GOVERNMENT OF MALAYSIA

FEASIBILITY STUDY REPORT

ON

LEBIR DAM PROJECT

VOLUME 2

MARCH 1989

JAPAN INTERNATIONAL COOPERATION AGENCY
(JICA)

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EQUIVALENTS AND ABBREVIATIONS

EQUIVALENTS AND ABBREVIATIONS

Measurement

1 - milli-micron (um)	= 1×10^{-9}m
1 - meter (m)	= 3.2808 feet
1 - feet (ft)	= 0.3047 meter
1 - kilometer (km)	= 0.6214 mile
1 - mile	= 1.6093 kilometers
1 - acre	- 4,046.85 m^2
1 - hectare (ha)	= 10,000 m^2
1 - square mile	= 2.58985 km^2
1 - cubic meter (m^3)	= 35.316 cubic feet
1 - liter (l)	= 0.2642 U.S.gallon
1 - U.S.gallon(gal)	= 3.785 liters
1 - barral (bbl)	= 158.987 liter
1 - million cubic meter (MCM)	
1 - gram (g)	= 0.00220 pound (lb)
1 - kilogram (kg)	= 1,000 gram
1 - metric ton (ton)	= 1,000 kilogram
1 - kilo volt (KV)	
1 - kilo volt-ampare (KVA)	
1 - kilowatt (KW)	= 1.341 horsepower
1 - kilowatt (KW)	= 1,000 watt
1 - megawatt (MW)	= 1,000 kilowatt
1 - kilowatt-hour (KWH)	= 3,412.1 BTU
1 - gigawatt-hour (GWH)	= 1,000,000 kilowatt-hour
1 - terawatt-hour (TWH)	= 1,000 gigawatt-hour
1 - British thermal unit(BTU)	= 2.931×10^{-4} kilowatt-hour
1 - million British thermal unit(MBTU)	= 1,000,000 British thermal unit
1 - cubic meter per second (m^3/s, m^3/sec or cms)	
1 - lugeon (Lu)	= 1l/min/m/10kgf/cm^2
1 - kilogram per square centimeter (kg/cm^2)	= 14.1935 pound per square inch (psi)
1 - meter in aqua (mAq)	

Domestic Organization

Drainage and Irrigation Department	DID (JPT)
Department of Statistics	DS
Deparment of Environment	DOE
Department of Forestry	DOF
Ecnomic Planning Unit	EPU
Federal Land Development Authority	FELDA
Federal Land Consolidation and Rehabilitation Authority	FELCRA
Geological Survey Department	GSD
Institute of Medical Research	IMR
Jabatan Orang Asli	JOA
Jabatan Kerja Raya	JKR(PWD)
Kelantan Agriculture Development Authority	KADA
Kelantan South Land and Regional Development Authority	KESEDAR
Ministry of Agriculture & Cooperation	MOAC
Ministry of Finance	MOF
National Electricity Board	NEB (LLN)
Public Works Department	PWD
State Economic Planning Unit	SEPU
State Development Department	SDD
Tourist Development Corporation	TDC
University Sains Malaysia	USM

International and Foreign Organizations

Asian Development Bank	ADB
American Association of State Highway officials	AASHO
Egneinering Expert Association of New Zealand, Inc.	ENEX
Japan International Cooperation Agency	JICA
Japan Society of Civil Engineer	JSCE

Others

Aluminum Cable Steel Reinforced	ACSR
Benefit Cost Ratio	B/C
Biochemical Oxygen Demand	BOD
Bench Mark	BM
Capital Recovery Factor	CRF
Cost, Insurance & Freight	CIF
Compacted & Undrained	$\overline{CU}$
Cohesion	c
Center to center	ctc
Chemical Oxygen Demand	COD
Degree centigrade	°C
Design Silt Level	DSL
Elevation above Mean Sea Level	EL.
Environment Impact Statement	EIS
Economic Internal Rate of Return	EIRR
Flood Water Level (Reservoir design flood level)	FWL
Free on Board	FOB
Financial Internal Rate of Return	FIRR
Fiscal Year	F.Y.
Foreign currency	F/C
Figure	Fig.
Geologic N62°E --- Strik 68°S --- dip	N62°E/68°S
Gravity Acceleration	g
Gross Domestic Product	GDP
Growth Production Value	GPV
High Water Level (Maximim Service Level)	HWL
Internal Friction Angle	ϕ
Irrigation Agriculture Development Program	IADP
Japanese Industrial Standard	JIS
Kelantan River Basin Study	KRBS
Kampung (Village)	Kg.
Low Water Level (Minimum Service Level)	LWL
Local currency	L/C

Main Transformer	M.Tr.
Minutes of Meeting	MOM, M/M
Mean Sea Level	MSL
Note of Discussion	NOD
Not available/Not Applicable	n.a.
Ocean Freight & Insurance	OF & I
Operation and Maintenance	O & M
Permeability Coefficient	k
Per unit	P.u
Probable Maximum Flood	PMF
Power Station	P/S
Production Cost	PC
Roller Compacted Concrete	RCC
Revolution per Minute	rpm
Ratio of Total Storage Volume of Reservoir to Total Annual Discharge of River	C/I
Rock Quality Designation	RQD
Seismic Coefficient	k
Standard Penetration Test	SPT
Standard System Kelantan	SSK
Substation	S/S
Sungai (River)	Sg.
Scope of Work	S/W
Sverdrup-Munk-Bretschneider Method	S.M.B.Method
Tailrace Water Level	TWL
Transmission Line	T/L
Velocity	V
Volume	V

CURRENCY EQUIVALENTS

US$ 1.00 = 2.5 Ringgit (M$)

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4. Topography and Geology

4. Topography and Geology

4.1. Topography and Geology of the Project Area

4.1.1. General Topography

The topographical features in the vicinity of the proposed dam site consist of a continuous mountain terrain of some 400 - 500 m elevation on the eastern side, from which the altitude of the eastern backbone mountain range begins a gradual decrease toward the lowlands of the Lebir river basin, as shown in Fig. 3-1. The main ridgeline of the backbone mountain ranges extends from the north-northwest to the south-southeast, in the same way as with the dominant topographical and geological features of the Malay Peninsula.

A closer look at the Lebir river indicates that it meanders repetitively, with a wave-length of a few kilometers before flowing into the South China Sea. The main dam is planned to be located at the downstream end of one of these S-shaped meandering streams, as shown in Fig. 4-4. On both banks of the river, there exist terrace plains of low elevation sporadically (relative height of some 20 m). The elevation of the river beds at and in the vicinity of the main dam site ranges from 24 m to 30 m, with the width extending for about 150 m, thereby forming a U-shaped valley.

Both the quarry site and saddle dam sites are planned to be located on the right bank of the river; the former to lie some 2 km north of and the latter some 2 km northeast of the proposed main dam site, in a straight line. The borrow site is a further 3 km east of the saddle dam site in a straight line.

Between the borrow site and the saddle dam sites, there exists a ridgeline with an alignment that is the most distinctive of all

those in the project area, extending from the north-northwest to south-southeast. Geologically, this ridgeline consists of granite on the eastern side, and Mesozoic and Palaeozoic sedimentary rocks on the western side.

The river basin upstream of the proposed dam site forms a valley with remarkably gentle slopes on both banks, forming a topographical feature most suitable for a large reservoir.

4.1.2. General Geology

Geological distributions in and around the project area are shown in Fig. 4-1 and Fig. 4-5. As shown, the proposed sites for the main dam, saddle dams and the quarry site are all located in a zone in which green rock groups are largely distributed. The green rocks, falling under the pyroclastic rock group, consist mainly of tuffs, and these have a characteristic of marked variations of rock facies in grain size (fine to coarse) and colour (green to purple). These tuffs are believed to have been deposited in shallow water, and often contain non-marine admixtures such as quartzose sands and gravels. These rocks are classified as tuffaceous sandstones or tuffaceous conglomerates in this study. It is also found that these rocks are locally interbedded with thin bands of quartzites and shales. Some of the green rock masses involve rocks which can be assumed to have originated from lava flows, but they are few in quantity.

Geological structure of the project area is harmonious with that generally recognized in a wider area. The geological strata in the area are generally trending north-northwest to south-southeast, but tectonic disturbances can often be observed locally.

The borrow site is located in a zone where granite is largely distributed. The granite group chiefly consists of biotites-amphiboles-granite. It is observed that the tuffaceous rocks on the geological boundary line with the granite group are transformed into metasomatic rocks due to intrusion of granitic masses. These rocks, decoloured and silicified by metasomatism, are harder than the surrounding rocks. The geological boundary between granite and tuffs forms the main ridgeline of the mountains extending from the north-northwest to south-southeast.

There also exist lower terrace deposits distributed sporadically along the Lebir River, and collusial deposits along the bottoms of the valley and gullies. These deposits are generally poor in sorting, and their matrices are largely composed of clayey materials. Rock debris in the deposits is large in size and hard. The thickness is, however, believed to be 5 meters or less.

4.1.3. Lebir Fault

The Lebir Fault is one of the typical tectonic lines in the northeastern region of the Malay Peninsula. The Fault stretches south-southeastwards along the geological boundary between the granite group and the sedimentary rock group of Mesozoic and Palaeozoic Ages, as shown in Fig. 4-2 and Table 4-1. It extends some 320 km and is 2 km wide at Kuala Kerai, with left lateral displacement of 20-40 km, according to Tija, H.D., 1969, Regional Implications of the Lebir Zone; Geol. Soc. Malaysia, Newsletter No. 19 P6-7 (1969) and Aw, P.C., Sungai Aring Area, South Kelantan; Geol. Survey Malaysia, Annual Report 1969 P103-107 (1969).

Fault outcrops are recognized mostly along the river bed of the aring tributary, where at least four such fault lines extend side by side in the south-southeast direction.

In the project, area, confirmation has yet to be made on the existence of fault outcrops belonging to the Lebir Fault, but it is often observed that there exist small-scale folding structures, where rock masses are often found to have been fractured.

Past records of seismic activities indicate that deep-focus earthquakes with magnitudes of 6 or greater have frequently occurred in a belt zone along the western shoreline of Sumatra, but practically no earthquakes of such magnitude have occurred along the Malay Peninsula. This is because the Sunda Islands including Sumatra are located at the northern part of the Benioff Zone composed of the Australian and Asian Plates. While Sumatra and the neighbouring Indonesian islands are located along the Benioff Zone and thereby subjected to frequent deep-focus earthquakes, the Malay Peninsula is located on the stable Sunda Continental Shelf which is seldom subjected to deep seismic activity.

Records of the seismic activities along the Malay Peninsula show that an earthquake with magnitude of 3.8 took place southeast of Kuala Lumpur in 1976, and small earthquakes with magnitudes of 2.5 to 4.6 have occurred 30 times since 1984 in an area near the Kenyir dam reservoir in Terengganu State.

4.2. Field Investigation

Field investigation was made of the main dam site, the saddle dam sites, the quarry site, the borrow site and the re-regulating pondage site. Surface geological survey, topographical survey, boring investigation, seismic prospecting and material tests were conducted. The location of each site is shown in Fig. 4-4.

4.2.1. Surface Geological Survey

Surface geological survey was made of the proposed site for the main dam, including upstream river beds over a distance of 5.5 km and downstream river beds over a distance of 4.5 km, the saddle dam sites, the quarry site and the borrow site of some 30 km^2, on the basis of aero-photographic maps of 1/10,000 scale.

The objective of the survey was to (1) identify geological distributions of outcrops by observation, so as to collect data to make geological maps, (2) investigate the topographical features to confirm the stability of slopes and their characteristics, and (3) investigate distributions, scale and nature of outcrops to estimate the weathering conditions of the base rocks.

The results of the geological survey are shown in Fig. 4-5.

(1) Main Dam Site

The main dam is to be located at the narrowest part of the Lebir River. The river bed at the dam site is well outcropped with rocks, but there exist no outcrops of rocks on the slope of both banks, with the exception of a small quantity of rounded gravels in cobble size on the low elevation terraces along the left bank. In and around the proposed dam site, there is a vast rubber tree plantation. The proposed sites for the waterways, penstocks, powerhouse and switchyard, too, are in the area of rubber tree plantation.

Logging is widely being carried out in a forest on and along the ridgeline on the right bank, and a few, heavily weathered brown rocks are recognized among the weathered resistates along the gullies exposed by logging.

The actual topographical features of the ridgeline seem to be somewhat leaner than that shown on the topographical maps with a scale of 1/10,000.

Upstream of the proposed dam site, the valley becomes largely widened, and there exists no alternative site suitable for a dam. Downstream of the dam site, rock outcrops on the river beds are generally poor and some of them have intensive anisotropic properties. It is therefore believed that on the downstream side, too, there could be no alternatives more suitable than the proposed dam site.

(2) Quarry Site

Hard exposed rocks are found along a cut slope of the forestry road and the valley. Weathering of base rocks in the quarry site is considered to be generally shallower than other alternative sites.

(3) Saddle Dam I

No rock outcrops are found on either side bank, except some, heavily weathered rock outcrops found over out-slopes of the forestry road. The right bank is formed by a lean ridgeline extending toward the upstream side. The left bank is formed by a tiny ridgeline extending towards an axis of the proposed saddle dam.

(4) Saddle Dam II

The dam is planned to be located at a place where the valley of the watershed makes its width narrowest. It is observed that the right bank is studded with hard outcrops of rock masses which are believed to be intrusive rocks.

(5) Borrow Site

It is observed that there exist heavily weathered disintegrated granites over cut slopes of the forestry road near the proposed borrow site, but no outcrops are found over the mountain slope.

4.2.2. Topographical Survey

(1) Aerophotographic Maps from the 1979 Investigation

Aerophotographic maps with a scale of 1/10,000 were prepared for the preliminary investigation done in 1979. These maps (28 sheets) cover an area of some 346 km^2, extending from Manek Urai to the points of upstream basins at EL.100 m. The topography covered by these maps involves the river basins of the main stream and tributaries up to EL.100-120 m. Larger scale (1/5,000) aerophotographic maps (3 sheets) were additionally prepared to cover an area at the Jeram Panjang dam site.

(2) Topographical Survey from the 1987 Investigation

(a) The survey involved the following work:

i. Installation of the datum points (8 points)

ii. Preparation of topographical maps at the Jeram Panjang dam site, covering an area of 1.9 km^2 (1/500 scale and 2 m contours)

iii. Preparation of topographical maps at the saddle dam site, covering an area of 0.4 km^2 (1/500 scale and 2 m contours)

iv. Preparation of topographical maps at the quarry site, covering an area of 0.9 km^2 (1/500 scale and 2 m contours)

v. Preparation of cross-sectional profiles of the river at 30 points downstream of the main dam site (vertical scale 1/100, Horizontal scale 1/500)

vi. Preparation of aerophotographic maps at the borrow site, covering an area of 4.2 km^2 (1/10,000 scale)

The survey locations are shown in Appendix Fig. 4-1.

(b) Referential Bench Marks

The existing bench marks referred to in carrying out the above-mentioned work are as follows;

No.	M.S.L. (m)	Location
BM 0225	40.203	Kuala Kerai - Gua Musang Highway
BM 0226	39.951	
BM 0060	28.831	Kuala Kerai

A check was made of the two bench marks set on the right bank downstream of the Jeram Panjang dam site for the 1979 investigation, but no substantial difference was confirmed as shown below:

No.	M.S.L.(m) set at the 1987 Investigation	M.S.L.(m) set at the 1979 Investigation
TBM-2	43.263	43.248
TBM-2-1	43.202	43.178

(c) Referential Datum Points

The three existing datum points set at Kg. Major and referred to in determining coordinates of the datum points for the 1987 investigation are as follows:

No.	S.S.K. (m)	
No. 1	N 589,822.136	E 474,700.189
No. 2	N 589,878.912	E 474,668.199
No. 3	N 589,928.065	E 474,757.323

(d) Data on Datum Points and Some of the provisional Bench Marks

Data on the datum points and some of the provisional bench marks set for the 1987 investigation are as shown in Table 4-2, and the locations of these points are as shown in Appendix Fig. 4-2.

(e) Survey Maps

The survey maps prepared from the survey works mentioned in (a) above are as follows:

	Title	Map No.	Number
a.	Main Dam Site	LDP-DS-01 to 30	30
b.	Saddle Dam Site	LDP-SD-01 to 08	8
c.	Quarry Site	LDP-QS-01 to 17	17
d.	River Cross-Section along Sg. Lebir	LDP-RS-01 to 30	30
e.	River Cross-Section at Main Dam Site	LDP-RSDS-01 to 58	58
f.	Additional Aerophotographic Map at Borrow Site	LDP-BR-01	1

The index of the survey maps is shown in Appendix Fig. 4-3.

4.2.3. Boring Investigations

Boring investigations were conducted in 1979 at three alternative dam sites; Tualang, Jeram Panjang (the proposed site for this investigation) and Kiak. Number of borings at that time totalled 14 with a total length of 326 m. This time, boring investigations were additionally conducted at the main dam site, the saddle dam sites, the quarry site and the borrow site, with a total length of 785 m. Details of these boring investigations, old and new, are as tabulated in Table 4-3(1) and Table 4-3(2).

The boring investigations were accompanied with Lugeon permeability tests at the respective sites except the borrow site and the quarry site. The permeability tests were conducted at intervals of 5 meters. At the borrow site, standard penetration tests were also conducted at intervals of 1.5 meters.

Observation results of the boring cores are summarized in Appendix Figs. 4-6 to 12, and pictures of the representative rocks are as attached in Appendix Fig. 4-20.

Boring machines used for the tests are of the specification details listed below:

Boring machine	:	KOKEN OE21 rotary boring machine
Pump	:	MARUYAMA MS 1503 (150 ltr./min. 30 kgf/cm^2 at maximum)
Core barrel	:	Soil - 76 mm dia. single core tube (with no water)
		Rock - 76 mm dia. triple core tube (with water)

Rod	:	40.5 mm dia. x 3 m long
Casing	:	88.9 mm dia. (NX size)

At the permeability tests, either air packer (76 mm dia. and 1.34 m long) or mechanical packer (76 mm dia. and 0.5 m long) was used depending on the rock properties. Injection pressures were in the order of 0, 1, 2, 4, 6, 8 and 10 kgf/cm^2 and vice versa in principle. Test data were analyzed according to the Technical Guide for Lugeon Permeability Tests (the Ministry of Construction, Japan, 1984).

The results of permeability test are as summarized in Appendix Figs. 4-13 to 19.

4.2.4. Seismic Prospecting Tests

(1) Test Method

The seismic prospecting tests were conducted by employing the 24-component refraction method. Geophones were installed at 5 meter intervals. Blasting points were set at four points per spread (at intervals of 25-30 meters), and remote blasting points set at one spread away from both ends of each traverse line.

Gelignites in use (0.14 kg per piece) for blasting were two to three pieces in number on each spread, three to four pieces at both ends of each traverse line, and six to eight pieces at each remote blasting point. Electric detonators were of the zero-delay type. The 24-component amplifier recorders (OYO MC seis-1500 seismic system) were used for the measurement.

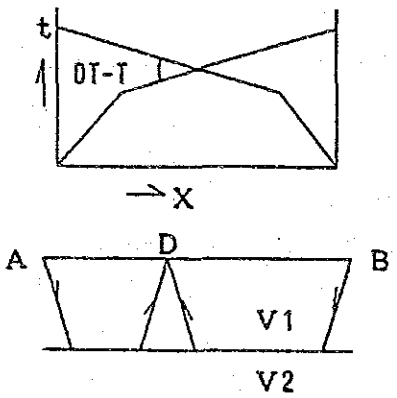
(2) Quantity

Quantitative data of seismic prospecting tests are as shown in Table 4-4.

(3) Method for Analysis

The first arrival time of each blasting to the measuring point was taken from the data on the field tests, and plotted on a figure with distance on the abscissas and time on the coordinates, from which a travel time curve was prepared on each traverse line. In analyzing the travel time curve, the Mean-Minus T method and the ABC method were used.

(a) Mean-Minus T Method

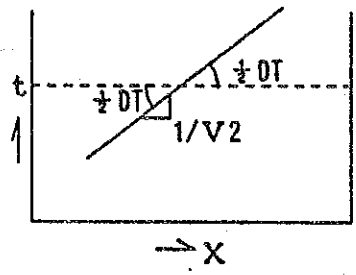


A time difference between the arrival time of blastings at Point A and Point B to the measuring Point D can be written as;

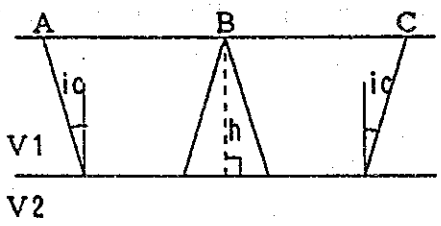
$$DT = TB - TA$$

where, TB is the arrival time from Point B to Point D
TA is the arrival time from Point A to Point D

The velocity of the refraction layer can be expressed as the inverse number of the inclination obtained by plotting 1/2 DT on each measuring point, as shown below:



(b) ABC Method



The depth to the lower layer (V2 layer) from the measuring point B to check the travel time from the blasting points A and C can be expressed as:

$$h = \frac{1}{2} \frac{(TAB + TCB - TAC)\ V1}{SINic}$$

where, ic is a critical angle, or SINic = V1/V2
TAB is the travel time from A to B.
TCB is the travel time from C to B.
TAC is the travel time from A to C.

(4) Summary of the Analysis

The results of the seismic prospecting tests are as shown in detail in the travel time curves and seismic velocity layer profiles in Appendix, but can be summarized as follows:

(a) Main Dam Site

The results of the seismic prospecting tests carried out at the proposed main dam site are shown as Table 4-5.

The velocity layers at the site can be grouped into three, and no low velocity layers corresponding to the faults or fractured zones can be detected by the tests.

(b) Saddle Dam I Site

The results of the seismic prospecting tests carried out at the proposed Saddle Dam I site are shown as Table 4-6.

The velocity layers at the site can be grouped into four. There exist zones with 2,800 - 4,000 m/s layers along Nos. 1, 3 and 4 spreads, lower than the surrounding layers, but whether these can be identified as the low velocity zones has yet to be confirmed due to lack of data for analysis.

(c) Quarry Site

The results of the seismic prospecting tests carried out at the proposed quarry site are shown as Table 4-7.

The velocity layers at the site can be grouped into three. Nos. 1, 2 velocity layers corresponding to sediments or weathered rocks are less than 20 m in depth at maximum. No low velocity zones have been found.

4.2.5. Material Tests

Material tests involved rock tests and soil tests. The rock tests were designed to investigate the quality and properties of rocks to be used for rock materials of rockfill dams, and aggregates for concrete work. The soil tests were to determine physical and mechanical characteristics and permeability of soils to be used for the core section of the rockfill dam. These tests were done in conformity with the Japanese Industrial Standards (JIS).

(1) Rock Tests

(a) Test Items

The following tests were conducted, by using boring cores (5.2 cm dia. and 10 cm long) as samples:

. Physical Tests: Specific gravity and absorbability (JIS A 1109, 1110)

. Uniaxial Compression Tests (JIS A 1108-76)

. Sodium Sulphate Tests (JIS A 1122-76)

(b) Samples

Eighteen pieces of rock samples were taken for the physical tests and uniaxial compression tests (thirteen pieces from the quarry site and five from the main dam site). Three pieces of rock samples were taken for the sodium sulphate tests (all from the quarry site). Shown in Table 4-8 is a summary of the rock samples.

(2) Soil Quality Tests

(a) Test Items

. Physical tests: Specific gravity, water content, Atterberg limits, unit volume and weight, void ratio and saturation degree (JIS A 1201, 1202, 1203, 1205, 1206)

. Sieving tests: (JIS A 1204-80)

. Compaction tests: (JIS A 1210-80)

. Consolidation tests: (JIS A 1217-80)

. Permeability tests: (JIS A 1218)

. Triaxial compression tests: Modified partly to meet the standards of the Japan Soil Engineering Society.

The consolidation tests, permeability tests and triaxial compression tests were done only on the samples of two species; granite and tuffaceous conglomerates, which are considered adequate for the use of core materials from the results of the physical tests, sieving tests and compaction tests.

(b) Samples

Sixteen soil samples were taken from three test pits and six cut slopes of forestry roads. Shown in Fig. 4-5 and Table 4-9 is a summary of the soil sample.

4.3. Topography and Geology of the Main Dam Site

4.3.1. Topography

The main dam is planned to be located at the downstream end of one of the S-shaped meanders, some 3 km upstream of Tualang Bridge.

(1) River Bed

The river bed at the proposed dam site is most remarkable in rock outcrop in the Lebir River basin. The river makes a slight curve to the left bank side near the proposed dam axis, and the gradient of the river bed is slightly steeper. The river width at the dam axis is about 150 m.

(2) Right Bank

The right bank is formed by a lean ridgeline extending from the north-northeast to the south-southwest. The ridge turns to the south at its end to which the dam is to be attached. The ridgy topography forms the continuation of rather flat elevations of EL. 110 - 117 m. The ridge is extremely lean with the thickness of the mountain being only 25 m at EL 95 m and 42 m at EL 90 m.

The mountain has gentle slopes corresponding to low elevation terraces sporadically at EL 45 m (relative height of some 20 m from the river bed).

(3) Left Bank

The left bank is formed by a topography with far more undulations than that of the right bank. The dam axis is to be situated on one of such undulations with a small

ridge extending eastward. Like the right bank, the mountain has gentle slopes corresponding to low elevation terraces up toward EL 50 m, and then gradually steeper slopes (gradient of 16°) up towards the peak, though a bit gentler than those of the right bank.

4.3.2. Geology

The base rocks underlying the proposed dam site consist mainly of green rock groups (green tuffs or purple tuffs), green tuffaceous sandstones and shales, all of which are believed to have been derived from sedimentary rocks formed during many periods from the late Palaeozoic to the early Mesozoic, and these are often interbedded with thin layers of tuffaceous conglomerates.

The bedding planes of different rock groups are either tightly closed or transitional. Anisotropy often observed in pyroclastic rocks or sedimentary rocks is found to be minor, and the bed rocks are generally massive.

Shown as Table 4-10 are the characteristics of rock groups underlying the proposed dam site.

It is observed that there are thin layers of tuffaceous conglomerate extending in the N62°E/68°S direction on the exposed river beds some 350 m downstream of the proposed dam axis. The direction is not in line with the dominant geological dip of the north-northwest to the south-southeast in the project site.

This is considered to be attributable to disturbances in layers due to local variations, which may have caused frequent slumps and formation of minor folding structures. Such local geology is, however, believed to be limited, disturbed and governed by the dominant regional geological trend, and as a whole to extend in the same NNW-SSE direction.

It is also observed that the rock masses are formed by the alteration of tuffaceous rocks with non-marine clastic rocks (sandstone, gravel, etc.) or shale. Besides, rock facies of these rock groups are found to be poor in continuity, and remarkable in variation laterally. It is therefore believed that these rock groups may have been deposited in the circumstances of shallow water, and frequently admixed with non-marine clastic sediments due primarily to volcanic ash deposition.

4.3.3. Civil Engineering Geology

The gist of the results of boring investigations carried out at the proposed main dam site is shown as Table 4-11, and a geological profile of the dam axis prepared thereon is shown as Fig. 4-7. Classification of rock masses used for the study is as given in Table 4-12.

Mentioned below are the weathering conditions, faults and joints and permeability of base rocks underlying the proposed main dam site:

(1) Weathering Conditions

(a) River Beds

Gravel layers on the river beds are estimated to be 3 m in thickness.

Weathered rock layers are so thin in depth that the CM class rock masses appear only at a depth of 5 m, and the CH class rock masses appear at a depth of 7 m, then the CH class rock masses continue to lie to a depth of 70 m, corresponding to the height of the main dam. However, these rock masses are as a whole subject to a weak hydrothermal alteration, and prone to crack along hair cracks.

Judging from the rock exposure on the river beds, a depth of about 7 m must be good enough to attain the best foundation for the core section.

(b) Left Bank

The base rocks at the crest section are found to have been heavily weathered to a depth of 5 m, and these can be classified as Class D similar to earth. The base rocks between 5 m to 7 m in depth, too, are weathered and can be grouped into Class CL. The CM class rocks then continue to a depth of 10 m, and the harder CH class rocks then exist at a depth of 10 m or deeper into the mountain masses.

The fact that the CM class and harder rocks lie at such shallow depth at this higher elevation indicates that weathering could in general be shallow on the base rocks of the left bank as a whole.

(c) Right Bank

Unlike the left bank and the river beds, weathering of rock masses of the right bank is found to be of considerable depth. The base rocks to a depth of 7 m are heavily weathered and can be grouped into Class D. Then, the CL class weathered rock masses continue to a depth of 21 m. The rocks classified as D are similar to earth in hardness and can be drilled with no water injection. The rocks classified as CL are hard but widely cracked, and loosened as a whole. The harder CM class rock masses can appear only at a depth of 21 m or deeper, and no harder rock masses than the CH class lie at a depth of 56 m. The rock masses are mainly comprised of tuffaceous sandstone, but rarely interbedded with thin layers of shale, which are often found to be fractured and weakened.

(2) Faults and Joints

(a) Faults

The existence of large-scale faults and fractured zones containing fault clays has not been confirmed by boring investigation and surface reconnaissance. There are rare cases where tuffaceous rock masses involve cracked zones of less than 1 m in thickness, made up of hard, non-clayey rocks. The seismic prospecting tests for the 1979 investigation, also, confirmed no low velocity layers to have been in existence along the traverse line of the proposed dam axis.

It is therefore considered that there is little possibility of the existence of large-scale faults in the base rocks underlying the main dam site.

(b) Joints

Both tuff groups and tuffaceous sandstone groups have sharp joints of more than three kinds. The surface reconnaissance indicates rock groups downstream of the dam site to have more systematic joints, while those on the dam axis and upstream of the dam site to have more irregular joints with tightly spaced cracks.

Tabulated in Table 4-13 are the average of the maximum core length of borings made at the dam site and EQD values. As shown, cracks are apt to have smaller spacings as a whole.

It is also found from boring core observation that cracks are often coated with films of calcite, chlorite or serpentine. This indicates that the cracks are apt to decrease shear resistance and are prone to exfoliation. The existence of such films is considered attributable to a weak hydrothermal alteration from which the base rock masses underlying the main dam site have suffered as a whole.

The directions of joints are, as shown in Fig. 4-6, rather dispersed. It is noted that there exists a relatively large number of cracks on a NW-SE strike with 40 - 70° dips to the NE or SW, while there are a small number of cracks on a NNE-SSW strike with rapid dips to the W.

(3) Permeability

(a) River Beds

The D-2 boring investigation indicates that the permeability of the bed rocks is 20 Lu or greater at a depth of 20 m, but lowers drastically to a few Lu at depths greater than 10 m.

(b) Left Bank

The D-1 boring investigation indicates that the permeability of the bed rocks is 20 Lu or greater at a depth of 15 m, but beyond this depth, it lowers drastically. It is noted, however, that there lie some zones at a depth of 50 - 60 m where the permeability ranges 4.4 Lu to 11.6 Lu. In these zones, there are many cracks in the rock masses, and those cracks are found to be coated with calcite films having worm-bore like spots. The rather higher permeability of bed rocks in these zones is considered attributable to the existence of such cracks.

The ground water level is believed to be at higher elevations. The water level of bore holes at D-1 located at a higher elevation (EL 107.889 m) than the proposed dam crest is at a depth of 7.2 m (EL 100.689 m).

(c) Right Bank

The D-3 boring investigation indicates that the bed rocks to a depth of 25 m are rather loosened with a permeability which comes down drastically and maintains 5 Lu or smaller to a depth of 60 m.

The ground water level is lower than that on the left bank, with the borehole water level being at a depth of 28.5 m (EL 86.609 m).

4.4. Topography and Geology at the Spillway Site

4.4.1. Topography

The flood spillway is planned to be laid down on a ridgy topography on the right bank some 400 m north of the proposed main dam site, by cutting open a lean ridgeline stretching to the southwest at right angles to the spillway. The ridgeline has peaks of EL 100 to 117 m and is extremely lean with only 60 m in thickness at EL 90 m on the proposed center line of the spillway.

The intake of the spillway is planned to be located at a saddle of the ridgeline. The slopes on the northwest side where the spillway is planned to be constructed form a topography with many small undulations and gullies that are believed to have been created by land collapses.

The slopes have an average gradient of 21° at EL 50 m or higher, but have a gentle gradient at lower elevations to form a terrace-like topography.

4.4.2. Geology

The general geology in and around the spillway site consists mainly of green rocks, similar to the geology at the dam site. The veneers or cover layers are generally thin.

4.4.3. Civil Engineering Geology

Table 4-14 shows the gist of the results of boring investigation carried out at the proposed spillway site, and Fig. 4-8 shows a geological profile of the spillway site along its center line.

(1) Weathering Conditions

The peak of the mountain ridge is covered for 20 m with rock masses that can be classified as Class D similar to earth in hardness, and for further 3 m with brittle, loosened rock masses that can be classified as Class CL. Beyond a depth of 23 m, there still lie the CM class rock masses, indicating the weathering condition of the base rocks to be relatively deepened. The harder CH class rock masses appear only at a depth of 29 m.

The slopes on the northwest side are covered for 5 m with heavily weathered rock masses that can be classified as D similar to earth in hardness, and for further 5 m with hard, brittle and loosened rock masses classified as CL. Beyond a depth of 10 m, however, the rock masses are hardened to a great extent.

The slopes on the southeast side are covered for 7 m with weathered rock masses including heavily weathered material. Rock masses at deeper layers are brittle and no harder rocks than the CH class exist to a depth of 22 m.

(2) Permeability

At the peak of the mountain ridge, rock masses have a permeability of 10 Lu or greater to a depth of 30 m, but at greater depths, the permeability decreases steadily, and finally to a few Lu at a depth of 35 m. On both side slopes, rock masses are classified as CM or harder classes, and are generally low in permeability, except for weathered rocks.

The ground water level is approximately the same as that on the right bank of the main dam site, with the borehole water level being at a depth of 20 m (EL 86 m).

4.5. Topography and Geology of the Diversion Tunnel Route

4.5.1. Topography

A temporary diversion tunnel is planned to be constructed under the mountain ridge topography stretching to the southwest direction, as is the case with the spillway. The tunnel is to be constructed some 50 - 90 m southwest of and almost in parallel with the spillway, and at a right angle to the direction of the ridgeline. The slopes on the southeast side of the ridgeline are formed by gullies which are believed to have been created by land collapses, and form a topography with steep gradients of 22° at EL 60 - 70 m or higher elevations. At lower elevations, however, the slopes as a whole form a convex shape topography in contour lines, though being accompanied with small undulations. The slopes on the northwest side of the ridge on the other hand form a concave topography with steep gradients of some 30° at medium and higher elevations and very gentle gradients at EL 50 m or lower elevations.

4.5.2. Geology

The general geology in and around the diversion tunnel route consists mainly of green rocks, similar to those of the main dam site. The slopes on both sides have almost no veneers, but at lower elevations where the inlet and outlet of the proposed tunnel are to be located, it is believed that there lie terrace deposits (near the northwest inlet) and collusive deposits (near the southeast outlet).

4.5.3. Civil Engineering Geology

The elevations of the proposed diversion tunnel are almost equal to the levels of the river beds (EL 40 m or so).

Overburdens of the mountains are some 75 m in the maximum thickness. The base rocks underlying the proposed tunnel route are in most parts to be classified as CM or harder classes. However, since the seismic prospecting tests carried out for the 1979 investigation did not confirm the existence of low velocity layers or large scale fault zones, the rock masses along the proposed diversion tunnel route are believed to be good and favourable in general from a civil engineering geological point of view.

The base rocks underlying the proposed inlet and outlet of the diversion tunnel are believed to be covered with deposits. In particular, a small overburden in the vicinity of the proposed inlet consists of colluvial deposits and heavily weathered rocks, thus indicating the rock masses in this location to be more or less unstable.

4.6. Topography and Geology of the Intake Site

4.6.1. Topography

The intake is planned to be located on a small ridge extending southward from the upstream side slope of the mountain ridge on the left bank where the proposed main dam is to be located.

The ridge decreases its altitute toward the southward direction, but the gradient of the slope is generally uniform, ranging from 13° to 15°. The ridge reaches a saddle at EL 70 m, and then connects to an independent hill with a peak having an altitude of EL 85 m.

4.6.2. Geology

The base rocks consist of green rock groups created during many periods from the end of the Palaeozoic to the early Mesozoic, as is the case with the main dam site. Veneers are believed to be thin in general.

4.6.3. Civil Engineering Geology

The results of seismic prospecting tests and D-1 boring investigation indicate the slopes to be uniform in weathering conditions, with soft and heavily weathered rock masses similar to earth in hardness and classified as Class D lying to a depth of 5 m, and heavily loosened, weathered rock masses classified as Class CL lying to a depth of 7 - 8 m.

Such shallow weathering conditions and stabilized geology of the slopes indicate there should be no particular problems on the base rocks underlying the proposed intake site from a civil engineering geological point of view.

4.7. Topography and Geology of the Penstock Route

4.7.1. Topography

The penstock is planned to be installed on the downstream side slope of a mountain ridge on the left bank where the proposed main dam is to be located. Small undulations lie on the slope formed by gullies which are believed to have been created by land collapses. The slope is generally uniform with a gradient of 16°.

4.7.2. Geology

The base rocks are comprised mainly of green tuff groups, as is the case with the main dam site. Veneers are believed to be extremely thin.

4.7.3. Civil Engineering Geology

There are no outcrops on the slope where the penstock is planned to be installed. Seismic prospecting tests reveal that the depth of weathered rock masses corresponding to D or CL classes is some 10 m at maximum, thus indicating harder rock masses exist at a relatively shallower depth.

There exist some remains of small scale, shallow collapses, but these pose no problems for the installation of structures. It is therefore considered that, from a civil engineering geological point of view, there are no particular problems on the base rocks underlying the proposed penstock route.

4.8. Topography and Geology of the Powerhouse and Switchyard Site

4.8.1. Topography

The powerhouse and switchyard are planned to be constructed on the gentle, terrace-like slope on the left bank of Lebir river at an altitude of 20 m from the river beds. A village named Kg J. Panjang is located on the slope and rubber plantations are being carried on in the surrounding area.

4.8.2. Geology

General geology of the proposed site is comprised mainly of green rock groups, as is the case with the main dam site. Low level terrace deposits, 4 m in thickness, are distributed on the slope.

4.8.3. Civil Engineering Geology

Shown in Table 4-15 is the gist of the results of boring investigation carried out at the proposed site.

The boring results indicate rock masses to change from the weathered zone to harder layers at a depth of some 9 m. Bed rocks to a depth of 9 m are made up of terrace deposits and heavily weathered rocks, and may in no way be suitable for the powerhouse foundation, but those at greater depths are hard enough for the purpose. It can be estimated that such geological conditions are uniform over the entire slope, and there are no particular problems on the base rocks underlying the proposed powerhouse and switchyard site from a civil engineering geological point of view.

4.9. Topography and Geology of the Tailrace Route

4.9.1. Topography

The tailrace is planned to be laid down on terraces at an altitude of 15 - 20 m from the river beds, as is the case with the powerhouse and switchyard site.

Rock exposures are noticeable on the river bank upstream of the proposed tailrace outlet, but not so at or in the vicinity of the outlet.

4.9.2. Geology

General geology of the proposed tailrace route consists chiefly of green rocks. Low level terrace deposits, 4 m in thickness, are distributed in an area where the route is planned to be laid down.

4.9.3. Civil Engineering Geology

Geological conditions of the proposed tailrace route are similar to those of the powerhouse and switchyard site, and harder rocks may appear at a relatively shallower depth. There should be no particular problems on the base rocks underlying the tailrace route from a civil engineering geological point of view.

The geological profile of the measurement line through the penstock, powerhouse and tailrace is shown as Fig. 4-9.

4.10. Topography and Geology of the Saddle Dam I and II Sites

4.10.1. Topography

(1) Saddle Dam I

(a) Right Bank

The dam axis is planned to make a right angle with a lean ridge extending from the north-northwest to the south-southeast. The ridge becomes narrower as it stretches southward. The mountain has a thickness of 65 m at EL 90 m at the dam axis.

The western side (dam side) slope of the mountain is less undulated in general, but forms a small gully at the dam axis.

(b) Left Bank

The left bank, too, is formed by a mountain ridge topography with its main ridgeline extending from the north-northwest to the south-southeast. The mountain is greater in thickness and altitude than the mountain on the right bank.

The eastern side (dam side) slope of the mountain is highly undulated, and the dam axis is to be situated at one of such undulations with a small ridgeline extending eastward. The ridge has a gentle slope at EL 70 m, and the general topography is gentler than that of the right bank.

(2) Saddle Dam II

The Saddle Dam II is planned to be mounted on a stream east of the proposed Saddle Dam I. It is to be located at

the point where the stream is divided into two, one running southward to join S. Pedah, and the other running westward to join a stream toward the Saddle Dam I site. The watershed makes a crook in the stream.

The topography of the valley forms a U-shape on a line of the proposed dam axis with relatively wide river beds and steep slopes on both banks.

4.10.2. Geology

(1) Saddle Dam I

The base rocks underlying the proposed Saddle Dam I site consist mainly of tuffaceous conglomerates and tuffaceous sandstones. They also include tuffs in small quantities.

Individual layers of major rock groups are several meters in thickness. The tuff group layers are thinner than others, and only 1 m or less. These layers are of the homecline structure having in general a NNW-SSE strike with 70° - 90° dips to the west. There exist, however, many disturbances due probably to local slumps and minor folding activities.

Shown in Table 4-16 are the characteristics of rock groups underlying the proposed dam site.

Veneers are generally thin, and several meters thick even in the valleys.

(2) Saddle Dam II

The base rocks underlying the proposed Saddle Dam II site are comprised mainly of tuffs and tuffaceous sandstones, and these are of intrusion meta-dacites.

Tuff beds and tuffaceous sandstone beds are closely alternated, and their grain size and colour are found variable locally. The trend of the base rock beds is generally in line with the dominant regional trend of NNW-SSE.

Meta-dacites are distributed on the right bank of the proposed Saddle Dam II site. Hard outcrops of rocks continue for about a dozen meters over the slope. The trend of the intrusion is not certain from the outcrop features, but is likely to be the same as the dominant regional trend of NNW-SSE. The intrusion rocks are believed to be some dozen meters in length.

The characteristics of rock groups underlying the proposed dam site are as shown in Table 4-17.

Veneers are extremely thin as are the cases with the proposed Saddle Dam I site.

4.10.3. Civil Engineering Geology

The results of boring investigation carried out at the proposed saddle dam sites are as tabulated in Table 4-18, and a geological profile of the Saddle Dam I axis is as shown in Fig. 4-10.

(1) Saddle Dam I

Problems from a civil engineering geological point of view, in the case where a rockfill dam of some 60 m in height is to be constructed on the proposed site, are that the base rocks are deep in weathering on both banks and high in permeability locally, as shown in Fig. 4-10.

(a) Weathered Rock Masses on Both Banks

It was confirmed by the surface reconnaissance that there exist no rock exposures nor residual rock boulders in large size on the slopes of both banks, except the forestry road near the river bed where the weathered rock masses are exposed on its surface and cut slope.

It was also confirmed by the boring investigations that the heavily weathered rock masses classified as Class D extend to a depth of some 20 m, and no rock masses classified as CM or harder classes are in existence to a depth of 27 m.

The fact that heavily weathered zones are well deepened on both banks of the Saddle Dam I site, and that rock masses forming such zones are weakened to the extent that rock debris can be easily broken by giving strong finger pressure or by hitting lightly with a hammer, indicates that it seems to be difficult to improve the water-tightness, and it is, therefore, necessary to make a thorough examination of the base rocks that are to serve as the foundation of the core section.

(b) Permeability

The Lugeon permeability tests at the S-3 boring carried out on the river bed reveal that the CH or B class hard rock masses involve some zones with a high permeability of 30 Lu or greater. A similar trend is seen at the S-1 boring on the left bank high elevation section.

The zones with such a high permeability are limited only to the left bank high elevation section and the river bed section, and to a depth of 30 - 35 m. This indicates that there may exist locally some zones where rock masses are well cracked.

The ground water level at the elevation of the proposed dam crest is at a depth of 28 m (EL 80 m) on the right bank, and at 32 m (EL 72 m) on the left bank, rising some 30 - 40 m from the river bed levels.

(2) Saddle Dam II

The left bank mountain of the Saddle Dam II is in itself the right bank mountain for the Saddle Dam I. The boring investigation indicates that the D class rock masses lie to a depth of 26 m.

The right bank mountain, however, is in a marked contrast to the left bank, as it contains the CM class hard rock masses at as shallow a depth as 7 m, due to the intrusion of meta-dacites.

The ground water level on both banks is generally low, almost the same as the river bed level even at high elevation sections.

4.11. Topography and Geology of the Quarry Site

4.11.1. Topography

The quarry site is located some 1.5 km north of the proposed main dam site on the northward extension of a lean ridge forming the right bank of the main dam.

The ridgeline of the mountain for the quarry site extends from the northwest to the southeast, with its peak as high as EL 227 m. The existing logging road is provided at around EL 85 m, some 140 m below the peak. The mountain forms a topography with relatively steep slopes on the northeast side of the ridgeline (the side for the quarry site) and gentle slopes on the southwest side, thus posing an unsymmetrical shape in a cross sectional profile.

4.11.2. Geology

The general geology of the quarry site consists mainly of green rock groups, similar to those at the main dam site. The green rock groups are rich in lithofacies, and are made up of alternated layers of green fine-grained tuffs, purple coarse-grained tuffs, purple fine-grained tuffs and purple tuffaceous breccias and tuffaceous conglomerates. Individual layers are some dozen meters thick on average and several meters at the thinnest. The trend of the rock beds are not certain for lack of outcrops, but are likely to be in line with the dominant regional geological trend of NNW-SSE.

Shown as Table 4-19 are the characteristics of rock groups underlying the quarry site.

4.11.3. Quantity and Quality of Materials

The results of boring investigation carried out at the quarry site are as shown in Table 4-20, and a geological profile on the traverse line of seismic prospecting tests done at the site is shown as Fig. 4-19.

(1) Quantity

The thickness of veneers of the D and CL class rock masses at the quarry site that are considered inadequate for the

use of rockfill materials or aggregates for concrete work is estimated to be 10 - 15 m on average as shown in Table 4-20. These rock beds are the thinnest of all candidate sites in the project area. Therefore, the proposed site is considered most adequate as the quarry site.

The quarry site is located on a mountain with the ridge stretching NW to SE, and the existing logging road runs in parallel with the ridge on its northeastern side slope. Having the least undulations, the slope poses no difficulties to be cut widely for the quarry site.

Besides, as there are no substantial differences of weathering conditions, regardless of elevations, the base rocks are considered uniform in nature.

It is therefore considered that there are no particular problems on the site from a quantitative point of view.

(2) Quality

The results of material tests using boring cores taken from the base rocks at the quarry site and the main dam site are as tabulated in Table 4-22.

The average values of the test results are as follows:

Physical tests	:	Specific value - 2.76 Absorption - 0.38 %
Uniaxial compression tests	:	443.3 kgf/cm^2
Sodium sulphate tests	:	2.9 %

The respective limits specified by the Japanese Standards are as follows:

Dam Concrete Standards (1980):

Specific value	:	2.60 or greater
Stability by sodium sulphate tests	:	12 % or less
Abrasion ratio by Los Angeles Abrasion Test Machine	:	40 % or less

JIS A 5005 (Standards for Concrete Aggregates):

Specific value	:	2.50 or greater
Absorption	:	3 % or less
Stability by sodium sulphate tests	:	12 % or less
Abrasion ratio	:	40 % or less

The test results satisfy these requirements well, indicating the rocks at the site to be very suitable for use as aggregates for concrete work. It is noted that the results of uniaxial compression tests are smaller than anticipated from the outward appearance of the boring cores. This may be attributable to the formation of fine hair cracks on the test samples. It appears that the cracks are prone to be easily broken as they are coated with slickenside films of calcite, green serpentine and chlorite, which may reduce shearing resistance of the crack surface.

It should also be noted that tuffs widely distributed in the project area are brittle as a whole and have three or more kinds of sharp joints. Table 4-21 shows the averages of RQD values and the maximum core length of borings done at the quarry site.

As shown in Table 4-21, the CM class or harder rocks at the quarry site tend to have cracks that are dense in spacing and easily breakable, and accordingly, it may be difficult to collect rockfill materials in large size from the site.

However, they have no unusual properties for use as rockfill materials or aggregates for concrete work. Also, the boring investigation made this time was only of the rock masses lying to a depth of 40 m, and harder rocks classified as B class are presumed to lie at greater depths. It is, therefore, considered that the proposed site should pose no particular problems as the quarry site for rockfill materials or aggregates.

4.12. Topography and Geology of the Borrow Site

4.12.1. Topography

(1) Borrow Site A

The proposed Borrow Site A is in a granite distribution area some 4 km east-northeast of the proposed main dam site. It is on the eastern side slope of a mountain ridge with the main ridgeline extending NNW to SSE with peak of EL 300 - 400 m, which forms a boundary between granites and Mesozoic and Palaeozoic sedimentary rocks.

The main ridgeline is crooked irregularly. The eastern side slope is rich in undulations caused by small collapses or erosions. There are also many saddles. Such topography in particular to the granite distribution area.

(2) Borrow Site B

The proposed Borrow Site B is in a tuffaceous conglomerate

distribution area some 1 km upstream of the proposed Saddle Dam I site. It is on a narrow ridge extending in west-south-west direction, and forming gentle mountain slopes.

4.12.2. Geology

(1) Borrow Site A

General geology underlying the borrow site consists mainly of medium and coarse grained biotite granites, and also partially of granodiorites.

The site is on the western edge of the granite intrusion rock masses. Being close by the boundary with Mesozoic and Palaeozoic sedimentary rock beds, the surrounding area abounds with tuffaceous xenoliths.

(2) Borrow Site B

Tuffaceous conglomerate is distributed in the surrounding area.

4.12.3. Civil Engineering Geology

The gist of the results of boring investigation carried out at the Borrow Site A is shown as Table 4-23. Quantitative estimation was made based on these test results. No boring tests were conducted at the Borrow Site B, but quantitative estimation was made based on the results of surface reconnaissance and data taken from the test pits.

(1) Quantity

(a) Borrow Site A

The boring results show that the base rocks are

heavily weathered, and such a heavily weathered zone classified as Class D extends to a depth of 15 - 20 m. These rocks are considered suitable for use as core materials. Weathering conditions to such extent are believed to be the same in the surrounding areas, and granites are widely distributed in and around these areas. Hence, it is considered that there should be no problems at the site from a quantitative point of view.

(b) Borrow Site B

Similar to the deep weathered layer over 20 m from the ground surface noted in the Saddle Dam I site, the tuffaceous conglomerate distributed in this area present the tendency of deep weathering.

However, further detail field investigation is required for confirmation of quantity available because the layers of tuffaceous conglomerate are generally narrow, and the mountain ridges are small in this surrounding area.

(2) Quality

Tabulated in Table 4-25 are results as soil quality tests made to check the adequacy for the use of core materials.

The following four kinds of samples were used for the tests:

(a) Granite samples taken from the Borrow Site A
(b) Tuffaceous conglomerate samples from the Borrow Site B left bank of the main dam site
(c) Terrace deposit samples taken from the upstream
(d) Tuff samples (coarse grained and fine grained)

All four kinds of samples were given physical tests and compaction tests. Dynamic tests, permeability tests and consolidation tests were applied only to those samples that are considered adequate for use as core materials judging from the results of physical tests and compaction tests.

There are no substantial differences in the physical characteristics among samples of granites, terrace deposits and tuffaceous conglomerates, but there are distinctive trends by sample in the compaction characteristics and the water content ratio. As indicated in the results of compaction tests shown in Fig. 4-13, tuffaceous conglomerates are greater in dry density and smaller in water content, while terrace deposits are smaller in dry density and greater in water content, and granites are intermediate in between.

As indicated in Table 4-24, there are few instances in Japan in which core materials used for rockfill dams exceed 30% both in the natural water content and optimum water content. Being greater in water content, therefore, the terrace deposit samples are not considered adequate for the core materials. It should also be noted that the natural water content of granite test pit samples is some 6% greater than the optimum water content.

Grain size distributions by sample are shown as Fig. 4-14. The terrace deposit samples and part of the granite samples are finer in grain size. The content ratio of fine grains of these samples is in the range of 21 - 66% in clayey size and 37 - 78% in silty size.

So far as this data is concerned, the terrace deposits may not be considered adequate for use as core materials. It should be noted in the case of the granite that the test

pit samples were taken from base rocks lying at a depth of 12 m, closer to the topsoils.

It should also be noted that the fine grained tuffs and tuffaceous sandstones are apt to be irregularly refracted in the sieving curve, as seen in the SP-5 and SP-6 samples.

The pertinent conclusion from the results of examination as mentioned above is that the SP-3 sample made up of granites and the TP-3-2 sample made up of tuffaceous conglomerates are considered most adequate for use as core materials.

4.13. Topography and Geology of the Re-regulating Pondage Dam Site

4.13.1. Topography

The re-regulating pondage dam is planned to be constructed at a point some 300 m downstream of Tualang Bridge.

Both banks of the river at the dam site form gentle slopes equivalent to river terraces at an altitude of 10 - 15 m from the river beds. A cross sectional profile of the river downstream of the site is wide in the river bed and shaped like the letter "U".

The river beds, particularly of the right bank, are good in rock exposure, but no outcrops are visible on both banks. A village named Kg. Lebut is located on the terrace of the right bank.

4.13.2. Geology

The base rocks underlying the proposed re-regulating pondage site consist mainly of green rocks that were created during