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

MALAYSIA

FEASIBILITY STUDY REPORT ON THE TEKAI HYDROELECTRIC POWER DEVELOPMENT PROJECT

Volume I Main Report

SEPTEMBER 1983

JAPAN INTERNATIONAL COOPERATION AGENCY

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JAPAN INTERNATIONAL COOPERATION AGENCY

PREFACE

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

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

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

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

Tokyo, August 1983

Keisuke Arita

Keisuke Arita
President
Japan International Cooperation Agency

FOREWORD

In 1980 the Government of Malaysia made a request to the Government of Japan asking for cooperation in carrying out a feasibility study for a hydroelectric power development project in the basin of the Tekai River, a tributary of the Tembeling River of the Pahang River Basin in Pahang State, West Malaysia. In response to this request, the Government of Japan entrusted the execution of the study to the Japan International Cooperation Agency (JICA).

Prior to carrying out this development project, JICA held consultations with authorities of the Malaysian Government regarding the contents of the project. JICA then dispatched a pre-feasibility survey team which spent 10 days from October 27 to November 5, 1980 in Malaysia confirming particulars such as the contents, background, etc. of the request made by the Malaysian Government and drawing up the scope of work to conduct the feasibility study. Further consultations on the details of the scope of work were carried out February 18 and 19, 1981 between the Economic Planning Unit (EPU) and National Electricity Board (NEB) representing the Malaysian Government and JICA and the parties concerned to establish final details (refer to Appendix 3: Scope of Work of the Feasibility Study of Tekai Hydroelectric Development Project, Pahang, Malaysia).

Based on the conclusions reached, it was decided to conduct a feasibility study consisting of the three stages mentioned below. A survey team composed of ten specialists was organized and immediately dispatched to Malaysia for a period of 25 days from March 1 through 25, 1981 in order to conduct field reconnaissance in preparation for preliminary site investigation and for the purpose of collecting data and information required for site investigation. The investigation results are described in the Inception Report.

(1) Preliminary Investigation Stage

The preliminary investigation stage is aimed at determining the optimum site.

(2) Detailed Field Investigation Stage

The detailed field investigation stage is aimed at carrying out field investigations of various kinds at the optimum site determined in the previous stage.

(3) Feasibility Design Stage

The feasibility design stage is aimed at carrying out design of the hydroelectric power development project at the optimum site and carrying out an economical feasibility study of the project in question.

In the preliminary investigation stage a survey team composed of specialists in the fields of dams, geology, hydrology, civil engineering for power generation, power demand forecasting, geological surveying, etc., was organized by JICA. The survey team visited Malaysia for a period of approximately five (5) months from mid June to late October, 1981 to undertake preliminary site investigations. During its stay in Malaysia, the survey team was engaged principally in supervision of drilling, seismic prospecting and aerial photography of the upper and lower sites. The team members also undertook a ground control survey in order to draw up aerial maps and longitudinal profiles and cross section surveys of the upper and lower sites as well as hydrological observations. In addition to this work, the survey team members collected data and information in their respective fields. Next, comparative analysis of the merits and demerits of the two proposed development sites, i.e., Upper Tekai Site and Lower Tekai Site, was carried out based on the results of the aforesaid investigations with the purpose of determining the most advantageous development method and the approximate scale of the development project. In addition, a preliminary analysis of the project was carried out with the purpose of providing basic data and information for subsequent stages of investigation and analysis. The results of the aforementioned investigations, analysis and discussions are described in the Interim Report (March 1982).

In the detailed field investigation stage, the survey team organized for the preliminary field investigation stage was reinforced with specialists in the fields of forestry, zoology and economics, taking into consideration the findings incorporated in the Interim Report. This reinforced survey team visited Malaysia for approximately 6 months from the mid May to late October, 1982 to carry out detailed field investigation.

In the feasibility design stage the development system and scale of the development project proposed in the Interim Report were reviewed taking into consideration the results of the aforementioned detailed field investigations. Further, detailed design corresponding to the optimum development scale was also carried out in the feasibility design stage. In addition, a works progress chart for implementation of this project, detailed estimation of construction costs, economic analysis, social and environmental impact study for the project are also discussed and conclusions presented in the final report of the following composition.

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

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UPPER TEKAI - (Center Core Rockfill Dam)

Crest Level	EL 166.20
Dam Crest Length	350 m
Dam Height	101 m
Dam Embankment Volume	$3.125 \times 10^6 m^3$
Spillway Crest Width	47.5 m
Maximum Spillway Discharge	1,504 m^3/s

STORAGE RESERVOIR

Catchment Area	1200 km^2
High Water Level	EL 157.00
Low Water Level	EL 147.00
Surcharge Water Level	EL 164.00
Gross Storage Capacity at H.W.L.	$2,040 \times 10^6 m^3$
Effective Storage Capacity	$680 \times 10^6 m^3$
Surface Area at H.W.L.	76 km^2

POWER STATION

Number of Units and Rated Capacity	2x75 MW
Average Annual Energy Output	194.8 GWH
Maximum Water Discharge	235 m^3/s

ESTIMATED CONSTRUCTION COST 289.451×10^6 M$

PRINCIPAL DIMENSIONS OF TEKAI PROJECT

LOWER TEKAI - (Concrete Gravity Dam)

Crest Level	EL 81.00
Dam Crest Length	160 m
Dam Height	38 m
Dam Embankment Volume	5.69×10^4 m^3
Spillway Crest Length	91.5 m
Maximum Spillway Discharge	1,100 m^3/s

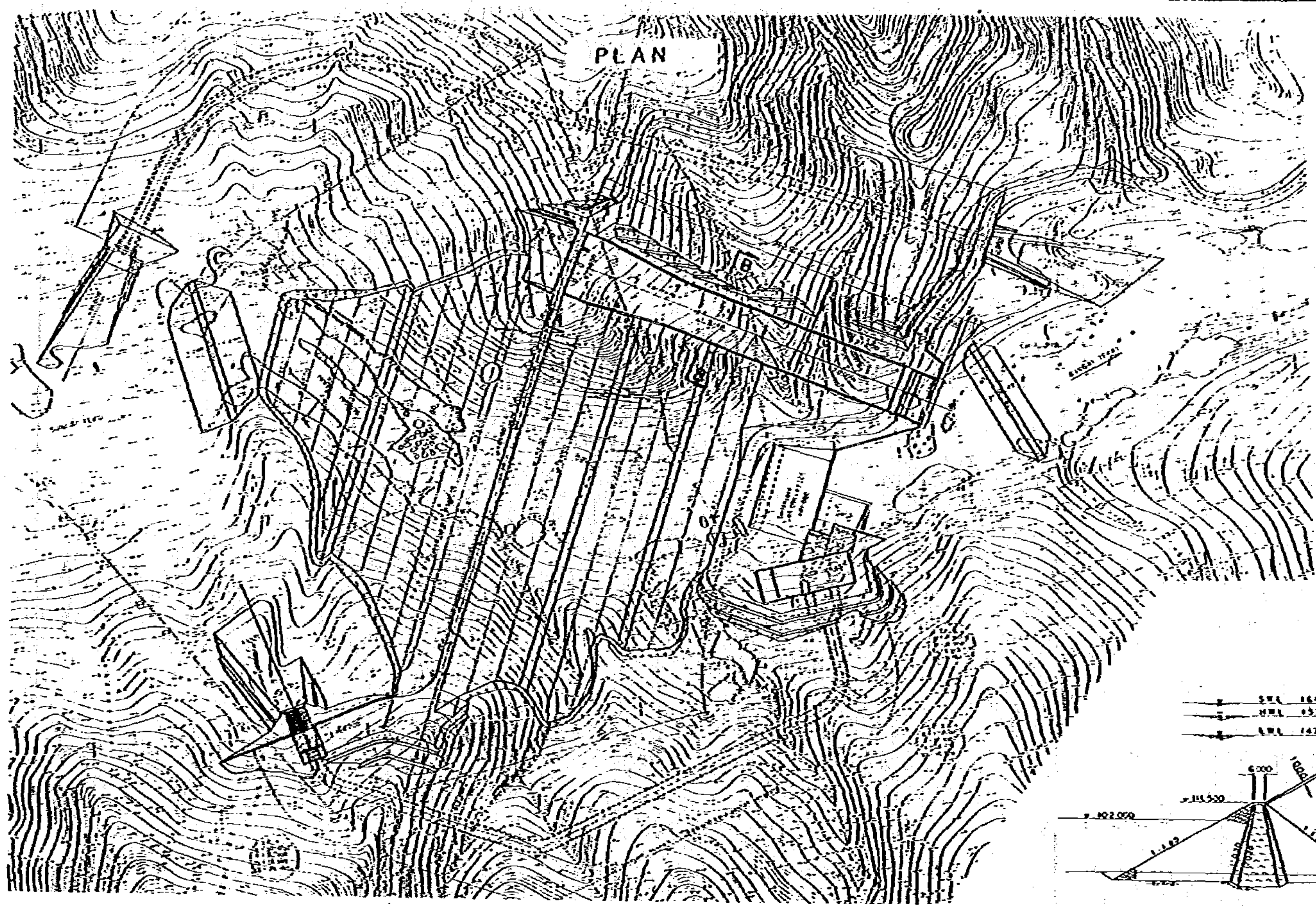
STORAGE RESERVOIR

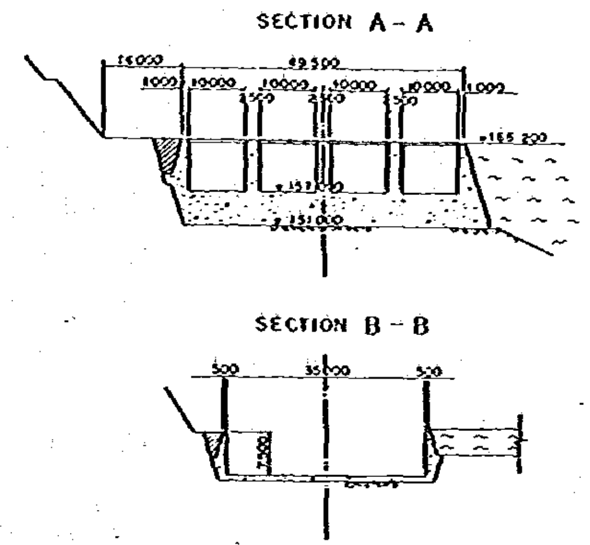
Catchment Area	1,380 km^2
High Water Level	EL 75.00
Low Water Level	EL 70.50
Surcharge Water Level	EL 79.00
Gross Storage Capacity at H.W.L.	41.5×10^6 m^3
Effective Storage Capacity	21.5×10^6 m^3
Surface Area at H.W.L.	6.1 km^2

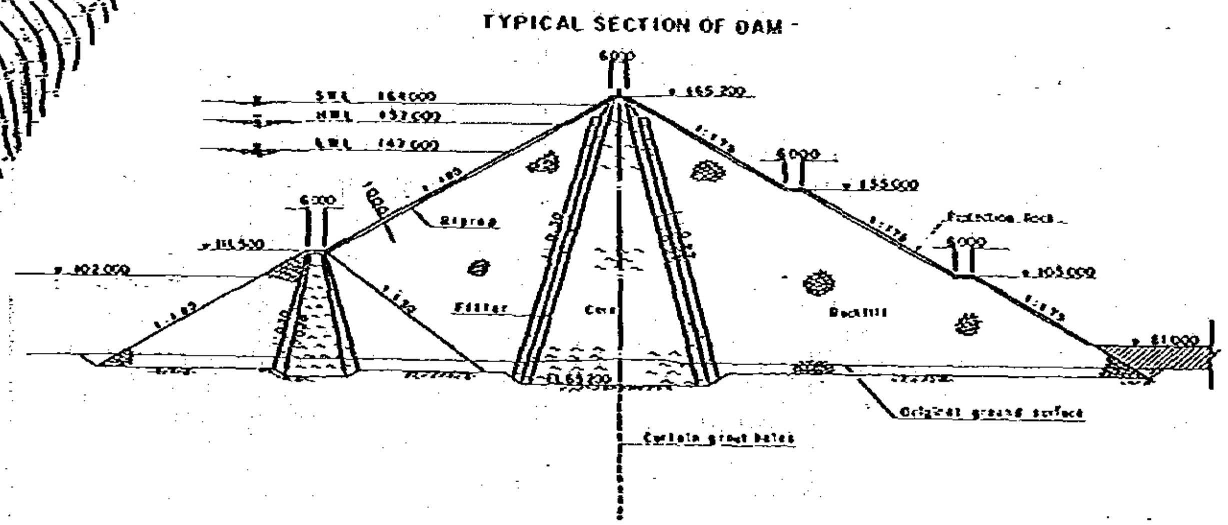
POWER STATION

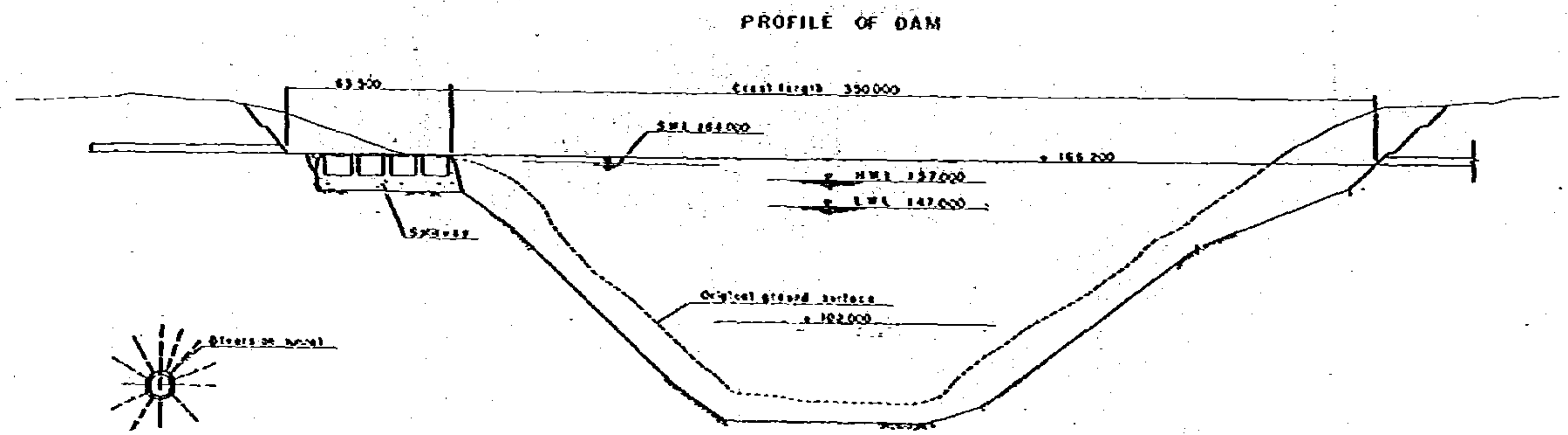
Number of Units and Rated Capacity	1 x 5.8 MW
Average Annual Energy Output	40.3 GWH
Maximum Water Discharge	40 m^3/s

ESTIMATED CONSTRUCTION COST 61.981×10^6 M$







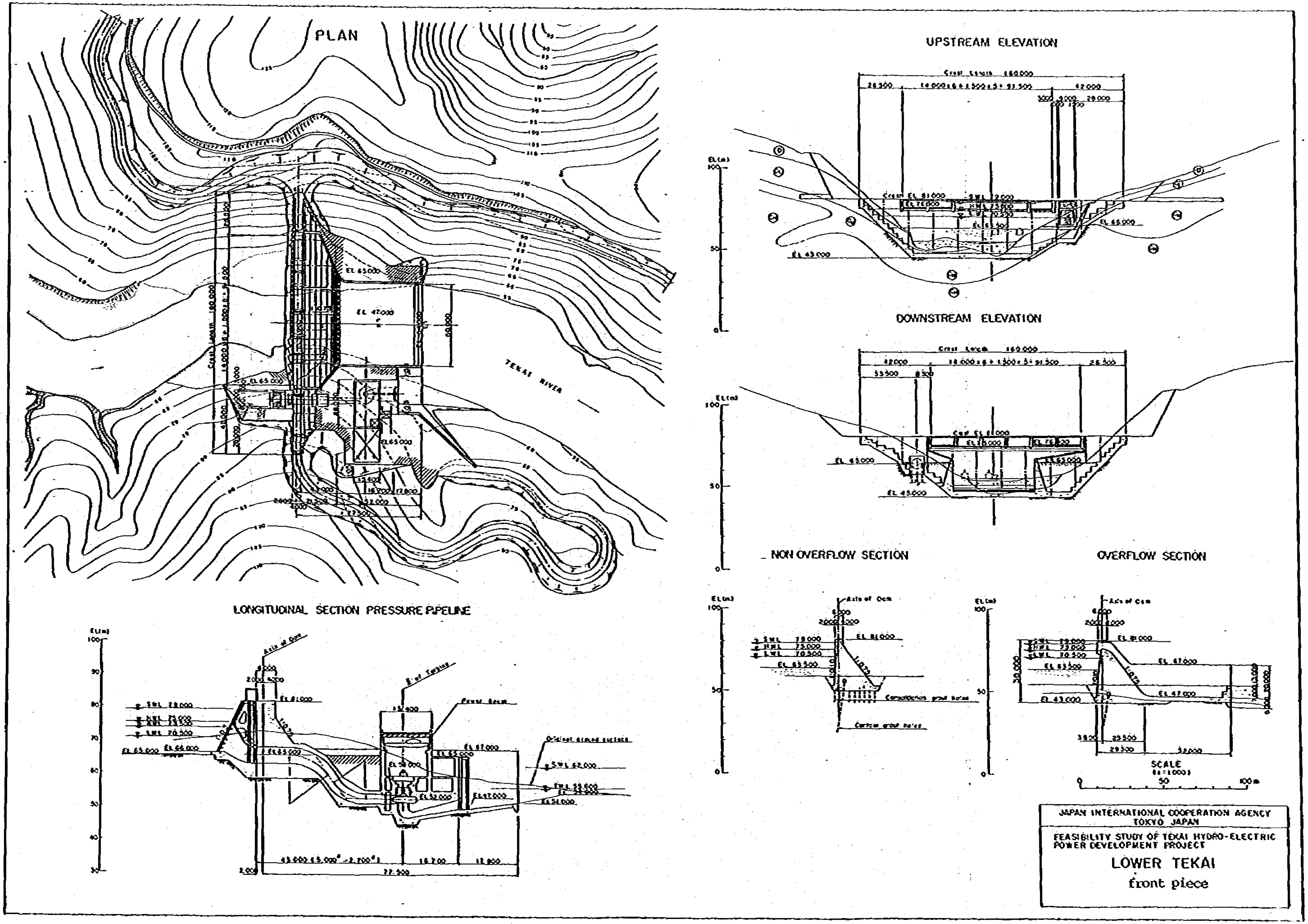


JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO JAPAN

FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

UPPER TEKAI

front piece



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FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

LOWER TEKAI

front piece

1. INTRODUCTION

1. Introduction

The mountainous regions of the Peninsular Malaysia, in an area of heavily rainfall have immense hydroelectric power development potential, thanks to the abundant availability of water. Furthermore, there are many sites suitable for construction of dams for hydroelectric power development in view of their global potentialities comprising factors such as topography, geology, and suitable reservoir area, etc.

In the study carried out (1981.3 - 1983.9) by the JICA survey team, emphasis is put on generation of hydroelectric power in the Tekai River, a tributary located on the upper reaches of the Pahang River, for the purpose of developing power resources in the region.

The Tekai site is located on the Tekai River, a tributary of the Tembeling River in the basin of the Pahang River, and consists of the Upper Tekai Site and Lower Tekai Site.

A rockfill dam with a catchment area of 1,200 km^2, maximum power generation of 150 MW and dam height of 101 m is planned for the Upper Tekai Site. On the other hand, a concrete gravity dam with a catchment area of 1,380 km^2, maximum power generation of 5.8 MW and dam height of 38 m is planned for the Lower Tekai Site. The latter is a base load power source that will function as a re-regulating dam for the Upper Tekai Dam as well as provide regulation of the residual area.

Electricity demand in Malaysia is expected to grow by a high annual average of 14.7% during the 1980-1985 period and the National Electricity Board (NEB) of Malaysia is pushing forward in development of power sources in order to cope with this demand. The present project is part of the development programme and is regarded as a particularly important hydroelectric power development required to promote economical development of Pahang State. In addition, the Tekai Hydroelectric Power Development Project is expected to make considerable contributions for regional development of Pahang State.

2. PROJECT AREA

2. Project Area

2.1 Location

The project site for the Tekai Hydro-electric Power Development Project is located on the lower reaches of the Tekai River, one of the tributaries of the Pahang River (catchment area of 28,500 km^2) which flows through the state of Pahang, the largest state in West Malaysia. The project area is located about 150 km northeast of the capital city of Kuala Lumpur and is adjacent to the southern border of the Taman Negara (National Park) on the northern tip of the state of Pahang (See Fig. I).

The Pahang River, the largest on the Peninsular Malaysia, flows in the mountainous interior of Pahang State, is joined by many tributaries, and flows south through the Jerantut District. It then turns east in the neighbourhood of Temerloh and finally reaches the South China Sea in the south of Kuantan.

The Tekai River is one tributary of the Tembeling River which is one of the main upper reaches of the Pahang River and joins the Jelai River at Kuala Tembeling. It flows mainly in the Trengganu coastal range which lies on a north-northwest, south-southeast axis with extensions from Ulu Trengganu District to Kuantan District. The Tekai River flows mainly west-northwest and joins the Tembeling River about 20 km upstream of Kuala Tembeling.

There are two possible dam sites, of which the lower site (catchment area of 1,380 km^2) is situated at 8.0 km upstream along the Tekai River from the Tembeling - Tekai junction, and the upper site (catchment area of 1,200 km^2) is situated about 18.5 km further upstream from the lower site.

Administratively, the sites belong to the Jerantut District. It takes about one and a half hours by jeep and boat from the administrative center of Jerantut to the lower project site. One reaches the upper site from the lower site by boat.

2.2 Topography and Geology

The Tekai River Basin is surrounded by the mountains of the Trengganu Coastal Range, which includes such lofty peaks as the G. Tapis (4,960 ft.), G. Dulang (3,488 ft.) and G. Ulu Bakar (4,561 ft.). These mountain ridges range NNW - SSE, nearly the same direction as the strike of stratum. Most of the rivers in the mountainous area also run in an identical direction; this directional agreement is a topographical characteristic of the region.

The direction taken by the mountain ranges and rivers has a close relationship with the geological structure of the area, with characteristics that clearly show the correlation between the geological structure and the topography.

In general, the mountainous area of the Tekai River Basin exhibits a gentle dip caused by marked weathering activity.

The gradient of the Tekai River is relatively steep in the mountainous area of its upper reaches, where it forms a steeply-graded river bed; in the lower reaches, however, the gradient becomes markedly gentle at about 1/1,000.

The geology of the Tekai River basin area can be roughly grouped into sedimentary rocks, metasediments, and granitic rocks (adamellite).

The sedimentary rocks are known as the Tembeling Group, and consist of Mesozoic conglomerate, sandstone, shale, and other rocks. These are grouped into four formations (Kerum formation, Laris conglomerate, Mangking sandstone, Terumus redbeds) according to their lithofacies.

These strata have a nearly NNW - SSE strike and, because they have large and small fold systems, they are distributed zonally.

The metasediments are known as the Bangak metasediments. These are distributed in the area interposed between the sedimentary rocks and granitic rocks (adamellite) found in the upper reaches of the Tekai River. They are distributed typically along the Bangak River and are thought to belong to Paleozoic in age. Although their metamorphic grade is low, these metasediments are generally foliated and have undergone contact metamorphism when in the vicinity of granite. They consist mainly of slate, phyllite, metaquartzite, metasandstone, semischist, and hornfels.

Granitic rocks are distributed in the Tekai River's upper reaches and are composed of adamellite. In the mountainous areas the granite has undergone deep weathering, the surface being formed of loose, coarse sand. The river bed deposits of the Tekai River are supplied largely from this granitic area.

The representative fold axes of the Tekai River Basin consist of Termus, Tekai and Penut anticline, and it is these that govern the Basin's geological structure.

Although the above large fold axes are not continuous in the neighborhood of the dam site, some small fold structures are distributed here and there. Of particular note is the irregular dip of the strata at the site of the upper dam site.

A number of small faults are distributed sporadically through the Tekai River Basin, but no large faults which could significantly govern the geological structure have been detected.

2.3 Climate and Hydrology

The climate of Peninsular Malaysia is generally influenced by the north-east monsoon and the south-west monsoon.

The north-east monsoon brings moisture from the South China Sea, usually from November to January, striking the north-eastern parts of the country first and then covering almost all of Peninsular Malaysia. All areas exposed to the north-east monsoon receive heavy rainfall, especially the northern part of the east coast. More than 1,000 mm of monthly rainfall has been experienced in the Kuala Trengganu region in the north-east. On the other hand, the northern part of the west coast, which is sheltered by the mountains of Central Malay, receives little rain during this season. Hours of sunshine during the monsoon season are the shortest of the year and air the temperature is highest in December and January throughout Peninsular Malaysia.

February and March are the driest months of the year. Relative humidity is at its lowest and hours of sunshine are the longest during these months.

Generally in April or May, the south-west monsoon reaches the west coast from across the Indian Ocean. The monsoon prevailing over Peninsular Malaysia in May-June causes fairly heavy rainfall on the west coast, but is not significant on the east coast. Due to the sheltering effect of Sumatra, rainfall during this season is less than that during the north-east monsoon season, with the exception of the northern part of the west coast. Maximum temperatures usually occur in April on the west coast and in May on the south and east coasts.

During the period between the two monsoons, from August to October, the northern part of the west coast has a peak in rainfall brought by western winds.

Rainfall

While there are many rain gauges along the Pahang River and near Raub, there are only a few rain gauges along S. Tembeling and its tributary. Two more gauges are in the catchment area above the Tekai damsite. But, due to the lack of rainfall data for the upper catchment, rainfall estimates were mainly derived from gauges in the surroundings of S. Tekai.

The average annual rainfall at Ulu. Tekai, derived with the result of correlation analysis with the Kangsar Station, was estimated at about 2,210 mm.

The probability calculation on the basis of data from 10 stations along the Tekai River Basin showed the five-day rainfall for a period of 10,000 years at about 840 mm.

Runoff

The estimation of damsite discharge in this project was based on observation data obtained at Penut downstream of the lower damsite.

But, due to the lack of measurements in the data, the estimation was finalized by supplement and correction through correlation analysis with surrounding stations and conversion from the rainfall of runoff model.

As a result, the annual mean discharge per day was estimated at 34.84 m^3/s for the upper damsite and 40.07 m^3/s for the lower damsite.

The max. discharge ever measured at the Penut Station was 1,620 m^3/s of the flood in November, 1975.

The design flood discharge of the upper damsite is 7,300 m^3/s while that of the lower damsite was determined at 1,100 m^3/s in view of the flood storage by the upper reservoir. (See Vol. I Hydrology)

2.4 Survey

(1) Collection of Data

The existing topographical maps covering the project area are 1:63,360 scale drawn up in 1975 and 1:25,000 scale (50-foot contour interval) drawn up in 1972, both by the Survey Department. These two maps cover Tekai River Basin, including the storage reservoir areas. Aerial photographs of the project area of 1:25,000 scale were taken in 1966 and 1967 by Lockwood Survey Corporation, Ltd., a Canadian aerial photogrametry company. Further, aerial photographs of 1:25,000 scale were also taken in 1975, but these do not cover the full project area. Existing maps, aerial photographs and relevant documents were obtained by the JICA Study Team via NEB - EPU - Survey Department.

(2) Aerial Photogrametry

Aerial photographs were taken at both dam sites and the environs of the lower storage reservoir where the access road will be constructed, with the purpose of drawing up topographical maps by aerial photogrametry. As for the environs of the upper storage reservoir, topographical map plotting work was carried out utilizing existing aerial photographs (1:25,000 scale, taken in 1972). The influence of the aging of the aerial photographs on the topographical mapping work is presumed to be negligible, because the project area is a mountainous region densely covered with virgin tropical jungle. Control point surveying was carried out by satellite Doppler survey and the accuracy was improved by conducting a traverse survey (L = 25 km) to assure conformity of the bench-marks between the upper dam and the lower dam sites.

Aerial photogrammetric maps of 1:10,000 scale covering the complete storage reservoir and 1:5,000 scale covering each dam site were produced.

(3) Topographic Survey

Control points were established by traverse survey at the upper and lower dam sites and a 1:500 scale topographical map drawn up by plane-table surveying.

Longitudinal profiling and cross sectioning were carried out at each dam site. Four bench-marks were established at the upper site and 2 bench-marks at the lower site. The true altitudes of these bench-marks were established by levelling survey from the national bench-mark located in Jerantut.

2.5 Outline of Pahang State

The State of Pahang is the third largest in Malaysia. Total area of the State is about 36,260 km^2 and it has a coastline extending over 200 km facing the South China Sea. Approximately 65 percent of the total area is covered with forest. Cultivated land is mainly in the basin of the Pahang River where approximately 566,000 ha. - only 16 percent of the total area - is under cultivation. Of this area, 257,000 ha. is devoted to rubber, 267,000 ha. to palm oil, 7,200 ha. is paddy and the remainder, other crops.

The State of Pahang is divided into ten administrative districts, and the state capital is situated in the city of Kuantan in the Kuantan district facing the South China Sea. The population of the state is 820,000 persons (1980 census), of which approximately 25 percent of live in the district of Kuantan. Other districts where the population density is relatively high are Pekan, Temerloh and Bentong, where industrial estates are located.

Natural population growth between 1971 and 1980 showed annual rate of 2.6 percent, which is less than 2.8 percent average for the country as a whole during this period. According to the Fourth Malaysia Plan (1981 - 1985), it is estimated that population growth will be average 2.4 percent annually.

This lower growth rate is due to low infant mortality and fertility brought about by improvements in culture, education, and medical care. During the 1970's, there was a population influx from the rural to the urban districts and migration from other states by implementation of industrial projects under the First, Second and Third Malaysia Plans, but this migration of population from within and outside the state is assumed to have taken off after the Fourth Malaysia Plan. The population growth rate in the 10-year period was 4.5 percent, but the growth rate is estimated to have decreased to 4.1 percent. The composition of the population is 58 percent Malay, 35 percent Chinese and 7 percent other races.

The national railway connects the towns of Kuala Lipis, Jerantut and Mentekab in the interior of Pahang with other major cities and towns in the Federation. Before the port of Kuantan was built, the railroad was the major means of shipping products out of the state. The road network includes of a freeway about 270 km long connecting the city of Kuantan with Kuala Lumpur. To the south, there is a road along the coastline leading to Singapore and to the north, a road along the coastline to Kota Baru. Within the state major roads are being newly constructed and existing roads upgraded and improved. There is a daily air service by Malaysian Airlines between Kuala Lumpur and Kuantan. Approximately 1,300,000 m^2 of land has been set aside to construct an airport at Jerantut. Improvement and expansion of a nationwide telecommunication system is underway. Subscriber trunk dialing by microwave and cable is in service and being expanded to cover the entire nation, including the states of Sabah and Sarawak. Development of a deep sea port has been undertaken by the Federal Government, located about 26 km to the north of the city of Kuantan. This port is at present partially operational for both the oil berth and the palm oil berth and is expected to become the major outlet for export of products of the eastern states when completed.

Historically, economic activities in the eastern states have lagged behind the western states. The major cause of this has been the undeveloped state of the infrastructures to provide impetus for economic growth. The economy of the state is basically agriculture and forestry, and all efforts are being made to promote improvements in productivity, plant breeding and use of fertilizer. The government is placing emphasis on the development of underdeveloped states having great potential and in order to promote industrialization and modernization to a level comparable to the advanced states. Since the 1960's the government has established and implemented First, Second, Third and Fourth Malaysia Plans.

In line with government policy, the state of Pahang has enacted and is actively implementing various economic policies. In order to transform itself from an economy based on agriculture and forestry, the state in aiming towards industrialization by utilizing agricultural and forestry resources as raw materials. To achieve this objective, the state has established industrial estates at Jebeng and Semanbu in Kuantan District, Pekan in Pekan District, Temerloh in Temerloh District and Bentong in Bentong District. In these industrial estates there are wood product manufacturing industries, rubber processing and manufacturing industries, palm oil refining and manufacturing industries and also export-oriented electronic and electrical equipment manufacturing industries, but these are all of small scale. The government is also actively inviting foreign manufacturers to invest in the country by enacting laws granting various privileges and tax relief. Anticipating development and growth of export-oriented industries in the future, the government has decided to set up a free trade zone in Gebeng Industrial Estate.

The State of Pahang is blessed with recreational and resort regions along its some 200 km long coastline and the highlands in the interior. The state has built recreational and resort facilities in these places and is actively attracting tourists, both domestic and from overseas.

Training and education of a labour force to support the industrialization program is essential. In the state there is a polytechnic school, six vocational schools and skills training institutes which can supply the skilled man-power needs of the industries in the state.

With the implementation of economic policies under the Malaysia Plans, the gross domestic product (GDP) and per capita GDP of the state has grown progressively. In 1971, the last year of the First Malaysia Plan, the GDP was 647 million M$ and per capita GDP was 1,170 M$ at 1970 prices. This per capita GDP was 1.0 as compared to the national average. In the Third Malaysia Plan ending in 1980, the state's target GDP was 1,218 million M$ and per capita GDP 1,486 M$ at 1970 prices. In the Fourth Malaysia Plan ending in 1985, the predicted GDP will be 2,491 million M$ and per capita GDP 2,558.8 M$ at 1970 prices. This per capita GDP ratio is 1.09 to the national average. During the period (1971 - 1980) inflation as indicated by the consumer price index averaged a rate of 5.8 percent annually.

The GDP composition by industry at the end of the First Malaysia Plan was agriculture and forestry 48.5 percent, manufacturing 6.5 percent and services 30.1 percent. However, according to the Third Malaysia Plan the composition was agriculture and forestry 38.2 percent, manufacturing 26.2 percent and services 25.9 percent. In the Fourth Malaysia Plan ending in 1985, the GDP composition should be agriculture and forestry 27.75 percent, manufacturing 29.1 percent and services 28.5 percent. It will be noted from the above given values that the economy of the state will be shifted from basically agriculture and forestry to manufacturing. In this context, it is hoped that the socio-economic infrastructures of the country can be implemented as quickly as possible.

3. HYDROELECTRIC DEVELOPMENT

3. Hydroelectric Development

3.1 Introduction

The JICA survey team visited Malaysia for approximately 5 months from June to October 1981 with the purpose of drawing up the hydroelectric power development project in the Tekai River and studying the feasibility of the project. During this period of stay in Malaysia the survey team carried out preliminary field investigations as well as examination of the project. Examination consisted of comparative study of the merits and demerits of the two development sites (Upper Tekai Site and Lower Tekai Site) proposed in the project area, in order to determine the most advantageous development system and the approximate scale of the development project. The contents of the study carried out of in the preliminary investigation stage are presented in the Interim Report of March 1982. According to this report integrated development consisting of an Upper Dam and a Lower Dam is considered most advantageous. The JICA survey team carried out a further additional detailed field investigation and collection of relevant data and information over a period of approximately 7 months from May to December, 1982, based on the development alternative proposed in the Interim Report. Review of the development project based on the newly obtained data and information from the detailed field investigation concludes that the series development is feasible.

3.2 Dam Site Geology

3.2.1 Geology of Upper Dam Site

The geology of the upper site is composed of Termus redbeds and Mangking sandstone. The Termus redbeds are distributed along the Termus River, in the area upstream of the dam site and are thought to belong to the Cretaceous period of Mesozoic in age.

Around the Upper Site, the Termus redbeds are composed mainly of shale (Tsh) and sandstone.

The shale (Tsh) consists of red and red-violet rock with well developed cleavage, intercalating massive mudstone in places. There is also much pronounced weathering, with weathered rock extending to great depths. The mountains following the Termus River, having distributions of this shale (Tsh), therefore take the form of gradual slopes.

The sandstone is composed mainly of fine quartzose sandstone, and is distributed only rarely around the Upper Site.

Mangking sandstone is distributed widely at the Upper Site. Thought to belong to Jurassic period of Mesozoic in age, it is composed mainly of sandstone and shale. Sandstone predominates at the upper sites, western end, while shale is distributed widely in the eastern sector upstream.

The sandstone is a hard, compact rock forming steep mountains along the Tekai River. The shale, in the other hand, shows developing cleavage and progressive weathering advancing toward the deeper strata.

The Mangking sandstone is distributed below the Termus redbeds, and is inferred to come into contact with the latter, having a NNW - SSE geological boundary of conformity, in the vicinity of the confluence of the Termus and Tekai Rivers.

The rocks at the upper site were classified according to their lithofacies into various groups described above. A further classification was then performed from the engineering geological standpoint, resulting in the four groups of sandstone, shale, alternation of sandstone and shale, conglomerate.

Overlying these strata are terrace deposits (Qtr) and river bed beposits (r) distributed along the river, and small talus deposits distributed on the mountainside slopes.

Terrace deposits (Qtr) are distributed widely around the confluence of the Tekai and Termus rivers and on a small scale along the Tekai River at the Upper Site. Terrace surface, consisting of comparatively firm clay, sand, and small gravel, is distributed along the left bank of the Tekai River some 250 m upstream of the dam site and also some 200 m downstream of the dam site.

River bed deposits are distributed along the Tekai River. These are composed of sand, silt and other material carried by the river. This is a relatively shallow stratum, measuring 5 m or less at the river bed.

Strata at the Upper Site generally strike in a NNW - SSE direction. Large and small fold axes, following the Termus syncline (Fig. I) are distributed here. Fold axis (synclines, anticlines) are distributed at the Upper Site, and for this reason the strata here do not have a uniform dip.

Small-scale faults, and fractured zones, are also distributed at the Upper Site. Also two large fractured zones have a NNE - SSW strike, distributed at the upper and lower reaches of the Upper Dam Site. One of these intersects the dam axis at a low angle and may continue to the right bank abutment.

On the basis of the geological conditions described above, a narrow point in the valley's width was selected as the site for the dam axis, and geological investigation was carried out on: a shale distribution area along the left bank of the Tekai River some 800 m upstream of the dam site, as an upper borrow site (Site A); a shale distribution area along the right bank of the Tekai River about 1 km downstream of the dam site, as an upper borrow site (Site B); and a sandstone distribution area at the latter location, as an upper quarry site (Site B).

3.2.2 Geological Engineering Assessment of Upper Dam Site

The upper dam site is considered to be a good site for a dam since it is composed of sandstone and shale, with its valley relatively narrow and endowed with a shallow layer of rock suited for the foundation of a fill dam. Although the strata have undergone folding, in the area upstream of the dam site they dip toward the upper reach. The center core type rock fill dam is considered suitable for this site since construction materials (core, rock, etc.) are available in its vicinity.

On the right bank of the dam site, there is inferred to exist a fractured zone of relatively large scale (estimated at 20 to 40 meters), which intersects with the dam axis at a low angle. Careful attention is therefore required in foundation treatment and the design structures.

3.2.3 Geology of Lower Dam Site

The geology of the lower site resembles that of the upper site in that it is composed of Hangking sandstone thought to date from Jurassic period of Mesozoic in age.

The Hangking sandstone found at the Lower Site consists mainly of sandstone and shale, with sandstone having the wider distribution.

The sandstone is a hard, compact rock that forms the right bank of the dam axis. The shale is distributed at the left bank of the dam axis; it shows developed clearage and deeper weathering than the sandstone.

The Mangking sandstone found at the Lower Site is classified into seven groups, as shown in the stratigraphy of the lower site.

Downstream of the dam site on the right bank is an area in which sandstone is widely distributed; this site was selected and investigated as the lower quarry site (Site C) for the lower dam (Fig. II). The lower quarry site is composed of the same Mangking sandstone as the dam site, also with sandstone predominating four strata of shale are interposed.

The rocks at the lower site were classified according to their lithofacies into the seven groups described above. A further classification was then performed from the engineering geological standpoint, resulting in the three groups of sandstone, shale, alternation of sandstone and shale.

Overlying these strata are terrace deposits (Qtr) and river bed deposits (r) distributed along the river, and small talus deposits distributed on the mountainside slopes.

Terrace deposits (qtr) are distributed on a small scale in the lower reach of the dam site and at bends in the river. These are composed of clay, sand, and small gravel.

River bed deposits (r) are distributed along the left bank of the dam axis; no other large distributions have been observed. The river bed deposits are composed mainly of quartzose sand and also contain silt and small gravel. They are also distributed along the river bed, having an irregular thickness ranging from 3 ∿ 10 m.

While almost no talus deposits are distributed at the dam site, they are distributed to a comparatively wide extent on the mountainside slopes at the lower quarry site (Site C).

The strata at the lower site generally strike in a NE ∿ SW direction. At the dam site, the strata dip 20° ∿ 40° to the southeast; at the lower quarry site at the lower reach of the dam site they dip 30° ∿ 60° to the southeast, exhibiting a monocline structure.

While the lower site does contain small-scale faults and fractured zones, there have been no confirmations of any faults which could govern the geological structure or of any fractured zones which could exert an effect upon structures.

3.2.4 Geological Engineering Assessment of Lower Dam Site

The lower dam site is composed of sandstone and shale. This site is geologically inclined upstream. Weathered zones in both right and left banks are a little deeper than those in the upper dam site.

A fill dam and concrete gravity dam are among the dam types possible here. But the latter brings greater advantages because of lower construction costs. As for the former no promising place has been found as yet for securing construction materials, especially core material.

3.3 Construction Materials

The JICA survey team investigated four quarry and borrow sites in the vicinity of the Upper and Lower Tekai Dam Sites to determine the availability of core material, rock material and aggregate for construction of the center-core type rock fill dam at the Upper Site, and the availability of aggregate for construction of the concrete gravity dam at the Lower Site. The 4 sites investigated are located in the vicinity of the Upper and Lower Tekai Dam Sites and are identified as sites A, B, C and D, respectively, from the upper reaches of the Tekai River. (Refer to the Fig.II)

Sites A and B are located upstream and downstream of the Upper Dam, and were examined for supply of core materials, rock materials and aggregate. The core material available at both sites consists of highly weathered sandstone and shale. Soil tests of samples collected from the pits excavated at both sites indicate that they have sufficient impermeability to be used as core material. The coefficient of permeability of these samples is of the order of 1×10^{-7} cm/sec. The difference between the natural moisture content and the optimum moisture content is minor. Accordingly, the core material can be transported directly from the borrow site to the upper dam site. The maximum dry density of the material is approximately 1.8 g/cm^3.

At site A the topography is rather steep and the highly weathered rock layer is thin, but at site B, which has plateau topography, the layer of highly weathered rock is thicker compared with site A. Site B appears more advantageous from the point of view of available deposit of core-material and ease of excavation work, therefore.

Fresh rock and slightly weathered rock located beneath the highly weathered rock for core material at sites A and B were examined for use as rock material and aggregate. Rock tests were carried out using boring cores obtained at these sites.

The results of these rock tests indicate that the sandstone has a specific gravity of approximately 2.55 and the percentage of water absorption does not exceed 3% at either site A or site B. The shale has a specific gravity of approximately 2.35 and a rather large percentage of water absorption. Normally, aggregate of good quality is required to have specific gravity larger than 2.5 and a percentage of water absorption rate below 3%. Accordingly, sandstone seems better than shale for use as aggregate and rock material.

On the other hand, the results of the geological investigations indicates that site B is the more advantageous because it contains more sandstone.

Site C is located on the right bank of downstream of the Lower Dam. Rock tests of boring cores were carried out to examine the potential of this site for sandstone and shale aggregate supply. The results of these rock tests are the same as those for sites A and B. Geological investigation indicates that the weathered rock layer at site C relatively thick.

Site D is located on the upper reaches of the Tembeling River downstream of the Lower Dam. Site D was examined for limestone aggregate. Geological investigations were not carried out at the site but the results of rock tests with boring cores obtained at the site are quite satisfactory, showing specific gravity of 2.7 and a percentage of water absorption not exceeding 0.3%.

Excavated material of the foundations, spillways, power stations, etc., of the upper and lower dam sites consists of sandstone and shale. Accordingly, this excavated material can be used good as rock material and for other purposes.

It is recommended that core material, rock material and aggregate should be excavated principally from site B, taking into consideration the results of the geological investigation and the ease of work, because it is located in the vicinity of the Upper Dam, which requires movement of a considerable volume of earth and needs a large deposit of core material. The volumes of core material, rock material and aggregate are indicated in the following table. The average thickness of avilable weathered layer for core material is assumed to be distributed respectively 5 m and 8 m in Quarry Site (Site B, B-1) and Borrow Site (Site B, B-2).

600,000 m^3 are necessary for core material and about 1,000,000 m^3 of site volume will be required by taking into consideration the yeild rate and bulking factor.

Approximately 56,000 m^2 from the borrow site and approximately 113,000 m^2 from the quarry site could be gathered in the borrow site in order to get 1,000,000 m^3 (8 m x 56,000 m^2 + 5 m x 113,000 m^2) of site volume.

		Demand	Supply		
		Volume of work (Va)	Natural Ground (Vb)		
		m^3	Site B-1	Site B-2	Remarks
Upper Damsite	Core	604,300	455,000	567,000	
	Rock	2,161,800			
	Filter	280,300			
	Riprap	79,000	2,475,800		
	Aggregate	131,100			
Lower Damsite	Aggregate	89,900			

V_a : Volume of work

V_b : Natural ground

Y_r : Yield rate (Core ... 70% Rock .. 80% Aggregate .. 60%)

B_f : Bulking factor (Core .. 0.85 Rock .. 1.4 Aggregate .. 1.65)

$$V_a = V_b \times Y_r \times B_f$$

3.4 Selection of Dam Site

3.4.1 Selection of Dam Site

The lower reaches of the Tekai River show a very gentle river gradient of the order of 1:1,000, and high potential for a large storage capacity through construction of dams. The area of the present development project is restricted exclusively to the lower reaches of the river with a view towards utilizing as effectively as possible the hydroelectric potential there. Two development sites (Upper site and Lower site) for construction of dams were selected based on the following premises.

- Outcropping of bedrock
- Satisfactory geological conditions
- Proximity of the river banks
- Topography suited for construction of a large storage reservoir in the upper river reaches

The upper site is located downstream of the confluence of the Tekai River and the Termus River. The river bed has rather steep topography and outcroppings of bedrock are observed from the river bed halfway up the slopes on both banks.

The lower site is located approximately 20 km downstream of the upper site and shows similar conditions.

In the present feasibility study the mission carried out field explorations from March 1 to 25, 1981 to confirm the appropriateness of these development sites.

Three dam axes, i.e., U-1, U-2 and U-3, were selected at the upper site and U-2 was taken up in the inception report as the top priority dam axis in view of the favourable topographical and geological conditions. Two dam axes, i.e., L-1 and L-2 were selected at the lower site and L-1 was taken up in the inception report as the top priority dam axis because of the favourable topographical and geological conditions.

3.4.2 Selection of Dam Axis

(1) Upper Dam

At the upper dam site the river is approximately 60 m wide and the slopes of the banks are very steep. Geological investigations conducted at this site indicate that it consists principally of sandstone and shale of the Mangking sandstone type, as well as sandstone and shale of the Termus sandstone type. The thickness of the weathered layer is approximately 10 m at the ridges and approximately 5 m on the slopes on both banks of the site. The sedimentary layer in the river bed is relatively thin, being of the order of 6 to 7 m and the foundation rock is extremely hard. Three dam axes, i.e., U-1, U-2 and U-3 were selected for comparison at the upper dam site, taking into consideration topography, geology, construction work conditions, etc.

Topographical and geological data obtained through field exploration of each dam axis and comprehensive evaluation of the data are shown in the following table. Topographical sections and plans of the dam axes are shown in Fig. 3.1.

Comparison of Upper Site Dam Axes (Results of Field Exploration)

Topographical and Geological Conditions \ Dam Axis			U-1	U-2	U-3
Topographical Conditions	Gradient of slope	Right bank	Rather steep (1:1.8)	Steep (1:1.5)	Rather steep (1:1.3 - 1:5.0)
		Left bank	Gentle slope (1:2.5)	Steep (1:1.4)	Gentle slope (1:1.8 - 1:8.0)
	River bed width		Wide (Approx. 80 m)	Narrow (Approx. 60 m)	Narrow (Approx. 40 m)
	Distance between banks at 165 m elevation		Rather large (Approx. 490 m)	Small (Approx. 320 m)	Large (Approx. 710 m)
	Topography in the environs of the centerline	Right bank	With formation of ridge	With formation of ridge	With formation of ridge
		Left bank	No formation of ridge (centerline should be curved upstream to follow the ridge)	With formation of ridge	Rather modest ridge
	Outcrop of bed-rock in the environs of the river bed	Right bank	No good (shale)	Satisfactory (sandstone)	Satisfactory (sandstone)
		Left bank	No good (shale)	Satsifactory (sandstone)	Satisfactory (sandstone)

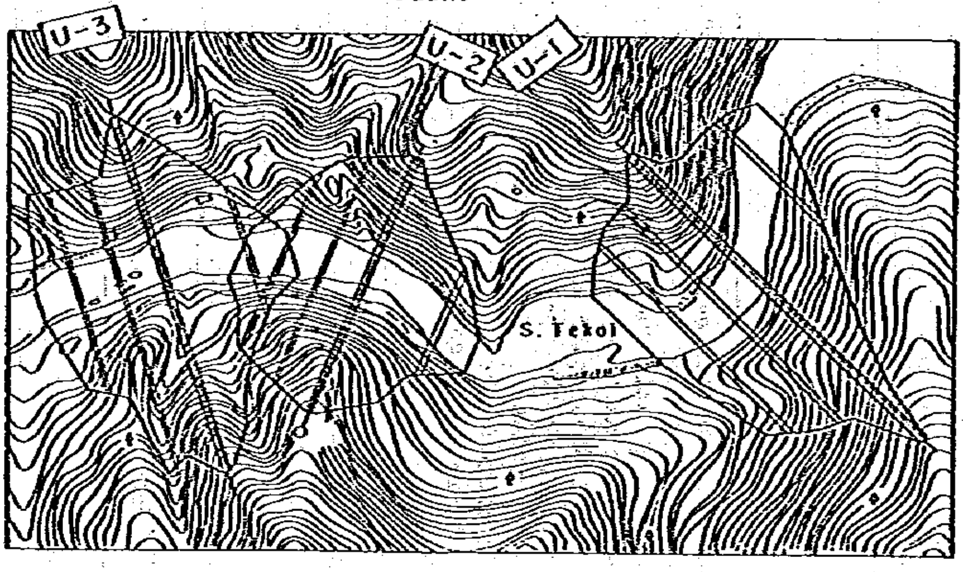
(to be cont'd)

(Cont'd)

Topographical and Geological Conditions \ Dam Axis			U-1	U-2	U-3
Geological conditions	Geological condition		Strata generally dips in upstream direction	Centerline inclined with regard to anticline axis and sincline axis. Strata dips principally to the upstream side. Fractured zone is inffered in right bank.	Centerline coincides with anticline axis. This anticline axis presents very frequent cracks (pronounced permeability).
	Excavation depth with abut (dam foundation rock depth)	Right bank	Approx. 10 m	Approx. 10 m	5 m - 10 m
		Left bank	Approx. 10 m	15 m - 20 m	15 m - 20 m (estimated)
Comprehensive evaluation	Topographical conditions		Inadequate	Optimum	Inadequate
	Geological conditions		Adequate	Adequate	Inadequate

Fig. 3.1 Selection of Dam Sites (Upper Dam Sites)

Plane



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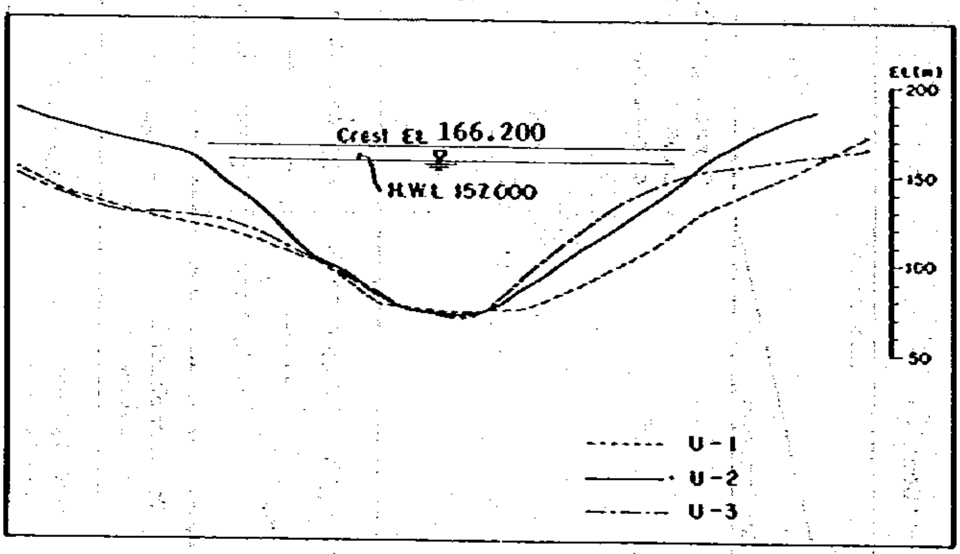


Comparison of the construction costs of dam layouts corresponding to each dam axis (Refer to Fig. 3-1) shows that the U-2 axis is the most advantageous alternative from the economic point of view because it has the shortest dam axis and the smallest dam body volume. The comparison was made assuming the construction of a rockfill dam with a height of 100 m.

Dam body volume and construction cost of each alternative are as follows:

Dam Body Volume

Item \ Dam Axis	Unit	U-1	U-2	U-3
Excavation	m^3	535,000	442,000	486,000
Core embankment	m^3	691,000	604,000	679,000
Filter embankment	m^3	330,000	280,000	324,000
Rock embankment	m^3	2,695,000	2,162,000	2,652,000
Riprap and protection rock	m^3	97,000	79,000	88,000

Dam Body Construction Cost

(Unit : M$)

Item \ Damsite		U-1	U-2	U-3
Earth excavation		2,541,000	2,100,000	2,309,000
Rock excavation		6,153,000	5,083,000	5,589,000
Dam body	Core embankment	9,674,000	8,460,000	9,506,000
	Filter embankment	5,775,000	4,905,000	5,670,000
	Rock embankment	41,773,000	33,508,000	41,106,000
	Riprap and rock protec-tection materials	1,795,000	1,462,000	1,628,000
	Drilling	3,640,000	2,873,000	3,640,000
	Cement grouting, etc.	5,670,000	4,455,000	5,670,000
Foundation treatment tunnel, etc.		2,779,000	2,354,000	2,682,000
Total		79,800,000	65,200,000	77,800,000

The U-2 alternative was selected as the optimum dam axis taking into consideration its merits and demerits regarding topography, geology, dam body construction cost and spillway, waterway and power station structure.

(2) Lower Dam

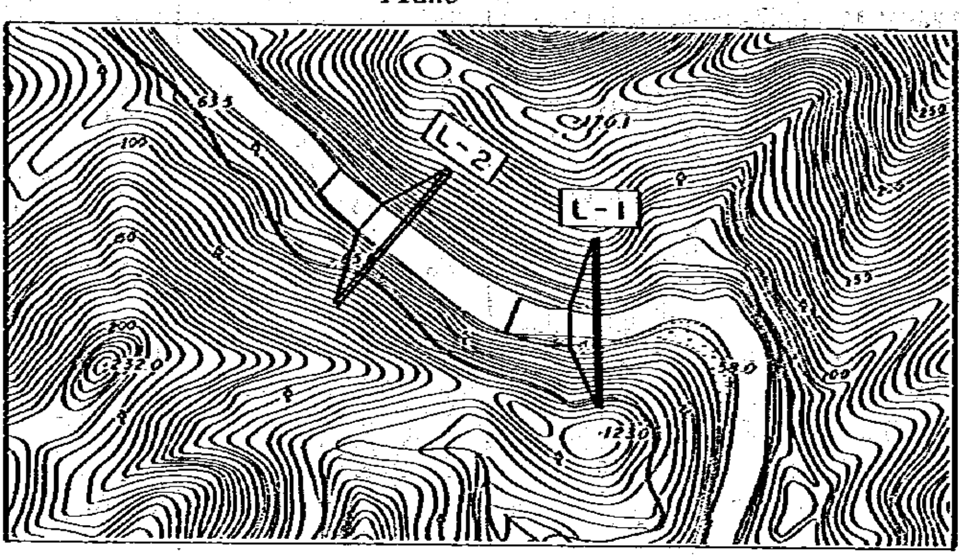
The topographical and geological conditions of the Lower Dam Site are similar to those at the Upper Dam Site. The geology of the Lower Dam Site consists of sandstone and shale of the Mangking sandstone type. Two dam axes, i.e., L-1 and L-2, were selected at the site taking into consideration the topography, geology, condition of construction work, etc. Topographical and geological data on these two dam axes (L-1 and L-2) obtained by field exploration and a comprehensive evaluation of these data are shown in the following table. The sections and plans of the dam axes are shown in Fig. 3.2.

Comparison of Lower Site Dam Axes (Results of field reconnaissance)

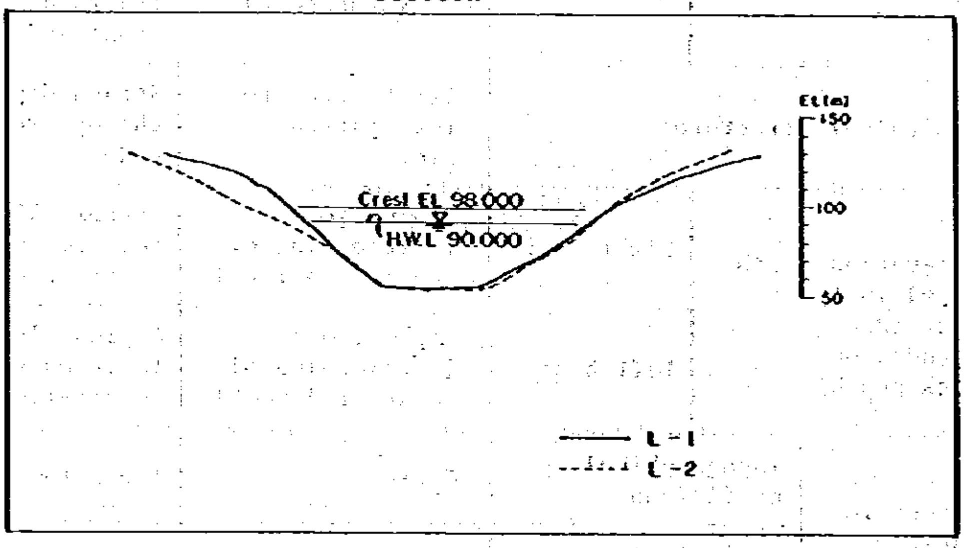
Topographical and Geological Conditions \ Dam Axis			L-1	L-2
Topographical Conditions	Slope gradient	Right bank	Steep (1:1.5)	Steep (1:1.5)
		Left bank	Steep (1:1.3)	Rather steep (1:1.8)
	River bed width		Narrow (Approx. 50 m)	Narrow (Approx. 50 m)
	Distance between banks at 80 m elevation		Small (Approx. 130m)	Rather small (Approx. 145m)
	Topography in the vicinity of the centerline	Right bank	With formation of modest ridge	With formation of insignificant ridge
		Left bank	Unchanged	Unchanged
Geological Conditions	Outcrop of foundation rock in the vicinity of the river bed	Right bank	Hard rock outcrop (sandstone)	Poor (alternate layers of sandstone and shale)
		Left bank	Satisfactory (sandstone)	Poor (alternate layers of sandstone and shale)
	Geological structure		Strata dips to the upstream side	Strata dips to the upstream side
	Excavation depth in vicinity of abut (Dam foundation rock depth)	Right bank	Approx. 10 m (outcropping of weathered rock)	Approx. 10 m (covered with weathered soil)
		Left bank	Approx. 10 m (outcropping of weathered rock)	Approx. 10 m (covered with weathered soil)
Comprehensive evaluation		Topographical conditions	Optimum	Adequate
		Geological conditions	Optimum	Adequate

Fig. 3.2 Selection of Dam Sites (Lower Dam Sites)

Plane



Section



As can be seen from the topographic maps of the two dam site alternatives, dam axis length and dam body section show no major differences. Further, the topographical configurations of the left and right banks are similar at both sites. Dam body construction costs at both sites are compared in the tables below. A comparative study was carried out assuming construction of 2 gravity-type concrete dam with a height of 55 m (Refer to Fig. 3.2). The volume of concrete required for construction of the dam body, including the spillway, and relevant costs assumed for comparison purposes is as follows.

Dam Body Concrete Volume

Item \ Dam Site	Unit	L-1	L-2
Earth excavation	m^3	84,100	85,000
Rock excavation	m^3	52,600	83,500
Concrete	m^3	173,300	207,000

Construction Cost

(Unit : M$)

Item \ Dam Site		L-1	L-2
Earth excavation		799,000	808,000
Rock excavation		1,210,000	1,461,000
Dam concrete		32,927,000	39,330,000
Grouting	Drilling	2,275,000	2,418,000
	Cement grouting	2,835,000	3,105,000
Foundation treatment tunnel, etc.		4,724,000	4,778,000
Total		44,770,000	51,900,000

Dam axis L-1 is selected as the optimum dam site as a result of comparison of construction costs.

3.5 Type of Dam

3.5.1 Upper Dam

Examination of hydrological data and hydroelectric output of the Tekai River basin points to construction of a dam with a height of 101 m and a 166.2 m dam crest elevation at the upper dam site. Each type of dam is examined in this section through comparative study of respective construction costs.

As for dam type, a rockfill dam and a concrete gravity type dam seem to be best in view of the topographical and geological conditions at the dam site. There is no site in the Tekai River basin with geological and topographical conditions suited to construction of an arch dam. On the other hand, construction of an earthfill dam is not recommended because it requires a sufficient slope gradient and the necessary volume of construction materials is not available nearby.

Such being the case, the merits and demerits of the following types of dam are comparatively examined in this section.

Rockfill Dam	Earth core impermeable wall type Concrete facing type Asphalt facing type
Concrete Dam	Concrete gravity dam

The layout and preliminary design of these dams are shown in Volume VI Drawings.

The outline of the equipment of these types of dam is given below.

(1) Rockfill Dam Alternative

The spillway is located on the left bank to attain the shortest route. As for power station equipment, the intake is positioned on the right bank immediately upstream of the dam and water is conveyed to the generators installed on the right bank downstream of the dam by means of one burried penstock. One 8 m dia. diversion tunnel is located on the left bank taking into consideration the topography of the dam site and the layout of other structures.

(2) Gravity Concrete Dam Alternative

In the concrete dam alternative the power station is built on the right bank immediately downstream of the dam as in the case of the rockfill dam alternative because the natural ground presents a gentle slope at this site. The intake is connected with two penstock installed in the dam body. Detailed design of the dam body and each structure is described in Volume V.

The costs of each alternative are comparatively examined in the following table. The spillway, diversion facilities, waterway structures and power house are included as accessory facilities. The comparative examination of the costs is given in Malaysian Dollars (M$).

(Unit : 10^3M$)

Structure	Type of Dam			
	Rockfill Dam			Concrete Gravity Dam
	Concrete Facing	Asphalt Facing	Earth Core	
Diversion and care of river	24,124	24,014	20,572	19,805.5
Dam	62,508	64,819	59,976	223,372
Spillway	21,200	21,200	21,200	8,000
Instake structure	3,512	3,512	3,512	9,034
Penstock	25,654	25,654	25,654	
Power house	20,450	20,450	20,450	20,030
Switching yard	1,330	1,330	1,330	1,330
Mechanical equipment	3,500	3,500	3,500	3,927
Total	162,278	164,479	156,194	285,498.5

The above data indicate that in the case of conrete dam the concrete cost is large compared with the rockfill dam because the dam body volume is very large. On the other hand, in the case of a rockfill dam the costs of an earth core impermeable wall and facing type show a difference, but the asphalt impermeable facing with 100 m dam height presents considerable demerit as to the ease of execution of the construction work and other relevant conditions compared with the earth core impermeable wall rockfill dam.

It is concluded that construction of an earth core impermeable wall rockfill dam is recommendable for this dam site. This alternative has the following merits.

i) Geological and topographical conditions of this dam site are best suited to construction of a rockfill dam.

ii) Rockfill and core material of good quality are available in abundance in the vicinity of the dam site.

iii) Most of the construction materials are locally available and therefore the project does not require external (foreign) sources for supply of construction materials. Furthermore, construction would not be influenced by the foreign exchange situation.

3.5.2 Lower Dam

The Lower Tekai Dam is to function as a regulating pondage for the upper site in the form of power source for peak hours and to regulate residual area. The optimum scale for the lower site seems to be 38 m dam height and approximately 81 m dam crest elevation. Comparative examination of construction costs of dam types was carried out in the same way as for the upper dam. Only the dam types shown below were studied because the arch dam and the earthfill dam are not suitable for the lower dam site.

Concrete Dam	Concrete Gravity Dam
Rockfill dam	Earth core impermeable wall type dam Concrete facing dam Asphalt facing dam

The layout and preliminary design of these dams are shown in Volume VI Drawings.

An outline of the equipment for each type of dam is given below.

(1) Concrete Gravity Dam Alternative

In the concrete dam the intake and penstock are attached to the dam body. Water is conveyed to the power house just downstream of the dam on the right bank of the river by means of these facilities. The study was carried out assuming construction of the concrete dam during the upper dam filling period by adopting sluice diversion.

(2) Rockfill Dam Alternative

The spillway is located on the right bank where the natural ground slopes gently. Water is conveyed from the intake on the left bank upstream of the dam to the power house on the left bank downstream side of the dam via one burried penstock.

One 6 m-diameter diversion tunnel is constructed on the right bank.

Construction costs of each type of dam with 81 m dam crest elevation are compared in the following table. Related facilities such as spillway, diversion tunnel, waterway structure, and power house are included in the costs.

(Unit : 10^3M$)

Structure	Type of Dam			
	Rockfill Dam			Concrete Gravity Dam
	Concrete Facing	Asphalt Facing	Earth Core	
Diversion and care of river	7,153	7,153	7,153	5,078
Dam	15,333	15,353	10,756	15,923
Spillway	9,427	9,427	9,427	4,950
Intake structure	1,210	1,210	1,210	1,600
Penstock	3,465	3,465	3,465	810
Power house	6,440	6,440	6,440	6,440
Mechanical Equipment	1,210	1,210	1,210	1,210
Total	44,238	44,258	39,661	36,011

The aforestated cost comparison points to construction of a gravity concrete dam at the lower site.

3.6 Optimum Project Size

To determine the optimum project size, including dam height, installed capacity and spillway crest length, a study was made to determine construction costs for a range of project sizes and the comparable power benefits in terms of kW and kW·H values. These costs and benefits were converted into annual equivalents and size comparisons made using B/C and B-C values. For these purposes, other benefits and costs of the Project were ignored.

The annual power benefits of kW and kWh are as follows.

1 kW = 142.7 M$

1 kWh = 0.19 M$

These values were based on the alternative thermal (gas turbine) power generation costs determined from the following factors (see Chapter 5 for details):

(1) Annual Capital Cost : M$86.79/kW

The capital cost of a gas turbine plant of M$660/kW, based on data supplied by NEB, was converted annual capital cost with a capital recovery factor (CRF) of 0.1315 assuming 15 years of economic life and a 10 percent discount rate.

(2) Operation and Maintenance Costs : M$40.72/kW

The operation and maintenance costs include personnel expenses, provision of spare parts and replacement of buckets and nozzles, which is assumed to occur every two years.

(3) Fuel Cost : M$0.1935/kW·H

Fuel consumption was obtained according the plant operation assumed in Chapter 5. The fuel oil unit cost of M$617/ton (US$30/bbl) is as based on Singapore f.o.b. prices for medium fuel oil in early 1982.

(4) Adjustment Factors for kW and kW·H Values : 1.119 and 0.982

These adjustment factors were calculated taking into account the differences in transmission loss rate, accident frequency, percentage of station usage and overhaul rate attributable to the hydro project and the alternative gas turbine units.

3.6.1 Upper Tekai

(1) Study of Dam Height

In the study of dam heights, 4 cases - 80 m, 90 m, 100 m and 110 m - were compared. In the comparison, the following basic conditions were applied.

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

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

(c) Effective drawdown is determined from the area-capacity curve plotted from the aerophotographic map of the reservoir area prepared in 1982.

(d) Reservoir maximum storage level is the ungated spillway crest elevation (spillway width - 40 m).

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

Table 3.1 Study of Dam Height

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

(Reference)

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

(2) Study of Drawdown (Effective Storage Capacity)

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

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

(b) Normal reservoir level is one-third of the effective storage capacity.

(c) There are slight differences between each case, but the tail-race water level was taken as 75.0 m for all cases.

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

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

Table 3.2 Study of Effective Depth

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

(3) Study of Installed Power Capacity

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

The following table indicates that an installed capacity of 150 MW is the most optimal.

Table 3.3 Study of Installed Power Capacity

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

(Reference)

Peak Plant Operation Time (hr)	11.1	5.5	3.6	2.7

(4) Study of Spillway Overflow Crest Length

In the foregoing studies the major features of the development scale were established. The last study is of the spillway. Comparison was made of three cases - 40 m, 60 m and 80 m - of overflow crest length as shown in the following table.

As a result of the study, a spillway crest length of 40 m is adopted.

Table 3.4 Study of Spillway Crest Length

Spillway Crest Length	(m)	40	60	80
Dam Height	(m)	100	100	100
Effective Storage Capacity	($10^6 m^3$)	680	680	680
High Water Level	(m)	157	157.5	158
Low Water Level	(m)	147	147.5	148
Effective Depth	(m)	10	10	10
Normal Water Level	(m)	153.7	154.2	154.7
Tailrace Water Level	(m)	75	75	75
Maximum Turbine Discharge	(m^3/s)	235	233.5	232
Installed Capacity	(mW)	150	150	150
Construction Cost	(10^6M$)	289	297	306
Annual Cost	(10^6M$)	35.83	36.71	37.77
Annual Energy Output	(10^6kWh)	194.8	196.2	197.3
Annual Benefit	(10^6M$)	58.42	58.69	58.90
B/C		1.63	1.60	1.56
B - C	(10^6M$)	22.59	21.98	21.13

(5) Optimum Upper Tekai Development Scheme

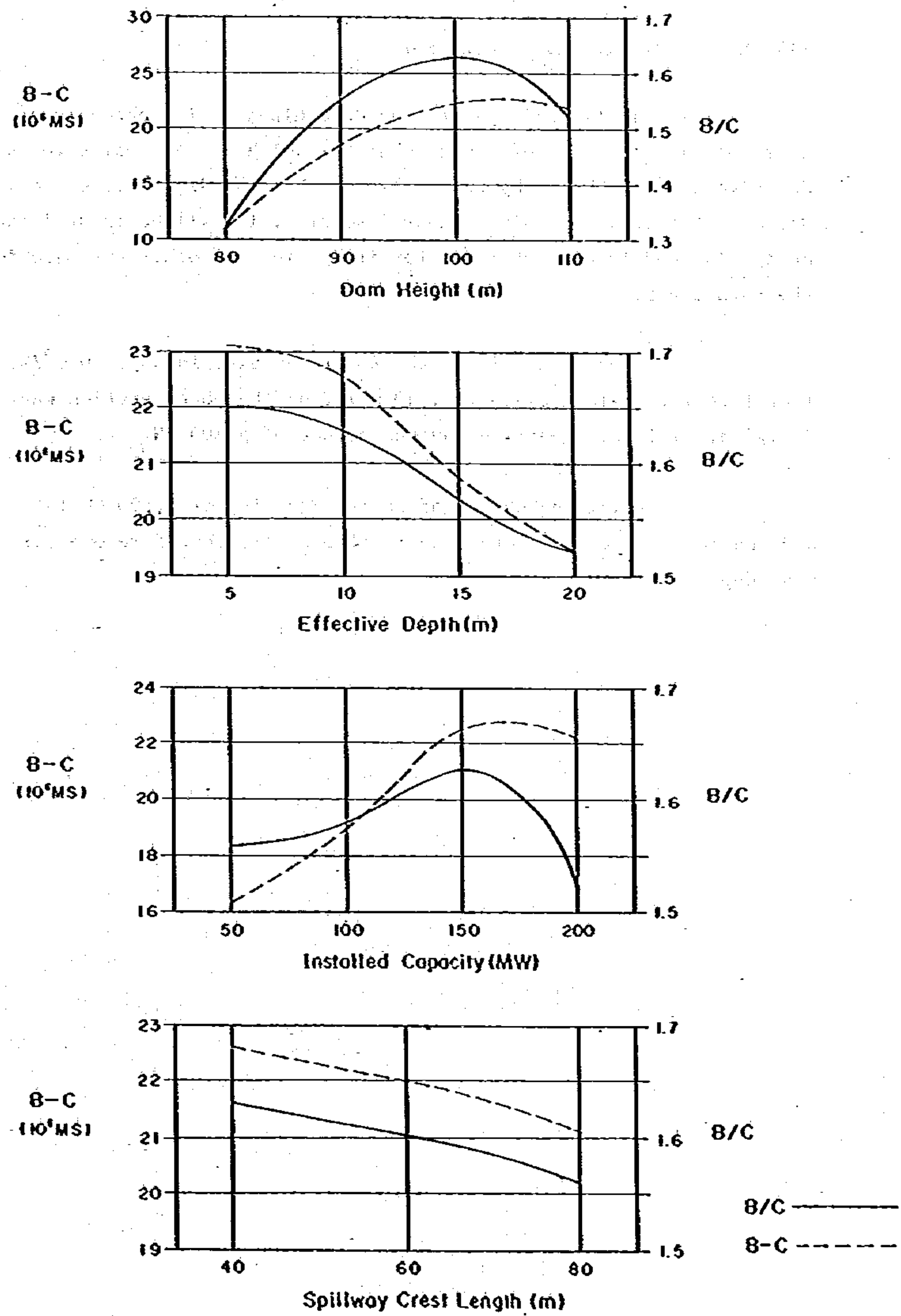
The results of the various comparison studies for the development of the Upper Tekai are graphically depicted on next page. A table is also attached giving the features of the optimum development scheme for the Upper Tekai.

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

Table 3.5 Optimum Upper Tekai Development Scheme

Dam	Type		Rock-fill
	Crest Elevation	m	166.2
	Foundation Elevation	m	65.2
	Dam Height	m	101
Reservoirs	High Water Level	m	157
	Low Water Level	m	147
	Effective Depth	m	10
	Normal Water Level	m	153.7
	Effective Storage Capacity	10^6 m^3	680
	Gross Storage Capacity	10^6 m^3	2,040
Tailrace Water Level		m	75
Rated Effective Head		m	75.1
Maximum Turbine Discharge		m^3/s	235
Installed Capacity		mW	150
Annual Energy Output		10^6 kWh	194.8
Construction Cost		10^6 M$	289
Annual Cost		10^6 M$	35.83
Annual Benefit		10^6 M$	58.42
B/C			1.63
B-C		10 M$	22.59

Fig. 3.3 B/C and B−C for Optimum Project Size (Upper Tekai)



3.7 Lower Tekai

(1) Most Suitable Development Form

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

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

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

4 . DESIGN AND CONSTRUCTION

4. Design and Construction

4.1 General

The Tekai Hydroelectric Power Development Project is to construct 2 (two) dams, Upper Tekai and Lower Tekai Dams, on the lower reaches of the Tekai River, a tributary of the Pahang River and to develop 150 MW of power at Upper Tekai, 5.8 MW of power at Lower Tekai Sites.

The Upper Dam Site is located approximately 1 km downstream of the confluence of the Termus and Tekai Rivers. In consideration of the topography and geology of the site and comparative study of various types, a rock-fill dam was adopted. The dam will be 101 m high, have a 350 m of crest length and a volume of approximately 3,125,000 m^3.

The reservoir created by the dam will have an effective storage capacity of 680 x $10^6 m^3$ which will regulate the annual inflow at the site. A maximum of 235 m^3/sec. of water will be conveyed through one pressure tunnel of diameters of 7.3 m - 4.6 m each about 580 m long, to a powerhouse located on the right bank immediately downstream of the dam.

The Upper Tekai Power Station will generate a maximum of 150 MW of electricity at a rated head of 75.1 m and produce 194,800 MWH of energy annually.

The lower dam, which will function as a re-regulating reservoir for the upper dam and for regulating the residual area, will develop 5.8 MW electric power.

The lower dam site is located approximately 20 km downstream from the upper and the high water level (E.L. 75.00 m) of the lower dam will be at the same level as the tailrace water level of the upper power station because the head between the upper and lower dams can fully be used. Available storage capacity is determined to be 21.5 x 10^6 m^3, which makes it possible to regulate the residual area. In view of the topographical and geological conditions of the site and comparison of various types, a concrete dam was adopted. The dam will be 38 m high, have a 160 m of the crest length and a volume of approximately 57,000 m^3.

A maximum of 40 m^3/s of water will be taken in and conveyed to a powerhouse, located on the right bank immediately downstream, through one penstock of diameters 5.0 m - 2.7 m having a length of approximately 50 m.

The Lower Tekai Power Staion will generate a maximum of 5.8 MW of electricity at a rated head of 17.2 m and produce 40,300 MWH of energy annually.

4.2 Civil Structure

4.2.1 Upper Tekai

(1) Slope Gradient

The Upper Tekai Dam is a central core wall rock fill dam of 101 m height and 350 m length. Dam stability analysis was carried out by examining the various design conditions and determining design constants. The stability calculations for sliding of the dam slope were carried out by applying the circular sliding method to the slope collapse. An upstream slope of 1:1.8 and a downstream slope of 1:1.75 are adopted in the design of the Upper Tekai Dam as a result of the stability calculations. This dam is provided with steps (6 m wide) on the downstream side. The steps are provided to facilitate construction and maintenance of the dam, but it contributes to slight moderation of the gradient of the downstream slope. The minimum safety factor at the highest water-level and at the design flood level is of the order of 1.44 to 2.22 under normal conditions both for the upstream side and the downstream side. This is considered to be a sufficient value. Further, the safety factor on the upstream side is 1.22 even in case of earthquake. Consequently the safety of the slope under these conditions was confirmed in the study. The principal factors be taken into consideration when determining the upstream slope gradient are the strength of the foundation, physical properties of the dam construction materials, variation of the reservoir level, the seismic coefficient, the condition under which of construction work is executed, etc.

Further details about stability calculations are given in Volume V Design and Construction Planning.

The dam slope gradient will finally be decided by carrying out stability calculations based on results of testing of the dam construction materials and further detailed examination of various design conditions.

In this case of dam type, impermeable center core type of fill dam, the slope gradient should be decided taking accounts of the seepage line which can be analysed by the finite element method (FEM).

Asphalt facing and the concrete facing type dams were examined with regard to the surface impermeable wall type rock fill dam. Resistance against sliding failure of the dam body becomes higher in these type dams because the reservoir level is intercepted on the upstream side of the dam. Further, the shearing strength becomes larger in these cases because the dam body is constructed by alternately laying relatively thin layers of rock and gravel, followed by compaction to minimize settlement. As a consequence, the slope gradient can be made steeper on both the upstream and downstream sides.

In the case of concrete and asphalt facing type dams, the slope gradients were assumed as follows.

	Upstream	Downstream
• Concrete facing dam	1 : 1.6	1 : 1.6
• Asphalt facing dam	1 : 1.7	1 : 1.6

(2) Zoning

A center core wall dam, a kind of zone-type dam, is proposed in this project. In zone-type dams the configuration of the impermeable zone, filter zone and shell zone are extremely important compared with surface impermeable facing type dams. The zoning of the Upper Tekai Dam was designed by stability analysis based on the soil test results and rock characterization available in the vicinity of dam site.

The position of the impermeable zone is determined first, then the configuration of the dam body is determined taking into consideration the physical properties of the soil and rock materials and the available quantities of each. Regarding to filter, zones consisting of fine grading materials are located close to the core, while zones with coarse material grading are located on the upstream and downstream sides. The impermeability requirement of the dam is satisfied due to the gradual change of grading of the materials from the core to the periphery of the dam body. Details on the various kinds of materials available in the vicinity of the site are described in "Design and Construction Planning" of Volume V.

For the semi-permeable zone, two filter layers of 3 m thickness consisting of fine and course grading material are provided at the boundary between the core material of the impermeable zone and the rock material of the permeable zone to prevent piping of core material, because the core material and the rock material show differences in grading.

The permeable zone has a decisive influence on the stability of the dam. Accordingly, it should be adequately arranged as a zone with sufficient slope gradient and thickness using materials having sufficient shearing strength.

Sandstone, which is a hard rock material fulfilling requirements, will be used for construction of the permeable zone of this dam. Construction materials are described in detail in Chap.3"construction material"and in Volume IV Geology.

(3) Dam Crest Elevation

Dam crest elevation was decided EL 166.200 taking into consideration of the wave height caused by wind and earthquake and additional freeboard. Calculations are shown in Volume V.

(4) Diversion Channel

For design flood discharge, the capacity of the diversion tunnel is assumed to be 850 m^3/s, (assuming a 100-year probability flood of 2,700 m^3/s and taking into consideration the surcharge effect). (Flood water level EL. 110.00 m). Each section is examined considering the topography of the dam site and the layout of other structures and assuming construction of a standard horseshoe-shaped tunnel on the left bank. An 8 m diameter tunnel is the most economic alternative in view of the relation between the tunnel diameter, the flow and water level.

Calculation of the diversion tunnel flow capacity is shown in Volume V.

(5) Coffer Dam

The river will diverted by the primary coffering during the dry season (4 months, river runoff Q max ≒ 20 m^3/s) at the damsite and banking of secondary coffering will be carried out.

The dam crest elevation of the secondary coffering is assumed to be EL. 111.50 m, adding a 1.50 m safety margin to flood water level EL 110.00 m obtained through examination of the diversion tunnel section. An earth core impermeable wall fill dam will be used for coffering purposes and this dam will be used as the upstream-side of the finished dam. As for the foundation treatment of the river bed, the sediment layer (approx. 6 m) will be removed by excavation and cut-off grouting will be done to prevent leakage.

(6) Spillway

The design flood at the upper dam site is 7,300 m^3/s, but the spillway is designed for a flood capacity of 1,504 m^3/s, taking into consideration the surcharge effect.

The spillway construction cost represents a considerable share of the cost of a fill dam, therefore, we examined the left bank alternative and the right bank alternative and adopted the former, taking into consideration factors related to the structure and economy. A free-overflow spillway is adopted in the design of the dam, taking into consideration unexpected problems due to careless operation of the gate, maintenance costs, etc. As for the overflow width, the 40 m alternative has been adopted after conducting comparative examination of the 40 m, 60 m and 80 m alternatives. The overflow depth is 7.00 m for this dam.

A horizontal apron type stilling basin is provided at the end of the spillway and the extremity of the stilling basin is provided with an sub-dam.

(7) Intake

An horizontal type intake is provided on the right bank just upstream of the dam, taking into consideration topography, geology and economy. The water velocity at the intake is assumed by be 0.5 m/s, taking into consideration the head loss and other relevant factors. An submerge depth was designed based on the hydraulic model test.

For further details refer to the "Design and Construction Section".

(8) Pressure Tunnel

The pressure tunnel is designed taking into consideration the topography, geology and cost comparison of various types and a pressure tunnel innerlined with steel pipe is adopted for this power station, due principally to topographical and economical reasons. The construction of two pressure tunnels was considered initially for reasons of maintenance, inspection and conditions of operation, but further studies indicate that the single pressure tunnel alternative is more economical and can replace the double pressure tunnel alternative without functional shortcomings. We therefore decided to adopt the 7.3 m diameter single penstock alternative. The penstock will be innerlined by a welded steel pipe, in view of its workability and other relevant peculiarities.

(9) Power Station and Switchyard

The power house is to be located on the right bank, just downstream of the dam. Barrel type is adopted in the power station in view of their economy, because geological investigation suggests that the foundation rock has satisfactory characteristics at the power station site.

The switchyard will be constructed on a banking just downstream of the dam, taking into consideration the requirements of proximity to the power station, layout of the power station access road and other structures.

(10) Foundation Treatment

Layers with low permeability (less than 2Lu) are distributed at relatively shallow depth on the right bank of the upper dam but on the left bank such layers are relatively deep. It is recommendable to carry out foundation treatment which can guarantee sufficient safety against leakage or piping. The target value for improvement of the bedrock by curtain grouting is assumed to be 2Lu.

As mentioned above, layers of less than 2Lu are distributed at very shallow depths in the river bed. Curtain grouting up to the third grouting will be carried out to a depth of 50 m, however, and the second grouting will be carried out down to 70 m in order to guarantee perfect impermeability.

With regard to the high permeability zones of 50Lu of more located under the overflow water level on the left bank and the fractured zone on the right bank, grouting tunnels will be constructed on both banks and grouting to the fourth grouting will be carried out (1.5 m pitch) in order to guarantee perfect impermeability.

Particularly in the fractured zone on the right bank, the high permeability zone has a possibility of reaching considerable depth. Therefore, it is recommendable to carry out curtain grouting (1.5 m pitch) to the river bed (equivalent to the hydrostatic pressure) in order to guarantee perfect impermeability.

4.2.2 Lower Tekai

(1) Slope Gradient

The Lower Tekai Dam is designed as a gravity concrete dam. The gradients of the upstream and downstream slopes are determined by the ordinary two-dimensional design consisting of a basic triangular cross section with unit width in the dam-axis direction.

The upstream and downstream slope gradients of the dam are examined based on the undermentioned premises and various design conditions.

- The resultants of the external forces shall actuate within the middle third of the horizontal cross section of the dam body.

- The shearing friction safety factor $F_S \geqq 4$.

- The resultant stresses shall not exceed the permissible compressible stress of the dam body and foundation.

For the Lower Tekai Dam the upstream gradient is 1:0.10 and the downstream slope gradient 1:0.75. The downstream slope gradient seems rather gentle for the dam height of 38 m, but this configuration is determined by the fact that the river gradient is rather gentle at the dam site (the Tekai River as a whole has a relatively gentle gradient) and by the influence of the back pressure caused by the small water level difference between the upstream and downstream sides.

Stability calculations for the overflow section are carried out for the following cases, assuming various conditions.

- High Water Level
- Flood Water Level
- Reservoir is Empty

The results of safety calculations evidence the following facts:

- The dam is safe against overturning because the resultant force is located within the middle third

- As for the sliding safety, n = 4.5 is the value at high water level

- Maximum compressive stress occurs on the upstream side when the dam is empty but its value is 69.1 t/m^2 and consequently both bearing capacity of the ground and concrete stress intensity are sufficient.

Refer to Volume V for further details of these stability calculations.

(2) Dam Crest Elevation

Dam crest elevation was decided EL 81.00 taking into consideration of the wave height caused by wind and earthquake and additional freeboard.

(3) Diversion Work

It is assumed that the lower dam will be constructed during the filling period of the upper reservoir. Such being the case the diversion method is adopted in view of its economical merits.

As for design flood discharge, a 10-year probability flood (200 m^3/s) of the residual area (180 km^2) is adopted.

The sluice diversion will be constructed with double sheet pile walls and by filling the space between them in order to guarantee stability and to prevent leakage because the sediment layer of the river bed is relatively thick (approx. 10 m).

(4) Spillway

The design flood discharge at lower dam is assumed to be 1,100 m^3/s, obtained by adding the residual area flood flow of 230 m^3/s to the upper dam flood flow of 875 m^3/s (1,000 year probable flood for lower dam). A free overflow spillway is adopted here, as in the case of the upper dam. The overflow depth is 4 m at the 42 m section located at the center of the spillway and 3.00 m at the two extremities. This configuration is adopted in order to assure stable water flow at the entrance of the stilling basin. A horizontal apron stilling basin is adopting for the lower dam.

(5) Intake

The intake is attached to the dam body. The elevation of the intake base is determined based on the hydraulic model test and taking into consideration the sedimentation.

(6) Penstock

A single penstock configuration with a 5 m diameter is adopted for the lower dam, for the same reasons as with the upper dam.

(7) Power Station and Switchyard

The power station is located on the right bank just downstream of the dam because the natural ground is gently sloped at that site. Results of geological investigation suggest that the foundation rock of the power house site has satisfactory characteristics, therefore, turbines and generators of the barrel type are adopted for the lower dam for reasons of economy.

(8) Foundation Treatment

The Lugeon value does not decrease even deep into the lower part, indicating that a high permeability zone about 10 Lu extends continuously into the depths. Permeable zones above 20 Lu are also distributed, which has been confirmed in LD-4.

Generally, curtain grouting is within the range of $L = \frac{H}{3} + \alpha$ ($\div \frac{H}{2}$, L : grout depth, H : dam height, α : allowance length). However, since a high permeability zone extends deep into the lower dam site, it is advisable to secure complete leakage by performing curtain grouting up to the third grouting (3-meter pitch), with the improvement target being set at 10 Lu.

Since the high permeability zone prevails in the lower dam site as a whole, it is necessary to dig a deeper pilot hole, thereby confirming that there is no high permeability zone in the depths of 80 m.

Consolidation grouting in the dam foundation shall be performed not only to increase the bearing capacity of foundation rock but also to ensure the water stop in the vicinity of rock joints. The depth of consolidation grouting shall be set at approximately 7 m, and it is advisable to perform adequate rock improvement.

4.3 Power Stations

4.3.1 General

The proposed location and preliminary arrangements for the Upper and Lower Tekai Power Stations are shown in Volume VI (Drawing). This arrangement, selected as the optimum installation, is suitable for the Upper Tekai station with 2 x 75 MW units and the Lower Tekai station with 1 x 5.8 MW unit.

4.3.2 Hydraulic System

(1) Upper Tekai Power Station

One pressure tunnel with 7.3 m internal diameter and 534.506 m in length which branches into two pressure tunnels with 4.6 m internal diameter and 43.852 m in length. The complete length of the pressure tunnel will be steel lined.

Turbine inlet valves are provided for each pipeline to operate one turbine during future maintenance. Remote control facilities to permit closing of the gates from the power station and control center will be incorporated. Provision will be made for stoplogs to be fitted upstream of the gates to permit maintenance of the gates. A crane will be provided at the intake structure to handle the stoplogs and the gates during maintenance. Intake trashracks will have mechanical raking equipment for cleaning.

Bulkhead-type draft tube gates will be provided, which can be lowered and raised only under balanced conditions, to enable each turbine to be isolated and drained for maintenance purposes. The draft tube gates will be handled by a small gantry crane from a gate deck external of the station on the downstream side.

A tailwater control weir should not be required downstream of the power station to maintain satisfactory tailwater levels.

(2) Lower Tekai Power Station

One penstock with 5.0 ~ 2.7 m diameter and 49,782 m in length is proposed. This will run through the dam body and connect one turbine located just downstream of the dam.

A turbine inlet valve is not included but a fixed-wheel gate will be provided at the intake. This gate will open hydraulically, close by gravity and be capable of closing under free discharge from the penstock. Remote control facilities to permit closing of the gates from the power station and control center will be incorporated, as will over-velocity protection, which will automatically trip the gates in the event flow in the penstock exceeds maximum discharge through the turbine.

Provision will be made for stoplugs to be fitted upstream of the gates to permit maintenance of the gates. A crane will be provided at the intake structure to handle the stoplugs and gates during maintenance. Intake trashracks will have mechanical raking equipment for cleaning.

Bulkhead-type draft tube gate will be provided, which can be lowered and raised only under balanced conditions, to enable each turbine to be isolated and drained for maintenance purposes. The draft tube gate will be handled by a small gantry crane from a gate deck external of the station on the downstream side.

A tailwater control weir should not be required downstream of the power station to maintain satisfactory tailwater levels.

4.3.3 Upper Tekai

The proposed power station is a conventional mass and reinforced concrete surface structure. The two units have a centreline spacing of about 18.6 m which allows adequate space for construction of the turbine spirals in position.

Inlet valve will be provided with consideration of keeping the width at a minimum.

The length and width of the powerhouse will be about 54.8 m and 31.0 m, respectively.

One overhead travelling crane will be provided, traversing the fuel length of the building.

It is proposed that the turbine spiral casing be fully embedded with a turbine floor level of about EL. 68.5 m. Structural crosswall between the machines are not required. There is adequate space for auxiliary turbine plant on this floor.

The main transformers and 132 KV switchyard will be located external to the station at ground-level at downstream of dam.

A drainage pump pit will be provided on the upstream side of the turbines, extending to a level below the invert of the draft tube, to collect station drainage water and also to permit draining of the turbines and draft tubes for maintenance. Drainage water will be pumped up to the tailrace.

Generating Equipment

(1) Turbine

1) Required quantity : Two (2) sets

2) Type : Vertical shaft, single runner, single flow Francis type turbine

3) Ratings

a. Head

Maximum net head	:	78.4 m
Rated head	:	75.1 m
Minimum net head	:	68.4 m

b. Turbine discharge

Maximum water discharge : 117.5 m^3/sec

c. Maximum output : 75,000 kW

d. Rated revolving speed : 187.5 rpm

(2) Generator

1) Required quantity : Two (2) sets

2) Type : 3-phase vertical shaft umbrella type synchronous generator with salient pole revolving field, self-ventilating and re-circulating type with water-cooled air coolers.

3) Ratings

a.	Capacity	: 88,200 kVA
b.	Frequency	: 50 Hz
c.	Rated voltage	: 13.2 kV
d.	Power factor (lagging)	: 0.85
e.	Rated speed	: 187.5 rpm

4) Excitation system

Static type complete with excitation transformer and voltage regulating equipment.

(3) Transformer

1) Main transformer proper

a. Required quantity : Two (2) sets
3-phase oil immersed, outdoor type

b. Type : Oil-forced air-forced cooling system

2) Rating

a. Voltage
Primary : 13.2 kV
Secondary : 132 kV $\pm$ 10%

b. Capacity : 88,200 kVA

c. Frequency : 50 Hz

(4) <u>Control and Protection System</u>

1) Control and relay panel

a. Required quantity : One (1) lot

b. Configuration

i) Operation and supervisory control panel for power generator and switchyard equipment

ii) Protection relay panel

iii) Synchronizing panel

iv) Auxiliary AC and DC power source panel for supervisory control

v) Control desk

(5) <u>Overhead Travelling Crane</u>

a. Quantity : One (1) set

b. Main lifting load : about 150t x 2 = 300t
Auxiliary lifting load: 5t hoist x 1

c. Span : about 21 m

4.3.4 Lower Tekai

The proposed power station is a conventional mass and reinforced concrete surface structure.

The length and width of the powerhouse will be about 28 m and 21.2 m, respectively.

One overhead travelling cranes will be provided, traversing the full length of the building.

It is proposed that the turbine spiral casing be fully embedded with a turbine floor level of about EL. 52.00 m.

The main transformers and 132 KV switchyard will be located external to the station at ground-level at downstream of dam.

A drainage pump pit will be provided on the upstream side of the turbines, extending to a level below the invert of the draft tube, to collect station drainage water and also to permit draining of the turbines and draft tubes for maintenance. Drainage water will be pumped up to the tailrace.

Generating Equipment

(1) Turbine

1) Required quantity : One (1) set

2) Type : Vertical shaft, single runner, single flow Kaplan type turbine

3) Ratings

a. Head

Maximum net head	:	18.7 m
Rated head	:	17.2 m
Minimum net head	:	14.2 m

b. Turbine discharge

Maximum water discharge : 40 m^3/sec

c. Maximum output : 5,800 kW

d. Rated revolving speed : 250 rpm

(2) Generator

1) Required quantity : One (1) set

2) Type : 3-phase vertical shaft umbrella type synchronous generator with salient pole revolving field, self-ventilating and re-circulating type with water-cooled air coolers.

3) Ratings

a. Capacity : 6,800 kVA
b. Frequency : 50 Hz
c. Rated voltage : 6.6 kV
d. Power factor (lagging) : 0.85
e. Rated speed : 250 rpm

4) Excitation system

Static type complete with excitation transformer and voltage regulating equipment.

(3) Transformer

1) Main transformer proper

a. Required quantity : One (1) set
3-phase oil immersed, outdoor type

b. Type : Self-forced air-natural cooling system

2) Rating

a. Voltage
Primary : 6.6 kV
Secondary : 132 kV ± 10%

b. Capacity : 6,800 kVA

c. Frequency : 50 Hz

(4) Control and Protection System

1) Control and relay panel

a. Required quantity : One (1) lot

b. Configuration

i) Operation and supervisory control panel for power generator and switchyard equipment

ii) Protection relay panel

iii) Synchronizing panel

iv) Auxiliary AC and DC power source panel for supervisory control

v) Control desk

(5) Overhead Travelling Crane

a. Quantity : One (1) set

b. Main lifting load : about 50/10 ton

c. Span : about 13 m

4.4 Access Facilities

4.4.1 General

Jerantut, approximately 150 km NE of Kuala Lumpur, is near-by the project site and is expected to function as a base for project construction. Over-land access from Kuala Lumpur to Jerantut is provided by a highway of approximately 180 km via Betong, Karak and Mentekab. This highway is fully paved and will be used as the main route for transportation after the start of construction work.

The base for sea transportation will be Gelang, a port located approximately 26 km north of Kuantan, the capital city of Pahang State. The port of Gelang is equipped with the facilities required to land large-scale plant equipment and construction materials.

The highway from Gelang to Jerantut via Maran and Temeloh is approximately 220 km long and is fully paved.

The road from Jerantut to the Project Area existing at present will be paved for use as a construction road and for maintenance purposes after completion of the Project.

As for the access road located in the Project Area (road providing access from the existing road to the upper dam site via the lower dam site) will be constructed on the left bank of the storage reservoir, taking into consideration cost, distance and work schedule.

4.4.2 Access to the Project Area

(1) Existing Road

There is a road of approximately 40 km from Jerantut to a point 2 km downstream of the lower dam site. It will be sufficient to pave this road without excavation for use not only for construction of the project but also for maintenance purposes after completion. The principal improvements this road requires are as follows:

- paving (laterite road currently existing) Approx. 35 km

The remaining 5 km of this road can be used as is. These improvements will cost apprixImately M$6,666,800.

(2) Newly Constructed Access Road

Construction of a new road is required in order to provide access from approximately 2 km downstream of the lower dam site to the lower dam site and on to the upper dam site.

The construction of a paved 6 m wide road with a 20 m minimum radius of curvature and 8% maximum slope gradient is considered in this case, in order to make possible use not only during construction of the project, but also for maintenance purposes after completion. As for the elevation of this access road, it is designed with road elevation of the order of EL. 85.0 m to EL. 90.0 m, because the lower dam storage reservoir has a high water level of 75.00 and a flood water level of 79.0 m.

As for the construction schedule, construction of the access road should be concluded prior to starting construction of the project itself. Of the temporary roads required for the upper dam, those accessing the aggregate plant site and the spoil site (including the road crossing the river) should be completed prior to starting construction of the dam body.

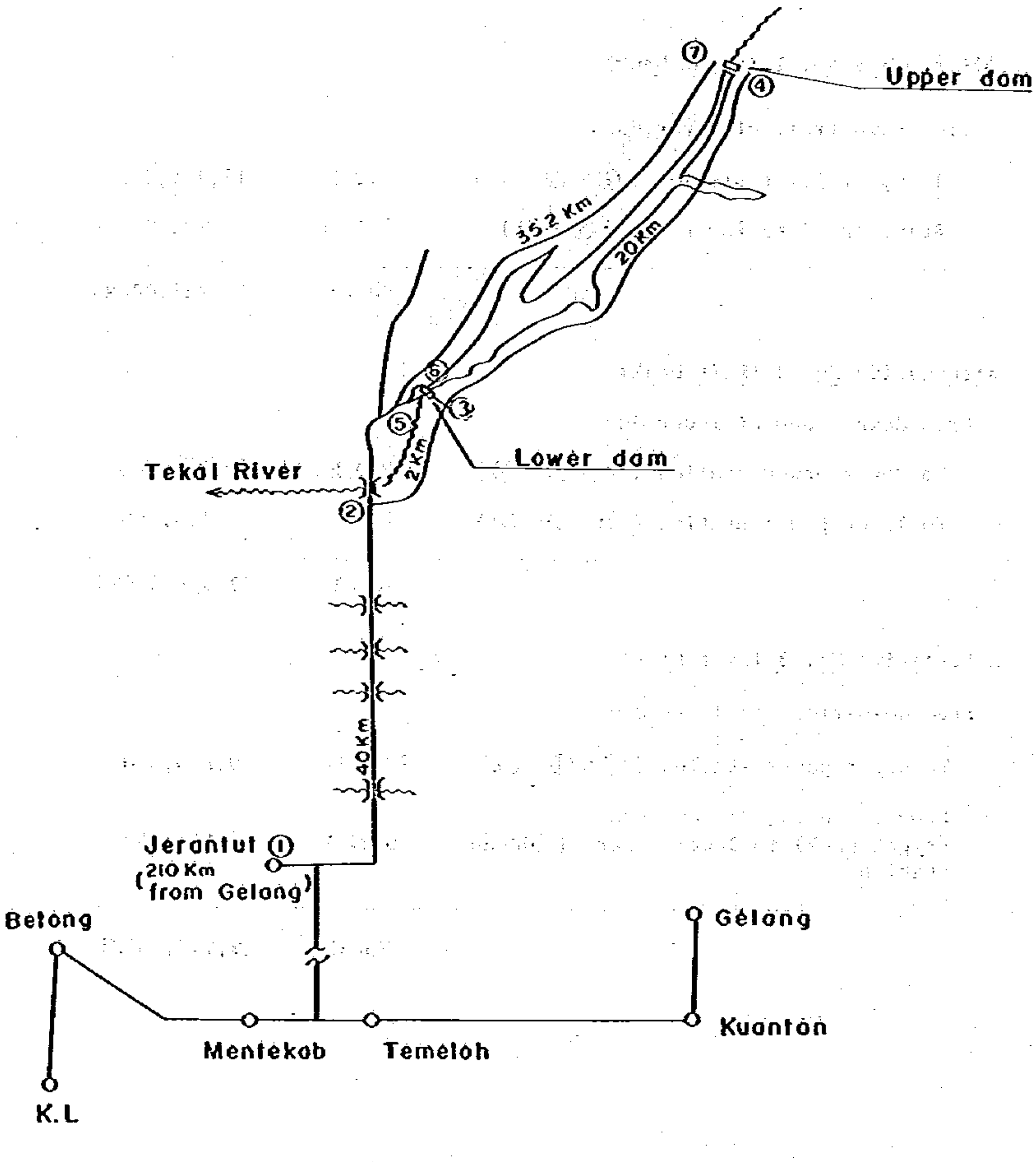
Three alternative routes, located on the left and right banks of the lower dam reservoir, are considered for the sake of determining the layout of the new road access to the upper dam. These 3 alternatives are compared in the followings (Table 4.1):

A sketch of the alternatives is shown in Fig. 4.1.

Table 4.1 Alternative Access Roads

Alternative No. 1 (Right bank)		
From downstream of lower dam		
To upper power station (②~⑤~⑦)	35.2 km	13,984,000
Branch road to lower dam (⑤~⑥)	0.4 km	274,000
	Total	14,258,000M$
Alternative No. 2 (Left bank)		
From downstream of lower dam		
To upper power station (②~③~④)	22.0 km	9,845,800
To lower power station (②~⑤~⑥)	2.5 km	2,130,780
	Total	11,976,580M$
Alternative No. 3 (Left bank)		
From downstream of lower dam		
To upper power station (②~③~④)	22.0 km	9,845,800
From dam crest of lower dam (right bank) to lower power station (③~⑥)	0.34 k	103,000
	Total	9,948,800M$

Fig. 4.1 Sketch of Alternative Access Road



Comparison of the 3 alternative routes shows that Alternative No. 3 is M$4,309,200 cheaper and 13 km shorter compared with Alternative No. 1. Alternative No. 3 is also M$2,027,780 cheaper than Alternative No. 2. Therefore, Alternative No. 3 is the most recommendable for construction of road access to the upper power station. The total construction cost of the access road, including the cost mentioned in 4.4.2(1) is M$16,615,600.

The optimum alternative for construction of the access road is shown in Figure II.

4.5 Quantity and Cost Estimates

4.5.1 General

In the estimate of construction costs for the alternatives for comparison purposes, the cost of civil works was calculated by multiplying the unit prices by the quantities of works obtained from the preliminary design of structures. For steel materials, the quantities in weight were calculated and then multiplied by applicable unit prices.

The estimated costs of turbines, generators and other electrical equipment include transportation and erection costs.

The unit construction prices are construction costs, material costs and labour costs prevailing in Malaysia as determined by the mission plus overhead costs.

The above costs are prices prevailing during the first half of 1982.

The estimated costs are broadly divided into the following categories. These are preparatory works, civil works, generating equipment, engineering services, government administration and contingencies.

Engineering services and government administration were estimated at 8% and 3%, respectively, of the sum of preparatory works, civil works and generating equipment costs.

Contingency funding was estimated to be 8% of the total cost.

4.5.2 Unit Construction Costs

In estimating the unit costs, construction planning and the construction schedule were considered and the types, number and operating hours of construction equipment for each category of works were estimated. The unit costs were calculated by adding labour costs, material costs, equipment costs and miscellaneous costs.

4.5.3 Construction Costs

The estimated construction cost for the optimum development scheme of the Upper Tekai is summarized on the following page and a breakdown is given in the attached sheets following the summary.

Table 4.2 Estimated Construction Cost for the Optimum Development Scheme

Upper Tekai

10^3 M$

1. Preparatory Works	32,256	
Access Road	16,513	
Temporary Facilities	15,743	
2. Civil Works	156,194	
Diversion and Care of River	20,572	
Dam	59,976	
Spillway	21,200	
Intake Structure	3,512	
Penstock	25,654	
Powerhouse	20,450	
Switchyard	1,330	
Mechanical Equipment	3,500	
3. Generating Equipment	53,000	
4. Engineering Service	19,316	(1)+(2)+(3)x8%
5. Government Administration	7,244	(1)+(2)+(3)x3%
6. Contingency	21,441	(1)+ ... +(5)x8%
7. Grand Total	289,451	

Table 4.3 Roads

M$16,512,650.-

Item	Unit	Quantity	Unit Cost	Cost (M$)	Remarks
(1)∿(2) 35 km					
Subbase Coursing	m^3	66,200	49.00	3,243,800	
Asphalt	m^2	210,000	16.30	3,423,000	
Sub-Total				6,666,800	
(2)∿(3) 2km					
Common Excavation	m^3	11,520	9.00	103,680	
Rock Excavation	"	2,880	20.50	59,040	
Subbase Coursing	"	3,600	49.00	176,400	
Asphalt	m^2	12,000	16.30	195,600	
Drainage Appliance	set	l.s. 1		32,580	
Sub-Total				567,300	
(3)∿(4) 20km					
Common Excavation	m^3	273,800	9.00	2,464,200	
Rock Excavation	"	68,400	20.50	1,402,200	4,032,150×0.2
Embankment	"	33,150	5.00	165,750	≒ 806,400
Subbase Coursing	"	36,000	49.00	1,764,000	
Asphalt	m^2	120,000	16.30	1,956,000	
Drainage Appliance	set	1		806,400	
Sub-Total				8,558,550	

(to be continued)