№ 19

THE ROYAL GOVERNMENT

OF

THE KINGDOM OF THAILAND

THE WATER SUPPLY PROJECT FOR THE REFUGEES IN THAILAND (PHASE III)

OCTOBER 1981

JAPAN INTERNATIONAL COOPERATION AGENCY

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JAPAN INTERNATIONAL COOPERATION AGENCY

国際協力事業団
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PREFACE

In response to the request of the Government of the Kingdom of Thailand, the Japanese Government decided to conduct a survey on the Water Supply Project for Refugees and entrusted the survey to the Japan International Cooperation Agency (JICA). The JICA sent to Thailand a survey team headed by Mr. Kazuhisa Matsuoka from 3rd June to 26th August, 1981.

The team had discussions with the officials concerned of the Government of the Kingdom of Thailand and conducted a field survey. After the team returned to Japan, further studies were made and the present report has been prepared.

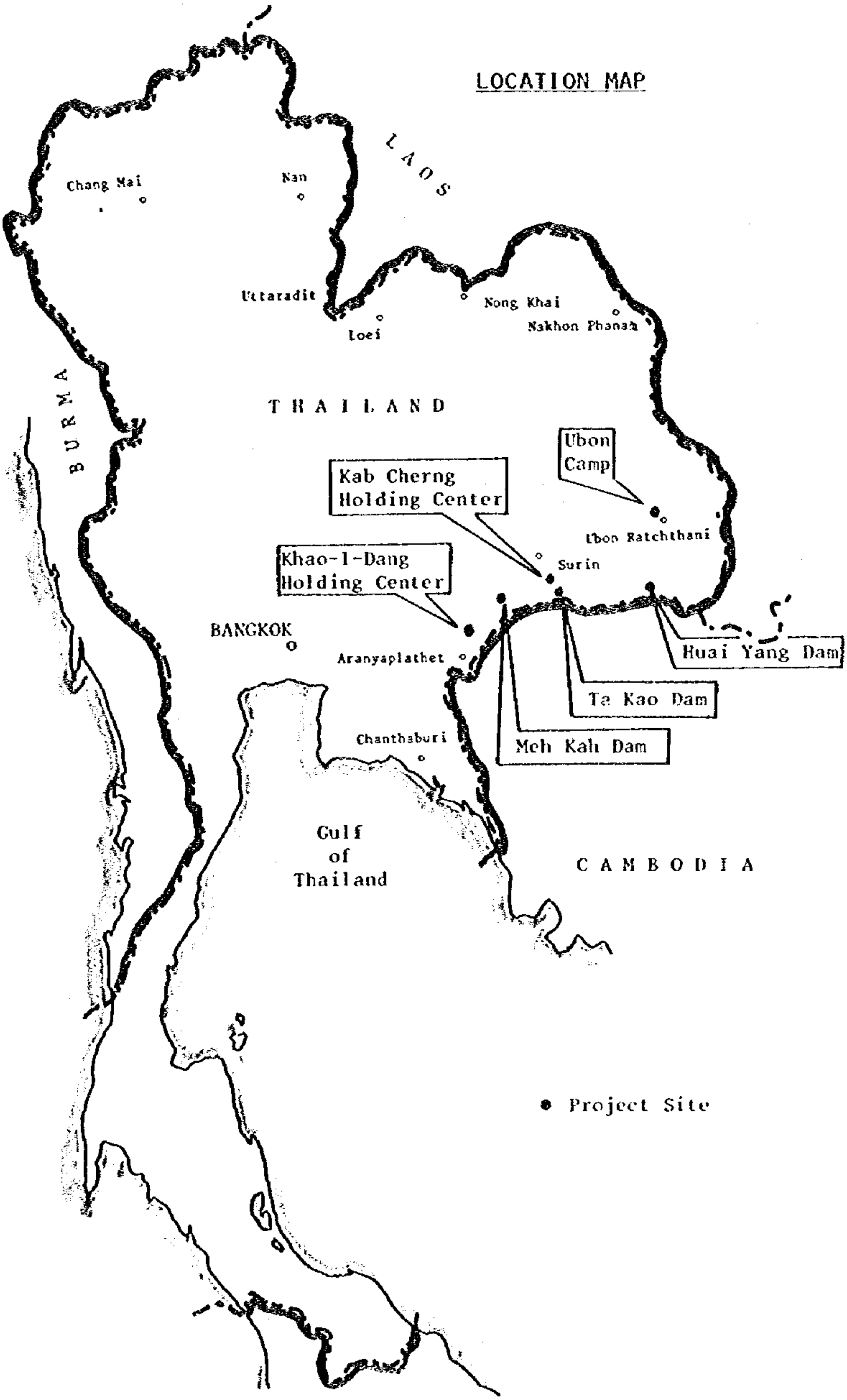
I hope that this report will serve for the development of the Project and contribute to the promotion of friendly relations between the two countries.

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

October, 1981

Keisuke Arita
President
Japan International Cooperation Agency

LOCATION MAP



SUMMARY

1. In Thailand, asistance to Cambodian and Laotian refugees and Thai inhabitants of the Cambodian border region, being affected by the Cambodian civil war, has been provided by UNHCR, biliteral aid agencies and various voluntary organizations. The Government of Japan has been offering technical cooperation and grant assistance to the Kingdom of Thailand in the fields of medical service and subsistence water supply since 1980.

Through the implementation of the subsistence water supply projects such as construction of the Sakeo intake barrage, the Huai Ta Kien reservoir and pipeline systems at the Phanat Nikhom Holding Center, a great number of refugees and the affected Thai villagers could enjoy the benefit from securing of the subsistence water supply. Most of the projects undertaken so for are mainly for the Cambodian refugees but any countermeasures for securing the supply of subsistence water have hardly been taken for the Laotian refugees and the affected Thai people so that those are suffering from a serious shortage of subsistence water. Under these circumstances, the Royal Government of Thailand has taken up a policy to promote a scheme for new village settlements for the affected Thai villagers and the scheme is progressing in line with the Government's policy to provide the stable living conditions to the villagers.

In January, 1981, the Royal Government of Thailand requested the Japanese grant assistance to a subsistance water supply project for the refugee camps in the Surin and Ubon regions as a step in the overall attempt to provide relief to the Cambodian and Laotian refugees and the new village settlers.

In March, 1981, upon receipt of this request, the Government of Japan dispatched a contact mission to survey the sites proposed under the projects and determine their priority. Based on the contact mission's recommendations, this survey has been conducted.

2. The scope of this survey is as follows:

i) Basic design study for the Meh Kah, Ta Kao, and Huai Yang dams;

ii) Groundwater development survey of deep wells at the Kab Cherng and Kao-I-Dang Refugee Centers, and improvement scheme for the existing shallow wells at the Ubon Holding Center; and

iii) Project formation survey for the subsistance water supply project for the Laotian refugee camps.

3. The details and results of the survey are as follows:

3.1 Basic Design of Dams

The fundamental plan was worked out for the construction of the Meh Kah and Ta Kao dams and for the renovation of the Huai Yang dam. The basic design for all three dam projects was completed. The location of each dam is as follows:

Name of Dam	Location	
	Province	District
Meh Kah	Buriram	Ban Kruat
Ta Kao	Surin	Kab Cherng
Huai Yang	Ubon Ratchathani	Nam Yun

(1) Particular of Dams

Main Feature of Three Dams

		Meh Kha Dam	Ta Kao Dam	Huai Yang Dam
Reservoir				
Catchment Area		21.0 km^2	34.6 km^2	31.3 km^2
Reservoir Area		1.10 km^2	2.08 km^2	0.15 km^2
Normal Water Level		EL. 250.50 m	EL. 203.00 m	EL. 189.30 m
Design Flood Level		EL. 251.50 m	EL. 203.85 m	EL. 190.50 m
Total Storage Capacity		4,600,000 m^3	8,600,000 m^3	300,000 m^3
Available Storage Capacity		4,500,000 m^3	8,440,000 m^3	300,000 m^3
Dead Storage Capacity		(EL. 240.00 m) 100,000 m^3	(EL. 196.00 m) 160,000 m^3	-
Dam				
Type		Homogeneous Type	Homogeneous Type	Homogeneous Type
Length of Crest		490 m	600 m	710 m
Elevation of Crest		EL. 253.70 m	EL. 206.30 m	EL. 191.40 m
Height of Dam		16.4 m	14.6 m	10.4 m
Crest Width of Dam		6.0 m	6.0 m	4.0 m
Slope	Upper-stream	1 on 3.0	1 on 3.0	1 on 2.5
	Down-stream	1 on 2.5	1 on 2.5	1 on 2.0
Volume Content of Dam		141,000 m^3	140,000 m^3	45,000 m^3
Spillway				
Type		Overflow Weir	Overflow Weir	Overflow Weir
Design Flood Discharge		44.0 m^3/s	31.3 m^3/s	184.0 m^3/s
Width of Spillway		22.0 m	20.0 m	70.0 m
Width of Emergency Spillway		--	--	35.0 m
Irrigation Area		250 ha	350 ha	360 ha
Construction Cost	Yen	524,000,000	526,000,000	277,000,000
	Baht	50,400,000	50,600,000	26,670,000

(2) Project Cost

Project cost for each dam is as follows:

unit: 1,000 Yen
(unit: 1,000 Baht)

Item \ Dam Name	Meh Kah Dam	Ta Kao Dam	Huai Yang Dam
A. Construction Cost	475,000 (45,700)	477,000 (45,900)	272,000 (26,190)
B. Engineering Service	49,000 (4,700)	49,000 (4,700)	5,000 (480)
Total	524,000 (50,400)	526,000 (50,600)	277,000 (26,670)

In the above listed figures, contingency is not included. The detailed design cost was estimated in the engineering services fee only for the Huai Yang Dam.

The exchange rate used in cost estimates was that of August, 1981. US$1.00 = ¥235 = B22.6, ฿1.00 = ¥10.4.

(3) Evaluation

All three dams, Meh Kah, Ta Kao and Huai Yang, are located in the Thai-Cambodian border region. At present, they are proposed to build in conjunction with the Royal Government of Thailand's vigorous promotion of the new village settlements for the Thai inhabitants of the border region. The construction of the dams are expected to increase agricultural production revenues, with the additional prospect for an increase in fisheries revenue. The expected annual increase in output by each dam is as follows:

Meh Kah Dam	฿2,779,000
Ta Kao Dam	฿3,945,000
Huai Yang Dam	฿955,000

The B/C ratio of the annual output increase (B) from each dam to the overall project cost (C) was computed as follows.

Meh Kah Dam	5.5 %
Ta Kao Dam	7.8 %
Huai Yang Dam	3.6 %

In evaluation of these figures would suggest the Meh Kah and Ta Kao dams to be economically feasible. However, the Huai Yang dam can not be compared effectively with the others.

(4) Construction Plan

- Construction of all three dams can be completed within one dry season (November - June). According to the construction schedule proposed in this report, if the detailed design work could begin in mid-October and be completed within one month. Construction would then be scheduled to commence in mid-November to be completed by the beginning of June, 1982, for a total of a 7.5-month construction period. Because construction can only be carried out during the dry season, close adherence to the construction schedule would be essential (see: Table Ia - Ic).

Table Ia Construction Schedule (Meh Kah Dam)

Work Item		Q.T.Y.	1981			1982					
			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.
Engineering Service Detailed Design Supervision			D/D		Supervision						
Preparatory Works											
Dam Body	Jungle clearing	67,600m²									
	Stripping	58,200m³									
	Excavation	13,300m³									
	Embankment	161,400m³									
	Riprap	10,400m²									
	Toe Drain	4,200m³									
Spillway	Earth Works	14,000m³									
	Concrete Works	2,100m³									
	Reprap	260m²									
Intake	Earth Works	2,900m³									
	Concrete Works	290m³									
	Steel pipe ϕ500	182m									
	Value	1									
Outlet Conduit	Earth Works	2,200m³									
	Concrete Works	75m³									
	Steel pipe ϕ200	105m									

Table Ib Construction Schedule (Ta Kao Dam)

Work Item		Q.T.Y.	1981 Oct.	1981 Nov.	1981 Dec.	1982 Jan.	1982 Feb.	1982 Mar.	1982 Arp.	1982 May	1982 Jun.
Engineering Service Detailed Design & Supervision			D/D			Supervision					
Preparatory Works											
Dam Body	Jungle clearing	153,300m^2									
	Stripping	70,500m^3									
	Excavation	13,800m^3									
	Embankment	156,000m^3									
	Riprap	11,400m^2									
	Toe Drain	3,450m^2									
Spillway	Earth Works	12,900^3									
	Concrete Works	2,000m^3									
	Riprap	250m^2									
Intake	Earth Works	2,200m^3									
	Concrete Works	270m^3									
	Steel pipe ∅500	157m									
	Value	1									
Outlet Concuit	Earth Works	2,000m^3									
	Concrete Works	65m^3									
	Steel Pipe ∅200	105m									

Table Ic Construction Schedule (Huai Yang Dam)

Work Item		Q.T.Y.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.
Engineering Service Detailed Design											
Preparatory Works											
Dam Body	Jungle Clearing	19,800m^2									
	Stripping	11,800m^3									
	Excavation	15,700m^3									
	Embankment	44,700m^3									
	Drain and Outlet	5,700m^3									
	Riprap	1,900m^3									
	Grass turf	4,000m^2									
Spillway	Earth Works	74,400m^3									
	Concrete Works	1,500m^3									
	Masonry	1,200m^2									
	Water Stop	300m^2									
Intake	Earth Works	600m^3									
	Concrete Works	50m^3									
	Masonry	22m^2									
	Steel Gate	1									

In implementing the project, the consultant will be assigned for construction supervision including detailed design, while the contractor for construction work under the control of the Ministry of Agriculture (Royal Irrigation Department), Royal Government of Thailand.

(5) Recommendations

In order for the construction of the three dams, each dam has been assigned a relative construction priority ranking as shown in the table below.

Comparative Item \ Name of Dam	Meh Kah	Ta Kao	Huai Yang
Economic Effect	2	1	3
Social Effect	1	2	3
Security of Construction Work	1	3	2
Easiness of Construction	2	1	3

As a result of the above comparative study, higher priority cannot be found between the Meh Kah and Ta Kao dams from the viewpoint of overall aspects but it would be made depending entirely upon the item to which due attention is paid. In comparison to the Huai Yang dam, on the other hand, the Meh Kha and Ta Kao dams are given higher priority in all the items mentioned above. Accordingly, the recommendable higher priority should be placed on the Meh Kah and Ta Kao dams.

3.2 Groundwater Development Survey

(1) Deep Wells at Kab Cherng and Kao-I-Dang Holding Centers

The groundwater development survey for deep wells was carried out at Kab Cherng and Kao-I-Dang Holding Centers.

i) Kab Cherng:

Two deep wells were bored, the water capacity was confirmed and pumps were installed, thereby completing construction.

ii) Khao-I-Dang:

The existing incomplete deep well was tested and found to be operable. Casing pipe was inserted and a pump was installed.

Details of the deep wells are as shown below:

Name of Camp	Well No.	Boring		Casing (m)	Submerged Pump			Water Capacity (ℓ/min)
		Diameter (inch)	Depth (m)		Bore (inch)	Horse Power	Installation Depth (m)	
Kab Cherng	J No.1	63/4	92.5	93.0	4	2	85	7
	J No.2	63/4	94.0	43.0	4	2	65	54
Kao-I-Dang		81.2	12.0	12.5	4	2	81	

With the completion of construction on the deep wells at Kab Cherng, all four wells will be fully operable, filling the present need for subsistence water for the 6,500 refugees.

(2) Improvement Plan for Shallow Wells at Ubon Holding Center

There are presenting 19,000 refugees in the Ubon Holding Center. All of their subsistence water is supplied by 200 shallow wells (163 public and 37 private wells) and four deep wells. In conducting this study, the location of the wells was plotted on a map of the Holding Center and, for each well, the amount of water used, the number of people using the well and the purpose of water usage was determined. The structural condition of the well, the level of water and the water quality of 36 points out of these wells were examined. Of the 163 public shallow wells, 98 were determined necessary in order to provide an adequate supply of drinking water.

The improvements planned are; the concrete paving of the area surrounding each well, the placement of reinforced concrete well covers and the installation of hand pumps.

The construction cost estimated for the implementation of this scheme amounts to Yen 100,000 (Baht 9,800) per well, with the total cost of Yen 10,000,000 (Bart 960,000) for the 98 wells.

A number of the wells have been inoperable due to pollution caused by the inflow of waste water. If the above-mentioned improvement measures were to be adopted, these wells would become usable and pure. Furthermore, disinfectant will be used so that the water would be safe for drinking.

The construction period would be three months. Commencement can take place in either the wet or dry season, however it should be undertaken as soon as possible. The local contractors for this work could be applicable in the light of their abilities.

3.3 Project Finding Investigation

Field surveys were completed at the four northern Laotian refugee camps of Nakhom Phanom, Pak Chom, Pua and Mae Jarim, taking into consideration the severe deprivation of subsistence water supply within the camps and their surrounding areas. The circumstances leading to a worsening of the environment and the requests of the local authorities were also taken into account in the determination of the necessity for work on the projects shown in Table-II.

The results of the investigation gave priority to Nakhom Phanom and Pak Chom refugee camps because they are to become permanent camps. However, in consideration of the lack of subsistence water at Pua and Mae Jarim camps, they are appropriate projects for economic cooperation efforts. Because there are numerous projects urgently in need of attention at these camps, immediate assistant countermeasures are essential.

Table II Project Formation for Laotian Refugees

Name of Camp	Province	Refugee Population		Proposed Project
		July 10, 1981	Future	
Nakhon Phanom (Ban Na Pho)	Nakhon Phanom	1,763	20,000	(1) Deep Well (Refugees) (2) Deep Well (nearby people) (3) Huai Leang Yai Reservoir (4) Huai Noi Reservoir (5) Huai Bang Koi Reservoir (6) Huai Kham Reservoir
Pak Chom (Ban viani)	Loei	31,077	50,000	(1) Deep Well (Refugees) (2) Huai Bao Diversion Weir (3) Huai Chom Diversion Weir (4) Agricultural Training Centre Nong Daeng Water Project
Pua (Ban Nam Yao	Nan	9,884		(1) Pump and Machines (2) Deep Well (nearby people) (3) Huai Mat Reservoir (4) Nam Yang Reservoir (5) Huai Khao Lam Reservoir
Mae Jorim (Sob Tuang)	Nan	8,114		(1) Pumping Station Improvement (2) Deep Well (nearby people and refugees) (3) Na Kha Reservoir (4) Huai Khai Reservoir

THE WATER SUPPLY PROJECT FOR THE REFUGEES IN THAILAND

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List of Drawing

Glossary and Abbreviation

DTEC	Department of Technical and Economic Cooperation
MOI	Ministry of Interior
MRD	Mineral Resources Department
RID	Royal Irrigation Department
SCH	Supreme Command Headquarter
UNHCR	United Nations High Commissioner for Refugees
Amphoe	An administrative subdivision equivalent to district
Ban	An administrative subdivision equivalent to village
Huai	A small stream or creek
Khlong	A small stream or canal
rai	Unit of an area equivalent to 1600 sq m
sq m	Square meter
sq km	Square kilometer
ha	hektare
km	Kilometer
m	Meter
ℓ	litre
kg	Kilogram

Metric System and Conversion Tables

1. Area and Length

1 rai	=	1600 square meters
1 foot	=	0.3048 meter
1 inch	=	2.54 centimeters

2. Exchange rates (as of 14 August, 1981)

us$ 1	=	฿ 22.6	=	¥ 235
฿ 1	=	us$ 0.044		
฿ 1	=	¥ 10.40		

CHAPTER 1 INTRODUCTION

In response to the request of the Royal Thai Government, the Japanese Government's assistance in securing subsistence water for Cambodian Refugees in Thailand has been made in conjunction with the Government's policy for the relief of Cambodian refugees. The following survey and projects have been excuted since December, 1979.

1) Survey of Water Supply to Cambodian Refugees
   (December 1979 - April 1980)

2) Intake Barrage Project for Water Supply to Sakeo No.2 Camp
   (April - December 1980)

3) Basic Design Survey for Construction of Small-Scale Dams
   (September - November 1980)

4) Construction of the Huay-Ta-Kien Reservoir and Related Facilities
   (January - August 1981)

5) Design Survey and Construction of Water Supply Project for Phanat Nikhom Holding Center and the Affected Thai Residents in Surrounding Area
   (May 1981 - February 1982)

In addition to the above, similar projects carried out by the UNHCR and other international agencies are progressing. However, along with the Cambodian refugees, many residents of the border areas and Laotian refugees still remain to be gravely troubled by the acute shortage of water.

In May, 1981, the Thai Government has requested the Japanese Government's Cooperation for the following projects:

1) Construction of several deep wells in the Kab Cherng Holding Center, and improvement of the existing deep wells in the Kao-I- Dang Holding Center.

2) Improvement of the existing wells in the Ubon Camp for Laotian refgees, aiming at protection against water pollution.

3) Construction of the Meh Kah, Ta Kao, and Huai Yang Dams.

With the inflow of Cambodian refugees over the Thai border, the Thai farmers of the border areas will be forced to shifted from their villages. To protect their livelihood, new village settlements are being constructed in Meh Kah, Ta Kao and Huai Yang. Plans for the construction of dams for these three settlements will need to be drawn so as to secure the subsistence and irrigation water supply to the new villages.

The Japan International Cooperation Agency dispatched a contact mission to Thailand for a two-week stay in mid-March 1981. The purpose of their study was to determine the priority of the projects requested. The Royal Thai Government then further requested an project formation investigation of the water supply for northern Laotian refugee camps in consideration of the urgency of their need for water. This study mission was dispatched, from June 4 - August 26, 1981, on the basis of the finding of the JICA Contact Mission and in response to the additional request for the investigation of the northern Laotian refugees.

Results of Investigations and Studies

1) Basic Design Study

Basic design studies were completed for the Meh Kah, Ta Kao, and Huai Yang Dams.

2) Groundwater Development Survey

a) Kab Cherng Holding Center

Two deep wells were bored, the water capacity was confirmed and pumps were installed.

b) Kao-I-Dang Holding Center

The existing incomplete deep well (which had been begin by the UNHCR) was tested and found to be operable. Casing was inserted and a pump was installed for completion.

c) Ubon Refugee Camp

Plans were developed for the improvement of 98 shallow wells in the Camp.

3) Project Formation Survey for the Subsistence Water Supply for the Northern Laotian Refugee Camps

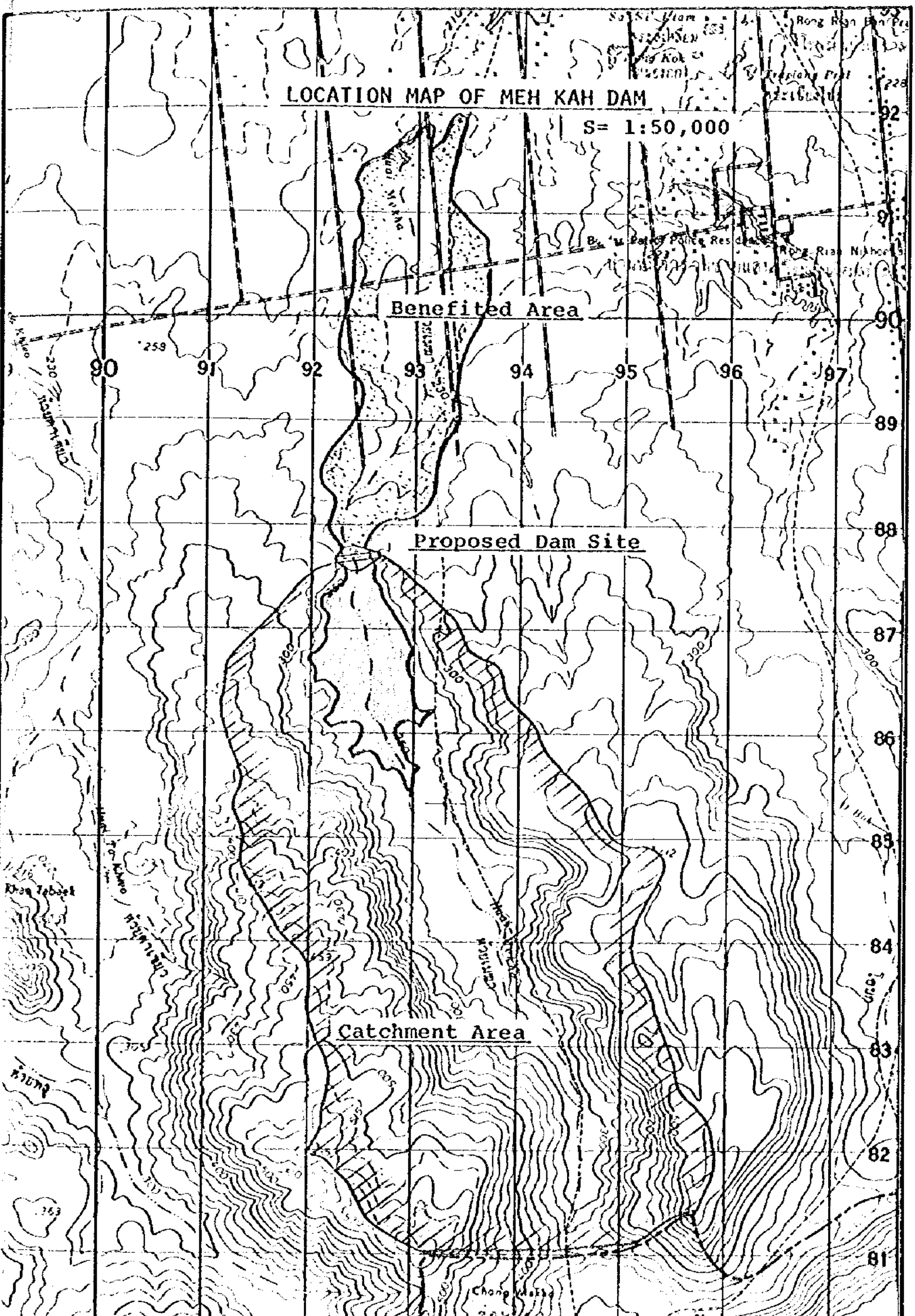
Field surveys were conducted on four northern Laotian refugee camps: Nakhom Phanom, Pak Chom, Mae Jarim and Pua.

This survey team submitted a draft final report to the Royal Thai Government on August 24, 1981. Included therein were the study results of the basic design of the dams, the improvement scheme of the shallow wells, and formation of the subsistence water supply projects for the northern Laotian refugee camps.

Upon returning to Japan, discussions on the findings of and the studies were held with the Japanese Government, resulting in the following final report.

Formation of the Survey Team is as follows:

Assignment	Name of Engineer	Position
Team Leader	Kazuhisa MATSUOKA	Japan International Cooperation Agency (JICA)
Executive Team Leader	Hiroshi YONEHARA	Japan Engineering Consultants Co., Ltd. (JEC)
Geologist	Kinzoo NARITA	JEC
Soil Mechanical Engineer	Yoshihiro INAGAKI	JEC
Hydrologist	Nobuyuki OKABE	JEC
Dam Design Engineer	Tetsuo SAKA	JEC
Dam Design Engineer	Kazuo MIBAYASHI	JEC
Hydraulic Design Engineer	Sumitada OKAMOTO	JEC
Structural Design Engineer	Koji SHINOHARA	JEC
Topographical Surveyor	Toshiaki SHIMAUCHI	JEC
Hydro-geological Engineer	Terukazu HAGIWARA	JEC
Procurement/Boring Engineer	Syozo MIYAJIMA	JEC
Boring/Mechanical Engineer	Syohachi SUZUKI	JEC
Well Improvement Engineer	Mitsugu TAKEO	JEC



LOCATION MAP OF MEH KAH DAM
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Benefited Area
Proposed Dam Site
Catchment Area
Chong Makha
Phanom Mae Kai
Rong Rian Nikhom
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CHAPTER 2 MEH KAH DAM CONSTRUCTION PROJECT

2.1 Project Area and Present Condition

2.1.1 Location

The project area is located in Amphoe Ban Kruat District, Buriram Province, in the eastern region of the Kingdom of Thailand.

It is situated at about 30 km point from Prakhom Chai where Provincial Highway No. 2075 starts for Lahan Sai.

The proposed Meh Kah dam, whose catchment area is rimmed by the Cambodian border, will benefit an area of about 1,600 ha including village under Government Renewal Scheme.

2.1.2 Topography

The catchment area varies in altitude from 300 m to 500 m. The ridges on the perimeter of this area form the border between Cambodia and north-eastern Thailand.

The Huai Meh Kah river, running north through the middle of the catchment area, turns into the Mae Nam Mun river after joining several streams and finally flows into the Mekhong river.

The benefit area extends between 200 m and 250 m in elevation on flat hills, spreding on both banks of the Huai Keh Kah river.

The proposed dam site of the altitude of 230 - 240 m is located at about 8 km from the border of Cambodia. The Huai Meh Kar river is perennial but during dry season its flow would not be very great.

2.1.3 Geology and Soils

The project area is composed of broad layers of the Khorat Group formed in the Mesozoic era. This Layer Group consists of (1) Phu Kradung formation of Jurassic - Triassic period, (2) Phan and Phra Wihan layers of the Jurassic period and (3) Salt and Khek Kruat layers of Cretaceous period. The Salt and Khek Kruat layer are predominant in the Khorat Group.

For the formations of the later Mesozoic era, volcanic rocks such as basalts, etc., which were formed through intrusion during the Tertiary - Quaternary period, can be found. Furthermore, river deposits are found as a new deposit at the surface layer.

Under the above-mentioned geological condition, the proposed site for Meh Kah dam and its benefited area are based on Phu Phan and Phra Wihan layers of Jurassic period and on lowlands or riverbed the Alluvium - Diluvium deposits comprising sandy or clayey soil, which were formed during the Quaternary period (see Fig. 2-1-1).

2.1.4 Present Socio-Economic Conditions

(1) Direct and Indirect Benefited Areas

With the completion of the project benefit area would involve four villages in total, viz. three new villages Saitri 3-10, 4-8 and 4-9 in the Amphoe Ban Kruat Province receiving direct benefit, and one village of Ban Yang receiving indirect benefit.

Area, Household and Population under Benefited Area

Name of Villages	Total Area (rai)	Total Household	Total Population
Saitri 3-10	2,187	65	417
Saitri 4-8	2,183	54	289
Saitri 4-9	5,625	46	194
Ban Yang	1,875	67	485
Total	11,870	232	1,385

(2) General Conditions of Direct Benefit Area

General conditions of the direct benefit area are shown on the following table.

General Conditions of Direct Benefit Area

Name of Village		Total Area (rai)	Cultivated Area (rai)			Other Area (rai)	Buffalo
			Paddy	Upland Crops	Total		
Saitri 3-10		2,187	400	1,300	1,700	487	76
Saitri 4-8		2,183	350	550	900	1,283	93
Saitri 4-9		5,625	350	910	1,260	4,365	137
Total	Rai	9,995	1,100	2,760	3,860	6,135	306
	ha	1,599	176	442	716	833	

This area has been reclaimed from thick jungle in only the last 15 years through the strenuous efforts of volunteering farmers under the Government promotion. In recent years, a village renewal project has been taken up as rural development for the nearby border villages and is on going.

This area, for the most hilly flat land, is mainly under upland crop cultivation, while the rest, along the Huai Meh Kah river, is under paddy cultivation.

The paddy fields produce once a year during the rainy season, while the upland areas are devoted for cultivation of tapioca. The average family member is high at about 5-7, because new settlers have been recently joined in development.

(3) Fundamental Structure of Agriculture

As mentioned above, this area has been reclaimed voluntarily by the local inhabitants in the last 15 years. They have the right to own as much land as they can develop by themselves, after registration. However, in recent years, only cultivation right and treat out right ownership have been awarded to the reclaimers. This case would be applicable to the nearby area of the dam site.

(4) Agro-Economic Analysis

The average per rai yields of main crops are: 300 kg of paddy and 1,800 kg of tapioca. While paddy is primarily for the cultivator's own consumption, upland crops is exclusively for marketing; the farm-gate price of paddy is ฿3.3/kg and that of tapioca, ฿0.4/kg.

For paddy cultivation, the average farmers need no cash expenditure as they plough with their own buffaloes and no fertilizer is applied to the field. They transplant and harvest with their own family labour or neighbourhood exchanging labour. For upland crop cultivation, on the other hand, some cash expenditure would be required: rental cost accounting for ฿200 per rai for the partial use of a tractor and extra labour cost at harvesting time.

Cash income per average farm-household would amount to ฿15,000/year: ฿2,000 from paddy and ฿13,000 from upland crops. These figures can be obtained depending entirely upon weather conditions and from the long-run point of view their average annual income would be estimated at about 80% of those given above.

2.2 Water Utilization Plan

2.2.1 Water Resources

The source of water for this project is the Huai Meh Kah river. The catchment area is 21 km^2. Since no discharge records are available for the Huai Meh Kah river, river discharge would be estimated in terms of the rainfall data.

2.2.2 Hydrology

The project area, which belongs to Buriram District, northern part of Thailand, is located approximately 8 km far from the border of Cambodia and spreading on a semi-flat plain, at approximately 240 m above sea level.

Since meteorological data are not available within the project area, these data recorded at the following four stations located in the visinity of the project area have been adopted as the basic figures for the project.

Lahan Sai	25 km from the dam site
Ban Kruat	10 km from the dam site
Prasat	50 km from the dam site
Surin	75 km from the dam site

Mean monthly and daily rainfall data recorded since 1952 have been collected as far as possible. For reference, meteorological records at Surin station are shown on the Table 2-2-1.

(1) Rainfall

The maximum probable daily rainfalls estimated by adopting the Gumbel Method at each station are as follows:

Maximum Probable Daily Rainfall (mm)

	Station			
Probability	Lahan Sai	Ban Kruat	Prasat	Surin
1/10	137	135	125	122
1/25	168	159	146	142
1/50	191	177	163	157
1/100	214	195	179	172
1/200	236	213	195	186

(2) Flood Discharge

The maximum flood discharge is estimated at 130 m^3/sec by adopting the design rainfall of 168 mm/day and the 1/25 probable daily rainfall recorded at Lahan Sai station, situating nearby the dam site and having plentiful rainfall data available.

In designing spilway, the flood follow-up calculation based on the assumed overflow width and depth of the spillway was made by taking into consideration its flood regulating capacity (the function of temporarily holding a part of the flood in surcharge portion of a reservoir). Consequently, spillway capacity, overflow width and depth are determined at 44 m^3/sec ,, 22 m and 1 m, respectively.

2.2.3 Water Requirements

The reservoir water can be utilized not only as subsistence water to villages but also for irrigating arable lands in and around the project area.

(1) Subsistence Water

Water requirements are calculated on the basis of:

Target Number of Family	200 families
Projected Family Constituent	7 persons per family
Target Population	1,400
Unit Amount of Water Supply	200 ℓ/day/head
Total Amount of Water Supply	280 m^3/day = 8,400 m^3/month

As there is no standard on subsistence water supply to villages, the losses due to conveyance and diversion presumed 10 % total amount of water supply are estimated at 311 m^3/day (3.6 ℓ/sec).

As the reservoir water would be brought to the suppling area through open earth canal, conveyance loss is computed at 20 % during the rainy season (May - October), while 30 % during the dry season (November - April).

Rainy Season	311/(1 - 0.2) = 390 m^3/day
Dry Season	311/(1 - 0.3) = 450 m^3/day

The reservoir water annually diverted to the villages would amount to:

390 m3 x 184 days + 450 m3 x 181 days = 153,210 m

Therefore, approximately 160,000 m^3 of the reservoir water per year would be utilized for villagers.

(2) Irrigation Requirements

The following assumptions would be made in calculating irrigation requirements.

1) Irrigation Efficiency

Irrigation efficiency is computed by taking into consideration season-wise losses due to application and conveyance in field as follows:

	Rainy Season	Dry Season
Field Efficiency	75 %	75 %
Conveyance Efficiency	80 %	70 %
Integrated Efficiency	60 %	52.5 %

2) Effective Rainfall

Effective rainfall as shown in Table 2-2-5 has been calculated by the following method on the basis of monthly rainfall (R, refer to Table 2-2-4).

Different Crops	Effective Rainfall	Upper Limit (mm/month)
Paddy	0.75 R	200
Upland Crops	0.75 R	120

3) Irrigation area under assumption of approximately 80 % of irrigation success would be determined in accordance with proposed dam storage capacity of 4.6 million m^3. Irrigation requirements can be obtained on the basis of the cropping pattern proposed under the project as shown in Figure 2-2-1. Moreover, the determination of the design storage capacity for the dam is described in Item 2.3 Dam Plan.

4) Irrigation Requirements

Irrigation Requirements can be computed as follows:

i) Net Water Requirements (N.W.R.)
= Crop Consumptive Use + Percolation + Water Requirements for Field Preparation

ii) Net Irrigation Requirements (N.I.R.)
= Net Water Requirements - Effective Rainfall

iii) Diversion Water Requirements (D.W.R.)
= Net Irrigation Requirements/Diversion Efficiency

Crop consumptive use is estimated at multiplying evapotranspiration value by crop consumption ratio of Penman Method. Percolation in field would be assumed to be 0.5 mm/day during the rainy season, while 1.0 mm/day during the dry season. Water requirements for land preparation, etc. would be assumed to be 200 mm for paddy and 40 mm for upland crops. Irrigation requirements for 1971 - 1980 have been estimated at the abovementioned assumptions.

(3) Diversion Requirements

Upon condition that proposed irrigation area is 250 ha and effective rainfall is valued by an average rainfall of the last 10 years, average monthly diversion requirements can be estimated as the total of irrigation requirements and subsistance water for villages.

Average Monthly Diversion Requirement

Rainy Season		Dry Season	
May	97,000 (m^3)	Nov.	422,000 (m^3)
Jun.	20,000	Dec.	165,000
Jul.	18,000	Jan.	517,000
Aug.	833,000	Feb.	676,000
Sep.	12,000	Mar.	668,000
Oct.	169,000	Apr.	468,000
sub-total	1,149,000	sub-total	2,916,000

Annual Total 4,065,000 m^3

2.2.4 Water Balance

(1) Assumptions

1) Inflow

Rainfall water in the catchment area of 21 km^2 would be collected into the proposed reservoir and its inflow to the reservoir can be estimated by adopting the following formulas:

Rainy Season (May - October)
Inflow = Monthly Rainfall x 21 km^2 x 24 %

Dry Season (November - April)
There would be no inflow available.

However, rainfall on the water surface of the reservoir is assumed to be 100 % effective.

Inflow = Monthly Rainfall x Water Surface in the Previous Month x 100

2) Losses in the Reservoir

Evaporation would be computed by multiplying 80 % of the pan evaporation value by the surface water area in the previous month.

Leakage is assumed to be 0.05 % per day (1.5 % per month) of the reservoir storage.

3) Rainfall Records

Rainfall records obtained at Lahan Sai have been supplemented by those available at the neighbouring stations. Monthly rainfall records from 1971 to 1980 have been used as basic data for this project.

(2) Calculation of Water Balance

Water balance for irrigation area from 1971 to 1980 is analyzed on the assumptions that the irrigation area is 250 ha; the whole area under paddy cultivation during the rainy season, while 100 ha for paddy and 150 ha under upland crops during dry season.

1) Effective Storage Potential and Water Requirements

Year	Effective Storage Potential	Water Requirements (Outflow)
1971/72	3,375,000 m^3	4,263,000 m^3
1972/73	4,334,000	4,107,000
1973/74	4,600,000	3,756,000
1974/75	3,209,000	3,914,000
1975/76	3,955,000	3,980,000
1976/77	3,714,000	3,899,000
1977/78	3,284,000	4,090,000
1978/79	4,341,000	4,400,000
1979/80	5,363,000	3,923,000
1980/81	5,363,000	3,923,000
Average	3,947,100	4,064,800

(Note: Design Storage Capacity = 4,600,000 m^3)

2) Months of Overflowing from Spillway

3 months out of 120 months

3) Effective Utilization Ratio

(Effective Storage Potential - Overflow)/Effective Storage Potential = 97 %

4) Months of Water Shortage

9 months out of 120 months

5) Crop-wise Irrigation Success Percentages

(refer to Figuare 2-2-2)

Rainy Season	Paddy (250 ha)	10/10 = 100 %
Dry Season	Paddy (100 ha)	4/9 = 44 %
Dry Season	Upland Crops-(1) (150 ha)	7/10 = 70 %
Dry Season	Upland Crops-(2) (150 ha)	4/9 = 44 %

2.3 Dam Plan

2.3.1 Dam Axis

The topographic map prepared by RID (s = 1/4,000) was used for determination of the proposed dam axis and it was finalized through surveying on the spot.

According to the topographic map, the dam axis proposed by RID was located in the downstream end of the narrowest portion of a valley where the valley widens towards its downstream. It was considered that, for the neighbouring area of the right bank side particularly, it widens most and in consequence the dam body would tend to slide at its abutments. As a result of field investigations, however, it was confirmed from the survey pegs remained that the dam axis proposed by RID was at about 30 m upstream distance from the dam axis read in the map and thereby causing no danger of silde. Therefore the dam axis proposed by RID would be deemed appropriate and it is adopted here.

2.3.2 Storage Capacity and Type

In determining storage capacity, the following two aspects are contemplated:

(i) Effective design storage potential which will be equalized to the balance between the inflow and reservoir loss (evaporation plus leakage), and

(ii) Comparative studies on the original plan worked out by RID.

Item (i): Inflow would be computed by the following formula in consideration of possibility only in rainy season.

In flow = Monthly Rainfall x 21 km^2 x 0.25

Effective design storage potential is calculated as shown in the following table on the assumption that run-off rate is set at 0.70. The rate, which can be defined as a ratio of losses due to evaporation and leakage to inflow, is determined at 0.70 by taking into consideration comparative study for the values of effective design storage potential in terms of water balance calculation.

Year	Inflow (1,000 m^3)	Effective Design Storage Capacity (1,000 m^3)
1971/72	4,475.6	3,132.9
1972/73	5,685.8	3,980.1
1973/74	6,258.0	4,380.6
1974/75	4,558.6	3,191.0
1975/76	5,298.3	3,708.8
1976/77	5,041.1	3,528.8
1977/78	4,384.3	3,069.0
1978/79	5,923.6	4,146.5
1979/80	4,235.2	2,964.6
1980/81	7,125.8	4,988.0
Agerage	5,298.8	3,709.2

As a result, the representative values are:

Maximum Storage Capacity (1980 - 81)	4,988 x 10^3 m^3
Minimum Storage Capacity (1979 - 80)	2,965 x 10^3 m^3
Average Storage Capacity (annual average)	3,709 x 10^3 m^3

Thus, it would be deemed appropriate that design storage capacity is 4,600 x 10^3 m^3 (EL. 250.5 m)

Item (ii): In the original plan worked out by RID, the optimal design storage capacity was determined at 6,400 x 10^3 m^3 (EL. 252 m) but, judging from the results mentioned above, the original plan would be found inefficient in operating the proposed dam.

Accordingly storage capacity, 4,600 x 103 m3 would be deemed appropriate as the optimal design storage one through these comparative studies (refer to Fig. 2-3-1).

In consideration of the geological formation at the dam foundation, materials available from borrow area, and the ease of construction work, homogeneous earth dam is surely most recommendable here.

(1) Dam Foundation Bedrock

As a result of the geological survey, it was confirmed that the foundation bedrock, which is mostly sandstone, is spreading over at levels below 2 - 5 m from the surface. Over this bedrock, there exist some layers of weathering and developed cracked conditions. While in heavy weathered zone coefficient of permeability would be $K = 10^{-3} - 10^{-5}$ cm/sec and N value, over 14 - 15, in normal weathered zone, $K = 10^{-5} - 10^{-6}$ cm/sec together with the N value of over 50.

On the top of the bedrock, silty sand and sandy silt are being distributed in loose condition, showing $K = 10^{-3} - 10^{-4}$ cm/sec and N value = 5 - 29.

As far as a particular nature of such permeability at the dam foundation is concerned, earth dam would be found the best and the most appropriate and safe enough comparing with other types.

(2) Embankment Materials

As a result of borrow area survey impermeable materials such as sandy clay are found to extend down from the depth of 0.4 m below the surface within the area of 200 - 600 m upstream from the dam axis, ensuring its sufficient quantity. However, as permeable materials can be hardly found in and around the spot, a homogeneous dam would be chosen out of other types.

(3) Easiness of Construction Work

A homogeneous earth dam is known to have such disadvantage as to make accelerated dam volume increase brought by the more gentle slop in every the higher dam. In this case, the dam height would be required as low as about 15 m, so the above disadvantage can be neglected by paying attention to ease of construction.

2.3.3 Foundation and Borrow Area

(1) Dam Foundation Bedrock

As a result of dam foundation study through 3 borings, 18 standard penetration tests, 5 field permeability tests, 7 test pits, sampling and soil tests, the following was concluded (see: Figure 2-3-2).

Geological Condition

Geological formation for the dam axis, as illustrated in Figure 2-3-3 consists of sandstone, shale and conglomerate corresponding to Phu Phan and Phra Wihan formations of the Mesozoic Jurassic period and, on the top of it river and talus cone deposits of the Quarternary Alluvium period are being distributed.

The foundation bedrock is composed primarily of sandstone, extending to a depth of 2-5 m, over which it is in an advanced stage of weathering and soft cracked conditions. Some open cracks can be found particularly on both abutments of the dam axis.

The river and talus cone deposits, which are being distributed over the bedrock, are all in unconsolidated condition and composed of sandy clay, silty fine sand and sandy silt, etc. containing stumps and formicaries in cracked conditions and extend to a depth of about 2 m below the surface.

Permeability of Foundation Bedrock

Layer-wise coefficients of permeability are as follows:

Name of Layer	Coefficient of Permeability (cm/sec)
As and Dt Layers	$K = 10^{-3} - 10^{-4}$
Heavy Weathered Zone (Foundation Bedrock)	$K = 10^{-3} - 10^{-5}$
Normal Weathered Zone (Foundation Bedrock)	$K = 10^{-5} - 10^{-6}$

Judging from the above results, the dam foundation was found to be permeable; and complete foundation treatment would be required through grouting or blanketing. Open cracks, which are found spreading over especially on both abutments of the dam axis, should be paid due attention.

Soil Constants of As and Ac Layers

As and Ac layers are in loose condition, showing N value of 5-15 and their soil constants are shown below:

Item	As Layer	Ac Layer
Specific Gravity (Gs)	2.66	2.65
Void Ratio (e)	0.54	0.39
Wet Density (γt. t/m^3)	1.95	2.13
Dry Density (γd. t/m^3)	1.74	1.91
Saturated Density (γsat. t/m^3)	2.09	1.19
Cohesion (Cu. t/m^2)	0	0.70
Angle of Internal Friction (Øu.)	20.00	20.00

The As and Ac layers are extending over in and around the riverbed (on lowland) in which the height of the dam embankment would be maximized so it would come to be the most unstable site for the dam body. Accordingly, to prevent such instability, stability analysis on the dam body is essential.

(2) Borrow Area

Borrow area was selected at 200-600 m upstream distance from the dam axis in consideration of topographic, geological and soil conditions through field investigations conducted in the vicinity of the proposed dam site and also by taking into consideration a dam construction planning (see: Figure 2-3-2).

10 test pits, sampling and soil tests have been carried out for a survey for the selected borrow area, the results of which are shown below.

Soil Condition

Name of Soil	Depth	Colour
Top Soil and Silty Sand	0 - 0.4 m	Yellowish Brown - Dark Yellowish Brown
Sandy Clay	Below 0.4 m	Gray Brown - Yellowish Gray Brown

Sandy clay, which is extending at the depth of 0.4 m or lower from the surface, can be used as embankment materials. Availability of this sandy clay is as follows:

Borrow Area	A = 90,000 m^2
Excavation Depth	H = 3.0 m
Volume	V = 270,000 m^3

The entire requirement for the embankment material is about 142,000 m^3 which is quantitatively sufficient.

Design Parameters for Embankment Material

Sandy clay, which is usable as embankment materials, shows the following design parameters:

Specific Gravity	Gs = 2.67
Design Moisture Ratio	W = 13.0 ± 1.5%
Design Dry Density	γd = 1.86 t/m^3
Design Wet Density	γt = 2.10 t/m^3
Saturated Density	γsat = 2.16 t/m^3
Coefficient of Permeability	K20 = 5.0 x 10^{-7} cm/sec

Cohesion	Cu = 6.0 t/m^2
Angle of Internal Friction	ϕu = 16°
Variation Rate of Earth Volume	(C) = 1.03

2.3.4 Dam Design

As a result of the following items, the typical cross section of the dam body is illustrated in Figure 2-3-4.

(1) Dam Foundation Design

As stated in 2.3.2, a homogeneous earth type dam was chosen to be built on permeable foundation bedrock; however, in order to minimize the seepage loss from the foundation and to drain seepage water easily out of the dam body, a drain would be required to combine suitably with any of other seepage control methods. Moreover, attention should be paid to protection for piping due to the liquefaction of loose sand layers.

The most popular methods to be taken up for making the minimization of seepage loss and securing the safety of the dam body are as follows:

a) Downstream Drain

The provision of a downstream drain would be indispensable for securing the safety of a homogeneous dam through lowering downstream phreatic line.

b) Broad Cut-off Core

The permeable foundation should be excavated as far as the impermeable layer and then it should be replaced by core earth. This should then be properly compacted.

c) Sheet Pile

It would be to great extent effective for homogeneous foundation such as fine sand and silt layers, etc., while ineffective for cobble-mixture or stratified foundation.

d) Grouting

One line seepage control curtain would be made through cement or special grouting in the dam centre or somewhere in the upstream area not far from the centre, aiming not only at diminishing leakage and uplift pressure for the downstream dam body, but also at finding cracks and permeable layers which can not be found by other means.

e) Impervious Blanket

It will be done through extending impervious horizontal blanket to the surface layer of a permeable foundation on the upstream side of the dam body, and guarantees lower construction cost and effective piping protection.

Among any combination of the above-mentioned, special grouting would be deemed more appropriate as a leakage and piping protection method by judging from the geological condition of the dam foundation bedrock as mentioned in 2.3.3. However, adoption of the grouting method would be difficult because it is attended with the following problems.

(i) As a result of marketing study, the special grouting method is hardly adoptable in Thailand. If it is adopted, imported grouting materials and machines along with dispatch of specialists will be required. Moreover, it could be found difficult to give appropriate instructions for mixing of chemical liquid and quantity needed.

(ii) From the geological point of view, grouting must cover the riverbed and the abutments and, thereby, required period for grouting would cause constructing the dam impossible within one dry season.

There would be a cut-off core as a perfect seepage control method in the alternative to grouting and it would be difficult to excavate by use of machine and making large amount earth work that about 10 m deep permeable layers and, in addition, about 6 m deep heavy weathered sandstone layers are existing.

Under this consideration, combination of the above-mentioned cut-off core, impervious blanket and drain would be adopted in terms of the following (a) to allow leakage running through the foundation bedrock within permissible range, (b) to protect piping and (c) to drain seepage water easily out of the dam body.

a) Cut-off Core

As the effect of piping on weathered rock formation is considered negligible, cut-off have to be done to the depth of 2-6 m of As, Ac and Dt layers in order to excavate the defect portion with 2-4 m bottom width and its slope 1:1 and to replace its cut-off portion by impermeable materials and thereafter compact it.

b) Impervious Blanket

After removing surface earth, cut-off earth has to be evenly laid and then compacted. From the results of piping as mentioned later, blanket would be 50 m long and extends up to the abutment portions. The thickness, which is subject to the water pressure of 1/10 as a standard value and the maximum depth of 10 m, has been determined at 1 m with even treatment.

c) Drain

Drain, facilitating to take out seepage water in permeable foundation or dam body, should be provided at the foot of the downstream of the dam body. Among some types such as toe, horizontal and chimney drain, toe-drain would be adopted by reason of a rather low dam, being about 13 m at the maximum, and easiness of its construction work.

It is necessary to keep as high as possible the K value of drain for permeable foundation bedrock and rock materials to be used for slope protection as an additional purpose on the downstream side of the dam body. The usable rock materials would be D50 = 30 cm in average diameter and D50 x 1.5 in the maximum and contain grains whose diameter is less than 2.5 cm in satisfactory distribution condition.

(2) Designing of a Typical Cross Section

1) Freeboard

The freeboard (Hf) shall be selected the larger one, out of two figures: value computed by the following equation, or the standard freeboard value of 2.0 m (for embankment not higher than 50 m).

$$Hf = R + \Delta h + hs + ht$$

where

R : wave runup

Δh : rise in water level by extraordinary flood

ht : freeboard allowance of a fill dam (1.0 m)

hs : freeboard allowance by spillway types (0.5 m for a gate type, and nil for others)

a) Wave runup R

Hight of wind wave adopted here is the significant wave (hw) computed in SMB method from fetch (F) and wind velocity (U).

And the value is modified by Saville's method to obtain the wave runup (R) from a chart, since it is a function of the fetch the wind velocity, the slope of embankment, and the roughness of slope of a dam relative surface roughness.

fetch : 1.9 km

wind velocity : from a record in Surin
maximum wind velocity 50 knots
= 25.7 m/sec.
10 minutes mean wind velocity
V = 25.7/1.5 = 17 m/sec. V = 20 m/sec.

roughness and inclination of the slope

riprap slope 1 : 3.0

from a chart of the wave runup

R = 1.15 m

b) Rise in Water Level by Flood (Δh)

The water level hightened by flood is computed in the following equation where the storage effect of a reservoir is taken into account.

$$\Delta h = \frac{2}{3} \frac{\alpha Qo}{Q} \frac{h}{1 + \frac{A.h}{Q.T}}$$

Qo : design flood discharge 44.0 m^3/sec

Q : discharge from the overflow section Q = Qo if a conduit is absent

α : rate of rise in water level flood 0.2 in an open channel spillway

h : design overflow depth 100 m

A : reservoir area at design flood water level 1.25 km^2

T : duration of extraordinary flood. Standard value ranging 1 - 3 hours, 2 hours is adopted.

$$\Delta h = \frac{2}{3} \times 0.2 \times \frac{1.00}{1 + \frac{1.25 \times 10^6 \times 1.00}{44.0 \times 2 \times 3,600}} = 0.03 \text{ m}$$

Hf = 1.15 + 0.03 + 1.0 + 0 = 2.18 m > 2.0 m

The freeboard of the dam hereby determined as Hf = 2.20 m

Therefore the crest elevation is, EL250.50 + 1.00 + 2.20 = EL253.70 m

2) Crest Width

The crest width (b) is computed in the following equation.

$b = 3.6 H^{1/3} - 3.0$ (m)

H : hight of dam 14.1 m from the riverbed

$b = 3.6 \times 15.9^{1/3} - 3.0 = 6.05$ m

Therefore the crest width is determined as b = 6.0 m

The crest part shall be paved with laterite of over 40 cm in thickness and be given the gradient of 2:100 at upstream and downstream in order to protect against cracks and erosion.

3) Extra - banking

Most of settlement of foundation and embanking materials would take place during the time of the embanking work, so that very little settlement would occur after the completion.

A long-term post-completion settlement of an earth-fill dam is only 0.2 - 0.4% of the dam height, or 3-6cm, so no extra banking will be required.

4) Slope Gradient

Slope gradient shall be determined finally after stability analysis on the basis of determination of various design values from tests results of the foundation and embanking materials and predictable conditions of construction work.

USBR recommends the following slope gradient for homogenous type dams built on a stable foundation, in case of embanking materials of CL and ML,

Upstream slope gradient	1 : 3.0
Downstream slope gradient	1 : 2.5

5) Protection of Upstream Slope

The upstream slope of riprap in order to protect the embankment materials against erosion by wind wave or by fluctuation in water level and against the occurrence of cracks caused by the variation of weather.

a) Quality of the Riprap

Hard rocks of solid texture sufficiently resistent against weathering shall be used for riprap.

b) Size of Riprap and Thickness of Riprap

The masonry materials for riprap should be of sufficient weight and size immovable by wind wave.

The standard riprap should be of well graded distribution including grain size under 2.5 cm, with the mean diameter (D50) 30 cm, and the maximum diameter D50 x 1.5 against wave height of 0.6 - 1.2 m.

Even though the minimum thickness of riprap is 30 cm, the pitching thickness in the present work shall be made 40 cm, considering the rocks available and convenience in execution.

c) Filter

A filter zone of well graded materials shall be provided under the riprap as a protection of embankment soil against flow loss through the voids among the riprap rock.

Filter zone, 30 cm thick, shall be made with crushed stone of 0.5 mm - 9 mm in diameter or natural gravel.

6) Protection of the Downstream Slope

The downstream slope of the embankment shall be lined by grass-plot as a protection against erosion by rain and cracks caused by variation in weather.

And draining ditches shall be provided on the downstream slope vertically and laterally at the interval of 5 m in order to prevent its errosion presumably caused by concentrated rain water falling over it.

The sectional dimensions of the draining ditch shall be 40 cm x 40 cm and the inner side of it shall be filled with crushed stones.

(3) Measures against Seepage Water

1) Phreatic Surface

Phreatic surface is obtained by Casagrande's method.

The assumption, here, is that the permeability of the body of the embankment is anisotropic.

The cross-sectional view showing the saturation line shall be deformed by contracting the horizontal dimension by multiply the real value with kv/kh.

kv : vertical coefficient of permeability

kh : horizontal coefficient of permeability

The value of kv/kh in the case of compaction by tamping roller is averagely $\frac{1}{5}$.

$$\frac{kv}{kh} = \frac{1}{5} \quad \sqrt{\frac{kv}{kh}} = 0.447$$

The deformed cross-section is as shown in the Fig.2-3-5. If the point A is the pole and x is the horizontal axis and y the vertical axis, the standard parabola of the phreatic surface is given by the following equation.

$$y = \sqrt{2y_o x + y_o^2}$$
$$y_o = \sqrt{h^2 + d^2} - d$$
$$y_o = \sqrt{10.00^2 + 21.25^2} - 21.25 = 2.24 \text{ m}$$
$$y = \sqrt{2 \times 2.24x + 2.24^2} = \sqrt{4.48x \times 5.02}$$
$$x = 0 \text{ m} \qquad y = 2.24 \text{ m}$$
$$y = 0 \text{ m} \qquad x = -y_o/2 = -1.12 \text{ m}$$

x (m)	2.0	4.0	6.0	10.0	15.0	21.25
y (m)	3.74	4.79	5.65	7.06	8.50	10.00

For representing the actual phreatic surface, the standard parabolic line thus obtained is modified as follows.

The flow line lies at a right angle to the slope at the point of inflow, and Co point falls for an oblique distance of Δa to C point at the point of outflows.

The value of Δa varies depending on the angle of slope at face of outflow and can be obtained from the following equation.

$$a + \Delta a = \frac{y_o}{1 - \cos\alpha} = 123\ 51$$

$y_o = 2.24$ m

$$a + \Delta a = \frac{2.24}{1 + 0.557} = 1.44 \text{ m}$$

from the chart

$$C = \frac{\Delta a}{a + \Delta a} = 0.17$$

$\Delta a = 0.17 \times 1.44 = 0.24$ m

Thus, the phreatic surface is obtained by reconstructing the deformed cross-section into the original cross-section following the modification of the standard parabolic line.

2) Amount of Leakage

a) Amount of Leakage from the Dam Body

The amount of leakage is computed by using modified coefficient of permeability, on the assumption that the impervious zone of the dam body is anisotropic. The modified coefficient of permeability is obtained from the following equation.

$\bar{k} = \sqrt{kh\ kv}$

$\bar{k}$: modified coefficient of permeability

kh : coefficient of permeability in horizontal direction

kv : coefficient of permeability in vertical direction

from the laboratory permeability test kv = 5×10^{-7} cm/sec.

since $\frac{kv}{kh} = \frac{1}{5}$, kh = 5kv

$\bar{k} = \sqrt{5}\ kv = 1.12 \times 10^{-6}$ cm/sec.

$= 9.67 \times 10^{-4}$ m/day

The amount of leakage from the dam body can be obtained from the following equation.

$Qb = \bar{k}\ y_o\ L$

$\bar{k}$ = modified coefficient of permeability 9.67×10^{-4} m/day

yo = $\sqrt{h^2 + d^2} - d = 2.24$ m

L = profile length of the dam body 300 m

$Qb = 9.67 \times 10^{-4} \times 2.24 \times 300 = 1$ m^3/day

b) Amount of Leakage from the Foundation

On the basis of the analysis on piping which might take place (treated later), a blanket shall be laid on the upper stream of the dam body.

The amount of leakage through the blanketed foundation is computed as follows.

$$q_f = \frac{k\ h\ d}{Xr + Xd}$$

$$Xr = \frac{e^{2aX} - 1}{a(e^{2aX} + 1)}$$

$$a = \sqrt{\frac{k1}{t\ k\ d}}$$

q_f = amount of permeation through the foundation

Xr = effective seepage path of the blanket

Xd = width of the impervious zone of the dam body

t = thickness of the blanket 1.0 m

d = thickness of the foundation layer 10.5 m

k1 = coefficient of permeability of the blanket
1×10^{-6} cm/sec. = 8.64×10^{-4} m/day

k = coefficient of permeability of the foundation
1.5×10^{-3} cm/sec. = 1.30 m/day

h = depth of water above the blanket 9.5 m

X = length of the blanket 50 m

$$a = \frac{8.64 \times 10^{-4}}{1.0 \times 1.30 \times 10.5} = 7.96 \times 10^{-3}$$

$$2aX = 2 \times 7.96 \times 10^{-3} \times 50 = 0.80$$

$$e^{2aX} = e^{0.80} = 2.23$$

$$Xr = \frac{2.23 - 1}{7.96 \times 10^{-3} \times (2.23 + 1)} \doteq 47.8 \text{ m}$$

$$q_f = \frac{1.30 \times 10.5 \times 9.5}{47.8 + 68} = 1.12 \text{ m}^3\text{/day/m}$$

If the longitudinal length of the dam body is 300 m, the amount of leakage through the foundation is,

$$Qf = 1.12 \times 300 = 336 \text{ m}^3\text{/day}$$

c) Total Amount of Leakage

$$Q = Qb + Qf$$
$$= 1 + 336 = 340\ m^3/day$$

The criterion amount of allowable leakage is 0.05%/day.

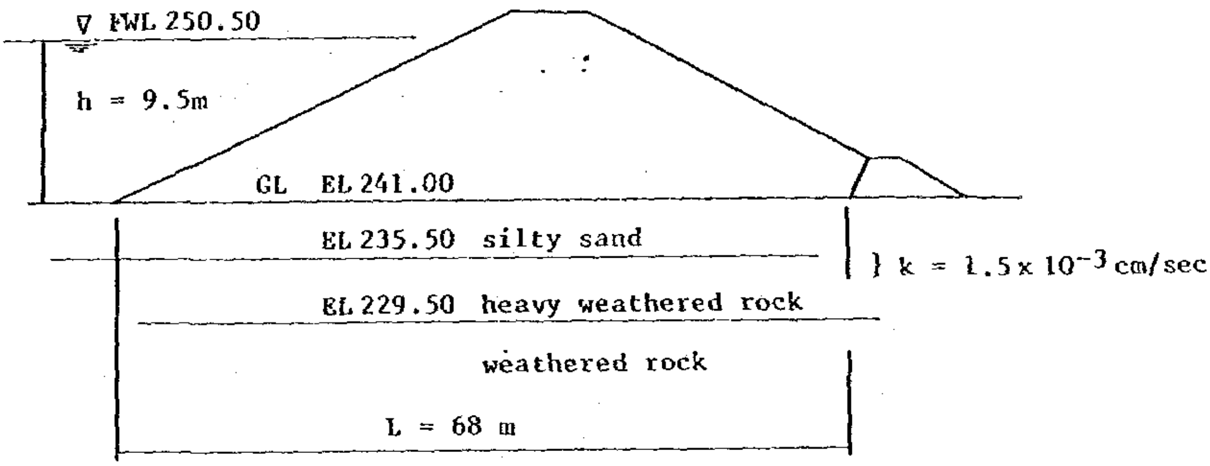
Effective storage amount = 4,500,000 m^3

Qa = 0.0005 x 4,500,000 = 2,250 m^3/day

Therefore, the amount of leakage is satisfactorily small.

d) Analysis of Possible Piping

The analysis of possible leakage is made by way of critical hydraulic gradient.



Analysis is made on possible piping in the loose silty sand with the N-value of 5-15 containing sand deposits at GL 0-5.5 m in the figure above.

critical hydraulic gradient (ic)

$$ic = \frac{Gs - 1}{1 + e}$$

Gs : specific gravity of soil particles known as

Gs = 2.66 from soil test

e : void ratio, know as e = 0.53 from soil test

$$ic : \frac{2.66 - 1}{1 + 0.53} = 1.08$$

hydraulic gradient (i)

$$i = \frac{h}{L}$$

L = Length of permeation route 68 m

h = difference in water level 9.5 m

$$i = \frac{9.5}{68} = 0.14$$

Safety factor (Fs)

$$Fs = \frac{ic}{i} = \frac{1.08}{0.14} = 7.7 < 8 - 12$$

The generally accepted safety factor is 8-12, and piping may occure if no particular measures are taken ("Shin-pen Doshitsu Rikigaku" by Fusayoshi Kawakami).

The works for the prevention of piping include sheet piles, curtain grouting, soil replacement by cut-offs, upstream blankets, etc., but it is decided to adopt cut-offs and upstream blankets in the present project on the basis of consideration on economy, convenience in execution, duration of the work, etc.

If the length of the blanket is 50 m, hydraulic gradient.

$$i = \frac{9.50}{68 + 50} = 0.08$$

safety factor

$$Fs = \frac{ic}{i} = \frac{1.08}{0.08} = 13.5 > 8 - 12$$

Therefore, the length of the blanket shall be 50 m.

Since the standard thickness of the blanket is 1/10 of the water head, the thickness of the conceived blanket shall be 1 m.

(4) Stability Analysis

1) Determination of Design Parameters

a) Embankment Material

Embankment construction will be conducted by the density control method. During the construction, density will be controlled under the following conditions:

The moisture content should be within ± 1.5% of its optimum and the dry density be 95% of its maximum. Design density has been determined from the results of the compaction test as follows:

Specific gravity	G =	2.67
Dry density	γd =	1.86 t/m^3
Optimum moisture content	w =	13.0%
Wet density	γt =	2.10 t/m^3
Saturated density	γsat =	2.16 t/m^3

A direct shear test has been made on the basis of the above mentioned moisture content through which the design strength has been determined. Assuming that the design strength is 70% of the empirical value, it results in the following:

Cohesion $C = 8.5 \times 0.7 \doteqdot 6.0\ t/m^2$

Angle of internal friction $\phi = 23 \times 0.7 \doteqdot 16°$

b) Foundation

The design density has been determined as follows:

Specific gravity	G =	2.66
Void ratio	e =	0.53

Dry density	γd =	1.74 t/m^3
Wet density	γt =	1.95 t/m^3
Saturated density	γsat =	2.09 t/m^3

A direct shear test was made on the undisturbed samples and the design strength has been determined. Assuming that the deisgn strength is 70% of the empirical value, the results are as follows:

Cohesion	C =	0 t/m^2
Angle of internal friction	ϕ =	28.5 x 0.7 ≑ 20°

c) Rock for Toe Drain

The design parameters of the rock for the toe drain were assumed as follows:

Specific gravity	Gs =	2.60
Wet density	γt =	1.65 t/m^3
Saturated density	γsat =	1.90 t/m^3
Cohesion	C =	0 t/m^2
Angle of internal friction	ϕ =	45°

2) Stability Analysis

A stability analysis of the finally adopted dam section (see Fig. 2-3-4) has been carried out for the three governing conditions viz. following fill completion (Case - 1), full reservoir (Case - 2) and rapid drawdown from maximum pool (Case - 3). Construction pore pressure is taken into account in Case - 1, the pore pressure of steady seepage in Case - 2 and 3.

As judging from a low coefficient of 5 x 10^{-7} cm/sec , it would be assumed that 50% of the uplift would be residual construction pore pressure.

Stability analysis has been made by electronic computer using the slice method. The allowable safety factor for all cases has been assumed to be 1.2. The results of stability analysis are illustrated in Figures 2-3-6, 2-3-7 and 2-3-8, and the computed safety factors are as follows:

		Upstream face	Downstream face
Case 1	Following fill completion	1.646	1.475
Case 2	Full reservoir	2.060	1.241
Case 3	Rapid drawdown	1.201	-

Therefore, this dam is deemed to be safe under any conditions.

2.3.5 Spillway Design

(1) Selection of Site

Spillway is proposed to be located at Point No.4 + 50.0 on the right bank side. This decision is based on considerations as to the topography and geology, plus safety and economy.

The bedrock is spreading to the depth of about 1.5 m below the ground surface providing 30 - 50N value in the neighbourhood of Point No.4 + 50.0. Furthermore, while rocks are scattering over on the left bank side, no rocks cannot be found on the right bank side. Judging from safety for the foundation of concrete structures and ease of construction, Point No.4 + 50.0 would be deemed most appropriate for the location of the spillway.

(2) Determination of Type

Overflow spillway (non-regulating type) following on open earth canal is adopted from the topographical, geological and O & M considerations, plus hydraulic and economic reasons, plus structural stability point of view.

(3) Determination of Spillway Scale

In this project, the reservoir area (1.1 km^2) is fairly large in comparison with the catchment area (21 km^2) and the reservoir will be equipped with non-regulating type spillway. Under these circumstances, an economical design would be sought by reducing the design overflow discharge by taking into account the flood regulating capacity of the spillway (the function of temporarily holding a part of the flood in the surcharge portion of the reservoir).

In designing a spillway, any combination of overflow depth and width can be thought in this project, however, floor follow-up calculation is made on the basis of the assumption that the overflow depth is set at 1 m from the economical point of view as well as of no restrictions given to such combination. As result, the overflow width is 22 m, and the overflow discharge, 44.0 m^3/sec. As this scale would be considered appropriate from the entire dam scale viewpoint, the overflow depth of the spillway, the overflow width and discharges are determined at 1 m, 22 m and 44.0 m^3/sec., respectively.

(4) Chute Portion

Chute portion is designed to have concrete structure because of functioning as a chute channel. The channel has 10 m width by narrowing its section after overflowing from the economical viewpoint. It has 1/20 gradient for the first 100 m while 1/2 gradient for next 8 m.

(5) Stilling Basin

Stilling basin is planned to follow chute portion so as to control velocity energy and lead to the river. Horizontal stilling basin III type with 9 m length is selected.

2.3.6 Design of Intake

(1) Selection of Site

Location of the intake work is determined on both bank sides according to the preliminary design done by RID, one of which was located at No.1 + 90.0 on the left bank side and the other at No.3 + 20.0 on the right bank side by taking into consideration the proposed conveyance canal route, and topographical and geological conditions.

Moreover, in order to discharge dead water staying below the bed height of the intake work during the end of dry season, conduit is provided at No.2 + 25.0.

(2) Determination of Type

The intake work has the following types: tower, conduit and valve type. Among these types, the tower and conduit types cause little effect on the dam body. As RID is desirous of adopting the valve type, it would be recommendable here. Steel pipe will be entirely armoured all round by ferro-concrete, and valve operation would be made on the downstream side.

(3) Amount of Intake Water

The maximum intake for subsistence water to be supplied to new villagers and irrigation water put together will be 0.6 m^3/sec , i.e. 0.3 m^3/sec on each bank side.

(4) Horizontal Conduit

Steel pipe (diameter: 500 mm) will be entirely armoured all round by ferro-concrete.

2.4 Construction Plan

2.4.1 General

Construction work shall be commenced in November, 1981, the forth-coming dry season and completed by June 1982, so that water will be stored during the next rainy season (May - October, 1982).

The detailed design should be prepared before commencement of the construction work, in consequence this study must be started immediately after the submission of this survey report.

All possible efforts, in this connection, should be made to manage necessary procedures with the Government of Thailand as smoothly and swiftly as possible.

Even though a preferable time of the commencement of the construction work would be October, it will be necessary to wait until the middle of November considering the time essential to making the detailed design and necessary procedures.

In the first instance the construction work shall be started by making temporary buildings like field office, labor's houses, material warehouse and in the next step it shall follow to repair and improve approach roads, as well as embankment of a cofferdam, and then the main works shall be started.

Land clearing for the dam sites and borrow-areas would be the first stage of the main works as stated in the schedule of construction work. In excuting this work the approval of the Department of Forestry is essential, so it is expected that necessary official procedures would be finished by the Government of Thailand.

To the possible extent use of construction machinery would be effective under the limitation of the construction period.

2.4.2 Construction Schedule

The optimum period of embankment works from November through March, as known from the precipitation record of the area would be fully manipulated for the maximum work progress, as a result the works could be finished by the latest deadline of April.

Almost all days are workable through the period from November to March, whereas in April commencement of rainy season would bring necessary measures against embankment works in case of schedule over-run.

Since the period of construction work is limited, it is necessary to embank the dam flowlessly without any idle time or delay.

Construction schedule of various works in the project is as shown in an attached sheet (Table 2-4-1) and the date of completion shall be June 30, 1982.

2.4.3 Construction Machinery

Amount of daily works are estimated from the construction schedule and then required construction machinery of suitable type and capacity as well as their combination would be determined.

List of Proposed Machinery

1. Excavator and Loader
   Bulldozer (15 - 20 t)   Tractor loader (1.0 - 1.5 m^3)
   Back hoe (0.45 - 1.0 m^3)   Motor scraper (1.0 - 1.5 m^3)
2. Truck
   Dump truck (6 - 12 t)
3. Motor grader and Compactor
   Motor grader (B = 3.0 m)   Tire roller (3 - 6 t)
   Tamping roller (3 - 6 t)   Tamper (100 kg.)
4. Road-sprinkler (6 t - 10 t)
5. Concrete mixer (0.3 - 1.0 m^3)
6. Disk harrow
7. Rock drill

2.5 Project Cost

2.5.1 General Description

The total cost of this project is estimated at ¥524,000,000 (₿50,400,000).

The project cost consists of construction cost and consultancy cost.

The construction cost consists of only dam embankment and related works (intake facility and spillway).

The consulting services comprise the detailed design and supervision for construction.

The proposed site of this project is identified to be government-owned land; some part is light jungle and the other field of cassavas, but it is deemed necessary to pay a certain amount of compensation for cultivators at the time of construction. This compensation considered as payable by the Government of Thailand is not included in this estimation.

The scope of construction work in this project is confined to dam consctruction to construction and does not include the canal system which would be preferably constructed by RID.

2.5.2 Construction Cost

In estimation of construction costs, unit costs were determined by referring to RID's standardized estimation and in addition past results on the same kind works in Thailand.

2.5.3 Detailed Design and Construction Supervision

These estimates are based on beginning the detailed design soon after the end of this study and completing it, if possible, by the end of October or, at latest, the beginning of November.

As for construction supervision, it is considered that 2 supervisors (3 ones for full-fledged period) should be resident at the job sites for $7\frac{1}{2}$ months, starting from the middle of November.

2.5.4 Project Cost

			¥	฿
A.	Construction Cost		¥	฿
	1.	Dam Body	252,996,000	24,326,550
	2.	Spillway	78,974,000	7,593,687
	3.	Intake Works	29,085,000	2,796,613
	4.	Outlet Conduit Pipe	7,988,000	768,104
	5.	Temporary Works	7,691,000	739,537
	6.	Indirect Construction Cost	22,594,000	2,172,509
		Sub-total	399,328,000	38,397,000
	7.	Miscellaneous Expenses	76,055,000	7,313,000
		Total	475,000,000	45,700,000

		¥	฿
B.	Detailed Design and Construction Supervision	49,000,000	4,700,000
	Grand Total	524,000,000	50,400,000

(No contingency is included in the above)

Exchange rates (as of 14 August, 1981)

US$1.00 = ฿22.6
US$1.00 = ¥235
฿1.00 = ¥10.4

2.6 Project Evaluation

2.6.1 Agricultural Production and Farm Income

(1) Agricultural Production

After the completion of the Meh Kah dam, the present one-shift cultivation of paddy 1,100 rai (176 ha) under Saitri 3-10, 4-8 and 4-9 would be converted into full season cultivation by supplying water, while upland crop cultivation land 460 rai (74 ha) out of 2,760 rai (440 ha) can be irrigated throughout a year, and as a result this land could be shifted to paddy field.

To raise the agricultural productivity by taking full advantage of this continuous supply of irrigation water to 1,560 rai (250 ha) in total, the local villagers would utilize the newly irrigable area according to the cropping pattern as shown in Figure 2-6-1. It is proposed to bring the entire area of the newly irrigable land under paddy cultivation during the rainy season, while during the dry season 40% (630 rai) of the total for paddy and the rest (930 rai) for upland crops.

The introduction of upland crops into 60% of the area during the dry season is to cope with shortage of irrigation water required for transplanting paddy in overall area, and thereby adding to farm income and conserving the soil fertility.

With the implementation of agricultural development for the benefited area, per rai yields of these farm products under the proposed cropping pattern are expected to be as follows:

a) Paddy (in rainy and dry seasons)	:	Paddy	500 kg/rai
b) Upland Crops	:	Peanuts	200 kg/rai
		Sesame	120 kg/rai
		Maize	300 kg/rai

(2) Farm Income

In the past, an average yield of paddy was about 300 kg/rai in the direct benefited area but it used to come down below half in the drought years. Thus, it may be assumed that an average yield of paddy would be allowed 250 kg/rai.

Accordingly, the present average yield would be computed as follows:

250 kg x 1,100 rai = 275 tons

฿ 3,300/ton x 275 tons = ฿ 907,500

After the completion of the dam, 1,560 rai of land comprising 1,100 rai for the existing entire paddy field and 460 rai for upland crop one would be surely applicable cultivation of HYV in every rainy season and expectedly would produce the average yield of 500 kg/rai.

During the wet season paddy production would amount to:

500 kg x 1,560 rai = 780 tons

฿ 3,300/ton x 780 tons = ฿ 2,574,000

Compared to that of without-project, it would bring the following results:

780 tons - 275 tons = 505 tons = ฿ 1,666,500

In the case of with-project it would increase by about 180%.

During the dry season, on the other hand, 630 rai of land would yield the production of 500 kg/rai, which is the same figure as for paddy in wet season, as shown below.

500 kg x 630 rai = 315 tons = ฿ 1,039,500

Crop - wise Benefits under 930 rai of Land

Crops	Area (rai)	Yield (kg/rai)	Production (ton)	Unit Price (฿ per ton)	Sales Value (฿ 1,000)
Peanuts	280	200	56.0	5,000	280
Sesame	280	120	33.6	10,000	336
Maize	370	300	222.0	2,500	555
Total	930				1,171

Assuming that the remaining 2,300 rai under upland crops is put under tapioca production and there would be no variation in its per rai yield and unit price, project benefits obtained from the benefited areas of the project would be concluded as follows:

Project Benefits by Agricultural Production Increase

Item	Paddy		Dry Season (Upland Crop)	Cassava	Total
	wet	dry			
Post - project	2,574	1,039.5	1,171	1,656	6,440.5
Pre - project	907.5	-	-	1,987.2	2,894.7
Benefit	1,666.5	1,039.5	1,171	331.2	3,545.8

In this evaluation costs for improved farming and operation and maintenance of infra-structures are not included, and furthermore benefit obtainable from productive use of oil seed cakes as feed-stuff and manure is neglected.

2.6.2 Project Benefits

Annual benefit of this project would be brought by increased agricultural production amounting about ฿ 3,546,000. However, for a more accurate estimation of its benefit, it is necessary to take into consideration cropping success percentage during the dry season (not 100% annually), that is:

Paddy Production in Dry Season	฿ 1,039,500 x 60% = ฿ 623,700	
Upland Crop Production "	฿ 1,171,000 x 70% = ฿ 819,700	

Accordingly, the annual benefit would amount to:

Paddy Production in Wet Season	฿ 1,666,500	
Paddy Production in Dry Season	฿ 623,700	
Upland Crop Production "	฿ 819,700	
Tapioca Production "	฿ 331,200	(due to cultivable land decrease)
Total	฿ 2,778,700	

2.7 Urgent Matters

2.7.1 Matters to be Settled Prior to the Commencement of Construction Works

The Government of Thailand is requested to take full consideration settled the following matters promptly, since it would be impossible to start the projected construction works even after the completion of the official international procedures between Thailand and Japan if they are left unsettled.

(1) Approval of cutting trees in the dam site and the area of future submersion.

(2) Cutting trees in the area of future submersion.

(3) Making compensation for farm crops and all other kinds of compensations for the dam site and the area of future submersion.

(4) Offering the land required for making field office, laborers houses machinery and material yard and other facilities for the construction work.

(5) Receiving agreement by the inhabitants concerned regarding the road reconstruction work for making an access road.

2.7.2 Note for Construction Work

There can be some change in the schedule and specification of the construction work at the time of execution, since the short period of survey necessarily limited the number of test drilling and test pits for data collection.

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FIGURES

Table 2-2-1 Climatological Data (Surin) for the Period 1951 - 1975

Temperature (°C)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	24.4	26.8	29.2	30.0	29.3	28.4	28.0	27.7	27.4	26.9	25.4	24.0	27.3
Ext. Maximum	36.6	38.2	40.8	41.6	39.7	38.8	37.4	37.1	36.7	35.8	36.2	35.8	41.6
Ext. Minimum	6.4	11.0	11.0	15.2	20.0	19.8	19.6	20.0	19.0	16.3	11.9	8.2	6.4

Relative Humidity (%)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	63.0	61.0	60.0	65.0	74.0	78.0	79.0	81.0	83.0	79.0	74.0	68.0	72.0
Mean Maximum	87.7	85.2	83.3	85.6	90.5	93.4	92.9	93.9	95.3	92.6	91.0	89.3	90.1
Mean Minimum	43.4	43.2	41.8	45.7	55.2	62.0	63.1	65.5	68.2	66.3	57.9	49.6	55.2

Climatological Data (Surin) for the Period 1951 - 1975

Evaporation (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean-Piché	143.1	140.3	163.4	145.3	110.8	83.1	80.6	69.0	59.0	81.5	102.1	125.0	1,303.2
Mean-Pan	202.6	194.4	229.6	218.0	207.9	178.1	188.1	164.8	145.3	179.4	188.2	192.9	2,289.3

Rainfall (mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Mean	2.0	11.0	30.6	84.1	175.4	159.2	190.8	194.4	276.3	132.8	22.3	1.6	1,280.5
Mean rainy days	0.7	2.1	4.3	8.4	14.3	17.6	17.8	19.6	20.8	11.6	3.5	0.6	121.3
Daily Maximum	12.8	57.7	40.1	108.9	106.3	114.4	97.6	94.5	104.5	132.1	39.6	19.5	132.1
Day/Year	26/54	12/70	24/64	12/68	25/51	12/70	18/61	6/58	28/73	6/60	14/66	26/66	6/60

Remark: Evaporation 1. Piché 1959 - 1975
2. Pan 1961 - 1975

Table 2-2-2 Monthly Diversion Water Requirements

1,000 m^3

Month / Year	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Total Annual	Total May ~ Oct.
1971	477	42	12	29	779	12	279	534	157	528	657	757	4,263	1,153
1972	328	212	12	29	934	12	142	410	166	528	762	572	4,107	1,341
1973	658	12	12	29	659	12	62	429	166	509	508	700	3,756	786
1974	610	245	45	12	834	12	25	334	166	452	565	614	3,914	1,173
1975	477	167	12	12	909	12	137	286	166	528	746	528	3,980	1,249
1976	229	70	59	12	895	12	12	439	166	528	762	715	3,899	1,060
1977	572	100	12	17	759	12	209	286	166	528	762	667	4,090	1,109
1978	429	12	12	12	820	12	395	486	166	528	762	766	4,400	1,263
1979	324	95	12	12	942	12	420	534	166	528	533	738	4,316	1,493
1980	572	12	12	12	800	12	12	486	166	515	704	620	3,923	860
Average	468	97	20	18	833	12	169	422	165	517	676	668	4,065	1,149
Cropping Calender	Upland Crop (2) 150 ha					Wet Season Paddy 250 ha				Dry Season Paddy 100 ha / Upland Crop (1) 150 ha				

Table 2-2-3 Area of Irrigation Required per 100 ha

N	--	--	--	2.5	2.5	--	--	--	0.6	3.8	0.6	--
LP	--	--	--	--	100	--	--	--	--	50	50	--
G	100	50	--	--	50	100	100	87.5	12.5	12.5	87.5	100
LP	50	--	--	--	--	--	--	--	50	50	--	50
G	100	100	87.5	12.5	--	--	--	--	12.5	87.5	100	100

N: Nursery LP: Land preparation (or puddling) G: Growing

Table 2-2-4 Monthly Rainfall at the Project Site (Lahan Sai--Meh Kah Dam)

in millimeter

Year \ Month	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Total: Rainy Season	Total: Dry Season	Total: Annual
1971/72	61.9	133.3	212.4	25.4	170.9	224.7	85.8	0.0	10.0	0.0	31.2	2.4	852.5	105.5	958.0
1972/73	102.3	65.5	415.3	33.0	79.0	361.2	129.0	38.0	0.0	0.0	0.0	55.0	1083.0	195.3	1278.3
1973/74	10.2	199.2	280.3	32.1	243.4	282.9	154.1	33.5	0.0	8.4	74.0	18.3	1192.0	144.4	1336.4
1974/75	24.1	51.8	77.3	192.2	137.0	243.2	166.8	64.5	0.0	35.2	57.5	42.8	868.3	224.1	1092.4
1975/76	60.9	84.3	221.9	276.6	91.6	204.8	130.0	79.0	0.0	0.0	3.7	66.5	1009.2	210.1	1219.3
1976/77	130.2	122.8	69.4	113.4	100.7	287.9	266.0	30.3	0.0	0.0	0.0	14.0	960.2	174.5	1134.7
1977/78	35.0	111.2	135.0	98.6	184.0	198.9	107.4	67.2	0.0	0.0	0.0	27.7	835.1	129.9	965.0
1978/79	75.3	273.6	122.6	220.7	146.9	316.2	48.3	14.0	0.0	0.0	0.0	0.0	1128.3	89.3	1217.6
1979/80	103.5	112.5	155.9	117.9	73.5	307.4	39.5	0.0	0.0	0.0	66.9	*8.3	806.7	178.7	985.4
1980/81	35.1	171.9	436.1	171.2	156.7	*204.4	*217.0	14.0	0.0	*6.3	15.5	*41.8	1357.3	112.7	1470.0
Total	638.5	1326.1	2126.2	1281.1	1383.7	2631.6	1343.9	340.5	10.0	49.9	248.8	276.8			
N	10	10	10	10	10	10	10	10	10	10	10	10			
Average	63.9	132.6	212.6	128.1	138.4	263.2	134.4	34.1	1.0	5.0	24.9	27.7	1009.3	156.6	1165.9

Notes: Rainy Season (May ∿ Oct.) Dry Season (Nov. ∿ Apr.),

*(Data at Ban Kruat) x 0.617 + 46.846 (Rainy Season) or (Data at Ban Kruat) x 0.811 + 6.272 (Dry Season)

Table 2-2-5 Monthly Effective Rainfall

in millimeter

Year \ Month	Apr.	May	Jun.	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Annual Paddy	Annual Upland
1971/72	46	100	(120) 159	19	(120) 128	(120) 169	64	0	8	0	23	2	718	622
1972/73	77	49	(120) 200	25	59	(120) 200	97	29	0	0	0	41	777	617
1973/74	8	(120) 149	(120) 200	24	(120) 183	(120) 200	116	25	0	6	56	14	981	729
1974/75	18	39	58	(120) 144	103	(120) 182	(120) 125	48	0	26	43	32	818	727
1975/76	46	63	(120) 166	(120) 200	69	(120) 154	98	59	0	0	3	50	908	748
1976/77	98	92	52	85	76	(120) 200	(120) 200	23	0	0	0	11	837	677
1977/78	26	83	101	74	(120) 138	(120) 149	81	50	0	0	0	21	723	676
1978/79	56	(120) 200	92	(120) 166	110	(120) 200	36	11	0	0	0	0	871	665
1979/80	78	84	117	88	55	(120) 200	30	0	0	0	50	6	708	628
1980/81	26	(120) 129	(120) 200	(120) 128	118	(120) 153	(120) 163	11	0	5	12	31	976	803
Total	479	(870) 988	(1020) 1345	(795) 953	(950) 1039	(1200) 1807	(882) 1010	256	8	37	187	208		
Average	48	(87) 99	(102) 135	(80) 95	(95) 104	(120) 181	(88) 101	26	1	4	19	21	834	691

Note: ( ) -- Effective rainfall of upland field

Table 2-2-6 Computation Sheet of Net Water Requirement for Wet Season Paddy

Item		Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Annual
Cropping Calender						Wet Season Paddy								
① Evapotranspiration	(mm/day)	3.6	3.2	2.6	2.6	2.4	2.4	2.7	2.6	2.6	2.8	3.3	3.8	
	(mm/mon)	107	98	78	79	73	70	84	78	80	85	92	116	1,040
② Crop Factor		-	-	-	1.00	1.13	1.30	1.33	1.20	1.10	-	-	-	-
③ Crop Consumptive Use	(mm/mon)	-	-	-	79	82	91	112	94	88	-	-	-	546
④ Percolation	(mm/mon)	30	16	15	16	16	15	16	30	31	31	28	31	275
⑤ Net Water Requirement	(mm/mon)	-	-	-	95	98	106	128	124	119	-	-	-	670
⑥ Water Requirement for Land Preparation	(mm)	-	-	-	-	200	-	-	-	-	-	-	-	
⑦ Net irrigation Area	(ha/100ha)	-	-	-	3	(100) 53	100	100	88	12	-	-	-	
⑧ Weighted NWR	(mm/mon)	-	-	-	3	252	106	128	109	14	-	-	-	612

Note: ⑤ is not including water Requirement for Land Preparation
⑦ ( ) is Land Preparation Area
⑧ = ⑤ x ⑦/100 + ⑥ x ⑦ ( )/100

Table 2-2-7 Computation Sheet of Net Water Requirement for Dry Season Paddy

Item		Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Annual
Cropping Calender												Dry Season Paddy		
① Evapotranspiration	(mm/day)	3.6	3.2	2.6	2.6	2.4	2.4	2.7	2.6	2.6	2.8	3.3	3.8	
	(mm/mon)	107	98	78	79	73	70	84	78	80	85	92	116	1,040
② Crop Factor		1.30	1.10	-	-	-	-	-	-	1.00	1.00	1.25	1.35	
③ Crop Consumptive Use	(mm/mon)	139	108	-	-	-	-	-	-	80	85	115	157	684
④ Percolation	(mm/mon)	30	16	15	16	16	15	16	30	31	31	28	31	275
⑤ Net Water Requirement	(mm/mon)	169	124	-	-	-	-	-	-	111	116	143	188	851
⑥ Water Requirement for Land Preparation	(mm)	-	-	-	-	-	-	-	-	-	200	200	-	
⑦ Net Irrigation Area	(ha/100ha)	100	50	-	-	-	-	-	-	1	(50) 16	(50) 88	100	
⑧ Weighted NWR	(mm/mon)	169	62	-	-	-	-	-	-	1	119	226	188	765

Note: ⑤ is not including water Requirement for Land Preparation
⑦ ( ) is Land Preparation Area
⑧ = ⑤ x ⑦/100 + ⑥ x ⑦ ( )/100

Table 2-2-8 Computation Sheet of Net Water Requirement for Upland Paddy

Item		Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Annual
Cropping Calender		Upland Crop								Upland Crop				
① Evapotranspiration	(mm/day)	3.6	3.2	2.6	2.6	2.4	2.4	2.7	2.6	2.6	2.8	3.3	3.8	
	(mm/mon)	107	98	78	79	73	70	84	78	80	85	92	116	1,040
② Crop Factor		0.7	0.9	0.75	0.55	-	-	-	-	0.55	0.7	0.9	0.75	
③ Crop Consumptive Use	(mm/mon)	75	88	59	43	-	-	-	-	44	60	83	87	539
④ Percolation	(mm/mon)	30	16	15	16	16	15	16	30	31	31	28	31	275
⑤ Net Water Requirement	(mm/mon)	105	104	74	59	-	-	-	-	75	91	111	118	737
⑥ Water Requirement for Land Preparation	(mm)	40	-	-	-	-	-	-	-	40	40	-	40	-
⑦ Net Irrigation Area	(ha/100ha)	(50) 100	100	88	12	-	-	-	-	(50) 12	(50) 88	100	(50) 100	-
⑧ Weighted NWR	(mm/mon)	125	104	65	7	-	-	-	-	29	100	111	138	679

Note: ⑤ is not including water Requirement for Land Preparation
⑦ ( ) is Land Preparation Area
⑧ = ⑤ x ⑦/100 + ⑥ x ⑦ ( )/100

Table 2-4-1 Construction Schedule (Meh Kah Dam)

Work Item		Q.T.Y.	1981			1982					
			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.
Engineering Service Detailed Design Supervision			D/D		Supervision						
Preparatory Works											
Dam Body	Jungle clearing	$67,600m^2$									
	Stripping	$58,200m^3$									
	Excavation	$13,300m^3$									
	Embankment	$161,400m^3$									
	Riprap	$10,400m^2$									
	Toe Drain	$4,200m^3$									
Spillway	Earth Works	$14,000m^3$									
	Concrete Works	$2,100m^3$									
	Reprap	$260m^2$									
Intake	Earth Works	$2,900m^3$									
	Concrete Works	$290m^3$									
	Steel pipe ϕ500	182m									
	Value	1									
Outlet Conduit	Earth Works	$2,200m^3$									
	Concrete Works	$75m^3$									
	Steel pipe ϕ200	105m									

Fig. 2-1-1 Geological Map

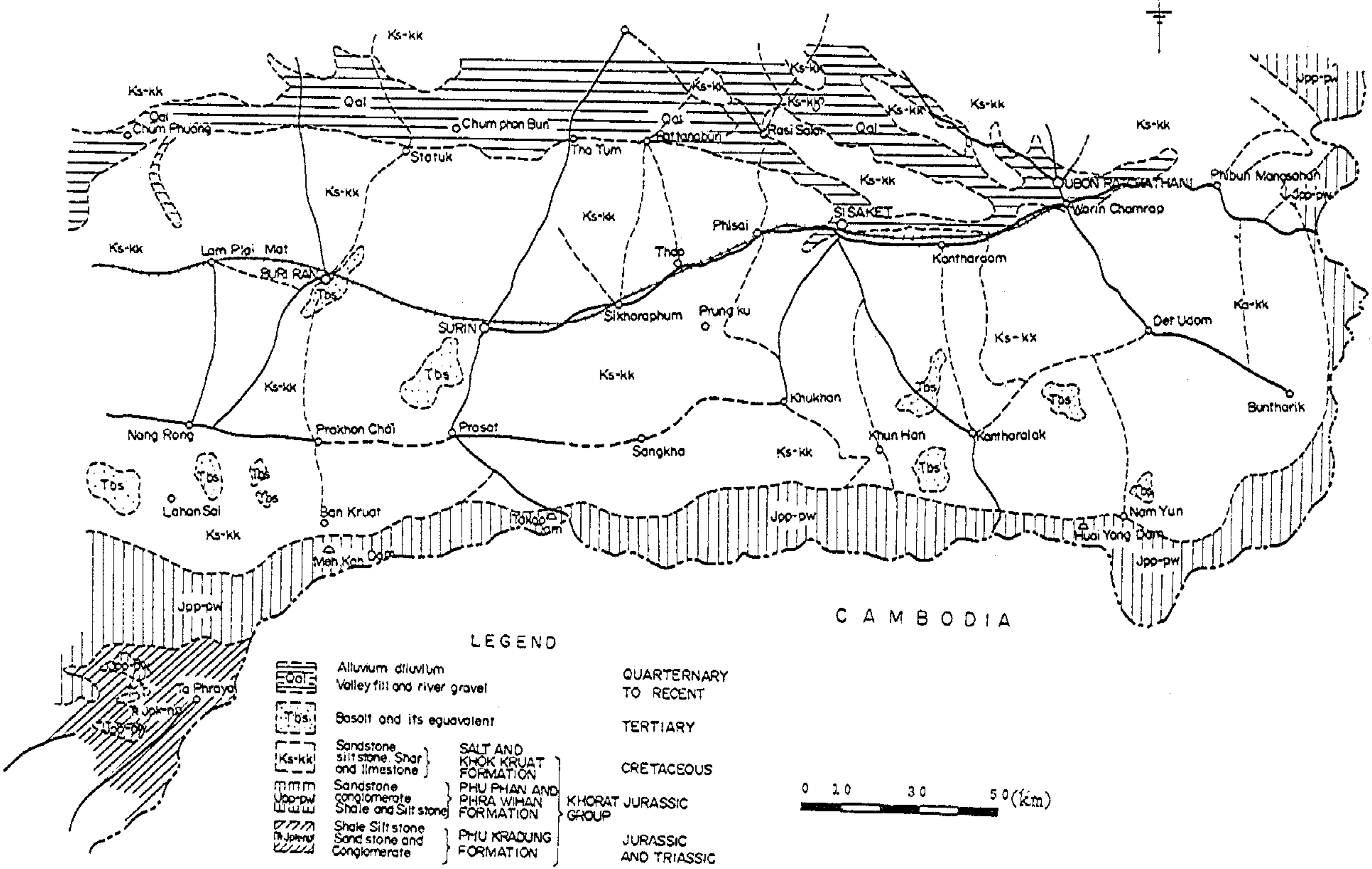
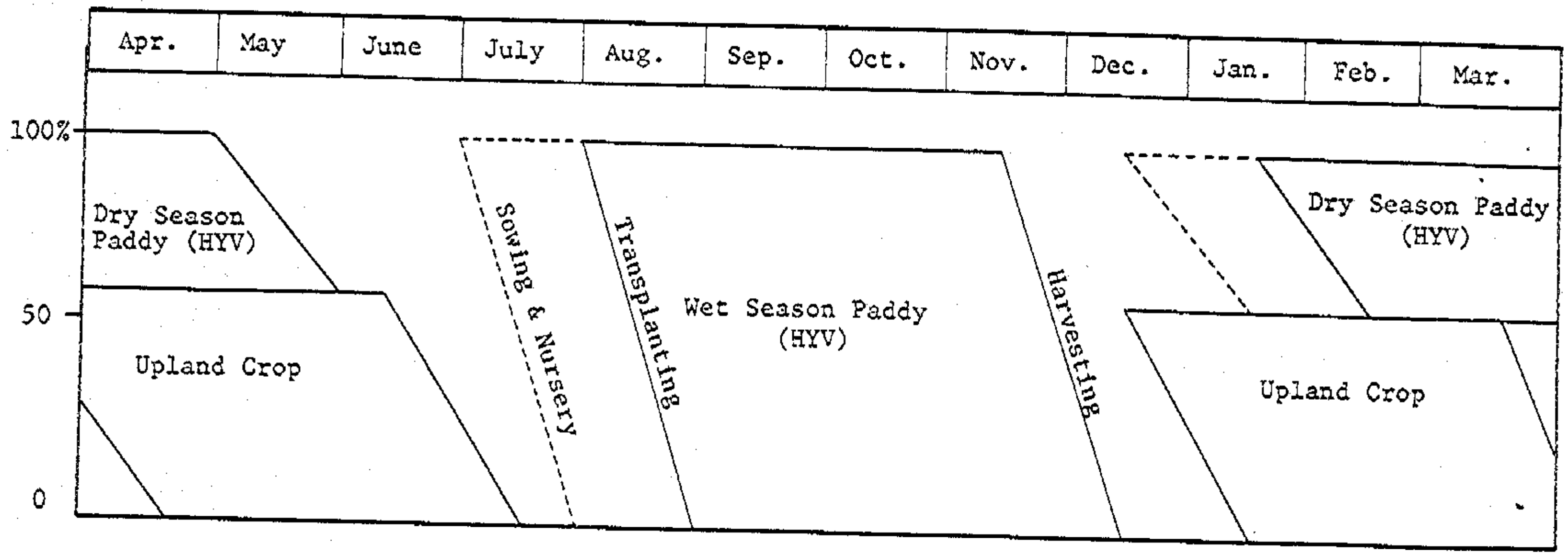


Fig. 2-2-1 Proposed Cropping Calender



Note: Nursery area is equivalent to 5% of cultivation area

Fig. 2-2-2 Water Balance Study (Meh Kah Dam)

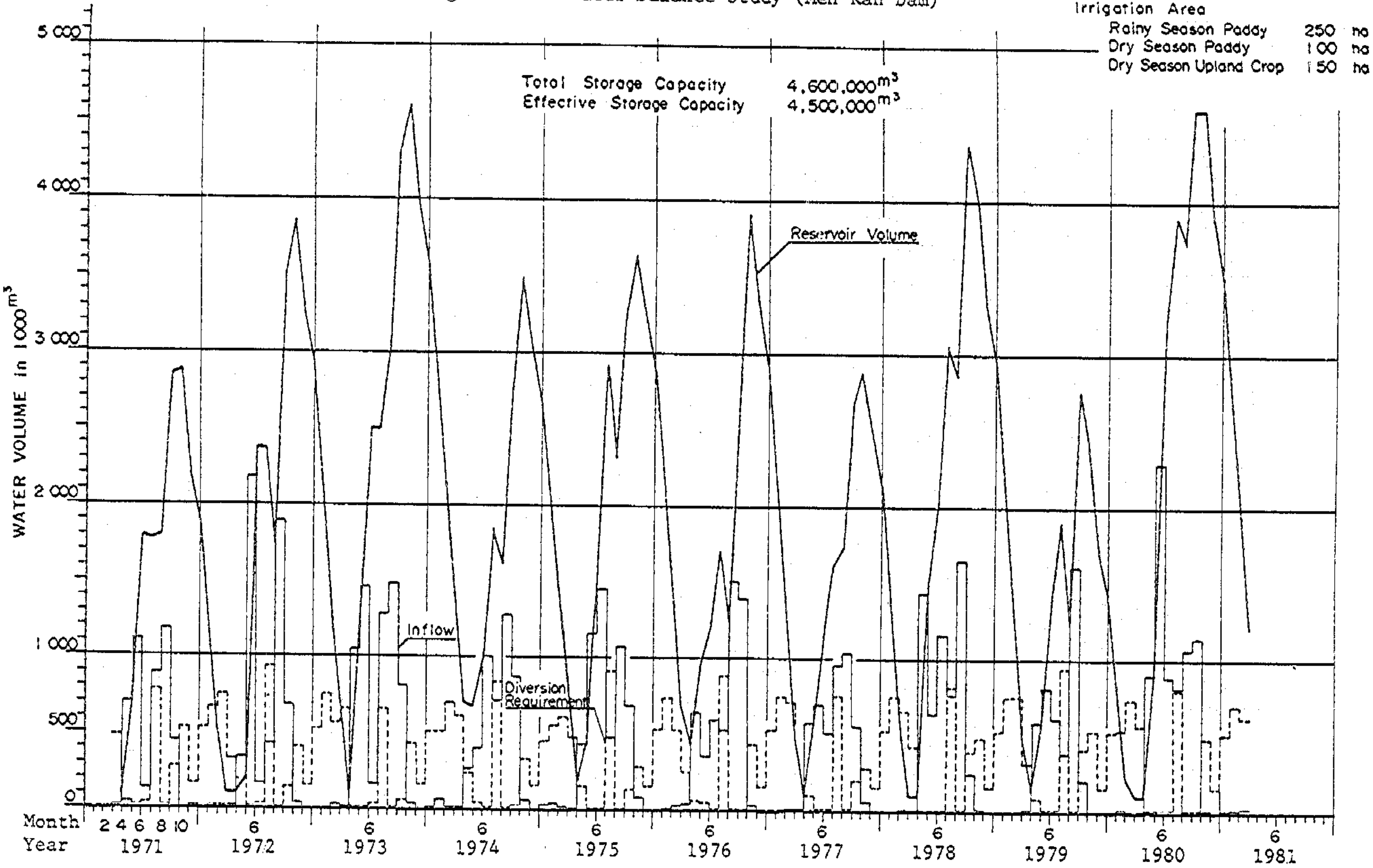
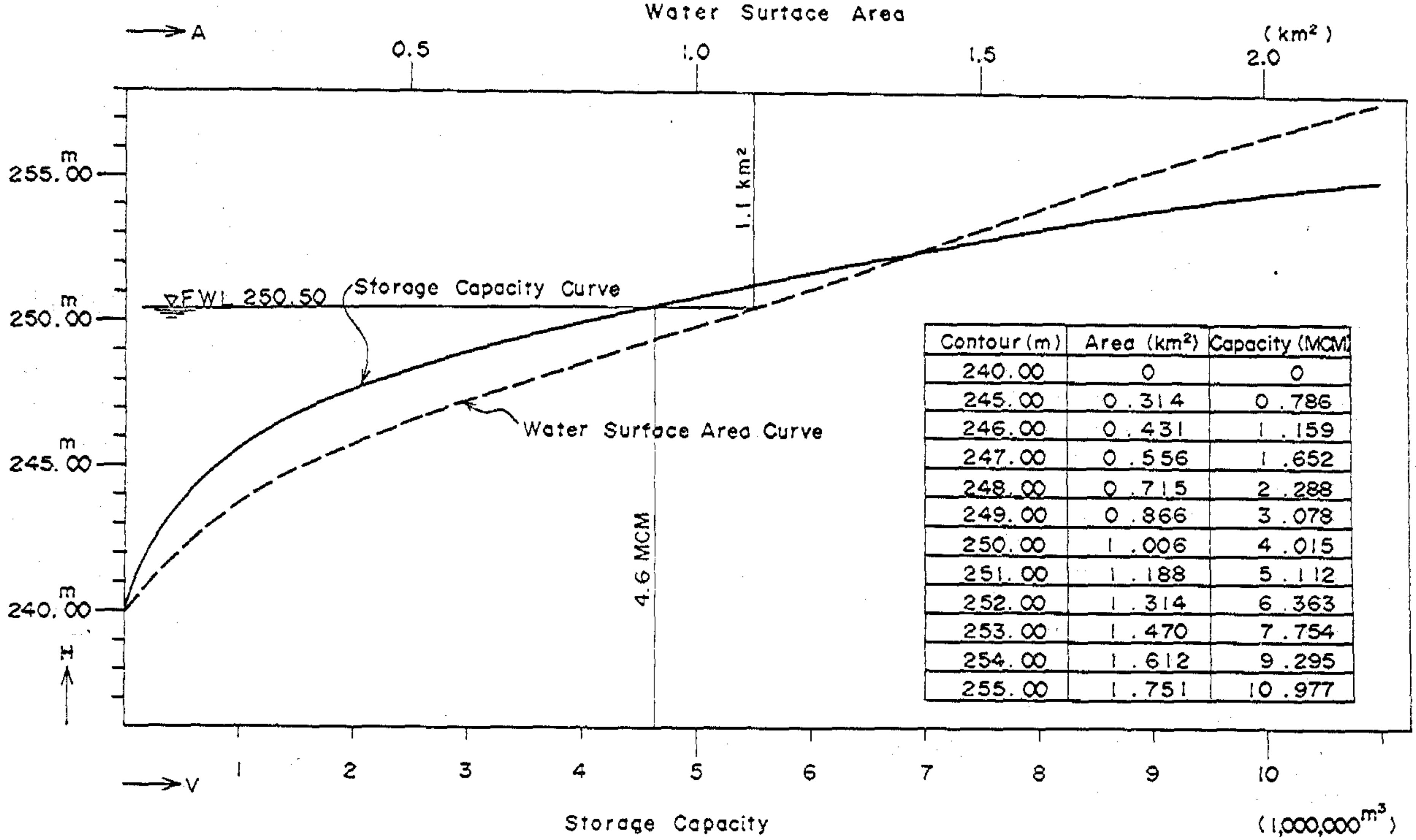


Fig. 2-3-1 H-V, H-A Curve (Meh Kah Dam)



Contour (m)	Area (km²)	Capacity (MCM)
240.00	0	0
245.00	0.314	0.786
246.00	0.431	1.159
247.00	0.556	1.652
248.00	0.715	2.288
249.00	0.866	3.078
250.00	1.006	4.015
251.00	1.188	5.112
252.00	1.314	6.363
253.00	1.470	7.754
254.00	1.612	9.295
255.00	1.751	10.977

Fig. 2-3-2 Location Map of Soil and Geological Investigation for Meh Kah Dam

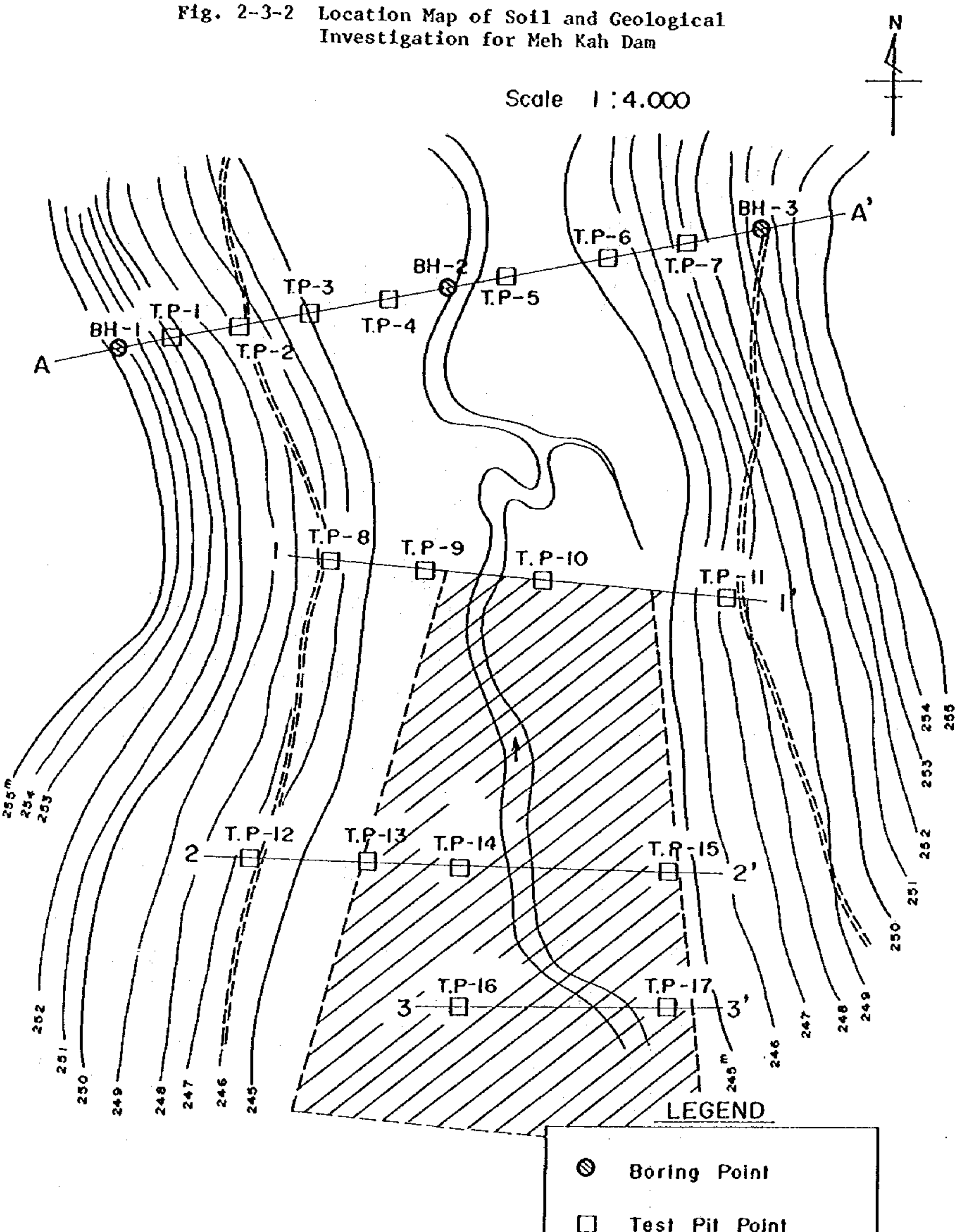


Fig. 2-3-3 Geological Section of the Meh Kah Dam Axis

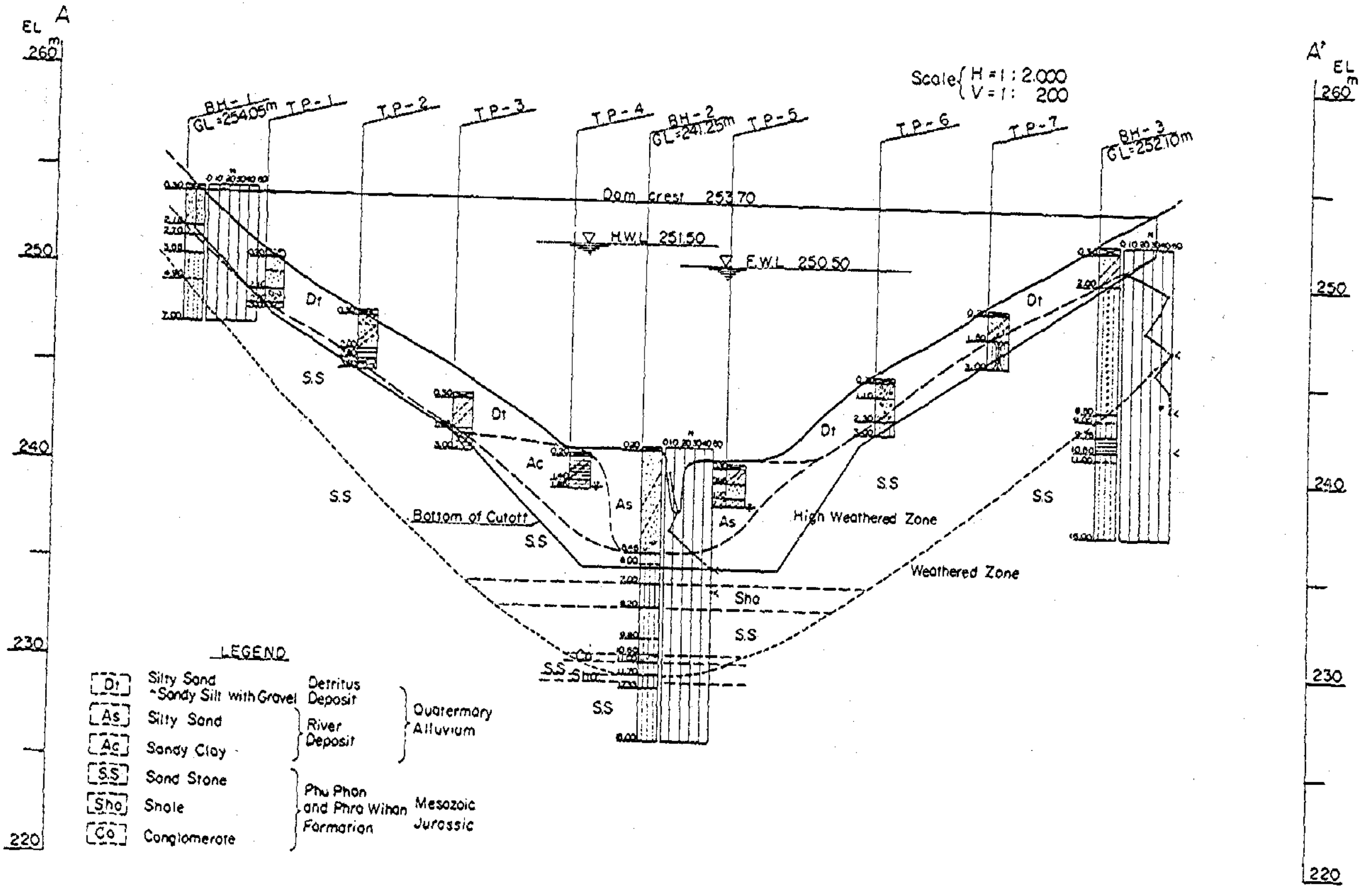


Fig. 2-3-4 Typical Section (Meh Kah Dam)

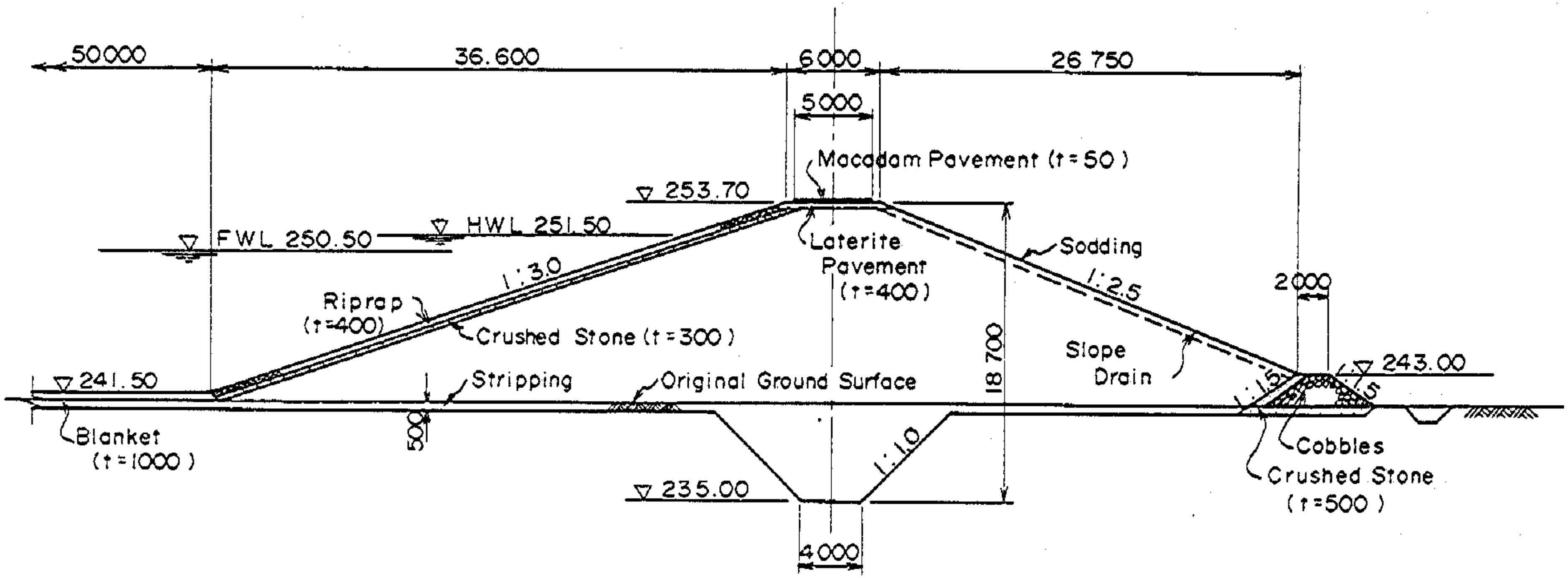


Fig. 2-3-5 Phreatic Surface

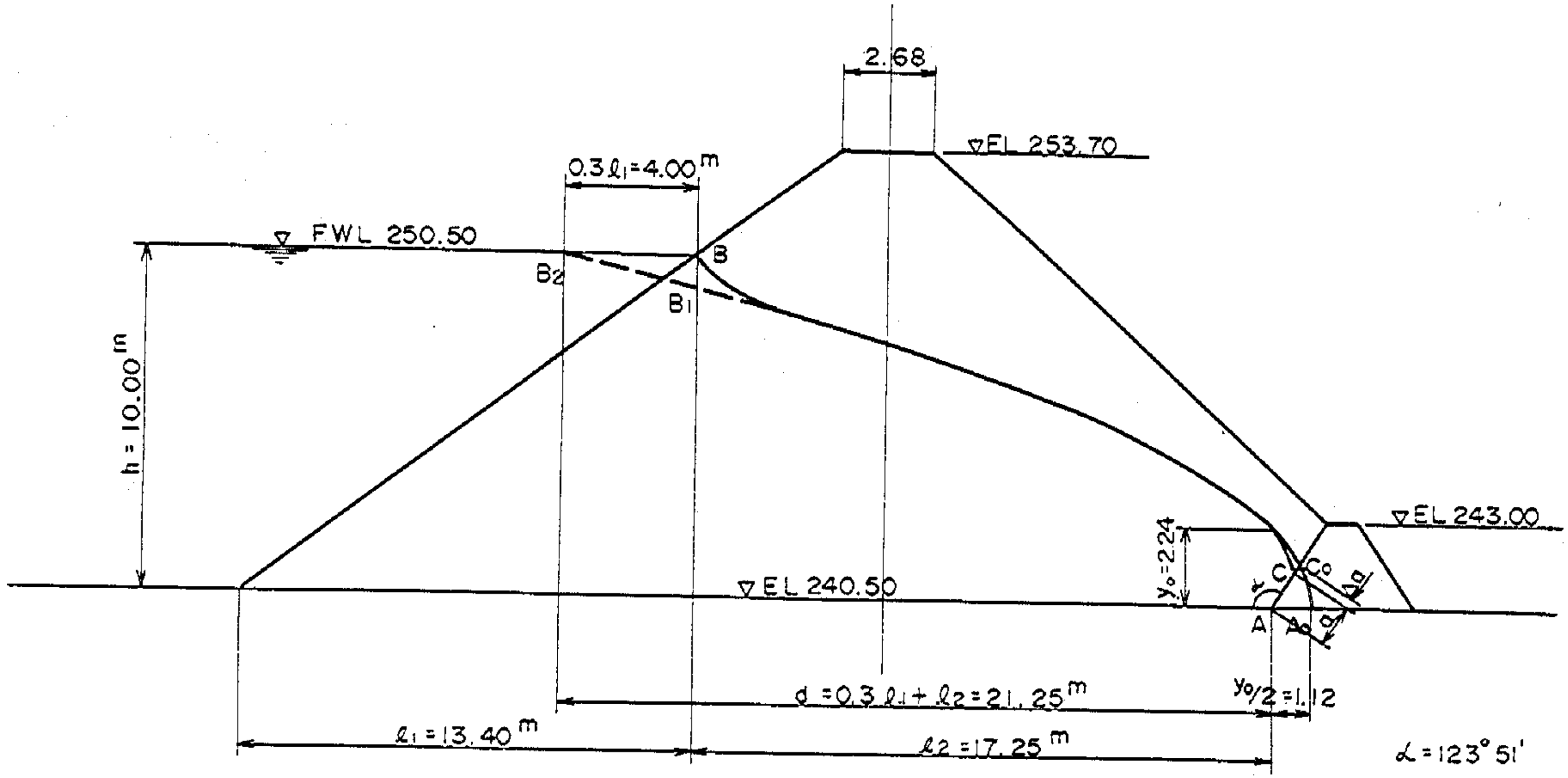


Fig. 2-3-6 Slope Stability Analysis (Meh Kah Dam)

Case 1 Fill Completed Condition

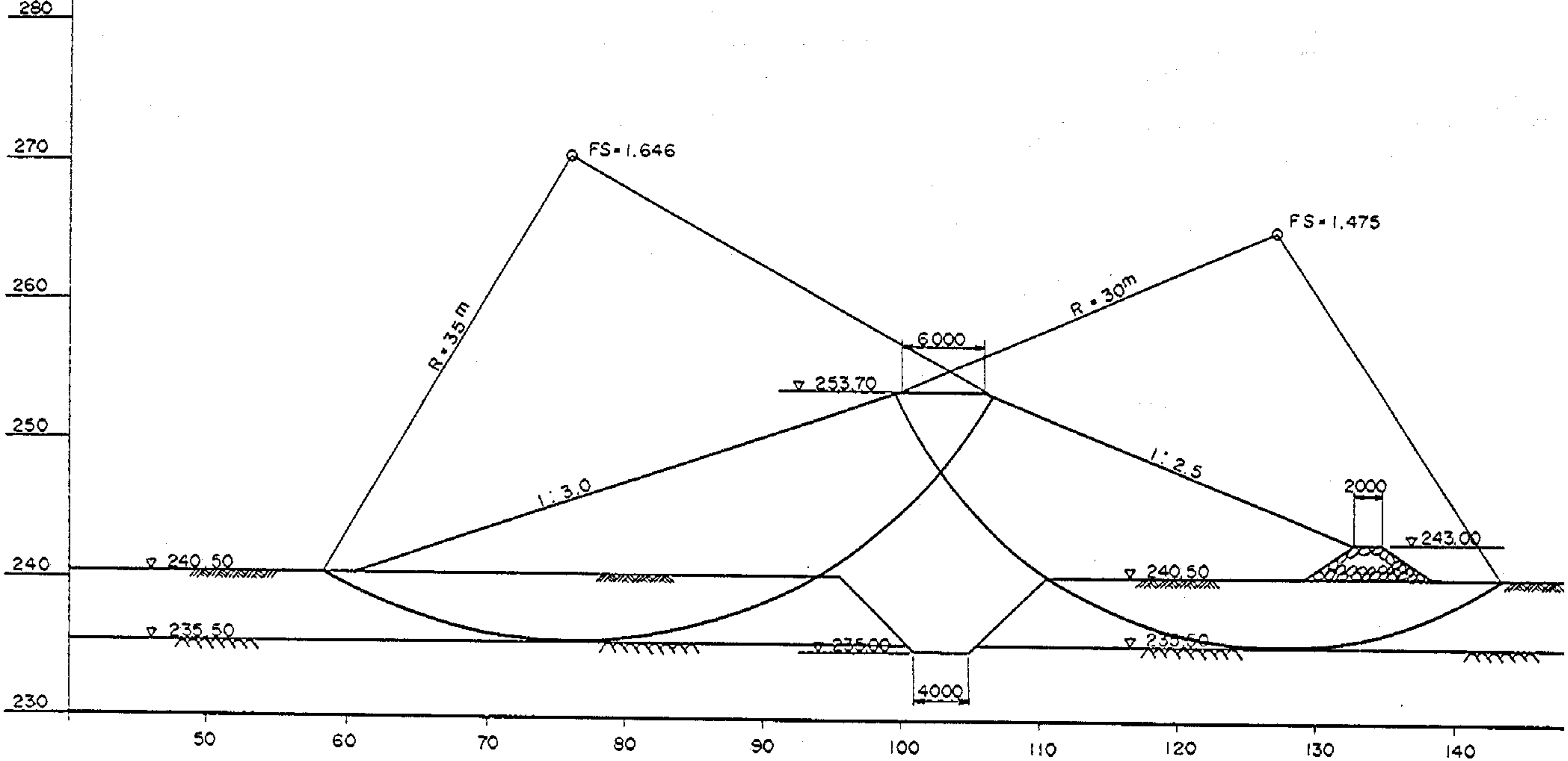


Fig. 2-3-7 Slope Stability Analysis (Meh Kah Dam)

Case 2 Full Reservoir Condition

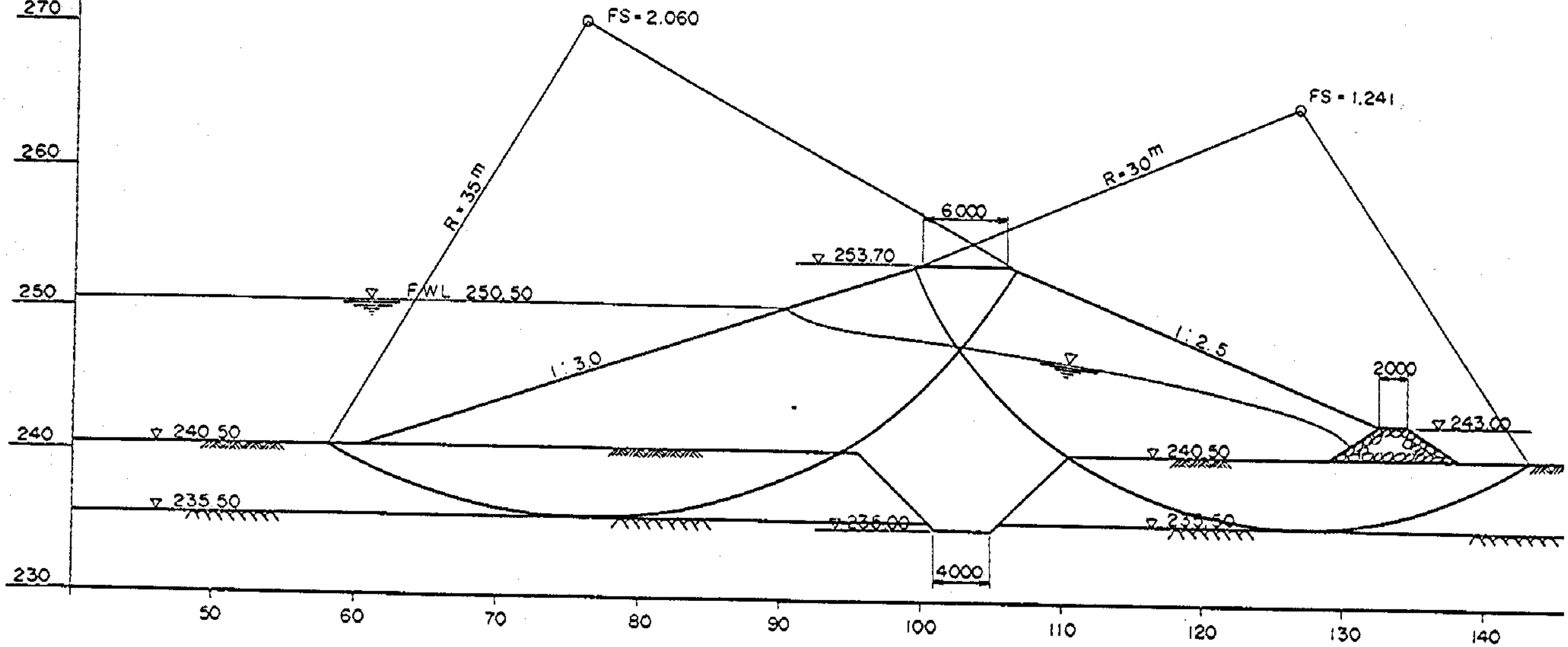


Fig. 2-3-8 Slope Stability Analysis (Meh Kah Dam)

Case 3 Rapid Drawdown Condition

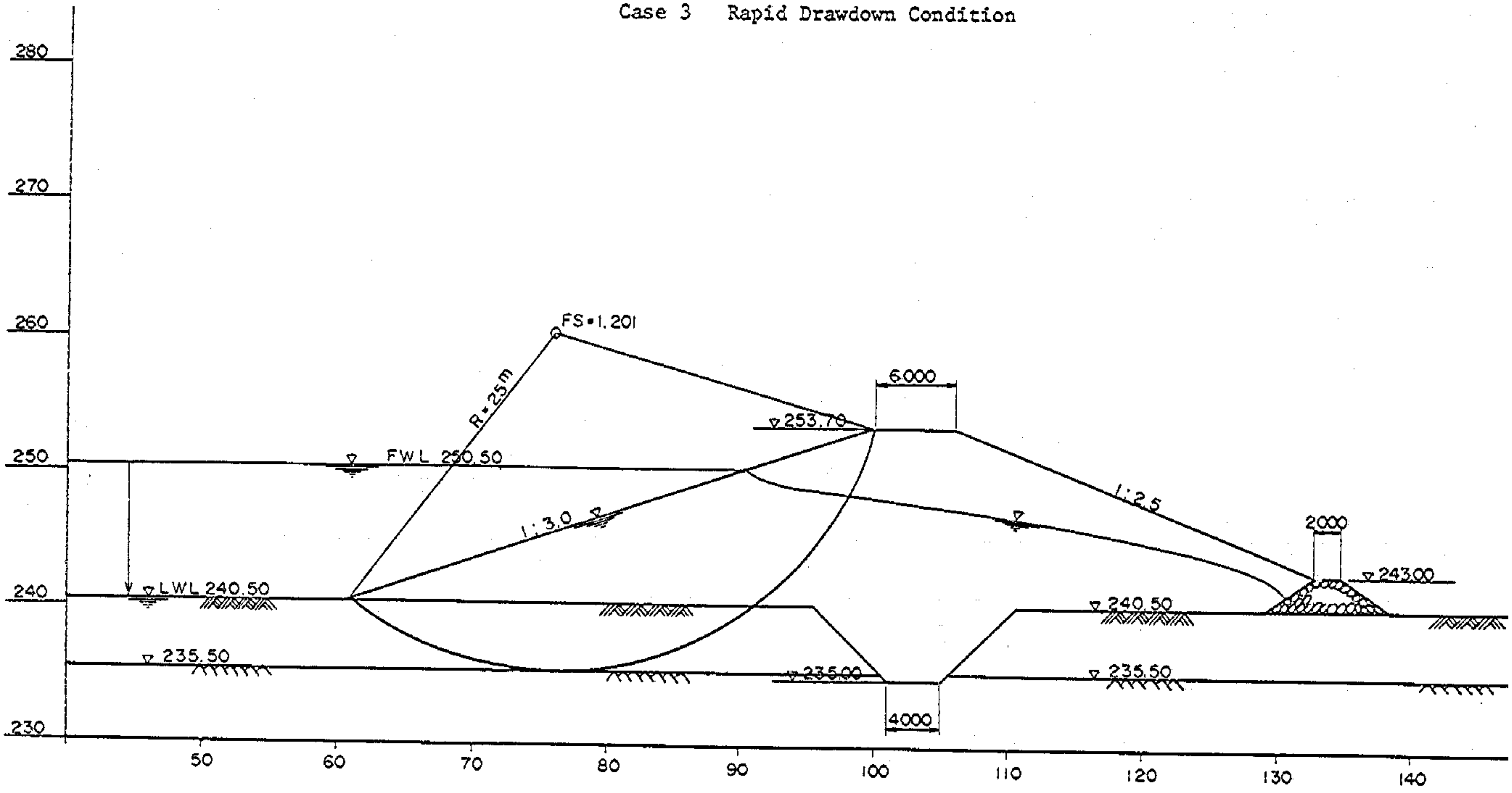
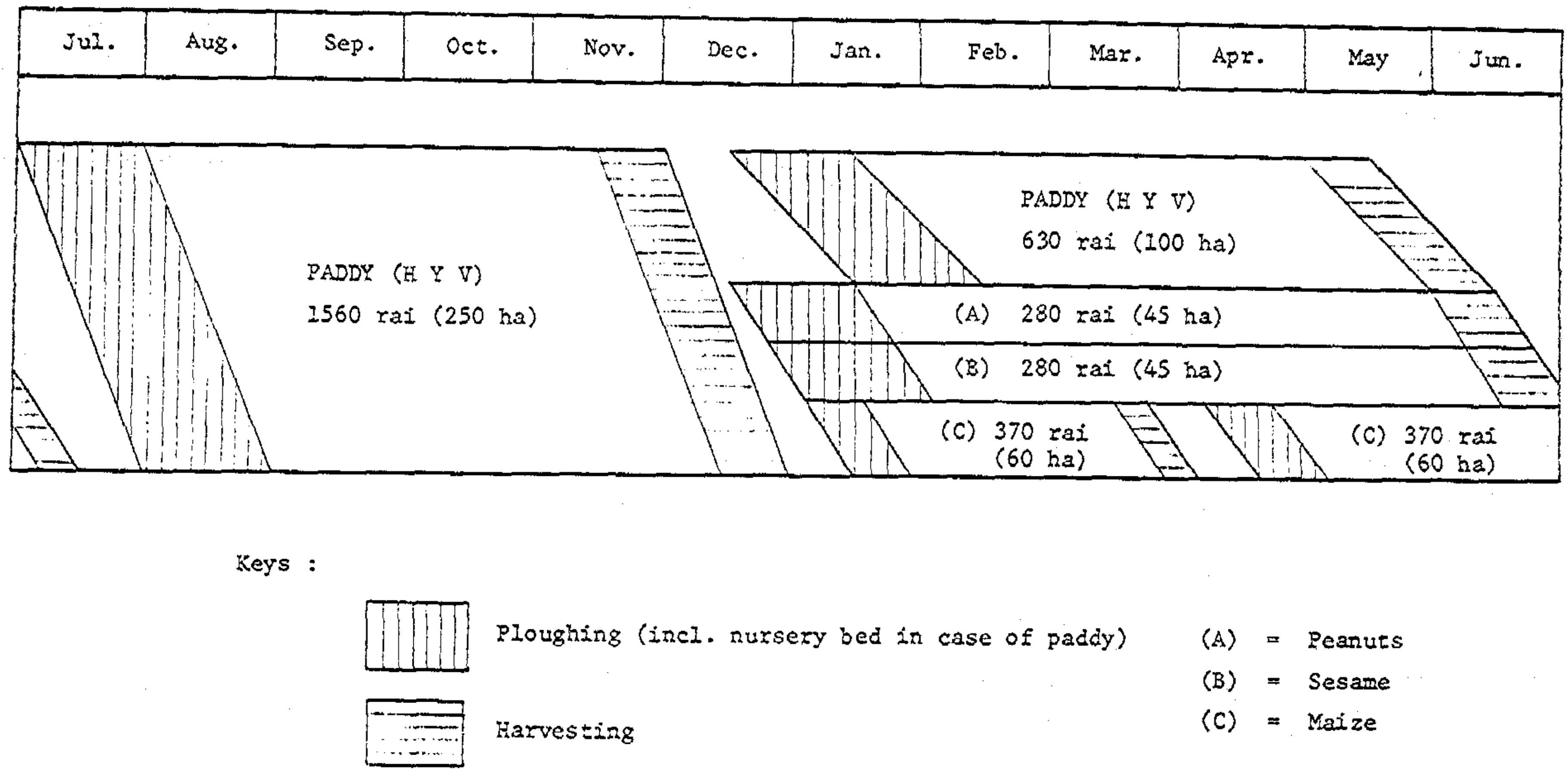
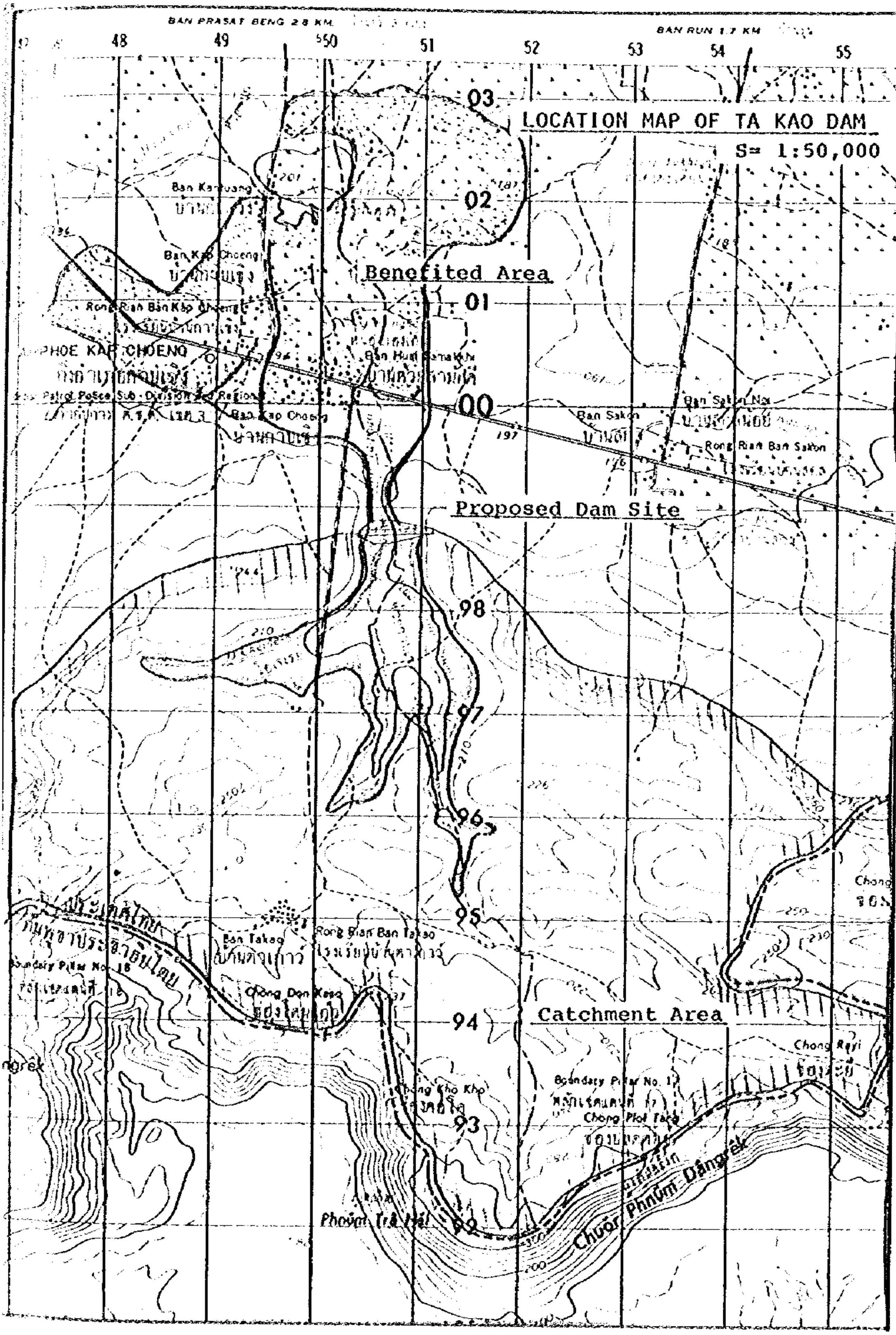


Fig. 2-6-1 Proposed Cropping Pattern





LOCATION MAP OF TA KAO DAM
S= 1:50,000
Benefited Area
Proposed Dam Site
Catchment Area
BAN PRASAT BENG 2.8 KM
BAN RUN 1.7 KM
48
49
50
51
52
53
54
55
03
02
01
00
98
97
96
95
94
93
92
Ban Kap Choeng
Ban Sakon
Rong Rian Ban Sakon
Ban Takao
Rong Rian Ban Takao
Chong Don Kaeo
Phnum Tra Hal
Chuor Phnum Dangrek
Chong Rayi
Chong Plot Tako
Boundary Pillar No. 17
Boundary Pillar No. 18
Ban Hua Samakkhi

CHAPTER 3 TAKAO DAM PROJECT

3.1 Project Area and Present Condition

3.1.1 Location

The project area is located in Amphoe Kap Choeng Province, Surin District, in the eastern region of the Kingdom of Thai.

National Highway No.214 between Surin and the Cambodian border runs at 1.5 km downstream from the dam site situating at 58 km point from Surin and at 28 km point from Prasat.

The boundary for the catchment area of the dam is adjacent to the Cambodian border and at 5 km point from the dam site.

Ban Ta Kao village accompanied by 70 households that had been located in the upstream of the dam site was shifted due to Cambodian civil war and now is being built as a new village under reclamation in the vicinity of Ban Kap Choeng village.

3.1.2 Topography

The catchment area is bordered by ridges of 250-300 m height in its boundary, forming the border between Cambodia and the eastern region of the country; and a flat plain is spreading to the north from the boundary.

The catchment area spreads in between 200-300 m high ridges and 190-210 m high lowlands, being penetrated by the Huai Ta Phao River which is joined by the small rivers such as the O Cheroe River, the Huai Kap Choeng (Kadon) River, etc.

The benefited area is extending between 180 m and 195 m in elevation on flat paddy and upland fields, spreading on both banks of the Huai Ta Phao River.

The Huai Ta Phao River seems to be perenial flow, however, there was not found running flow during the survey period.

3.1.3 Geology and Soils

No description can be made here for this project area due to having almost homogeneous geological composition as for Meh Kah dam approximately 60 km far from this area. For your reference, see 2.1.3.

3.1.4 Present Socio-Economic Conditions

(1) Direct and Indirect Benefited Area

Benefited area after the completion of the project would involve four villages in the downstream of the Takao dam proposed under the project, i.e. three villages of Ban Kap Choeng, Ban Huai Samakkhi and New Ban Takao in the Amphoe Kap Choeng Province for direct benefited ones, while one village of Ban Kantuang for indirect one.

Area, Household and Population under Benefited Area

Name of Villages	Total Area (rai)	Total Household	Total Population
Ban Kap Choeng	1,747	266	1,195
(New) Ban Takao	1,000	70	413
Ban Huai Samakkhi	4,500	105	507
Ban Kantuang	4,000	115	565
Total	11,247	556	2,680

(2) Direct Benefited Area

General conditions of the direct benefited area are shown on the following Table:

General of Direct Benefited Area

Name of Villages	Total Area (rai)	Cultivated Area (rai)			Other Area (rai)	Buffalo
		Paddy	Upland Crops	Total		
Ban Kap Choeng	1,747	1,560	110	1,670	77	256
Ban Huai Samakkhi	4,500	2,025	890	2,915	1,585	111
(New) Ban Takao	(1,000)	(450)	(500)	(950)	(50)	45
Total	6,247 (7,247)	3,585 (4,035)	1,000 (1,500)	4,585 (5,535)	1,662 (1,712)	412

( ) in table implies under reclamation.

Under fruitless condition, this area comprising three villages has been reclaimed gradually by setters form the other provinces : 20% of villagers is Laotian.

Under the influence of the Cambodian civil war, Ban Takao village which had located at 1 km distance from the border was fully shifted to the vicinity of Ban Kap Choeng village where a new village project is on-going.

The area is put under paddy cultivation on a plain. Paddy field remains to produce rice once a year during the rainy season and the upland one mainly devoted for cultivation of tapioca, sesame and hemp. Average family member is about five.

(3) Fundamental Structure of Agriculture

Long term reclamation for this area, which involves old villages, has been brought about by farm population increase together with settlers from the northern Roi region. Accordingly, Ban Kap Choeng village, one of the old villages, has been well developed, by reclaiming the most part of arable lands to paddy cultivation land whose landownership by the villagers are authorized while in Ban Huai Samakkhi village, newly formed by population increase, the majority of the villagers have no landownership but cultivation right.

For Ban Taka village, the villagers who were newly shifted from other villages are making efforts to land reclamation.

Average farm-household has 20 rai of land, consisting of 15 rai for paddy and 5 rai for upland crops, while the biggest, about 50 rai of land.

(4) Agro-Economic Analysis

Average per rai yields of main crops are : 300 kg of paddy, 1,800 kg of tapioca and 225 kg. of hemp. While paddy is primarily for the cultivator's own consumption except for small surplus for marketing, upland crops is exclusively for marketing; the farm-gate price of paddy is ฿ 3.3/kg. that of tapioca, ฿ 0.4/kg. and that of hemp, ฿ 5/kg.

For paddy cultivation, the average farmers need no cash expenditure as they plough with their own buffaloes, apply their own manure and little chemical fertilizer to the field, and transplant and harvest with their own family labour and neighbourhood cooperation. For upland crop cultivation, no cash expenditure is also required due to all farming works to be done by their own family labour.

Cash income per average farm-household would amount to ฿ 15,000/year; ฿ 10,000 from paddy and ฿ 5,000 from upland crops. Under almost the same situation, of the region covered by Meh Kah dam, farm income would become lower than those estimation, if the specific draught year's income can be clarified.

3.2 Water Utilization Plan

3.2.1 Water Resources

The source of water for this project is the Huai Tak Phao River which is joined by the small tributaries. The dam proposed in the project would be constructed in the downstream of such confluence. The catchment area is 34.6 km^2.

Since there is no river flow data available for the Huai Tak Phao River, discharge would be calculated according to rainfall data.

3.2.2 Hydrology

The project area, which belongs to Surin District, the eastern part of the Kingdom of Thai, is located at 5 km. point from the border of Cambodia and torming a semi-flat plain with approximately 200 m above sea level.

As meteorological data are not available within the project area, those recorded at the following three stations adjacent to the area are adopted as the basic figures for the project.

Ban Kruat	55 km.	from the	dam site
Prasat	30 km.	"	"
Surin	50 km.	"	"

Mean monthly and daily rainfall data recorded since 1952 have been collected as many as possible. For reference, meteorological records at Surin station are shown in Table 3-2-1.

(1) Rainfall

The maximum probable daily rainfalls estimated by adopting the Gumbel Method at each station are as follows:

Maximum Probable Daily Rainfall (mm)

Probability	Station		
	Ban Kruat	Prasat	Surin
1/10	135	125	122
1/25	159	146	142
1/50	177	163	157
1/100	195	179	172
1/200	213	195	186

(2) Flood Discharge

The maximum flood discharge is estimated at 138 m^3/sec. by adopting the design rainfall of 146 mm./day and the 1/25 probable daily rainfall recorded at prasat Station, situation nearby the dam site and having plentiful rainfall data available.

In designing spillway, the flood follow-up calculation based on the assumed overflow width and depth of the spillway was made by taking into account its flood regulating capacity (the function of temporarity holding a part of the flood in surcharge portion of a reservoir). Consequently, spillway capacity, overflow width and depth are determined at 31.35 m^3/sec., 20 m and 0.85 m respectively.

3.2.3 Water Requirements

The reservoir water can be utilized not only as subsistence water to villages but also for irrigating arable lands in and around the project area.

(1) Subsistence Water

Water requirements are calculated on the basis of:

Number of Family	560 families
Projected Family Constituent	7 persons per family
Population	3,920
Unit Amount of Water Supply	200 l/day/head
Total Amount of Water Supply	784 m^3/day = 23,520 m^3/m

As there is no standard on subsistence water supply to villagers, the losses due to conveyance and diversion presumed 10% of total amount of water supply are estimated at 871 m^3/day (10.1 l/sec.).

As the reservoir water would be brought to the supplying area through earthern canal, conveyance loss is computed at 20% during the rainy season (May - October), while 30% during the dry season (November - April).

Rainy Season	871/(1-0.2) = 1,090 m^3 /day
Dry Season	871/(1-0.2) = 1,250 m^3 /day

The reservoir water annually diverted to the villages would amount to 1,090 x 184 days + 1,250 x 181 days = 426,810 m^3

Therefore, approximately 430,000 m^3 of the reservoir water per year would be utilized.

(2) Irrigation Requirements

The following assumptions would be made in calculating irrigation requirements:

1) Irrigation Efficiency

Irrigation efficiency is computed by taking into consideration season-wise losses due to application and conveyance in field as follows:

	Rainy Season	Dry Season
Field Efficiency	75%	75%
Conveyance Efficiency	80%	70%
Integrated Efficiency	60%	52.5%

2) Effective Rainfall

Effective rainfall as shown in Table 3-2-4 has been calculated by the following method on the basis of monthly rainfall (R, see : Table 3-2-3).

Different Crops	Effective Rainfall	Upper Limit (mm./month)
Paddy	0.75 R	200
Upland Crops	0.75 R	120

3) Cropping Pattern

Irrigation area under assumption of approximately 80% of irrigation success percentage would be determined in accordance with the proposed dam storage capacity of 8.6 million m^3. Irrigation requirements can be made on the basis of the cropping pattern proposed under the project as shown in Figure 3-2-1. Moreover, the determination of the design storage capacity for the dam is mentioned in Chapter Dam Plan.

4) Calculation of Irrigation Requirements

Irrigation requirements can be computed as follows:

(i) Net Water Requirements (N.W.R.)
= Crop Consumptive Use + Percolation
+ Water Requirements for Field Preparation

(ii) Net Irrigation Requirements (N.I.R.)
= Net Water Requirements - Effective Rainfall

(iii) Diversion Water Requirements (D.W.R.)
= Net Irrigation Requirement/Diversion Efficiency

Crop consumptive use is estimated at multiplying the evapotranspiration value by crop consumption ratio of the Penman Method. Percolation in field would be assumed to be 0.5 mm./day during the rainy season, while 1.0 mm./day during the dry season, Water requirements for land preparation, etc. would be assumed to be 200 mm. for paddy and 40 mm. for upland crops. Irrigation requirements for 1968-1977 have been estimated at the above-mentioned assumptions.

(3) Diversion Requirements

Upon condition that proposed irrigation area is 350 ha, and effective rainfall is determined by use of rainfall of the last 10 years, average monthly diversion requirements can be estimated as totalling both irrigation and subsistance water requirements.

Average Monthly Diversion Requirement

May	153,000m^3	Nov.	619,000m^3
Jun.	35,000	Dec.	247,000
Jul.	36,000	Jan.	744,000
Aug.	1,148,000	Feb.	1,088,000
Sep.	44,000	Mar.	954,000
Oct.	336,000	Apr.	684,000
Rainy Season	1,752,000	Dry Season	4,336,000

Anual Total 6,088,000m^3

3.2.4 Water Balance

(1) Assumptions

1) Inflow

Rainfall water in the catchment area of 34.6 km^2. would be collected into the proposed reservoir and its inflow to the reservoir can be estimated by adopting the following formulas:

Rainy Season (May - October)
Inflow = Monthly Rainfall x 34.6km^2. x 25%

Dry Season (November - April)
There would be no inflow available.

However, rainfall water on the water surface of the reservoir is assumed to be 100% effective.

Inflow = Monthly Rainfall x Water Surface in the Previous Month x 100%

2) Losses in the Reservoir

Evaporation

Evaporation would be computed by multiplying 80% of the pan evaporation value by the surface water area in the previous month.

Leakage

Leakage is assumed to be 0.05% per day (1.5% per month) of the reservoir storage.

Leakage = Storage Capacity in the Previous Month x 1.5%

3) Rainfall Records

Rainfall records obtained at Prasat have been replenished by those available at the neighbouring stations. Monthly rainfall records from 1968 to 1977 have been used as vasical data for this project.

(2) Calculation of Water Balance

Water balance for irrigation area from 1968 to 1977 is analized on the assumptions that the irrigable area is 350 ha; the whole area is under paddy cultivation during the rainy season, while 150 ha for paddy and 200 ha under upland crops during the dry season.

1) Effective Storage Potential and Water Requirements

Year	Effective Storage Potential	Water Requirements (Out flow)
1968/69	7,395,000 m^3	6,135,000 m^3
1969/70	5,398,000	6,347,000
1970/71	7,832,000	6,177,000
1971/72	2,886,000	6,585,000
1972/73	5,294,000	5,842,000
1973/74	5,279,000	5,952,000
1974/75	4,971,000	5,823,000
1975/76	6,685,000	6,292,000
1976/77	9,437,000	5,412,000
1977/78	4,430,000	6,347,000
Average	5,960,700	6,091,200

Note: The design storage capacity of the dam is 8,600,000 m^3

2) Months of Overflowing from Spillway

3 months out of 120 months

3) Effective Utilization Ratio

(Effective Storage Potential - Overflow)/
Effective Storage Potential = 96%

4) Months of Water Shortage

9 months out of 120 months

5) Crop-wise Irrigation Success Percentages (see : Figure 3-2-1)

Rainy Season	Paddy (350 ha)	10/10	= 100%
Dry Season	Paddy (150 ha)	4/9	= 44%
" "	Upland Crop (1) (200 ha)	9/10	= 90%
" "	Upland Crop (2) (200 ha)	4/9	= 44%

3.3 Dam Plan

3.3.1 Dam Axis

The survey for this dam was said to be conducted about 10 years ago, nevertheless records at that time could not only be obtained but also its executive agency is unknown.

As there was no available topographic map which covers the neighbouring area of the dam site, the proposed dam axis was determined in terms of a topographic map (s = 1/50,000) through field investigation.

Since such a dam centre - line as to have been marked 10 years ago was luckily found on the right side abutment, the investigation was carried out over the nearby area which extends form the marked line as a centre. As a result of the investigation, it is located on the narrowest portion of a valley and the valley widens towards its upstream and downstream. From topographical point of view, therefore, this point would be deemed as the most appropriate site for the proposed dam.

3.3.2 Storage Capacity and Type

In determining storage capacity, the following two aspects was taken account:

(i) Effective design storage potential which will be equalized to the balance between the inflow and reservoir loss (evaporation plus leakage), and

(ii) Comparative study on the original plan worked out by RID.

Item (i)

Inflow would be computed by the following formula under consideration of possibility only in rainy season.

Inflow = Monthly Rainfall x 34.6 km^2. x 0.25

The effective design storage potential was calculated as shown in the following Table on the assumption that run-off rate is set at 0.70. The rate, which can be defined as a ratio or losses due to evaporation and leakage to inflow, was determined at 0.70 by taking into consideration comparative study for the values of effective design storage potential in terms of water balance calculation.

Estimation of Effective Design Storage Capacity

Year	Inflow (1,000 m^3)	Effective Design Storage Capacity (1,000 m^3)
1968/69	10,401.6	7,281.1
1969/70	8,031.5	5,622.1
1970/71	11,340.2	7,938.1
1971/72	5,256.6	3,679.6
1972/73	7,205.5	5,043.9
1973/74	7,664.8	5,365.4
1974/75	7,224.5	5,057.2
1975/76	9,629.2	6,740.4
1976/77	12,944.7	9,061.3
1977/78	7,799.7	5,459.8
Average	8,749.5	6,124.7

As a result, the representative values are:

Maximum Storage Capacity (1976-77)	$9,061 \times 10^3\ m^3$
Minimum Storage Capacity (1971-72)	$3,680 \times 10^3\ m^3$
Average Storage Capacity (Annual Average)	$6,125 \times 10^3\ m^3$

Thus, it would be deemed appropriate that the optmal design storage capacity is about $8,600 \times 10^3\ m^3$ (EL. 203.0 m)

Item (ii)

In the original plan worked out by RID, the design storage capacity was determined at $11,800 \times 10^3\ m^3$ (EL. 204.4 m) but, judging from the results mentioned above, the original plan is considered to be inefficient in operating the proposed dam.

Accordingly, $8,600 \times 10^3\ m^3$ would be deemed appropriate as the design storage capacity as the results of comparative study, (Refer to Fig. 3-3-1).

In consideration of the geological condition at the dam foundation, embankment materials available from borrow area and the easiness of construction work, homogeneous earth dam is recommendable as the most suitable dam type.

(1) Dam Foundation Bedrock

As a result of geological survey, it was confirmed that the foundation bedrock is spreading over at levels below 2.5 m from the surface. Over this bedrock, it is in development of weathering and cracked conditions. In heavy weathered zone, water permeability coefficient would be $K = 40^{-3}$ cm./sec. and N value over 26-50, while in normal weathered zone $K = 10^{-4}$ cm./sec. together with the N value of over 50.

On the top of the bedrock, silty sand, sandy silt and sandy clay and being distributed in loose condition, showing $K = 10^{-3} - 10^{-4}$ cm./sec. and N value = 3-24.

As far as such permeability condition at the dam foundation is concerned, earth dam would be found the most appropriate dam type among other ones.

(2) Embankment Materials

As a result of borrow area survey impermeable materials such as sandy silt - sandy clay are extended to the depth of 0.4 m below the surface within the reservoir in 200-600 upstream from the dam axis, and ensure its sufficient quantity. As permeable materials can be hardly found in and around the area, homogeneous dam is believed to be more advantageous.

(3) Easiness of Construction Work

Homogeneous earth dam is known to have such advantage as to make accelerated dam volume increase brought by required more gentle slop in every its height.

In this proposed dam height would be required as low as about 11 m, so above disadvantage can be neglected in the light of easiness of condition.

3.3.3 Foundation and Borrow Area

(1) Dam Foundation Bedrock

As a result of dam foundation study through 3 borings, 17 standard penetration tests, 5 field permeability tests, 8 test pits, sampling and soil tests, the following was concluded (see : Figure 3-3-2 and the report attached hereto for details).

<u>Geological Condition</u>

Geological formation for the dam axis, as illustrated in Figure 3-3-3, consists of sandstone corresponding to Phu Phan and Phro Wihan Formations of the Mesozoic Jurassic period and, on top of it river and talus cone deposits of the Quarternary Alluvium period are distributed.

The foundation bedrock is extending to the depth of 2-5 m below the surface, over which it is in development conditions of weathering, soft and cracked conditions. The river and talus cone deposits, which are distributed over the bedrock, are all in unconsolidated condition and composed of silty sand, sandy clay, etc. containing stumps and formicaries in cracked conditions are spreading over to the depth of about 2 m below the surface.

Pemeability of Foundation Bedrock

Layer-wise coefficients of permeability are as follows:

Name of Layer	Coefficient of Permeability (cm/sec)
As and Dt Layer	$K = 10^{-3} - 10^{-4}$
Highly Weathered Zone (Foundation Bedrock)	$K = 10^{-3}$
Normal Weathered Zone (Foundation Bedrock)	$K = 10^{-4}$

Judging from the above results, the dam foundation was found appropriate in its permeability; and complete foundation treatment would be required through grouting or blanketing. Open cracks, which are found spreading over especially on both abutments of the dam axis, should be paid due attention.

Soil Constants of As and Ac Layers

As and Ac layers are in loose condition, showing N value of 3-13 and their soil constants are shown below.

Item	As Layer	Ac Layer
Specific Gravity (Gs)	2.64	2.66
Void Ratio (e)	0.53	0.49
Wet Density (jt - t/m^3)	1.99	1.95
Dry Density (jt - t/m^3)	1.73	1.78
Saturated Density (jt - t/m^3)	2.07	2.11
Cohension (Cu - t/m^2)	0	0
Angle of Internal Friction (Ø)	21.00	21.00

As and Ac layers are extending over in and around the riverbed (on low land) in which the height of the dam embankment would be maximized, so it would come to the most unstable site for dam construction. Accordingly, to prevent such instability, stability analysis on the dam body is essential.

(2) Borrow Area

Borrow area was selected at 200 - 500 m upstream distance from the dam axis in consideration of topographic, geological and soil conditions through field investigations conducted in the vicinity of the proposed dam site and also by taking into account a dam construction planning (see : Figure 3-3-2).

10 test pits, sampling and soil tests have been carried out for a survey for the selected borrow area, the results of which are shown below (see : the report attached hereto for details).

Soil Condition

Name of Soil	Depth	Colour
Surface Soil and Silty Sand	0 - 0.4 m	Dark Brown-Yellowish Gray
Sandy Silt Sandy Clay	Below 0.4 m	Yellowish Gray-Gray

Sandy silt - sandy clay, which are extending at the depth of 0.4 m or lower below the surface, can be used as embankment materials. Availability of this sandy silt - sandy clay is as follows:

Borrow Area	$A = 125{,}000\ m^2$
Excavation Depth	$H = 2.5\ m$
Volume	$V = 312{,}500\ m^3$

The entire requirement for embankment material is about 142,000 m^3 which is quantitatively sufficient.

Design Parameters for Embankment Material

Sandy clay, which is usable as embankment materials, shows the following design parameters:

Specific Gravity	$Gs = 2.66$
Design Moisture Ratio	$W = 15.5 \pm 1.5\%$
Design Dry Density	$\gamma d = 1.73\ t/m^3$
Design Wet Density	$\gamma t = 2.00\ t/m^3$
Saturated Density	$\gamma\ sat = 2.08\ t/m^3$
Coefficient of Permeability	$K20 = 5.0 \times 10^{-6}$ cm/sec
Cohesion	$Cu = 4.0\ t/m^3$
Angle of Internal Friction	$\phi u = 17$
Variation Rate of Earth Volume	(c) = 0.96

3.3.4 Dam Design

As a result of the following items, the typical cross section of the dam body is illustrated in Figure 3-3-4.

(1) Dam Foundation Design

As stated in 4-3-2, homogeneous earth type was chosen to built itself on permeable foundation bedrock; however, in order to make minimized the seepage loss from the foundation and to drain seepage water easily out of the dam body, drain would be required to combine itself suitably with any of other seepage control methods. Furthermore, due attention should be paid to protection for piping due to the liquefaction of loose layers.

The most popular methods among multifarious ones to be taken up for making the minimization of seepage loss and securing the safety of the dam body are as follows:

a) Downstream Drain

The provision of downstream drain should be indispensable for securing the safety of a homogeneous dam through lowering downstream phreatic line.

b) Board Cut-off Core

Permeable foundation should be excavated to arrive impermeable layer and there upon be replaced by core earth.

It can be managed to protect against seepage by compacting its portion.

c) Sheet Pile

It would be to great extent effective for homogeneous foundation such as fine sand and silt layers, etc., which ineffective for cobble-mixture of stratified foundation.

d) Grouting

One line seepage control curtain would be made through cement or special grouting in the dam centre or somewhere in the upstream area not far from the centre, aiming not only at diminishing leakage and uplift pressure for the downstream dam body, but also at finding cracks and permeable layers which cannot be found by other means.

e) Impervious Blanket

It will be done through extending impervious horizontal blanket to the surface layer of a permeable foundation in the upstream side of the dam body, and guarantees lower construction cost and effective piping protection.

Among any combination of the above-mentioned, special grouting would be deemed more appropriate as a leakage and piping protection method by judging from the geological condition of the dam foundation bedrock as mentioned in 3-3-3. However, the adoption of the grouting method would be difficult because it is attended with the following issues:

(1) As a result of marketing study, the special grouting method is hardly adoptable in the Kingdom of Thai. If it is adopted, imported grouting materials and machines along with dispatch of specialists will be required. Moreover, it could be found

difficult to give appropriate instructions for mixing of chemical liquid, and required quantity.

(ii) Judging from the geological condition of the dam foundation, grouting must cover the whole riverbed and the abutments and, thereby, required period for grouting would cause dam construction impossible within one dry season.

There would be a cut-off core as a perfect seepage control method in the alternative of grouting but it would cause difficulty in excavation by use of machine and making gib amount earth work due to the special geological condition that about 8 m deep permeable layers and, in addition, about 2-5 m deep heavy weathered sandstone layers are existing.

Under this consideration, combination of the above mentioned cut-off core, impervious blanket and drain would be adopted in terms of the following:
(a) to allow leakage running through the foundation bedrock within permissible range, (b) to protect piping, and (c) to drain seepage water easily out of the dam body.

1) Cut-off Core

As an effect of piping on weathered rock formation is considered negligible, cut-off have to be done to the depth of 2-6 m of As and Dt layers in order to excavate the defect portion with 2-4 m bottom width and its slope 1:1 and to replace its cut-off portion by impermeable materials and thereafter compacted.

2) Impervious Blanket

After removing surface earth of the reservoir bed, cut-off earth has to be evenly laid and then compacted. From the results of piping as mentioned later, blanket would be 30 m long and extends up to the abutment portions. The thickness, which is subject to the water pressure of 1/10 as a standard value (the maximum depth of about 8 m) has been determined at 1 m with even treatment.

3) Drain

Drain, facilitating to take out seepage water in permeable foundation or the dam body, should be indispensably provided at the foot of the downstream of the dam body. Among some types such as toe, horizontal and chimney drains, toe-drain would be adopted by reason of a rather low dam, being about 11 m at the maximum, and easiness of its construction work.

It is necessary to keep as high as possible the K value of drain for permeable foundation bedrock and rock materials would be used for slope protection as an additional purpose on the downstream side of the dam body. The usable rock materials would be D 50 = 30 cm in average diameter and D 50 x 1.5 in the maximum and contain grains whose diameter is less than 2.5 cm in satisfactory distribution condition.

(2) Designing of a Typical Cross Section

1) Freeboard

The freeboard (Hf) shall be selected the larger one out of two figures : value computed by the following equation, or the standard freeboard value of 2.0 m (for embankment not higher than 50 m).

$$Hf = R + \Delta h + hs + ht$$

where

R : height of wind wave (including creeping)

Δh : rise in water level by extraordinary flood

ht : freeboard allowance of a fill dam (1.0 m)

hs : freeboard allowance of spillway types (0.5 m for a gate type, and 0 m for other)

a) Hight of Wind Wave R

Hight of wind wave adopted here is the significant wave hw computed in SMB method from fetch (F) and wind velocity (U).

And the value is modified by Saville's method to obtain the height of wind wave R including creeping obtained from a chart, since the rate of creeping can greatly vary depending on the inclination and roughness of slope of dam.

fetch	:	3.0 km.
wind velocity	:	from a record in Surin
		maximum wind velocity 50 knots
		= 25.7 m/sec.
		10 minutes mean wind velocity
		V = 25.7/1.5 = 17 m/sec.
		V = 20 m/sec.

roughness and inclination of the slope

pitched slope, 1:30

from a chart of the height of wind wave

R = 1.42 m

b) Rise in Water Level by Extraordinary Flood (Δh)

The water level hightened by flood is computed in the following equation where the storage effect of a reservoir is taken into account.

$$\Delta h = \frac{2}{3} \frac{\alpha Qo}{Q} \cdot \frac{h}{1 + \frac{A.h}{Q.T}}$$

Qo	:	design flood discharge 44.0 m^3/sec.
Q	:	discharge from the overflow section Q = Qo if a conduit is absent
α	:	rate of rise in water level b flood 0.2 in an open channel spillway
h	:	design overflow depth 0.85 m
A	:	reservoir area at design flood water level 2.3 km^2
T	:	duration of extraordinary flood. Standard value ranging 1-3 hours, 2 hours are adopted

$$\Delta h = \frac{2}{3} \times 0.2 \times \frac{0.85}{1 + \frac{2.3 \times 10^6 \times 0.85}{31.35 \times 2 \times 3,600}} = 0.01 \text{ m}$$

Hf = 1.42 + 0.01 + 0 + 1,00 = 2.43 m > 2.0 m

The freeboard of the dam hereby determined as Hf

= 2.45 m

Therefore the crest elevation,

EL 203.00 + 0.85 + 2.45 = EL 206.30 m

2) Crest Width

The crest width (b) is computed in the following equation.

$b = 3.6 H^{1/3} - 3.0$ (m)

H : hight of dam 14.1 m from the riverbed

$b = 3.6 \times 14.1^{1/3} - 3.0 = 5.70$ m

Therefore the crest width is determined as b = 6.0 m

The crest part shall be paved with laterite of over 40 cm in thickness and be given the gradient of 2:100 at upstream and downstream in order to protect against cracks and errosion.

3) Extra - banking

Most of the settlement of foundation and the embanking materials would take place during the time of the embanking work, so that very little settlement would occur after the completion.

A long-term post-completion settlement of an earth-fill dam is only 0.2 - 0.4% of the dam height,or 3-6 m, so no extra banking will be required.

4) Slope Gradient

Slope gradient shall be determined finally after stability analysis on the basis of determination of various design values from tests results of the foundation and embanking materials and predictable conditions of construction work.

USBR recommends the following slope gradient for homogenous type dams built on a stable foundation, in case of embanking materials of CL, and ML.

upstream slope gradient 1 : 30
downstream slope gradient 1 : 25

5) Protection of Upstream Slope

The upstream slope is lined with pitching in order to protect the embankment materials against errosion by wind wave or by fluctuation water level and against the occurrence of cracks caused by the variation of weather.

a) Quality of the Pitching Rocks

Hard rocks of solid texture sufficiently resistent against weathering shall be used for pitching.

b) Size of Pitching Rocks and Thickness of Pitching

The masonary materials for pitching should be of sufficient weight and size immovable by wind wave.

The standard pitching rocks should be of well graded distribution including grain size under 2.5 cm, with the mean diameter (D50) 30 cm, and the maximum diameter D50 x 1.5 against the wave height of 0.6 - 1.2 m.

Even though the minimum thickness of pitching (under this condition) is 30 cm, the pitching thickness in the present work shall be made 40 cm considering the rocks available and convenience in execution.

c) Filter

A filter zone of well graded materials shall be provided under the pitching as a protection of embankment soil against flow loss through the gaps among the pitching rocks.

Filter zone, 30 cm thick, shall be made with crushed stone of 0.5 mm - 9 mm in diameter or natural gravel.

6) Protection of the Downstream Slope

The downstream slope of the embankment shall be lined by grassplot as a protection against errosion by rain and cracks caused by variation in weather.

And draining ditches shall be provided on the downstream slope vertically and laterally at the interval of 5 m in order to prevent its errosion presumably caused by concentrated rain water running over it.

The sectional dimensions of the draining ditch shall be 40 cm x 40 cm and the inner side of it shall be filled with crushed stones.

(3) Measures against Seepage Water

1) Phreatic surface

Phreatic surface is obtained by Casagrande's method

The assumption, here, is that the permeability of the body of the embankment is anisotropic.

The cross-sectional view showing the phreatic surface shall be deformed by contracting the horizontal dimension by multiplying the real value with $\sqrt{kv/kh}$.

kv : vertical coefficient of permeability

kh : horizontal coefficient of permeability

The value of kv/kh in the case of compaction by tamping roller is averagely $\frac{1}{5}$

$$\frac{kv}{kh} = \frac{1}{5} \sqrt{\frac{kv}{kh}} = 0.447$$

The deformed cross-section is as shown in the Figure 3-3-5. If the point A is the pole and x is the horizontal axis and y the vertical axis, the standard parabola of the phreatic surface is given by the following equation.

$$y = \sqrt{2\ yox + yo^2}$$

$$yo = \sqrt{h^2 + d^2} - d$$

$$yo = \sqrt{8.50^2 - 19.20^2} - 19.20 = 1.80\ m$$

$$y = \sqrt{2 \times 1.80x + 1.80^2} = \sqrt{3.60x + 3.24}$$

$x = 0$ m $\quad y = 1.80$ m

$y = 0$ m $\quad x = -yo/2 = 0.90$ m

x (m)	2.0	4.0	6.0	10.0	14.0	19.20
y (m)	3.23	4.20	9.98	6.26	7.32	8.50

For representing the actual phreatic surface, the standard parabolic line thus obtained is modified as follows.

The stream line lies at a right angle to the slope at the point of inflow, and Co point falls for an oblique distance of Δa to C point at the point of outflow.

The value of Δa varies depending on the angle of slope at face of outflow and can be obtained from the following equation.

$$a + \Delta a = \frac{yo}{1 - \cos\alpha}$$

$$\alpha = 123°51'$$

$$yo = 1.80 \text{ m}$$

$$a + \Delta a = \frac{1.80}{1 + 0.557} = 1.16 \text{ m}$$

$$c = \frac{a}{a + \Delta a} = 0.17$$

$$\Delta a = 0.17 \times 1.16 = 0.20 \text{ m}$$

Thus, the phreatic surface is obtained by reconstructing the deformed cross-section into the original cross-section following the modification of the standard parabolic line.

2) Amount of Leakage

a) Amount of Leakage from the Dam Body

The amount of leakage is computed by using modified coefficient of permeability (k), on the assumption that the impervious zone of the dam body is anisotropic. The modified coefficient of permeability is obtained from the following equation.

$$k = \sqrt{kh\ kv}$$

k : modified coefficient of permeability

kh : coefficient of permeability in horizontal direction

kv : coefficient of permeability in vertical direction from the laboratory permeability test kv

$= 1 \times 10^{-6}$ cm/sec.

since $\frac{kv}{kn} = \frac{1}{5}$, $kn = 5\ kv$

$k = 5\ kv = 2.24 \times 10^{-6}$ cm/sec.

$= 1.94 \times 10^{-3}$ m/day

The amount of leakage from the dam body can be obtained from the following equation.

$Q = k.yo.L$

k = modified coefficient of permeability
1.94×10^{-3} m/day

$yo = \sqrt{h^2 + d^2} - d = 1.80$ m

L = profile length of the dam body 350 m

$Q = 1.94 \times 10^{-3} \times 1.80 \times 350$

$= 1\ m^3/day$

b) Amount of Leakage from the Foundation

On the basis of the analysis on piping which might take place (treated later), a blanket shall be laid on the upper stream of the dam body.

The amount of leakage through the blanketed foundation is computed as follows:

$$qf = \frac{k\ d\ h}{Xr + Xd}$$

$$Xr = \frac{e^{2aX} - 1}{a(e^{2aX} + 1)}$$

$$a = \sqrt{\frac{kl}{t\ k\ d}}$$

qf = amount of permeability through the foundation

Xr = effective seepage path of the blanket

Xd = width of the impervious zone of the dam body 60 m

t = thickness of the blanket 1.0 m

d = thickness of the foundation layer 7.0 m

kl = coefficient of permeability of the blanket
5×6^{-6}cm/sec = 0.86 m/day

h = depth of water above the blanket 8.0 m

X = length of the blanket 30 m

$$a = \sqrt{\frac{4.32 \times 10^{-3}}{1.0 \times 0.86 \times 7.0}} = 0.027$$

$$2aX = 2 \times 0.027 \times 30 = 1.62$$

$$e^{2aX} = 5.05$$

$$Xr = \frac{5.05 - 1}{0.027 \times (5.05 + 1)} = 24.8 \text{ m}$$

$$gf = \frac{0.86 \times 7.0 \times 8.0}{24.8 + 60} = 0.57 \text{ m}^3/\text{day/m}$$

If the profile length of the dam body is 350 m, the amount of leakage through the foundation is,

$$Qf = 0.57 \times 350 = 200 \text{ m}^3/\text{day}$$

c) Total Amount of Leakage

$$Q = Qb + Qf$$

$$= 1 + 200 \doteqdot 200 \text{ m}^3/\text{day}$$

The criterion amount of allowable leakage is 0.05%/day

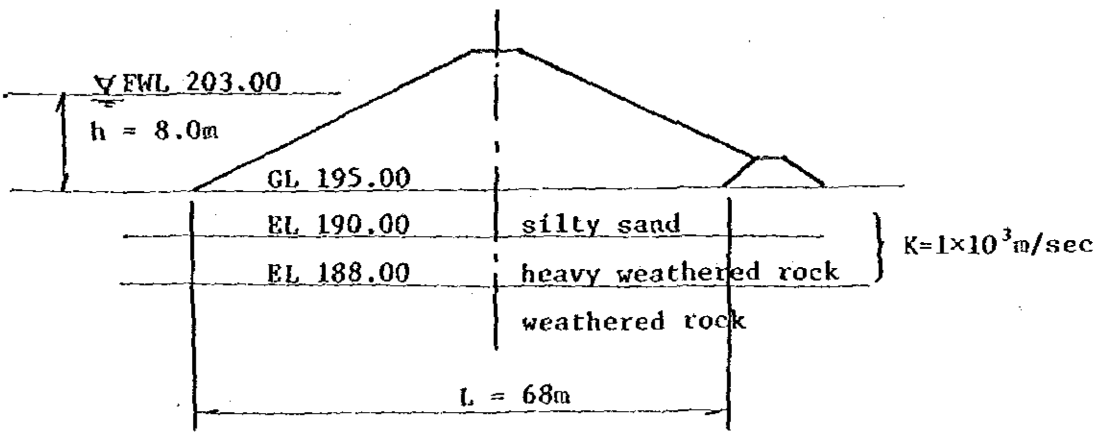
Effective storage amount $V = 8,440,000 \text{ m}^3$

$$Qa = 0.0005 \times 8,440,000 = 4,220 \text{ m}^3/\text{day}$$

Therefore, the amount of leakage is satisfactorily small.

3) Analysis of Possible Piping

The analysis of possible piping is made by way of critical hydraulic gradient.



Analysis is made on possible piping in the loose silty sand with the N-value of 5-15 containing sand deposits at GL 0 - 5.0 m in the figure above.

critical hydraulic gradient (ic)

$$ic = \frac{Gs - 1}{1 + e}$$

Gs : specific gravity of soil particles known as Gs = 2.64 from soil test

e : void ratio, known as e = 0.52 from soil test

$$ic \fallingdotseq \frac{2.64 - 1}{1 + 0.52} = 1.08$$

hydraulic gradient (i)

$$i = \frac{h}{L}$$

L : length of permeability route 60 m

h : difference in water level 8.0 m

$$i = \frac{8.0}{60} = 0.13$$

Safety Factor (Fs)

$$Fs = \frac{ic}{i} = \frac{1.08}{0.13} = 8.3$$

The generally accepted safety factor is 8-12, and piping may occure if no particular measures are taken ("Shin-pen Doshitsu Rikigaku" by Fusayoshi Kawakami)

The works for the prevention of piping include sheet piles, curtain grouting, soil replacement by cut-offs, upstream blankets, etc., but it is decided to adopt cut-offs and upstream blankets in the present project on the basis of consideration on economy, convenience in execution, duration of the work, etc.

If the length of the blanket is 30 m, hydraulic gradient

$$i30 = \frac{8.0}{60 + 30} = 0.089$$

safety factor

$$Fs = \frac{ic}{is0} = \frac{1.08}{0.089} = 12.1 > 8 - 12$$

Therefore, the length of the blanket shall be 30 m.

Since the standard thickness of the blanket is 1/10 of the water head the thickness of the conceived blanket shall be 1 m.

(4) Stability Analysis

1) Determination of Design Parameters

a) Embankment material

Embankment construction will be conducted by the density control method. During the construction period, density will be controlled under the following conditions:

The moisture content should be within ± 1.5% of its optimum and the dry density be 95% of its maximum. Design density has been determined from the results of the compaction test as follows:

Specific gravity	$G = 2.66$
Dry density	$\gamma d = 1.73\ t/m^3$
Optimum moisture content	$w = 15.5\%$
Wet density	$\gamma t = 2.00\ t/m^3$
Saturated density	$\gamma sat = 2.08\ t/m^3$

A direct shear test has been made on the basis of the above mentioned moisture content through which the design strength has been determined. Assuming that the design strength is 70% of the empirical value, it results in the following:

Cohesion	$C = 4.0\ t/m^2$
Angle of internal friction	$P = 17°$

b) Foundation

The design density has been determined as follows:

Specific gravity	G = 2.64
Void ratio	e = 0.53
Dry density	γd = 1.73 t/m^3
Wet density	γt = 1.99 t/m^3
Saturated density	γsat = 2.07 t/m^3

A direct shear test was made on the undisturbed samples and the design strength has been determined. Assuming that the design strength is 70% of the empirical value, the results are as follows:

Cohesion	C = 0 t/m^2
Angle of internal friction	P = 21°

c) *Rock for toe drain*

The design parameters of the rock for the toe drain were assumed as follows:

Specific gravity	Gs = 2.60
Wet density	γt = 1.65 t/m^2
Saturated density	γsat = 1.90 t/m^3
Cohesion	C = 0 t/m^2
Angle of internal friction	P = 45°

2) Stability Analysis

A stability analysis of the finally adopted dam section (see Figure 3-3-4) has been carried out for the three governing conditions viz. following fill completion (Case - 1), full reservoir (Case - 2) and rapid drawdown from maximum pool (Case - 3). Construction pore pressure is taken into account in Case - 1, the pore pressure of steady seepage in Case - 2 and 3. As judging from a low coefficient of 5×10^{-7} cm/sec, it would be assumed that 50% of the uplift would be residual construction pore pressure.

Stability analysis has been made by electronic compute, using the slice method. The allowable safety factor for all cases has been assumed to be 1.2.

The results of stability analysis are illustrated in Figures 3-3-6, 3-3-7 and 3-3-8, and the computed safety factors are as follows:

		Upstream face	Downstream face
Case 1	Following fill completion	1,547	1,411
Case 2	Full reservoir	1,927	1,206
Case 3	Rapid drawdown	1,244	-

Therefore, this dam is deemed to be safe under any conditions.

3.3.5 Spillway Design

(1) Selection of Site

Spillway is proposed to be located at Point No.5 + 60.0 on the right bank side. This decision is based on considerations as to the topography and geology, plus safety and economy.

The bedrock is spreading to the depth of about 2 m below the surface providing over 25 N value and good foundation in the neighbourhood of Point No.5 + 60.0. Furthermore, while the left bank side is being heavy forest, the right bank side is being covered with weeds and slightly slanting with little relief from the topographical viewpoint. Judging from safety for the foundation of concrete structures and ease of construction, Point No.5 + 60.0 would be deemed most appropriate for the location of the spillway.

(2) Determination of Type

Overflow spillway (non-regulating type) following on open earth canal is adopted from the topographical, geological and O & M considerations, plus hydraulic and economic reasons, plus structural stability point of view.

(3) Determination of Spillway Scale

In this project, the reservoir area (2.08 km^2.) is fairly large in comparison with the catchment area (34.6 km^2.) and the reservoir will be equipped with non-regulating type spillway. Under these circumstances, an economical design would be sought by reducing the design overflow discharge by taking into consideration the flood regulating capacity of the spillway.

In designing a spillway, any combination of overflow depth and width can be thought; in this project, however, flood follow-up calculation is made on the basis of the assumption that the overflow depth has been set at 0.85 m from the economical point of view as well as of the scale of Meh Kah dam regardless of such combination. As result, the overflow width is 20 m, and the overflow discharge, 31.3 m^3/sec. As this scale would be considered appropriate from the entire dam scale viewpoint, the overflow depth of the spillway, the overflow width and discharge have been determined at 0.85 m, 20 m and 31.3 m^3/sec.

(4) Chute Portion

Chute portion is designed to have concrete structure because of functioning as a chute channel. The channel has 10 m width by narrowing its section after overflowing from the economical viewpoint. It has 1/26.3 gradient for the first 100 m, while 1/2 gradient for the next 7 m.

(5) Scour Protection Stilling Basin

Scour protection stilling basin is planned to follow the chute portion so as to reduce high velocity energy and lead to the river. Horizontal stilling basin III type is adopted with 7 m length.

3.3.6 Design of Intake Work

(1) Selection of Site

Location of the intake work is determined on both bank sides according to the plan worked out by RID, one of which was located at No.2 + 15.0 on the left bank side and the other at No.4 + 30.0 on the right bank side by taking into account the proposed conveyance canal route, and topographical and geological conditions. Moreover, in order to discharge dead water staying below the bed height of the intake work during the end of dry season, conduit is provided at No.3 + 45.0.

(2) Determination of Type

The same as for Meh Kah dam.

(3) Amount of Intake Water

The maximum intake water for both new villagers and irrigation is 0.8 m^3/sec., i.e. 0.4 m^3/sec. on each bank side.

(4) Horizontal Conduit

Steel pipe (diameter : 200 mm) will be entirely armoured all round by ferro-concrete.

3.4 Construction Plan

The construction work shall be commenced in November, 1981, the coming dry season and completed by the end of June 1982, so that water will be stored during the next rainy season (May - October, 1982).

The construction plan, procedures for construction work, etc. are all the same as in the case of Mah Kah dam, which is stated in 2-4. The construction schedule and machinery used are also the same as in case of Mah Kah dam.

The construction schedule is shown in Table 3-4-1.

3.5 Project Cost

3.5.1 General Description

The total cost of this project is estimated at ¥526,000,000 (฿ 50,600,000).

The project cost consists of construction cost and consultancy fee.

The construction cost consists of the cost of dam construction and the cost of related works (intake facilities and spillway) only.

The consultancy fee covers the costs for detailed design and supervision.

The proposed site of this project is all government-owned land of light jungle so there would be no need of compensation for plants.

3.5.2 Construction Cost

The dam site is located not so far from the site of Mah Kah dam, the unit costs of construction are assumed to be the same as in case of Mah Kah dam.

3.5.3 Cost of Detailed Design and Construction Supervision

The construction scale and schedule being mostly identical to the case of Meh Kah dam, the same figureas in Meh Ka dam construction is adopted.

3.5.4 Project Cost

A. Construction Cost

		¥	฿
1.	Dam Body	263,040,000	25,292,333
2.	Spillway	76,103,000	7,317,552
3.	Intake Works	26,091,000	2,508,730
4.	Outlet Conduit Pipe	7,484,000	719,613
5.	Temporary Works	5,314,000	510,967
6.	Indirect Construction Cost	22,680,000	2,180,805
	Sub - total	400,712,000	38,530,000
7.	Miscellaneous Expenses	76,326,000	7,339,000
	Total	477,038,000	45,869,000
		= 477,000,000	= 45,900,000
B.	Detailed Design and Construction Supervision	49,000,000	4,700,000
	Grand Total	526,000,000	50,600,000

(No contingency is included in the above)

Exchange rates (as of 14 August, 1981)

1 U.S.$ = 22.6 ฿

1 U.S.$ = 235 ¥

1 ฿ = 10.4 ¥

3.6 Project Evaluation

3.6.1 Agricultural Production and Farm Income

(1) Agricultural Production

After the completion of Ta Kao dam, at least 2,190 rai (350 ha) out of 4,035 rai of paddy field under three villages of Ban Kap Choeng, Ban Huai Samakkhi and New Ban Ta Kao where paddy is cultivated throughout a year.

To attain the upliftment of agricultural development for the project area by taking full advantage of such irrigation feasibility, the annual land use programme for the said irrigable land would be worked out in accordance with the proposed cropping pattern as shown in Figure 3-6-1. It is to put the whole area of the newly irrigable land under paddy cultivation during the rainy season, while during dry season about 40% (940 rai) out of it under paddy cultivation and the rest 60% (1,250 rai) under upland crop cultivation.

This cropping pattern was adopted under the same consideration as for Meh Kah dam.

With the implementation of agricultural development for the benefited areas, crop-wise yield attainable under the proposed cropping pattern as shown in Figure 3-6-1 are as follows:

1)	Paddy (in rainy and dry seasons):	Paddy	500 kg/rai
2)	Upland Crops:	Hemp	225 kg/rai
		Peanuts	200 kg/rai
		Sesame	120 kg/rai
		Maize	300 kg/rai

(2) Farm Income

As it is reported that in the direct benefited area the present average yield of paddy would be about 300 kg/rai and, but it would come down below half during the draught years. Its yield should be adjusted at estimation of approximately 250 kg/rai.

Accordingly, the present average yield would be computed as follows:

250 kg x 4,035 rai = 1,008.8 tons

฿ 3,300/ton x 1,008.8 tons = 3,329,000

After the completion of the dam, 2,190 rai out of 4,035 rai of the existing paddy field including under reclamation land would assure the paddy cultivation of HYV in every rainy season and expectedly produce average yield of 500 kg/rai.

During the rainy season paddy production would amount to:

Irrigable Area	500 kg x 2,190 rai	=	1,095 tons
Existing Area	250 kg x 1,845 rai	=	461.3 tons
	Total		1,556.3 tons

฿ 3,300/ton x 1,556.3 tons = ฿ 5,135,800

In comparison with without-project condition, it would bring the following results:

1,556.3 tons - 1,008.8 tons = 547.5 tons (฿ 1,806,800)

In case of with-project condition, it would increase by about 54%

During the dry season, on the other hand, 940 rai of land would yield the production of 500 kg/rai, which is the same figure as for paddy during the rainy season, as shown below.

500 kg x 940 rai = 470 tons (฿ 1,551,000)

Crop-wide benefits accruable under 1,250 rai of land can be represented on the following Table.

Crop-wise Benefits Under 1,250 Rai of Land

Crops	Area (rai)	Yield (kg/rai)	Production (ton)	Unit Price (฿ per ton)	Sales Value (฿ 1,000)
Hemp	370	225	83.3	5,000	416
Peanuts	370	200	74.0	5,000	370
Maize	370	300	222.0	2,500	555
Sesame	140	120	16.8	10,000	168
Total	1,250				1,509

Assuming that 1,500 rai of the current land use including under reclamation, which are extending on a slanting portion of plateau, would remain unchanged in its per rai yield, and therefore unit, price, crop-wise benefits under 1,500 rai of the current land use would be as follows:

Crop-wise Benefits Under 1,500 Rai of Land

Crops	Area (rai)	Yield (kg/rai)	Production (ton)	Unit Price (฿ per ton)	Sales Value (฿ 1,000)
Hemp	300	225	67.5	5,000	337.5
Peanuts	200	200	40	5,000	200
Maize	100	300	90	2,500	225
Sesame	100	120	12	10,000	120
Cassava	800	1,800	1,440	400	576
Total	1,500				1,458.5

As a result, project benefits to be born in the benefited areas of the project would be estimated according to without-and with-project conditions as follows:

Project Benefits by Agricultural Production Increase

Item	Paddy rain	Paddy dry	Dry Season (Upland Crops)	Existing Upland Crops	Total
With-Project	5,135.8	1,551	1,509	1,458.9	9,645.3
Without-Project	3,329	-	-	1,458.5	4,787.5
Benefit	1,806.8	1,551	1,509	-	4,866.8

In this evaluation, costs for improved farming and operation and maintenance of infrastructures are not included and furthermore benefit obtainable from productive use of oil seed cakes as feed-stuff and manure is neglected.

3.6.2 Project Benefits

Benefits to be born by the implementation of the project would be not only an annual agricultural production increase amounting to about ฿ 4,867,000, but also farm income for selling agricultural products as well as labour force saving due to subsistance water utilization would form part of the benefits.

However, for a more accurate estimation of project benefits, it is necessary to take into account success percentage during dry season (not 100% annually), that is:

Paddy Production during the Dry Season

฿ 1,551,000 x 60% = ฿ 930,600

Upland Crop Production during the Dry Season

฿ 1,509,000 x 80% = ฿ 1,207,200

Accordingly, the annual benefits of agricultural products would be

Paddy Production during the Rainy Season	฿ 1,806,800
Paddy Production during the Dry Season	฿ 930,600
Upland Crop Production during the Dry Season	฿ 1,207,200
Total	฿ 3,944,600

The total project cost of 50,600,000 Baht shall be recovered in about 13 years with the estimated anual benefits of 3,944,600 Baht

50,600,000 - 3,944,600 = 12.8 (years)

The interest accruing to the investment is not included here.

3.7 Urgent Matters

3.7.1 Matters to be Settled Prior to the Commencement of Construction Works

The Government of Thailand is requested to take full consideration settled the following matters promptly, since it would be impossible to start the projected construction works even after the completion of the official international procedures between Thailand and Japan if they are left unsettled.

(1) Approval of cutting trees in the dam site and the area of future submersion.

(2) Cutting trees in the area of future submersion.

(3) Making compensation for farm crops and all other kinds of compensations for the dam site and the area of future submersion.

(4) Offering the land required for making field office, laborers houses machinery and material yard and other facilities for the construction work.

(5) In the case of Ta Kao dam, the topographical survey of the area of future submersion could not be conducted due to the official prohibition of entry into the area in the vicinity of the national border.

So estimated storage capacity of the dam is not highly accurate. It is desired that necessary topographical survey to be conducted when the circumstances allow it.

3.7.2 Note for Construction Work

There can be some change in the schedule and specification of the construction work at the time of execution, since the short period of survey necessarily limited the number of test drilling and test pits for data collection.