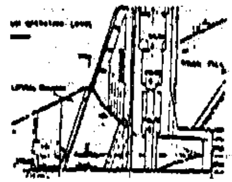
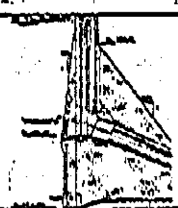
| Power station | Type of dam | Dam height (m) | Available depth (m) | Turbine discharge ($m^3/s$) | Headrace channel | | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.) | | Submergence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Dia. $D_1$ (m) | Velocity of flow (m/s) | | | $H_1$ (m) | $H_2$ (m) | | | |
| Salto Osorio (Brazil) | Fill | 56 | 9.0 | 193 | 7.4 | 4.5 | 7.5 | 389 | 7 | 4.2 | 0.56 |  | 0.8 |
| Capivara (Brazil) | Fill | 60 | 5.0 | 375 | 11.0 | 4.7 | 16.0 | 321 | 39 | 29.0 | 1.80 |  | 1.0 |
| Icumbiara (Brazil) | Fill | 92 | 6.2 | 500 | 10.0 | 6.4 | 6.2 | 495 | 27 | 3.1 | 0.50 |  | 1.6 |
| Salto Santiago (Brazil) | Fill | 80 | 25.0 | 350 | 7.6 | 7.7 | 11.0 | 481 | 10 | 5.0 | 0.45 |  | 0.92 |

Table 2-14

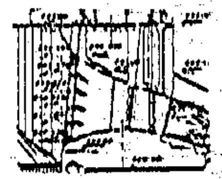
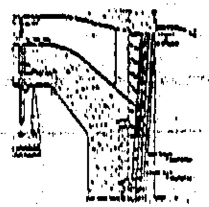
| Power station | Type of dam | Dam height (m) | Available depth (m) | Turbine discharge ($m^3/s$) | Headrace channel: Dia. $D_1$ (m) | Headrace channel: Velocity of flow (m/s) | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.): $H_1$ (m) | Depth from LWL (Ref. to Fig.): $H_2$ (m) | Submergence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Paulo Afonso (Brazil) | Fill | 34 | 6.0 | 385 | 8.5 | 6.6 | 9.7 | 247 | 15 | 7.2 | 0.74 |  | 1.2 |
| Sao Simao (Brazil) | Fill | 72 | 20.0 | 420 | 9.5 | 6.1 | 11.0 | 381 | 17 | 10.0 | 0.90 |  | 1.1 |
| Esteito (Brazil) | Fill | 80 | 7.7 | 335 | 8.2 | 6.3 | 10.8 | 619 | 12 | 4.2 | 0.38 |   | 0.9 |
| Ilha Solteria (Brazil) | Concrete Cravity | 24 | 9.0 | 390 | 8.5 | 6.7 | 9.6 | 300 | 20 | 14.4 | 0.66 |  | 1.3 |

Table 2-15

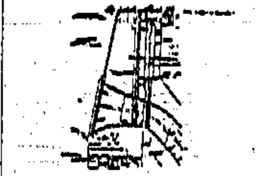
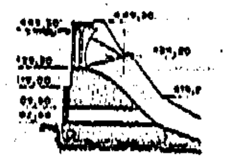
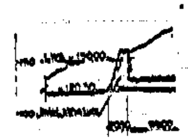
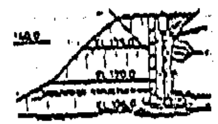
| Power station | Type of dam | Dam height (m) | Available depth (m) | Turbine discharge ($m^3/s$) | Headrace channel Dia. $D_1$ (m) | Headrace channel Velocity of flow (m/s) | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.) $H_1$ (m) | Depth from LWL (Ref. to Fig.) $H_2$ (m) | Submergence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Agua Vermelha (Brazil) | Gravity | 67 | 11.5 | 480 | 10.0 | 6.1 | 12.0 | 373 | 27 | 17.0 | 1.4 |  | 1.5 |
| Marimbond (Brazil) | Gravity | 45 | 21.0 | 360 | 8.5 | 5.2 | 9.0 | 426 | 13 | 8.0 | 0.90 |  | 1.6 |
| Hasan Ugurlu (Turkey) | Fill | 175 | 40.0 | 520 | 8.4 | 9.3 | 9.0 | 150 | 25 | 20.5 | 2.20 |  | 0.74 |
| Trengganu Kenyir Dam (Malaysia) | Fill | 150 | 17.0 | 410 | 3.75 (x4) | 2.3 | 4.2 | 115 | 8 | 5.8 | 1.4 |  | 1.3 |

Table 2-16

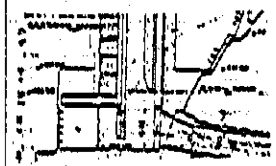
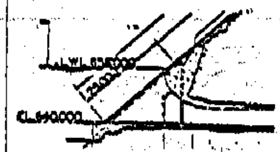
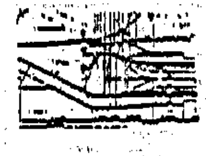
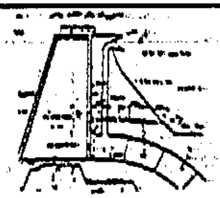
| Power station | Type of dam | Dam height (m) | Available depth (m) | Turbine discharge ($m^3/s$) | Headrace channel: Dia. $D_1$ (m) | Headrace channel: Velocity of flow (m/s) | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.): $H_1$ (m) | Depth from LWL (Ref. to Fig.): $H_2$ (m) | Submergence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tedorigawa No.1 (Japan) | Fill | 153 | 60.0 | 180 | 7.0 | 4.7 | 10.0 | 422 | 30 | 23.0 | 2.3 |  | 1.1 |
| Shimo-Kotori (Japan) | Fill | 119 | 47.0 | 65 | 4.8 | 3.6 | 6.2 | 655 | 13 | 8.8 | 1.4 |  | 0.4 |
| Horoka (Japan) | Fill | 33 | 4.0 | | 2.7 | | 3.4 | 605 | 5 | 2.6 | 0.76 |  | |
| Shioya (Japan) | Gravity | 17 | 5.0 | 18 | 2.7 (x4) | 3.1 | 3.0 | 336 | 5 | 3.0 | 1.0 |  | 0.9 |

Table 2-17

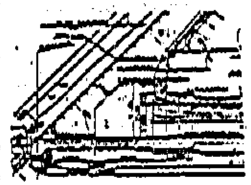
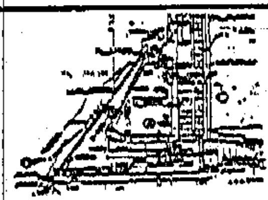
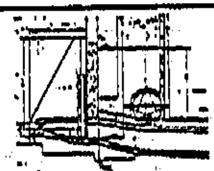
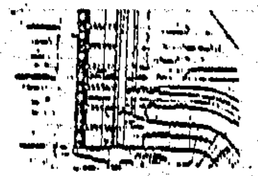
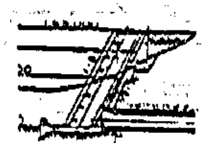
| Power station | Type of dam | Dam height (m) | Avail-able depth (m) | Turbine dis-charge (m³/s) | Headrace channel Dia. $D_1$ (m) | Headrace channel Velocity of flow (m/s) | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.) $H_1$ (m) | Depth from LWL (Ref. to Fig.) $H_2$ (m) | Submer-gence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tagokura (Japan) | Gravity | 140 | 52.0 | | 5.0 | | 6.0 | 458 | 20 | 14.0 | 2.3 |  | |
| Tozugawa No.1 (Japan) | Gravity | 101 | 30.0 | 60 | 5.2 | 2.8 | 5.6 | 265 | 15 | 11.0 | 1.9 |  | 0.6 |
| Sakuma (Japan) | Gravity | 155 | 40.0 | 306 | 7.0 | 7.9 | 10.0 | 220 | 22 | 18.1 | 1.8 |  | 1.0 |
| Sendaigawa No.1 (Japan) | Gravity | 57 | 30.0 | | 7.0 | | 8.6 | 130 | 6 | 8.8 | 0.8 |  | |

Table 2-18

| Power station | Type of dam | Dam height (m) | Available depth (m) | Turbine discharge ($m^3/s$) | Headrace channel: Dia. $D_1$ (m) | Headrace channel: Velocity of flow (m/s) | Intake dia. $D_2$ (m) | L.W.L. (m) | Depth from LWL (Ref. to Fig.): $H_1$ (m) | Depth from LWL (Ref. to Fig.): $H_2$ (m) | Submergence depth $H_2/D_2$ | Intake cross section | Velocity of flow at the vicinity of the screen $V_3$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sudagai (Japan) | Gravity | 59 | 25.0 | 65 | 3.8 | 2.3 | 5.5 | 718 | 9 | 4.5 | 0.8 |  | 0.82 |
| | | | | | | | | | | | | | |
| Upper Tekai (Malaysia) | Fill | 100 | 10.0 | 231 | 4.7<br>6.0 | 5.1 | 7.0 | 147 | 17 | 13.0 | 1.9 |  | 0.5 |
| Lower Tekai (Malaysia) | Gravity | 36 | 4.5 | 80 | 3.6 (x2) | 3.9 | 3.8 | 70.5 | 5.3 | 3.0 | 0.8 |  | 1.0 |

## 2.8 Pressure Tunnel

### 2.8.1 Type and Layout

Three alternatives referring to the configuration and layout of the pressure tunnel were studied in the economical points of view.

- Alternative 1

  Three turbine & generators with two pressure tunnel (two tunnel after bifurcation of one tunnel)

- Alternative 2

  Two turbine & generator sets with pressure tunnel of one and two after bifurcation.

- Alternative 3

  Two turbine & generator sets with headrace tunnel and penstock with surge tank

The specifications of each alternative are as follows.

(1) Two pressure tunnel plan

a) Unit 1, 2

Inner diameter 6.0 m x Length 479.887 m

After bifurcation — Inner diameter 3.8 m x Length 40.379 m (Generator No.1)

Inner diameter 3.8 m x Length 46.155 m (Generator No.2)

b) Unit 3

Inner diameter 3.8 m x Length 589.711 m

After gradual contraction : Inner diameter 3.8 m x Length 34.000 m (Generator No.3)

(2) Single pressure tunnel plan (Inner steel lined waterway)

- Before bifurcation: Inner diameter 7.3 m x Length 534.506 m
- After bifurcation : Inner diameter 4.6 m x Length 43.852 m

(3) Single pressure tunnel plan (Headrace tunnel and penstock with surge tank)

- Headrace tunnel : Inner diameter 7.3 m x Length 370 m
- Penstock
  - Before bifurcation : Inner diameter 7.3 m x Length 152.592 m
  - After bifurcation : Inner diameter 6.0∿4.6 m x Length 95.207 m
- Surge tank
  - Port diameter 19.5 m x Height 43.85 m
  - Orifice diameter 4.7 m

The construction costs of the 3 alternatives are compared in the following table. As can be seen from the table, the single pressure tunnel plan (2) (Inner steel lined tunnel) is the most preferable one.

Table 2-19 Comparison of Pressure Tunnel

(Unit : M$)

| Item | | Pressure Tunnel 2 Generator,3 D=6.0m ℓ=524m D=4.7m ℓ=637m | Pressure Tunnel 1 Generator,2 D=7.3m ℓ=578m | Headrace Tunnel & Penstock with Surge Tank D=7.3m ℓ=593m |
|---|---|---|---|---|
| Pressure Tunnel | Tunnel Excavation (Horizontal) | 3,425,400 | 2,819,900 | 1,156,500 |
| | " (Inclined) | 704,000 | 737,000 | 555,500 |
| | Plug Concrete (Horizontal) | 3,841,000 | 2,622,000 | 1,122,400 |
| | " (Inclined) | 621,000 | 667,000 | 395,600 |
| | Reinforcement Bar | 374,600 | 291,100 | 226,000 |
| | Sub-Total | 8,966,000 | 7,137,000 | 3,456,000 |
| Headrace | Tunnel Excavation | - | - | 2,326,850 |
| | Lining Concrete | - | - | 2,415,000 |
| | Reinforcement Bar | - | - | 1,162,150 |
| | Sub-Total | - | - | 5,904,000 |
| Surge Tank | Shaft Excavation | - | - | 1,721,500 |
| | Concrete Open | - | - | 453,150 |
| | Concrete Shaft | - | - | 1,538,700 |
| | Reinforcement Bar | - | - | 1,383,850 |
| | Open Excavation Common | - | - | 28,800 |
| | Open Excavation Rock | - | - | 759,000 |
| | Sub-Total | - | - | 5,885,000 |
| Grouting | Drilling | 470,900 | 247,000 | 623,480 |
| | Grouting | 733,100 | 384,750 | 972,000 |
| | Contact Grouting | 525,000 | 367,500 | 414,750 |
| | Miscellaneous | 14,000 | 7,750 | 16,770 |
| | Sub-Total | 1,743,000 | 1,007,000 | 2,027,000 |
| Pipe | Steel Pipe | | | |
| | Sub-Total | 17,550,000 | 16,200,000 | 7,236,000 |
| Access Road | | - | - | 100,000 |
| Access Tunnel (L=150m) | | 1,310,000 | 1,310,000 | 1,310,000 |
| Total | | 29,569,000 | 25,654,000 | 25,918,000 |

### 2.8.2 Economical Diameter of Pressure Tunnel

(1) Outline

The inner diameter of the pressure tunnel is determined from the economical point of view by comparing the construction cost and the benefit caused by the head loss.

(2) Results

The optimum value of the inner diameter of the pressure tunnel is considered 7.3 m, from Fig. 2-27. The increase/decrease of benefits and costs corresponding to each pressure tunnel diameter are listed in Table 2-20.

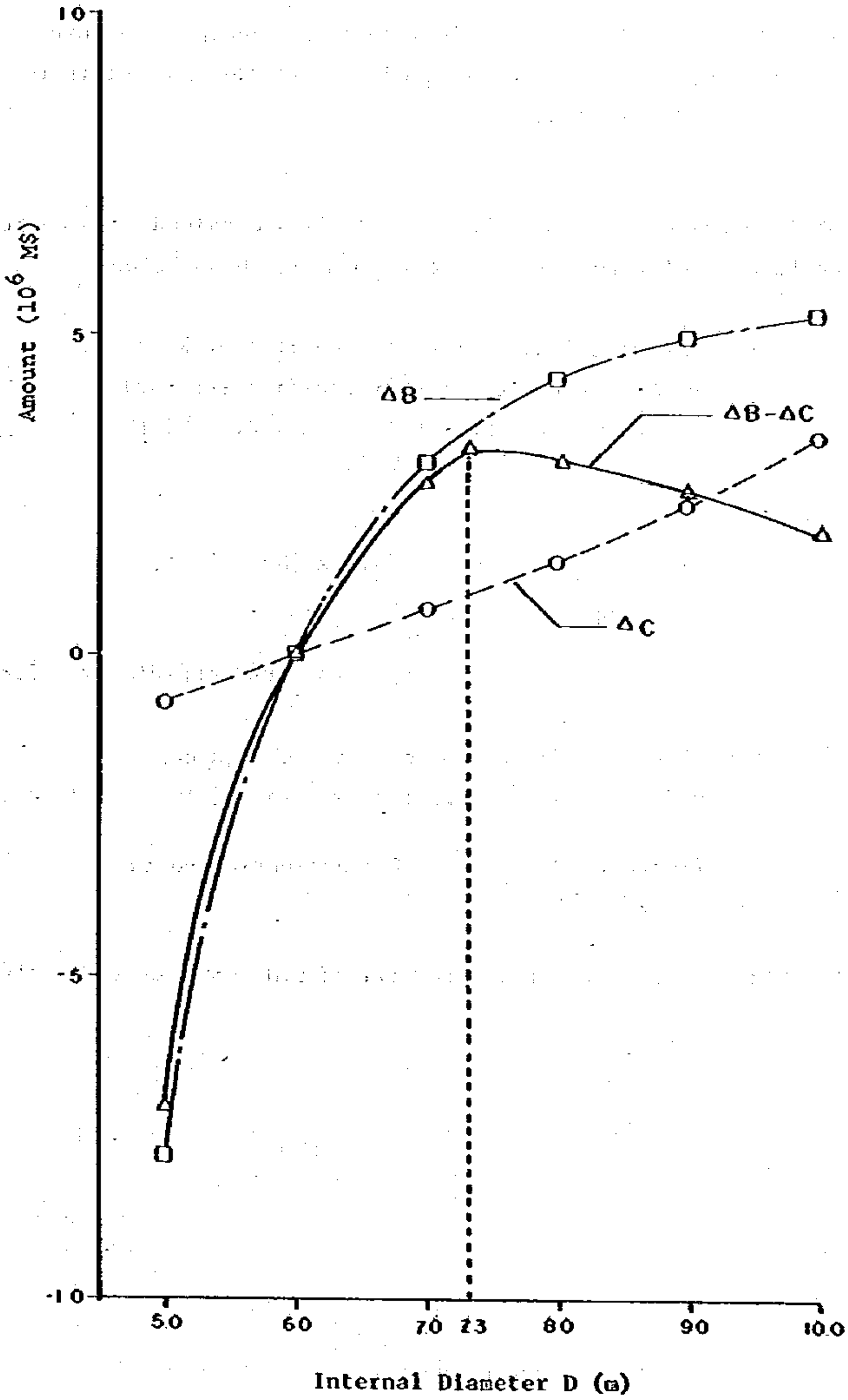
Table 2-20 Increase/Decrease of Benefits and Costs

(Unit : $10^6$ M$)

| Item | | Inner Diameter D (m) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| Increase/decrease of benefit | ΔB | -7.83 | 0 | 3.00 | 4.34 | 5.01 | 5.36 |
| Increase/decrease of cost | ΔC | -0.77 | 0 | 0.71 | 1.48 | 2.37 | 3.40 |
| ΔB - ΔC | | -7.06 | 0 | 2.99 | 2.86 | 2.64 | 1.96 |

(The standard value of inner diameter is 6.0 m)

Fig. 2-27 Relationship between Internal Diameter of Pipe and ΔB, ΔC, and ΔB - ΔC



(3) Methods of comparison

When the increase and decrease in benefits and annual costs by changing the internal diameter of an pressure tunnel of a certain reference internal diameter are defined as $\Delta B$ and $\Delta C$, the internal diameter which makes the value of $\Delta B - \Delta C$ the largest is the most economical internal diameter.

$\Delta B$ is obtained from head losses and $\Delta C$ is obtained from construction costs according to the respective formula shown below.

$$\Delta B = (\Delta KWh) \times (\text{Unit price of benefits per KWh}) + (\Delta KW) \times (\text{Unit price of benefits per KWh})$$
$$= \Delta KWh \times 0.190 \text{ H\$/KWh} + KW \times 142.7 \text{ H\$/KW}$$

where;

$$\Delta KWh = 9.8 \times \eta_G \times \Sigma Q \times 3.6 \times [\text{Head loss}]$$
$$\Delta KW = 9.8 \times \eta_G \times Q_{max} \times [\quad " \quad]$$

$\eta_G$ : Combined efficiency = 0.87

$$\Delta C = [\text{Increment/decrement of construction cost}] \times (1 + \text{interest during construction}) \times (\text{Annual cost rate})$$

$$= [\text{Increment/decrement of construction cost}] \times 1.2 \times 0.11586$$

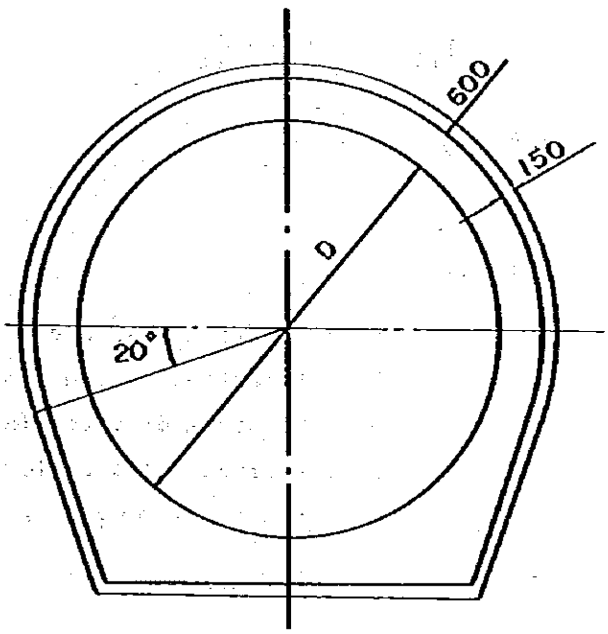
In this study, an internal diameter of 6.0 m was used as reference.

(4) Specifications of the pressure tunnel

- The length of the pressure tunnel is 578.358 m and bifurcates at 534.506 m from its inlet.
- The standard cross section of the pressure tunnel for determining the optimum inner diameter is shown in Fig. 2-28. The calculations are carried out for 6 cases, i.e., 5 m, 6 m, 7 m, 8 m, 9 m and 10 m of inner diameter.
- The design head is 85.2 m hydrostatic pressure and the water-hammer pressure is assumed to be 30% of the hydrostatic pressure (26.56 m).
- Other details are specified as the occasion demands.

Fig. 2-28 Cross Section Studied

(D = 4, 5, 6, 7, 8, 9 and 10 m)



(5) Calculation of benefits

The head loss is calculated first so that the benefit is calculated by means of the expression described in section (3).

1) Head loss (Turbine discharge Q = 235.0 $m^3/s$

The head loss is calculated by means of the understated expression

$$h_r = h_{r1} + h_{r2} + h_{r3} + h_{r4}$$

where;

$h_r$ : Total head loss (m)
$h_{r1}$ : Head loss due to friction (m)
$h_{r2}$ : Head loss due to bifurcation (m)
$h_{r3}$ : Head loss due to bend (m)
$h_{r4}$ : Other head losses

a) Head loss due to friction ($h_{r1}$)

$$h_{r1} = f \cdot \frac{L}{D} \cdot \frac{V^2}{2g}$$

$$f = \frac{124.5\ n^2}{D^{1/3}}$$

where;

f : Friction loss coefficient
L : Extension of the penstock = 578.358 m
D : Penstock diameter (m)
V : Average speed of low in the penstock (m/s)
n : Manning's roughness coefficient = 0.012

b) Head loss due to bifurcation ($h_{r2}$)

$$h_{r2} = f_B \cdot \frac{V_o^2}{2g}$$

where;

$f_B$ : Bifurcation loss coefficient = 0.5

$V_o$ : Average speed of flow in the penstock before bifurcation (m/s)

c) Head loss due to bend ($h_{r3}$)

$$h_{r3} = f_{b1} \cdot f_{b2} \cdot \frac{V^2}{2g}$$

$$f_{b1} = 0.131 + 0.1632 \left( \frac{D}{\rho} \right)^{7/2}$$

$$f_{b2} = \left( \frac{\theta°}{90°} \right)^{1/2}$$

where;

$f_{b1}$ : Coefficient of loss determined by the retio between the radius of curvature $\rho$ of the bend and the penstock diameter D. The center angle of the bend shall be 90°.

$f_{b2}$ : Ratio between the loss when the bend center angle $\theta$ has an arbitrary value and the loss when the center angle is 90°.

V : Average velocity of flow in the penstock (m/s).

d) Calculation results

The values calculated by using the aforestated expressions are listed in Table 2-21. The relation between the inner diameter of the penstock and the h-ad loss is shown in Fig. 2-29.

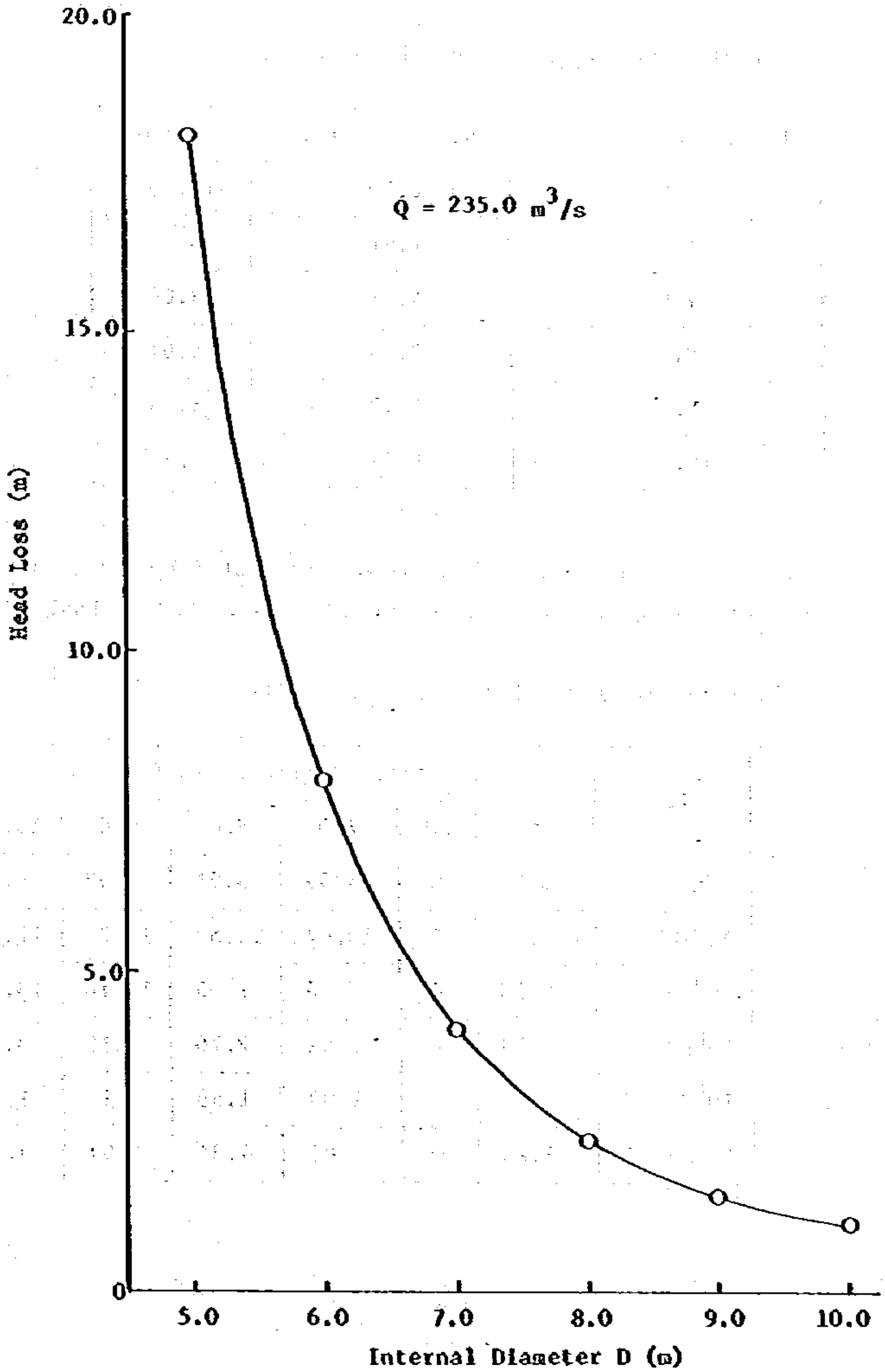
Table 2-21 Head Loss

(Unit : mm)

| | Inner Diameter D (m) | | | | | |
|---|---|---|---|---|---|---|
| | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| Head loss due to friction ($h_{r1}$) | 8.564 | 3.239 | 1.423 | 0.699 | 0.372 | 0.212 |
| Head loss due to bifurcation ($h_{r2}$) | 3.531 | 1.703 | 0.919 | 0.539 | 0.336 | 0.221 |
| Head loss due to bend ($h_{r3}$) | 3.787 | 1.952 | 1.165 | 0.782 | 0.579 | 0.462 |
| Otherr losses of head ($h_{r4}$) | 2.218 | 1.076 | 0.573 | 0.340 | 0.203 | 0.135 |
| Totali | 18.100 | 7.970 | 4.080 | 2.360 | 1.490 | 1.030 |

Turbine discharge $Q = 235.0\ m^3/s$

Fig. 2-29 Relationship between Internal Diameter of Pressure Tunnel and Head Losses



ii) Benefits

The variation of the loss of head ΔH calculated by assuming a reference inner diameter of 6.0 m is presented in Table 2-22.

Table 2-22 Variation of the Head Loss

| Inner Diameter (m) | Head Loss (m) | ΔH (m) |
|---|---|---|
| 5.0 | 18.10 | -10.13 |
| 6.0 | 7.97 | 0 |
| 7.0 | 4.08 | 3.89 |
| 8.0 | 2.36 | 5.61 |
| 9.0 | 1.49 | 6.48 |
| 10.0 | 1.03 | 6.94 |

The increase/decrease of benefit ΔB calculated by means of the expression presented in section (3) is listed in Table 2-23.

Table 2-23 Increase/Decrease in Benefits

| | Unit | Inner Diameter D (m) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| ΔH | m | -10.13 | 0 | 3.89 | 5.61 | 6.48 | 6.94 |
| ΔKWh | $10^6$KWh | -26.22 | 0 | 10.07 | 14.52 | 16.77 | 17.96 |
| ΔKW | $10^3$KW | -19.95 | 0 | 7.66 | 11.05 | 12.76 | 13.67 |
| ΔKWh x 0.190 | $10^6$M$ | -4.98 | 0 | 1.91 | 2.76 | 3.19 | 3.41 |
| ΔKW x 190.20 | $10^6$M$ | -2.85 | 0 | 1.09 | 1.58 | 1.82 | 1.95 |
| ΔB | $10^6$M$ | -7.83 | 0 | 3.00 | 4.34 | 5.01 | 5.36 |

(6) Calculation of annual costs

The quantity of steel pipe and the volume of civil works for each different internal diameter were estimated and the construction costs were calculated. Thus, the increase and decrease in annual costs were obtained.

1) Weight of steel pipe

a) Study on pipe thickness

∘ Design internal pressure

The design internal pressure shall be the sum of the hydrostatic pressure and the water-hammer pressure. Here, the water-hammer pressure is assumed to be 30% of the hydrostatic pressure.

. Hydrostatic pressure : $Ho = 157.000 - 71.800 = 85.200$ m

. Water-hammer pressure : $Zo = 85.200 \times 0.3 = 25.560$ m

Maximum design internal pressure : $Ho + Zo = 110.760$ m

The design internal pressure for each point can be obtained by the following formula.

$$H = H_x + Z_x$$

$H$ : Design internal pressure for each point

$H_x$ : Hydrostatic pressure at each point

$Z_x$ : Water-hammer pressure at each point

Here, $Z_x$ is assumed to be largest at the center of valve and zero at the starting point of pressure tunnel, and to change linearly in proportion to the tunnel length in-between.

- Pipe thickness

As the material for penstock, steel material for welded construction (SM41) is to be used.

The following formula is used in calculating the pipe thickness.

$$to = \frac{H(Do + \varepsilon)}{2\sigma_a} + \varepsilon$$

where;

to : Pipe thickness (cm)

H : Design internal pressure (kg/cm$^2$)

Do : Internal diameter of steel pipe (cm)

$\varepsilon$ : Extra thickness against corrosion and abrasion = 0.15 cm

$\sigma_a$ : Allowable tensile stress = 1750 kg/cm$^2$

However, this is provided that the value shall be not less than the minimum pipe thickness indicated by the following formula.

$$t = \frac{Do + 800}{400} \quad (mm)$$

Calculations are made for the cases of pipe diameter (Do) being 4 m, 5 m, 6 m, 7 m, 8 m, 9 m and 10 m.

∘ Weight of steel pipe

The following formula is used in calculation.

$$W' = \{(Do + 2t)^2 - Do^2\} \cdot \gamma s \frac{\pi}{4}$$

$$W = W' \cdot L \cdot \alpha$$

where;

$W'$ : Weight of iron pipe per 1 m (t/m)

$W$ : Total weight (t)

$Do$ : Internal diameter (m)

$t$ : Pipe thickness (m)

$\gamma_s$ : Unit weight by volume of steel material 7.85 $t/m^3$

$L$ : Extended length of pipeline

$\alpha$ : Premium rate of iron pipe volume due to accessories = 1.14

c) Calculation results

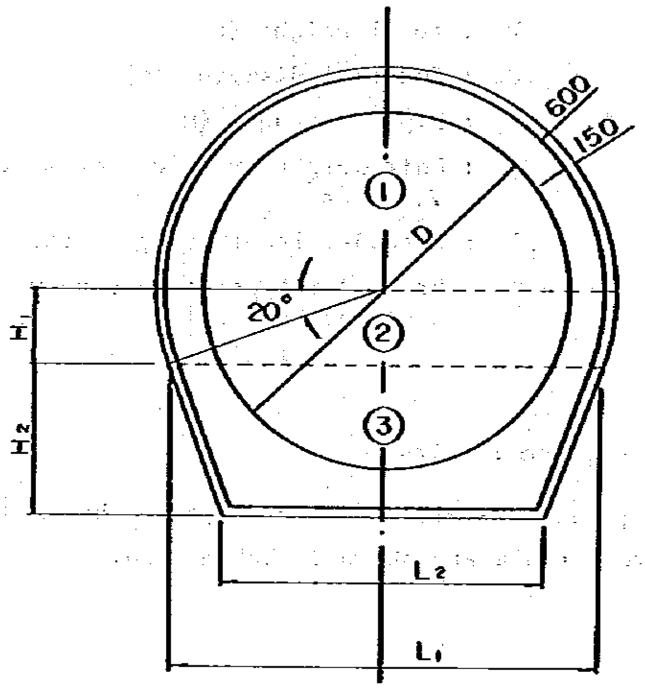
The pipe thickness and steel pipe weight obtained by the above formula are shown in Table 2-24.

Table 2-24 Steel Pipe Weight

| Inner Diameter (m) | Average Wall Thickness (mm) | Weight of Steel Pipe per Meter (t/m) | Total Weight of Steel Pipe (t) |
|---|---|---|---|
| 5.0 | 17.698 | 2.189 | 1553 |
| 6.0 | 21.356 | 3.171 | 2249 |
| 7.0 | 24.209 | 4.194 | 2975 |
| 8.0 | 26.781 | 5.302 | 3761 |
| 9.0 | 29.637 | 6.600 | 4682 |
| 10.0 | 32.968 | 8.157 | 5786 |

ii) Volume of civil works for construction of penstock

a) The volume of civil works required to construct the penstock with cross section shown in the figure below is calculated for inner diameters Do of 4 m, 5 m, 6 m, 7 m, 8 m, 9 m and 10 m.




① Upper semi-circle

② Intermediate cross section

③ Base of penstock

Table 2-25 Comparison of Civil Works per Unit Length of Penstock

| | Unit | Inner Diameter D (m) | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 4.6 | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| Excavation | $m^3$ | 30.887 | 35.070 | 46.619 | 59.973 | 74.914 | 91.515 | 109.777 |
| Concrete | $m^3$ | 14.268 | 15.436 | 18.417 | 21.488 | 24.648 | 27.898 | 31.237 |

Table 2-26 Volume of Pressure Tunnel Construction Works per 1 Meter

| Inner Diameter D (m) | $H_1$ (m) | $H_2$ (m) | $L_1$ (m) | $L_2$ (m) | (1) ($m^2$) | (2) ($m^2$) | (3) ($m^2$) | Excavation Volume (A) (1)+(2)+(3) ($m^3$) | $A' = \frac{\pi}{4}D^2$ ($m^2$) | Volume of Concrete A-A' ($m^3$) |
|---|---|---|---|---|---|---|---|---|---|---|
| 4.6 | 1.043 | 2.007 | 5.732 | 4.271 | 14.612 | 6.237 | 10.038 | 30.887 | 16.619 | 14.268 |
| 5.0 | 1.112 | 2.138 | 6.108 | 4.551 | 16.592 | 7.082 | 11.397 | 35.070 | 19.635 | 15.436 |
| 6.0 | 1.283 | 2.467 | 7.048 | 5.252 | 22.089 | 9.428 | 15.174 | 46.691 | 28.274 | 18.417 |
| 7.0 | 1.454 | 2.796 | 7.987 | 5.952 | 28.373 | 12.110 | 19.490 | 59.973 | 38.485 | 21.488 |
| 8.0 | 1.625 | 3.125 | 8.927 | 6.652 | 35.441 | 15.127 | 24.345 | 74.914 | 50.265 | 24.648 |
| 9.0 | 1.796 | 3.454 | 9.867 | 7.352 | 43.295 | 18.480 | 29.741 | 91.515 | 63.617 | 27.898 |
| 10.0 | 1.967 | 3.783 | 10.807 | 8.052 | 51.934 | 22.167 | 35.675 | 109.777 | 78.540 | 31.237 |

b) The total volume of civil works required to construct the pressure tunnel calculated from the volumes of work per unit length are shown in Table 2-27.

Table 2-27 Total Volume of Civil Works for Construction of the Pressure Tunnel

| Item | Type of Work | Unit | Inner Diameter D (m) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| Tunnel excavation | | | | | | | | |
| | Horizontal | $m^3$ | 19522 | 25358 | 32029 | 39533 | 47870 | 57042 |
| | Inclined | " | 3132 | 4169 | 5355 | 6690 | 8172 | 9803 |
| Plug concrete | | | | | | | | |
| | Horizontal | " | 8470 | 9938 | 11451 | 13009 | 14609 | 16254 |
| | Inclined | " | 1352 | 1613 | 1882 | 2159 | 2444 | 2736 |
| Reinforcement bar | | t | 64 | 69 | 73 | 78 | 82 | 87 |
| Grouting | | | | | | | | |
| | Drilling | m | 1867 | 1867 | 1867 | 1867 | 1867 | 1867 |
| | Consolidation grout | t | 280 | 280 | 280 | 280 | 280 | 280 |
| | Contact grout | $m^3$ | 914 | 983 | 1027 | 1059 | 1086 | 1106 |

iii) Pressure tunnel construction cost

The cost for construction of the pressure tunnel is calculated by multiplying the volumes of civil works and the quantities of steel pipes with the respective unit costs, for diameters varying from 5.0 m to 10.0 at 1 m steps.

Table 2-28 shows the unit prices used to carry out the aforesaid calculation.

Table 2-28 Unit Prices

| | Unit Prices (M$) |
|---|---|
| Tunnel excavation (horizontal) | 86.5/$m^3$ |
| Tunnel excavation (inclined) | 110.0/$m^3$ |
| Plug concrete (horizontal) | 230.0/$m^3$ |
| Plug concrete (inclined) | 230.0/$m^3$ |
| Reinforcement bar | 1800.0/t |
| Drilling | 130.0/m |
| Consolidation grout | 1350.0/t |
| Contact grout | 1130.0/$m^3$ |
| Steel pipe | 5400.0/t |

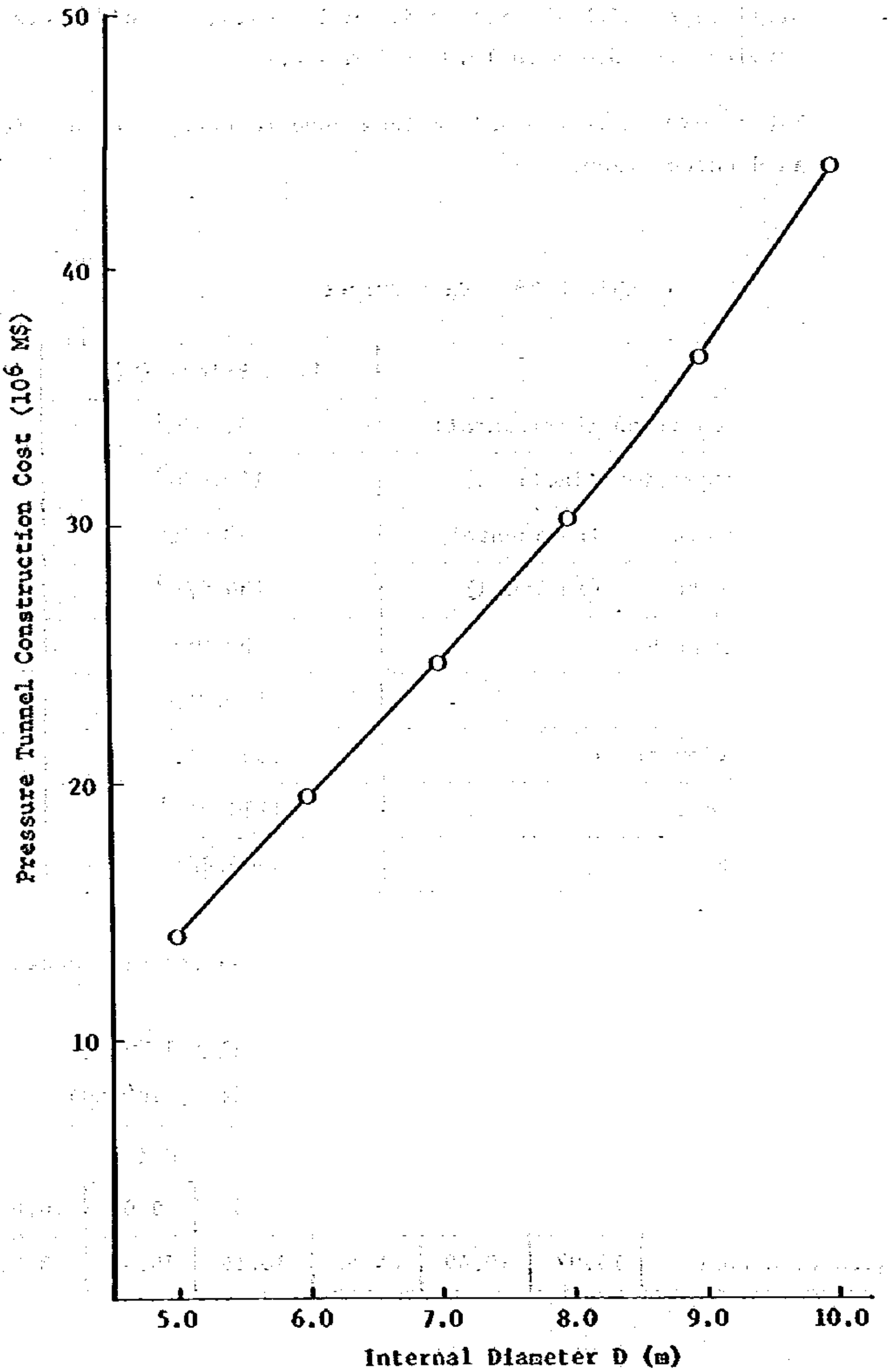
Table 2-29 presents the pressure tunnel construction costs.

Table 2-29 Total Pressure Tunnel Construction Costs

(Unit : $10^6$ M$)

| | Inner Diameter D (m) | | | | | |
|---|---|---|---|---|---|---|
| | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 |
| Construction cost | 13.97 | 19.49 | 24.62 | 30.16 | 36.53 | 43.98 |

Fig. 2-30 Relationship between Internal Diameter of Pressure Tunnel and Construction Cost



iv) Annual cost

The increase/decrease of the pressure tunnel construction cost calculated by assuming a reference inner diameter of 6.0 m is presented in Table 2-30.

Table 2-30 Increase/Decrease of Construction Cost

Unit $10^6$M$

| Inner Diameter (m) | Pressure tunnel Construction Cost | Increase/Decrease of Pressure Tunnel Construction Cost |
|---|---|---|
| 5.0 | 13.97 | -5.52 |
| 6.0 | 19.49 | 0 |
| 7.0 | 24.62 | 5.13 |
| 8.0 | 30.16 | 10.67 |
| 9.0 | 36.53 | 17.04 |
| 10.0 | 43.98 | 24.49 |

Table 2-31 presents the increase/decrease of annual cost ($\Delta C$) calculated by using the expression given in section (3).

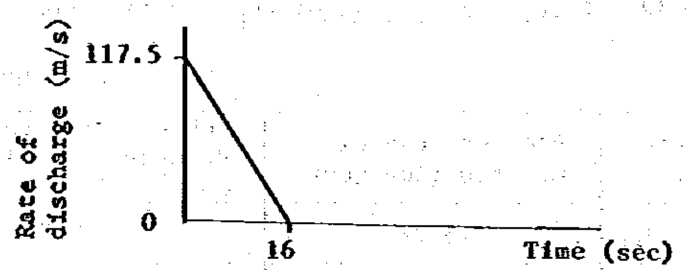
Table 2-31 Increase/Decrease of Annual Expenditure

| Inner Diameter (m) | $\Delta C$ ($10^6$M$) |
|---|---|
| 5.0 | -0.77 |
| 6.0 | 0 |
| 7.0 | 0.71 |
| 8.0 | 1.48 |
| 9.0 | 2.37 |
| 10.0 | 3.40 |

### 2.8.3 Calculation of Water Hammer Pressure

(1) Opening and rate of discharge

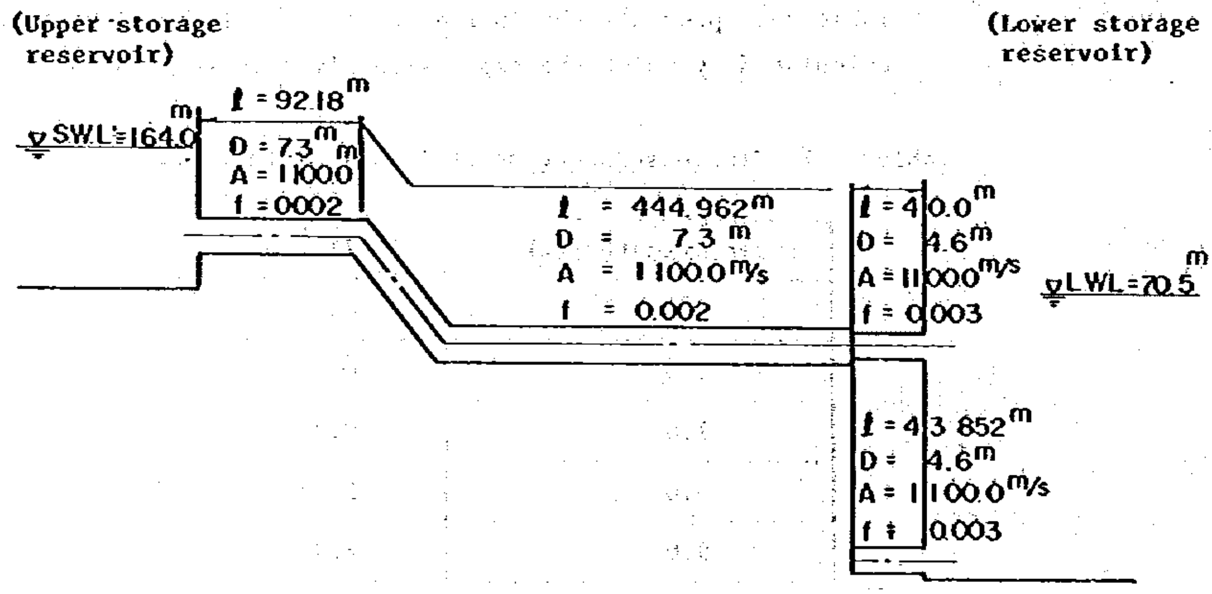
The valve is assumed to cut off the flow within 16 seconds.




(2) Upper reservoir water level and lower reservoir water level

| | |
|---|---|
| Upper reservoir water level | S.W.L. = 164.0 m |
| Lower reservoir water level | L.W.L. = 70.5 m |

(3) Numerical values of the waterway parameters



(4) Results

(Unit : EL m)

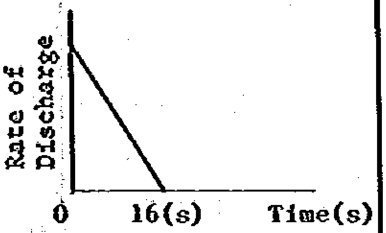
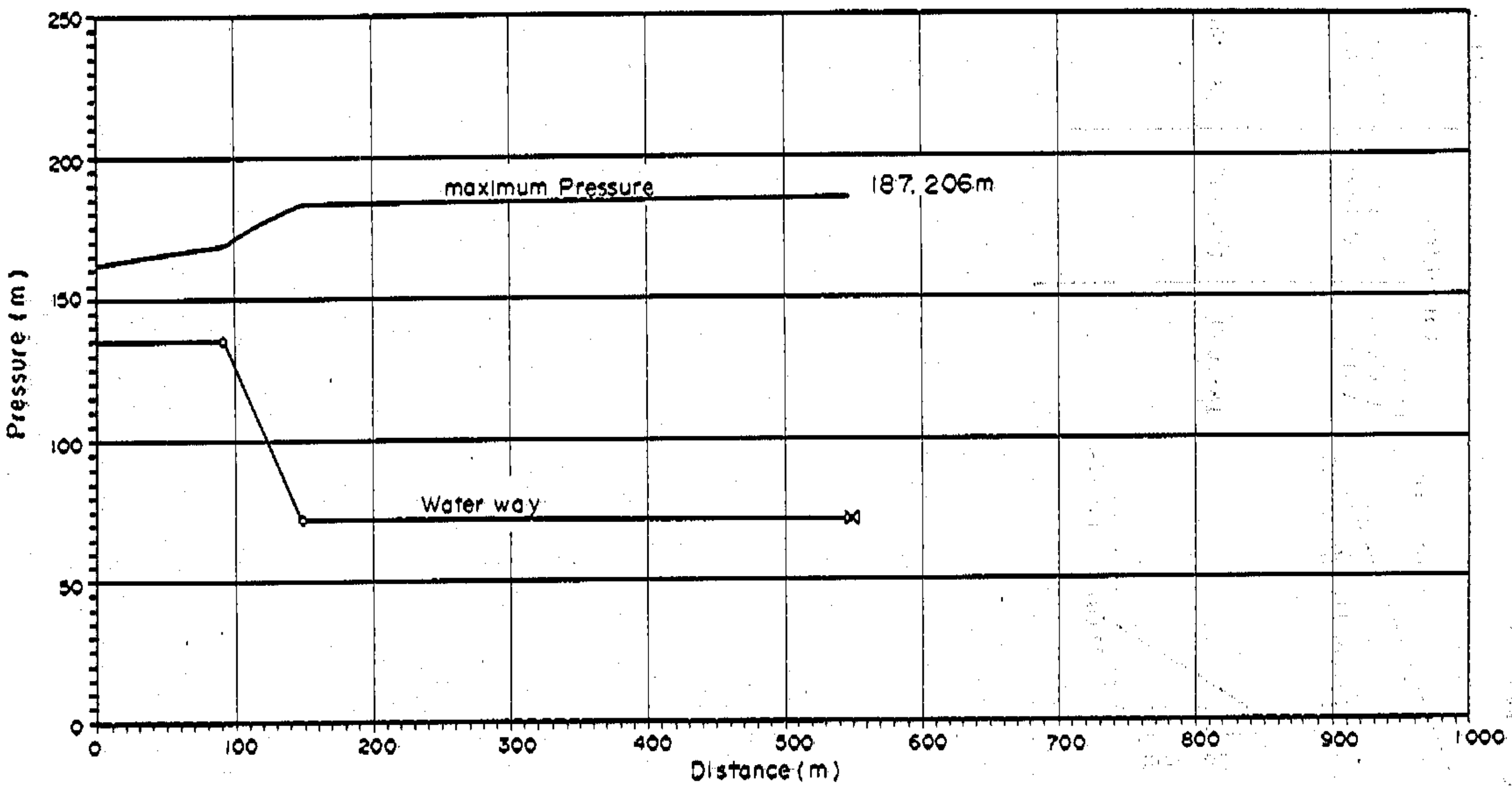
| Pressure / Cut Off Condition | Bifurcation | | Turbine Center | |
|---|---|---|---|---|
| | Maximum Value | Minimum Value | Maximum Value | Minimum Value |
|  | 185.000 | 143.258 | 187.268 | 140.897 |

Fig. 2-31 Maximum Pressure Along The Water Way



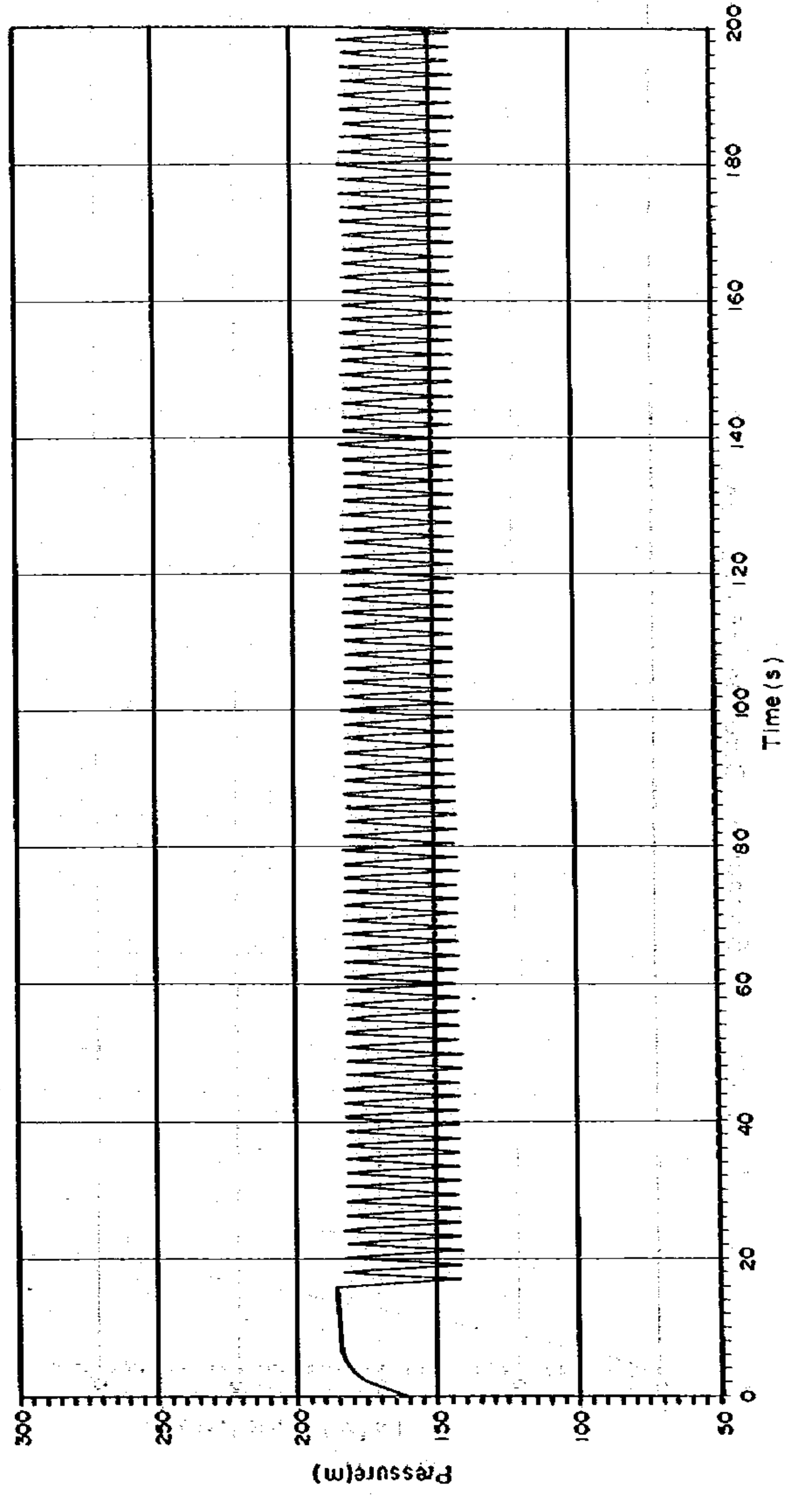
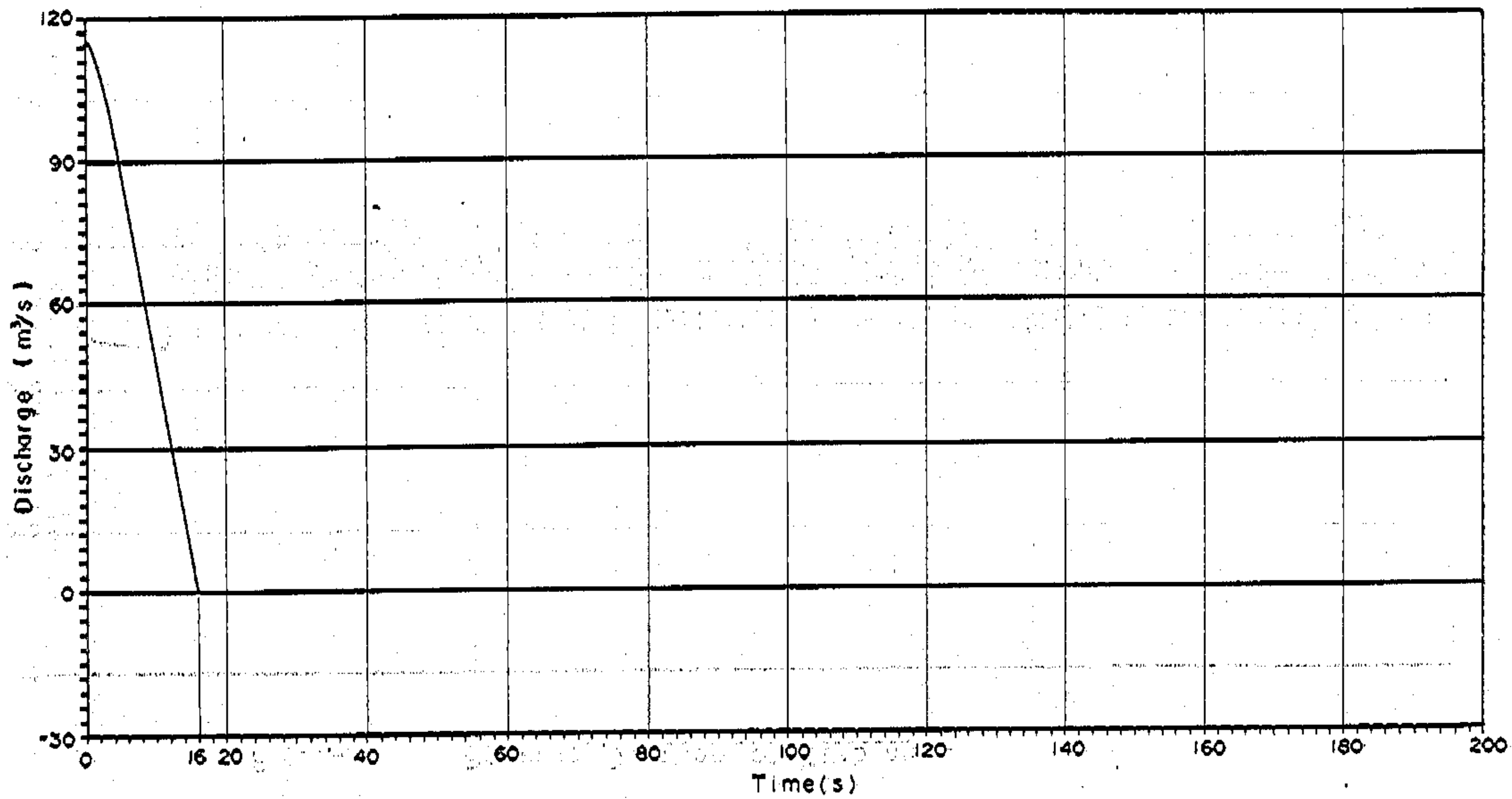



Fig. 2-32 Pressure Duration at Turbine

Fig. 2-33 Discharge Duratron of One Unit



## 2.9 Diversion Tunnel

The diversion works are designed by assuming that the flood to be handled is equal to the 100-year probability flood discharge. It is planned that the diversion channel construction cost will be cut down by making use of the surcharge capacity of the coffer dam. The basic specifications of the diversion works are as follows.

- Flood discharge to be handled: 100-year flood
  (peak flow = 2,700 $m^3/s$)

- Specifications of the diversion tunnel

| | |
|---|---|
| Cross section configuration | : Standard horseshoe shape |
| Inner diameter | : 8 m |
| Length | : 716.68 m ≑ 717 m |
| Inlet elevation | : EL 75.00 |
| Outlet elevation | : EL 72.50 |

### (1) Calculation of discharge

The discharge is calculated by means of the following expression.

$$Q = A \cdot V$$

$$V = \sqrt{\frac{2 \cdot g \cdot \Delta H}{1 + f_e + f_b + f' \frac{L}{R}}}$$

where;

| | |
|---|---|
| $Q$ | : Discharge ($m^3/s$) |
| $V$ | : Velocity of discharge (m/s) |
| $g$ | : Acceleration of gravity (= 9.8 $m/s^2$) |
| $\Delta H$ | : Head difference (m) |
| $f_e$ | : Coefficient of loss due to the inlet (= 0.2) |
| $f_b$ | : Coefficient of loss due to bend and curve (= 0.125) |
| $f' \frac{L}{R}$ | : Coefficient of loss due to friction (= $\frac{2 \cdot g \cdot n^2}{R^{4/3}} \cdot L$) |
| m | : Coefficient of roughness (= 0.013) |

R : Hydraulic radius (= A/P)

L : Waterway length (m)

P : Perimeter (m)

$$A = A_o - \{\tilde{\theta}_o - (\frac{h_o}{r_o})\sqrt{1 - (\frac{h_o}{r_o})^2}\}\, r_o^2$$

$$P = P_o - 2 \cdot \tilde{\theta}_o \cdot r_o$$

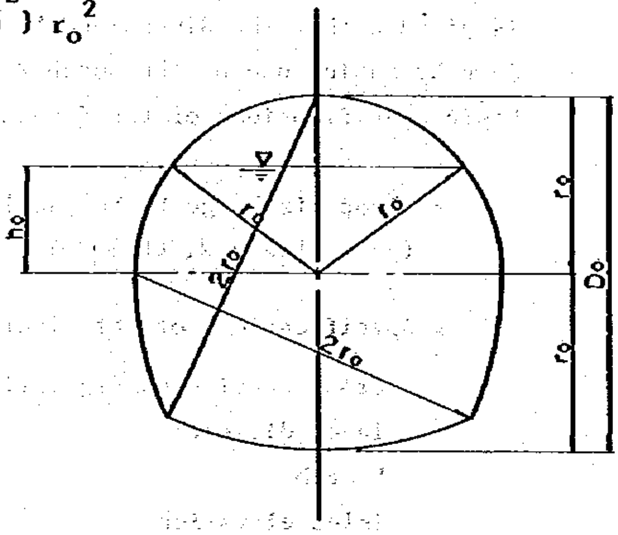
where;

$$\tilde{\theta}_o = \text{rad } \theta_o$$

$$\sin(90° - \theta_o) = \frac{h_o}{r_o}$$

$$A_o = 3.317 \cdot r_o^2$$

$$P_o = 6.543 \cdot r_o$$




(2) Diversion channel water level versus discharge rate

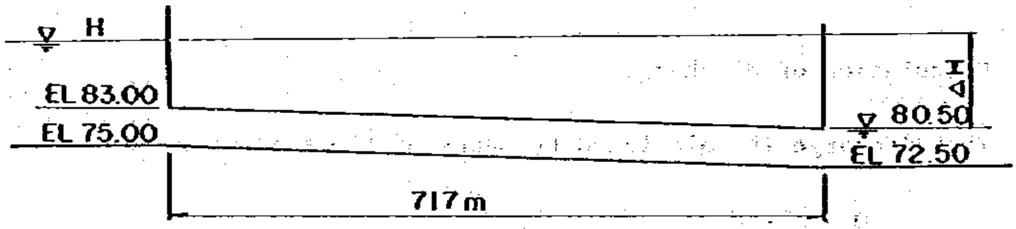


Table 2-32 Calculation of Discharge Volume

| H | ΔH | $h_o$ | $\theta_o$ | P | A | R | $f'\frac{L}{R}$ | V | Q |
|---|---|---|---|---|---|---|---|---|---|
| 75.5 | | -3.5 | 2.636 | 5.082 | 4.115 | 0.810 | 3.147 | 3.310 | 13.6 |
| 76.0 | | -3.0 | 2.419 | 6.821 | 6.433 | 0.943 | 2.568 | 3.548 | 22.8 |
| 76.5 | | -2.5 | 2.246 | 8.205 | 9.331 | 1.137 | 2.001 | 3.838 | 35.8 |
| 77.5 | | -1.5 | 1.955 | 10.530 | 16.337 | 1.541 | 1.334 | 4.292 | 69.7 |
| 78.0 | | -1.0 | 1.823 | 11.584 | 20.023 | 1.729 | 1.145 | 4.454 | 89.2 |
| 79.0 | | 0 | 1.571 | 13.606 | 27.939 | 2.054 | 0.910 | 5.682 | 130.8 |
| 80.0 | 2.5 | 1.0 | 1.318 | 15.627 | 35.855 | 2.294 | 0.785 | 4.819 | 172.8 |
| 81.0 | | 2.0 | 1.047 | 17.794 | 43.245 | 2.430 | 0.727 | 4.387 | 211.3 |
| 82.0 | | 3.0 | 0.723 | 20.390 | 49.446 | 2.425 | 0.729 | 4.884 | 241.5 |
| 82.2 | | 3.2 | 0.643 | 21.024 | 50.456 | 2.400 | 0.739 | 4.872 | 245.8 |
| 82.4 | | 3.4 | 0.555 | 21.734 | 51.359 | 2.363 | 0.755 | 4.854 | 249.3 |
| 82.6 | | 3.6 | 0.451 | 22.564 | 52.132 | 2.310 | 0.778 | 4.828 | 251.7 |
| 82.8 | | 3.8 | 0.318 | 23.632 | 52.737 | 2.232 | 0.814 | 4.786 | 252.4 |
| 82.9 | | 3.9 | 0.224 | 24.379 | 52.953 | 2.172 | 0.844 | 4.753 | 251.7 |
| 83.0 | | 4.0 | 0 | 26.172 | 53.072 | 2.028 | 0.925 | 4.666 | 247.7 |
| 85.0 | 4.5 | | | | | | | 6.261 | 332.3 |
| 86.0 | 5.5 | | | | | | | 6.921 | 367.3 |
| 88.0 | 7.5 | | | | | | | 8.082 | 428.9 |
| 90.0 | 9.5 | | | | | | | 9.096 | 482.8 |
| 92.0 | 11.5 | | | | | | | 10.008 | 531.2 |
| 94.0 | 13.5 | | | | | | | 10.844 | 575.5 |
| 96.0 | 15.5 | | | | | | | 11.619 | 616.6 |
| 98.0 | 17.5 | | | | | | | 12.346 | 655.2 |
| 100.0 | 19.5 | | | | | | | 13.032 | 691.7 |
| 102.0 | 21.5 | | | | | | | 13.684 | 726.3 |
| 104.0 | 23.5 | | | | | | | 14.307 | 759.3 |
| 105.0 | 24.5 | | | | | | | 14.608 | 775.3 |
| 106.0 | 25.5 | | | | | | | 14.903 | 790.0 |
| 107.0 | 26.5 | | | | | | | 15.192 | 806.3 |
| 108.0 | 27.5 | | | | | | | 15.476 | 821.4 |
| 109.0 | 28.5 | | | | | | | 15.755 | 836.2 |
| 110.0 | 29.5 | | | | | | | 16.029 | 850.7 |
| 111.0 | 30.5 | | | | | | | 16.299 | 865.0 |
| 112.0 | 31.5 | | | | | | | 16.546 | 879.1 |

The "water level versus discharge rate" curve of the diversion tunnel is shown in Fig. 2-34. On the other hand, the relation between the peak flood flow and probability year is shown in Fig. 2-35.

When the calculations are carried out by assuming the 100-year probability discharge rate (peak discharge rate = 2,700 $m^3/s$), the maximum discharge of the diversion tunnel becomes 850 $m^3/s$, and the maximum water level rise becomes 110.000 m. Consequently, the elevation of the coffer dam is considered EL111.500, by taking a margin of safety of 1.50 m. (Storage reservoir wind wave 1.2 m + margin of safety 0.3 = 1.50 m).

Fig. 2-34 Water Level - Discharge Curve of Diversion Tunnel

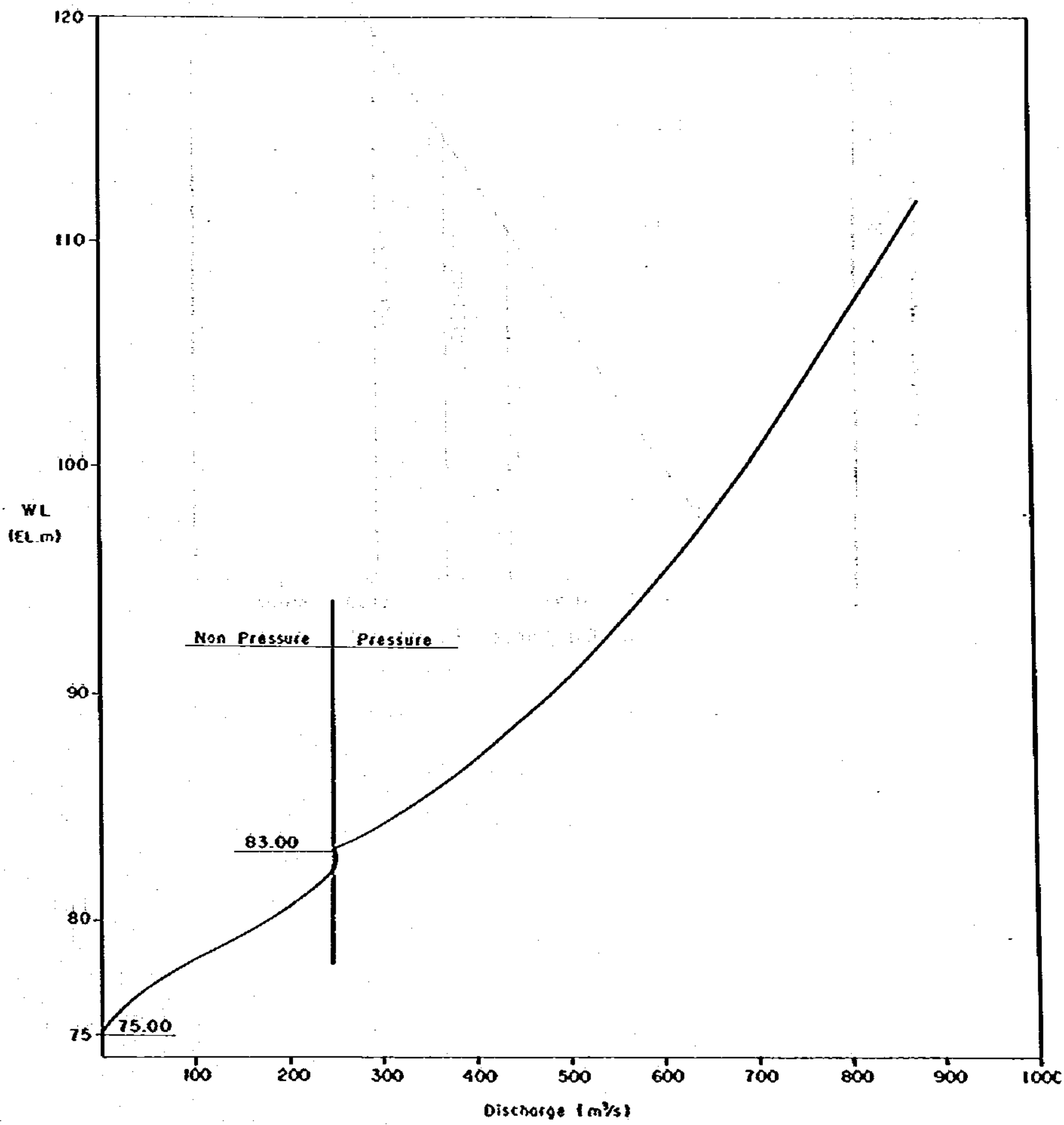


Fig 2-35 Probable Flood Discharge

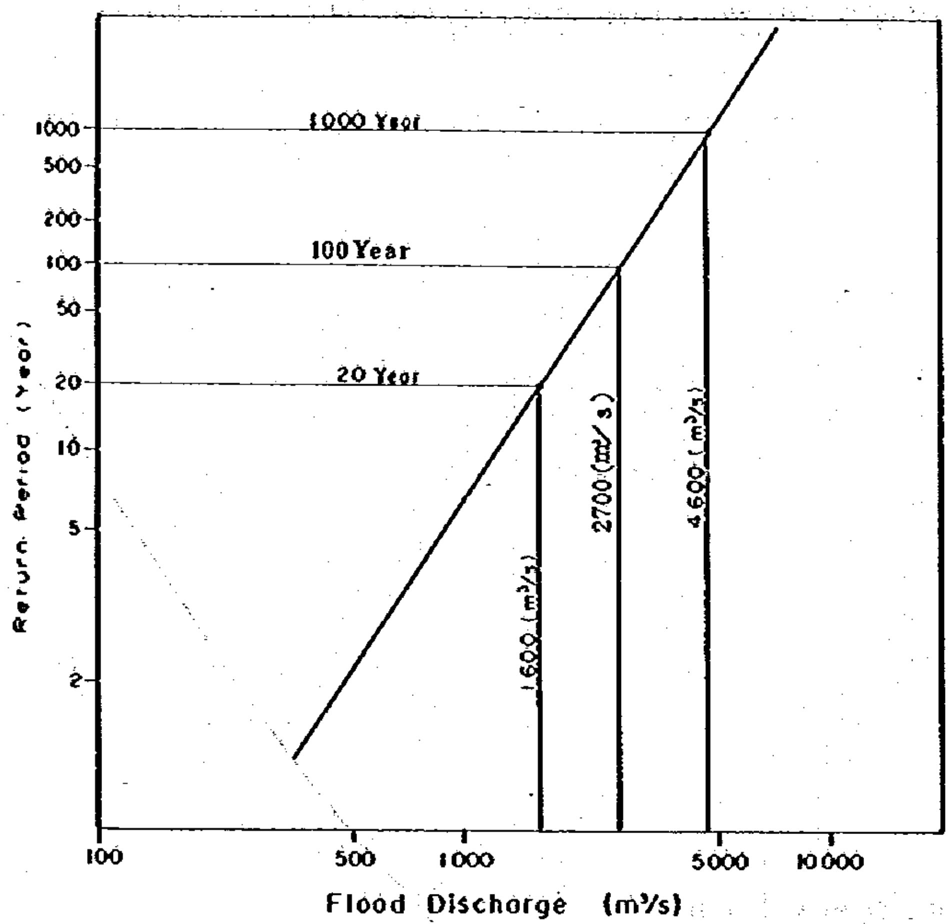


Fig. 2 — 36 Discharge and Water Level during Flood