HIS MAJESTY'S GOVERNMENT OF NEPAL

FEASIBILITY REPORT ON SAPT GANDAKI HYDROELECTRIC POWER DEVELOPMENT PROJECT

VOL. III

ANNEX

JANUARY 1983

JAPAN INTERNATIONAL COOPERATION AGENCY

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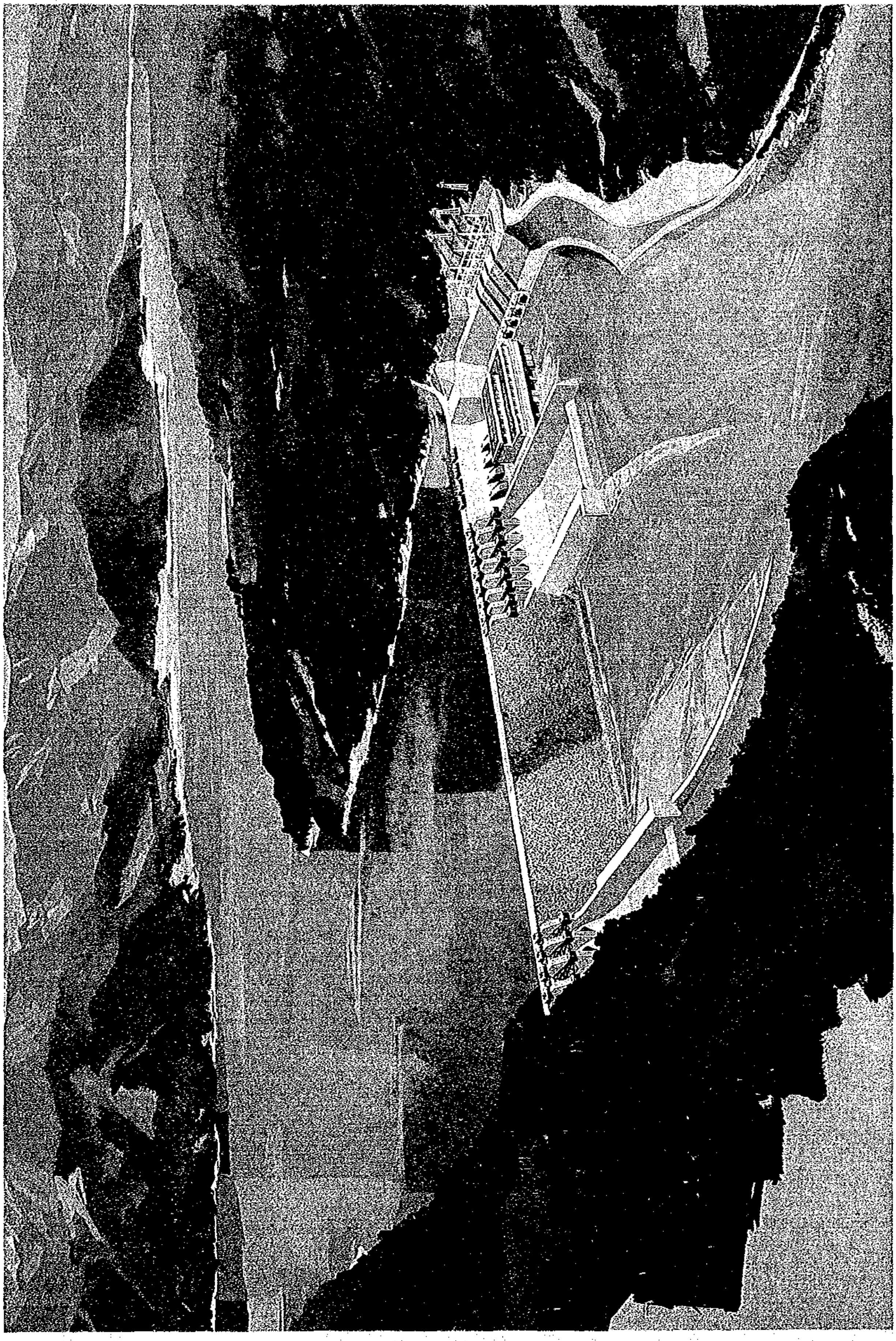
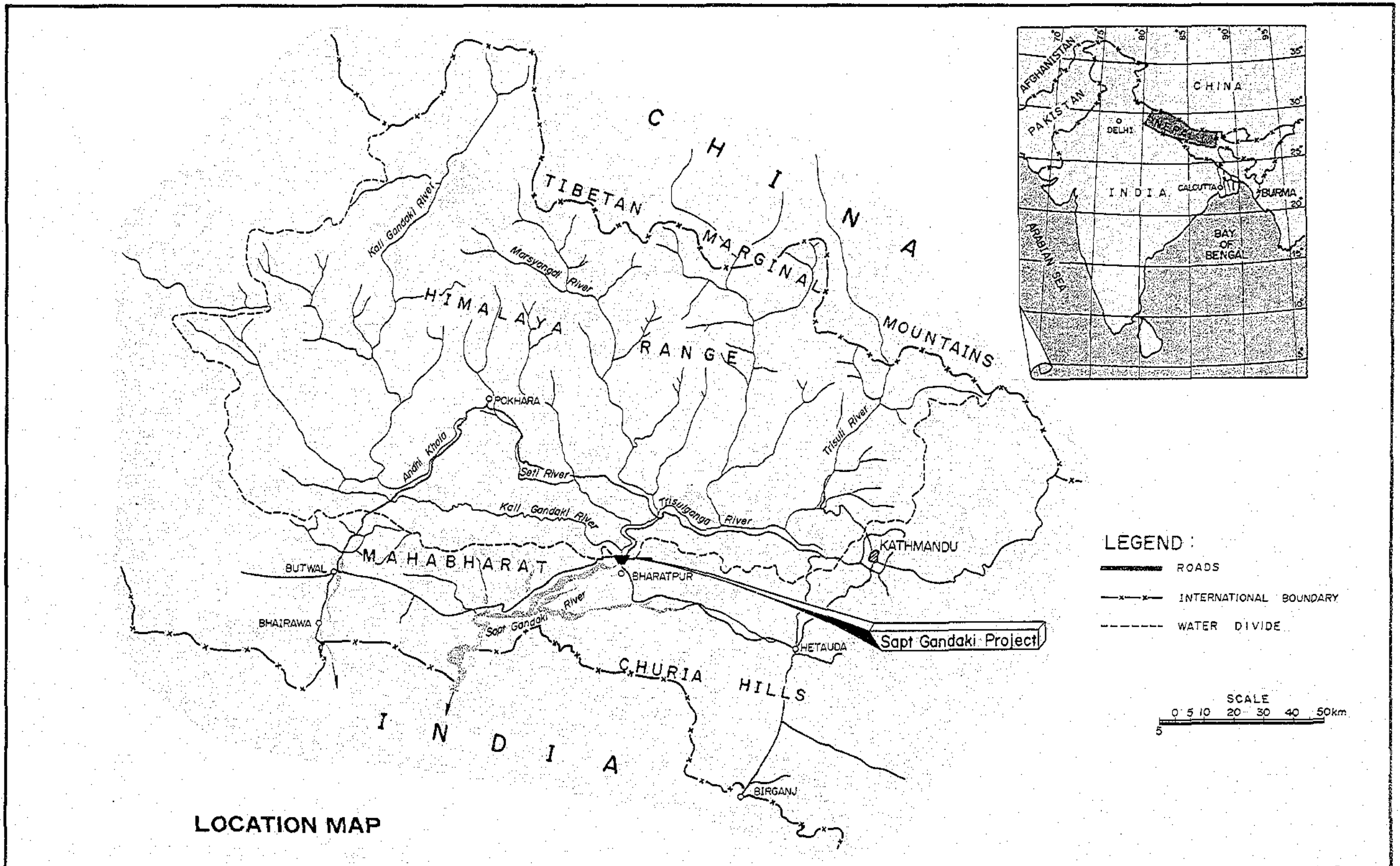




LOCATION MAP

ANNEX (E)

GEOLOGY

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

LIST OF FIGURES

LIST OF ATTACHMENTS

NO.	TITLE
I	SEISMIC REFRACTION PROFILE; TIME-DISTANCE PLOT AND INTERPRETATION (16 SHEETS; SL-1 TO 17 AND SLG-1 TO 4)
II	DRILL LOG (47 SHEETS; DG-1 TO 3, B80-1 TO 3, B81-1 TO 18 AND TG-1 TO 7)
III	RECORD OF WATER PRESSURE TEST (37 SHEETS)

I. INTRODUCTION

The damsite of the Sapt Gandaki Project was raised up first as the Dev-Ghat project in the report of "Master Plan of Hydroelectric Power Development in Nepal" (hereinafter called as the master plan) in 1974. Afterwards (in 1979), the prefeasibility study on Sapt Gandaki Hydroelectric Project (hereinafter called as the prefeasibility study) was also carried out.

In the master plan, a moderate scale embankment dam was planned to be built assuming the bedrock condition might be poor. A concrete gravity dam with a central overflow spillway, however, was recommended in the prefeasibility study in view of huge floods of the Sapt Gandaki River based on the assumption that the foundation is composed of competent rocks overlain by a gravel layer and a weathered rock zone of each 5 m in thickness.

Therefore, the investigation in the feasibility study was rather focused on getting an exact information about strength of the bedrock as well as the thickness of the riverbed deposit on purpose to conclude the best project layout including a dam type and to study the project feasibility.

Geological investigation for the feasibility study on Sapt Gandaki Hydroelectric Power Development Project launched out on February 1, 1981. The field survey was divided into two stages, i.e. the first stage from February 1 to March 31, 1981 and the second stage from October 1, 1981 to April 30, 1982.

In the first stage, geological mapping in the scale of 1 to 10,000 of the reservoir area, 125 linear meters of core drilling with water pressure test and 2,500 meters of seismic refraction profiles have been done. More detail and essential part of the survey has been conducted in the second stage which consisted of geological mapping in the scale of 1 to 2,000 of the damsite, 816.5 linear meters of core drilling, 9,975 meters of seismic refraction profiles, test grouting, excavation of test adits and test trenches and rock tests.

As a result of the investigation, the riverbed deposit is so deep as 30.5 m at the core drilling B81-11 and more than 36 m at the B81-6, which are deeper than both assumptions in the prefeasibility study (assumed as 5 m) and in the master plan (assumed as 15 to 30 m).

The strength of the bedrock against the sliding of a concrete gravity dam has been revealed as $\tau\,(kg/cm^2) = 8 + \sigma \cdot \tan 40°$ which is so marginal for the foundation of a high concrete gravity dam as to necessitate a large base, so that the determination of the dam type requires a detailed examination.

Furthermore the terrace deposit on the left bank has been proved to be so wide and thick as to be seriously concerned in the selection of damsite.

Survey items performed, applied method, obtained data and engineering assessment based on the geological investigations carried out are discussed in detail in this ANNEX (E).

II. REGIONAL GEOLOGY

2.1 General

Nepal is situated in longitude 80° to 88°E and latitude 26° to 30°N with a rectangle boundary. It can be geomorphologically divided into several zones extending east-west direction, i.e. Tibetan arid zone, Himalaya range, Mahabharat range (or Lesser Himalaya), Siwalik hilly zone and Terai plain from north to south.

In the Gandaki basin, Mustang area is in the Tibetan arid zone and in the Himalaya range are the Langtang Himal (7,264 m), Manasulu (8,125 m), Himalchuli (7,864 m), Ganesh-Himal (7,406 m) Dhaulagiri (8,172 m) and Annapurna Himal (8,078 m). The Mahabharat range is composed of 2,000 to 3,000 m high mountains and inner basins which oftenly subdivided into the Lesser Himalaya, the Midlands and the Mahabharat range. Old cities and towns such as Tansen, Pokhara, Gorkha, Dhading, etc. are located in this range.

The Mahabarat range and the Siwalik zone are plainly bounded by the Main Boundary Thrust not only forming the southern steep slope of the Mahabharat mountains but also showing remarkable contrast of the altitude in both sides. The Siwalik and the Terai is transitional such as the Siwalik being once buried under the Chitwan Valley (the Inner Terai) and again formed the Churia Hills along the southern boundary of the Chitwan Valley. The Churia Hills have altitudes of 700 to 800 m and consists of the Upper Siwalik rocks. In further south, the main Terai plain spreads widely as the northern part of the huge Gangatic Plain.

The Trisulganga River and the Kali Gandaki River as shown in the location map head in the Tibetan Marginal Mountains, then pass through an area of granitic rocks and ancient metamorphic rocks of the Mahabharat range gathering many tributaries and finally meet at 1 km upstream of the Sapt Gandaki Damsite, after which it is called the Sapt Gandaki River. The damsite area is underlain by the Tertiary sedimentary rocks of the Siwalik Group which bounds on the above mentioned metamorphic rocks along the east-west trending Main Boundary Thrust running about 5 km north of the damsite.

2.2 Stratigraphy

As shown in Fig.-E.1 "GEOLOGICAL MAP OF RESERVOIR AREA", the oldest stratigraphic unit within the reservoir area consists of a variety of Paleozoic metamorphic rocks which are exposed in the northern part of the Main Boundary Thrust. On the other hand in the southern side of the Main Boundary Thrust, the Tertiary sedimentary rocks of the Siwalik Group widely spread up to the flat alluvial plain of the Chitwan Valley. The stratigraphy of strata in the reservoir area is shown in Illus.-2.1.

The Siwalik Group of this area consists of (1) banded sandstones and slate, (2) banded sandstones and mudstones, (3) massive and pebbly sandstones and (4) conglomerates from lower to upper horizons, which are called Bsl Formation, Bst Formation, Sst Formation and Cgl Formation respectively in this report. Bsl Formation occupies the strip bounded by the Main Boundary Thrust in the north and the inner Siwalik fault in the south. The upper part of Bsl Formation, and the lower part of Bst Formation are petrographically similar. The Sapt Gandaki damsite is located on the gradual transition zone between Bst Formation and Sst Formation.

The other unit of geological facies is alluvial terrace deposits which develop on various elevations and are classified into four terraces as shown in Fig.-E.2. It is remarkable that the terrace IV is thickly developed in front of the Main Boundary Thrust and the inner Siwalik fault and includes thick cemented conglomerates and breccias. The layers of cemented conglomerates and/or breccias are observed not only in the terrace IV deposits, but also in other terrace deposits except the terrace III which widely extends on the left bank of the damsite.

2.3 Folding and Faulting

The most striking structural feature is the Main Boundary Thrust which is located in the northern part of the reservoir area and bounds the Paleozoic Midland Group and the Tertiary Siwalik Group. There are two other prominent faults called the inner Siwalik fault and the foot-hill fault in this report, which trend east to west as the Main Boundary Fault does. An anticline extends in the northern side of the foothill fault plunging to the east. Strikes of bedding, an axis of folding and directions of faulting are consistently oriented in the east to west direction.

III. GEOLOGICAL INVESTIGATION

3.1 General

The damsite is located in the straight channel of the Sapt Gandaki River which traverses the Siwalik Hills and flows into the Terai Plain. At the damsite, the bedrock is exposed on both banks and the river is 200 m wide at the flow level. Three alternative dam-axes are proposed within 1 km of this stretch and named A-, B- and C-site from upstream to downstream.

The bedrock consists of the banded sandstone and mudstone formation (Bst Formation) and the massive pebbly sandstone formation (Sst For mation) of the tertiary Siwalik group as shown in Fig.-E.2 "GEOLOGICAL MAP AND SECTION OF DAMSITE". Bst Formation is composed of orderly repetitions of thick sedimentary cycles, each of which consists of graded sandstones, laminated siltstones, intraformational breccias and greenish mudstones in order from lower to upper layers. Thickness of the component layers varies in each cycle. In this report, the zones of thick sandstone and thin other banded layers are denoted by the name of massive sandstone members (Ms members), and the zones of this sandstone and rather thick banded layers by the name of alternation of sandstone, silstone and mudstone members (At members).

In the damsite, Bst Formation contains eight Ms members and the same number of At members which are alternated and numbered Ms-1 to Ms-8 and At-0 to At-7 from lower to upper zone respectively.

On the left bank are three stages of terrace at the elevations 260 m, 230 m and 200 m which are called the higher terrace, middle terrace and lower terrace respectively hereinafter. Among them, the middle terrace has substantial significance for the dam construction due to its wide and thick deposits lying within the range of dam foundation.

Geological mapping, seismic refraction prospecting, core drilling with permeability tests, test grouting, excavation of test adits and trenches and rock tests of in-situ rock masses and rock samples in laboratory have been carried out for the feasibility study during the period of February and March 1981 (Stage I) and October 1981 to April 1982 (Stage II).

The location map of geological survey in the damsite is shown in Fig.-E.3. List of survey works and actual progress done in the field are shown in Table-E.3.1 and Table-E.3.2.

As shown in Table-E.3.2, the field work was carried out in the dry season which is the period from October to the next April. Particularly works in the river, e.g. core drilling and seismic refraction prospecting in the river portion, were concentrated in January to March, because the river flow is still too large in the early half of the dry season.

Although the survey in Stage I was focused on the damsite A which had been proposed in the prefeasibility report, the survey area was extended in Stage II to cover alternative damsites B and C which were assumed in the downstream stretch within 1 km from the damsite A for finding the most suitable damsite.

3.2 Seismic Refraction Prospecting

Seismic refraction prospecting method has been applied for overall investigation of damsites and gravel deposits for aggregate materials. As shown in Table-E.3.3, 21 profiles of 12,475 m long in total have been surveyed, out of which 4 profiles of 2,715 m in total were done in the gravel deposits and the remainder was in the damsites. Profile lines were arranged in such a manner as shown in Fig.-E.4 for the damsites and Fig.-E.5 for the gravel deposits.

3.2.1 Operation in the Field

The arrangement of shots and detectors was planned to be laid out on a line (profile shooting). Before the field recording began, ground surface profile were surveyed and shot and detector stations were marked by stakes.

In a cycle of operation, the distance range was arranged in such a manner that five or seven shots of 50 to 60 m intervals were picked up by 21 detectors which were spread at regular intervals of 5 m as illustrated in Illus.-3.1.

The shot locations and detector spreads of one operation cycle were moved progressively to give complete coverage over the refraction profile lines as shown in the Fig.-E.3 "LOCATION MAP OF INVESTIGATION" except the river channel area. In the river channel, shots were done at 5 m intervals in water and detectors were set at several points on both banks and gravel bar.

Main instruments and materials used are shown in Table-E.3.4.

3.2.2 Profile Line Arrangement in the Damsite

Since three alternative damsites named the damsite A, damsite B and damsite C from upstream to downstream site were proposed, seismic refraction profile lines were arranged to form a grid with two lines in each site across the river and three lines parallel to the river as shown in the Fig.-E.4. Out of three parallel lines, one was on the right bank and the other two lines were on the left bank because of fluvial terraces widely spread on the left bank.

In the damsite A, since the bedrock was overlain by a thick terrace deposit even at the left end of the profile SL-1, the profile SL-6 was added in the direction towards further hill side. SL-7 was also added in the damsite A for the purpose of checking the left end area of SL-1 where a low velocity layer was detected as well as thick overburden.

The refraction profile line SL-5 was laid out on the gravel bar located about 300 m upstream of the damsite A in order to obtain data of thickness of the gravel deposit. SL-14 was laid out to take a refraction profile along the test adit TA-2.

3.2.3 Time-Distance Plot and Profile Interpretation

Travel time of the seismic wave (primary wave) was read from recording paper to the accuracy of 1/1,000 second, and plotted on the time-distance graph. From this graph, the profile of velocity layers was drawn mainly by means of Hagiwara's method. The time-distance curves and the profile interpretations are attached at the end of this volume as "SEISMIC REFRACTION PROFILE: TIME-DISTANCE PLOT AND INTERPRETATION".

Correlation between velocity layers and geology as shown in the Table-E.3.5 is deduced from other geological data obtained in surface observation, core drilling, test aditting, etc. as well as refraction profile. This correlation table is explanatory of the geological conditions of refraction profiles.

For the damsite, the distribution plan of speed of the deepest layer, which is the fifth layer in the correlation table and generally represents the fresh bedrock, is shown in Fig.-E.6. Velocities of deepest layers are in the range from 2.6 km/sec to 3.9 km/sec, except low velocity zones which are the segments of remarkably low speed such as 1.5 km/sec to 1.6 km/sec zone in the profile SL-7, 8 and 9.

The low velocity zone may represent defect of the bedrock such as a fractured zone and other anomalies. In the case of damsite A left bank where several low velocity zones are distributed, it seems that they have continuity and infer the depression of old river channel, though it must be surveyed by core drilling. Then the continuity of them and similar low velocity zones are shown in the figure as probable trends of the low velocity zones. Other low velocity zones such as on the right bank possibly correlate to some cracky or fault zones.

Since the variety of speed of the deepest layer can be assumed to reflect the property of underlying bedrock, the 2.8 km/sec zone in the river segment of the SL-1 line appears to extend toward the similar velocity zones of SL-2, SL-13, SL-10, SL-17 and SL-15 southeastwardly as shown in Fig.-E.6. In the same manner, the 3.5 km/sec to 3.6 km/sec zone on the right bank of the SL-15 appears to extend as shown in Fig.-E.6.

The traces of 2.8 km/sec zone and 3.5 to 3.6 km/sec zone appear to correlate with Ms-8 Member and At-3 Member in the geological map respectively.

3.3 Core Drilling

As shwon in Table-E.3.6, 21 holes of 941.5 m in total linear length were core-drilled during the feasibility study stage. Since the terrace deposits on the left bank of damsite A, which was the proposed damsite in the prefeasibility study, were revealed so thick by the core drilling of B81-1 and B81-10 in the early stage of survey, the original layout of drill holes in which the drill holes had been arranged so as to investigate damsite A principally was revised in the course of the investigation, i.e. some holes were shifted from damsite A to alternative damsite B and C and some were added. Conclusively 10 holes were drilled in the damsite A, 9 holes in the damsite B and 4 holes in the damsite C. The layout of holes drilled is shown in Fig.-E.3 "LOCATION MAP OF INVESTIGATION".

Coring was carefully done and recovered core samples were well arranged in core boxes in order and inspected by a geologist to draw up logs. The core samples are kept in the warehouse in Birganj by the Electricity Department, HMG.

3.3.1 Drilling Operation

The following drills and pumps were used for core drilling.

<u>Drills;</u>	TONE UD-5	2 sets
	KOKEN OE-2L	1 set
<u>Pumps;</u>	KOKEN MG-5A	2 sets
	KOKEN MG-10	2 sets
	YAMMER F (for power sprayer)	1 set

Core drilling was carried out by means of a double tube core barrel with a diamond coring bit of 66 mm in diameter. The collar of hole was drilled and protected by larger size casing pipes of 114 mm in diameter for the drilling in the river and of 86 mm or 116 mm on both banks.

It is noted that the drilling through the riverbed deposits was most difficult in the drilling operation of this time since the riverbed deposits were so thick in addition to so hard boulders contained.

Especially the drilling of No.B81-6 hole did not reach the bedrock even at the depth of 36 meter.

3.3.2 Logging

Data obtained through careful inspections of the drilling cores are summarized in "DRILL LOG" which is attached at the end of this volume.

Rock classification

For logging, the following classification of rocks were applied in aid of assessing dam foundation in the project site.

Class	Description
A :	Fresh and hard rock without any discontinuities in sufficient extent at least a few meters. No segment of drilled core corresponded to the class A, so that the class A has been only an ideal ranking so far.
B :	Fresh and hard rock and cores being recovered in long shape. Or fresh and moderately hard without discontinuities in a meter or more. Generally RQD indicates 100%. Rather high cohesion such as more than 10 kg/cm^2 may be expected as a mass.
C_H :	Fresh rock. Hard to moderately hard and cores being recovered generally in long but including some defects such as crossing cracks and/or cracks with clay seams with a frequency of once in a few meters. Then the percentages of core-recovery and RQD generally indicate 100% and more than 70% respectively. In the case of soft rock, the core of this class is so long as RQD showing 100% without defect. The rock classified as C_H generally appears as good as being sufficient for the foundation of a high fill dam. For construction of a concrete gravity dam, the trend of discontinuity in the hard rock and mechanical property of soft rock may be seriously concerned.

C_M : In the case of hard rock, cracks and/or thinly bedded planes are included in places, so that cores are frequently taken in the shape less than 10 cm and/or fragmental. Consequently RQD becomes low to such a degree as 30 to 70%.

Core recovery is, however, as high as 90 to 100%, because fractured and/or clayey materials are rarely included. In the case of weathered and/or soft rock, core are long with high percentage of RQD.

In general the rock of class C_M is not favorable for the foundation of a high dam. Particularly if it widely spreads, dam construction will be restricted in its scale or some special device will be required.

C_L : Rocks are cracky and/or including a remarkable clay layer or weak zones and cores are rarely taken more than 10 cm long. Consequently the percentage of RQD becomes low or zero.

Rocks, which are massive but so weathered and/or weak that they are unable to retain their shape when being taken out of a core barrel or easily crashed out by hand, are also classified here. The rock of class C_L is unfavorable for dam construction and usually required to be removed and/or replaced to a certain extent. Therefore if it spreads widely, dam construction will be seriously restricted.

D : Fractured and/or decomposed rocks. In the case of no core having been taken, the corresponding segment is classified here. Rocks decomposed by weathering will be removed and fractured zones as well as fault zones must be removed and/or replaced by suitable materials and treated by grouting etc. to a good extent.

Generally the rock of class D must be paid careful attention for dam construction.

In the above index of rock classification, hard rock means the rock of which compressive strength under the moistended condition appears to be more than 200 kg/cm^2 and moderatley hard rock to be ranging from 80 kg/cm^2 to 200 kg/cm^2 and soft rock to be less than 80 kg/cm^2. This classification of rock strength is exclusively applied to rocks in the damsite, not on quarry rocks and others.

RQD

RQD is an index termed the Rock Quality Designation which is obtained by examining the core recovered from a borehole, discarding sections of core less than 10 cm long, and expressing the remainder as a percentage of the total length drilled.

Then it can be formulated as the following:

$$RQD = \frac{\text{Total length of cores longer than 10 cm}}{\text{Total length drilled}} \times 100\%$$

In the logging of this survey, the core was examined in each one meter for RQD, except a fraction of upper end of rock zone of a hole. Then it was calculated by the following equation modified the above.

$$RQD = \frac{\text{Total length of cores longer than 10 cm}}{\text{100 cm}} \times 100\%$$

3.3.3 Results

The data obtained from core drilling are shown in the drilling logs, which are self explanatory. They have been utilized for processing geological profiles which displays the general idea on the damsite geology together with such other field data as of water pressure test, seismic refraction prospecting, test adit, etc.

Some representative pieces of boring core have been taken out and sent to Tokyo for the purpose of performing rock tests in laboratory, which are discussed in the section 3.8 hereinafter. The segments from which rock samples have been taken out are listed in Table-E.3.7.

3.4 Permeability Test

In-situ permeability of rock was measured by the method of the "water pressure test" (or Lugeon test) in boreholes drilled. Testing methods applied for terrace and riverbed deposits differed from the above "water pressure test". The test methods for the terrace and riverbed deposits are called "pumping-in test" and "open-end pipe test" respectively.

The water pressure test was applied in 112 sections of 17 boreholes and the pumping-in test and the open-end pipe test were in 2 sections each.

3.4.1 Testing Method

For the water pressure test (Lugeon test), a plug seal (rubber packer) was inserted at about 5 m from the base of the hole and then water applied under pressure through a pipe extending through the plug to the base of the hole. The flow of water is measured, at various pressures as shown in Table-E.3.8. The permeability of the rock was assessed in both terms of Lugeon units and permeability coefficient. One Lugeon unit is defined as a flow of 1 lit/min per lienar-meter of hole at a standard applied pressure of 10 kg/cm^2; one Lugeon equals to about 10^{-5} cm/sec, the precise equivalent being dependent upon the diameter of test borehole and the depth to the water table. Then the Lugeon units and the permeability coefficient were calculated by the following equations, Lugeon units

$$Lu = \frac{Q}{L \cdot P} \times 10$$

where, Lu = the Lugeon units
Q = the constant rate of flow into the hole (lit/min)
L = the length of test section (m)
P = the pumping pressure applied (kg/cm^2)

Permeability coefficient (from the packer test in "Earth Manual" USBR)

$$k = \frac{Q}{2 \cdot L \cdot H} \times \log_e \frac{L}{r} \text{ (cm/sec)}$$

where, k = the permeability (cm/sec)

Q = the costant rate of flow into the hole (cm^3/sec)

L = the length of test section (cm)

H = the differential head of water (cm)

H = H(gravity) + H(pressure)

r = the radius of hole tested.

The pumping-in test was the method applied in the drill holes Nos.B81-1 and B81-10 for determination of the permeability coefficients of terrace deposits which were too soft and weak to set such a rubber packer as used in the water pressure test. The borehole B81-1 was drilled on the left bank of damsite A into a terrace deposit by a 86 mm diameter core barrel. When it reached deep to 19 m without casing pipe, the test was done by injecting water into the hole, adjusting to maintain a constant head of water at the collar of the hole and the rate of flow being measured after it became constant. For the calculation of permeability coefficient, the same equation with the water pressure test was applied. In the drillhole B81-10, the pumping-in test was done similarly to the drillhole B81-1, but the test section was confined within the depth 36.2 m to 37.8 m due to casing pipe inserted upper than 36.2 m.

The open-end pipe test was applied in the drillhole No.B81-13; it was drilled in the river channel of the damsite B through a riverbed sand and gravel deposit which was too loose to be tested by not only the water pressure test but also the pumping-in test. The test was made through the open end of a pipe casing (inside diameter 101.6 mm) which had been sunk to the depth tested. Before commencement of test, the inside of casing pipe was cleared of sand and fragments deep to the bottom by drilling with 66 mm diameter core barrel and then the test was begun by adding clear water into the hole to maintain a constant head with or without pumping pressure. Measurement of constant head, constant rate of flow into the hole, size of casing pipe, and elevations of top and bottom of casing were recorded. The permeability was obtained from the following equation quoted from "Earth Manual" by USBR.

$$k = \frac{Q}{5.5 \cdot r \cdot H}$$

where, k = the permeability (cm/sec),

Q = the constant rate of flow into the hole (cm^3/sec),

r = the internal radius of casing (cm), and

H = the differential head of water (cm).

3.4.2 Permeability Determined

The data and results are shown in the tables attached at the end of this volume as "RECORDS OF WATER PRESSURE TESTS", of which summary is shown in Table-E.3.9 on 112 stages of the water pressure test in the bedrock and in Table-E.3.10 on 4 stages of the pumping-in test and open-end pipe test in the terrace and riverbed deposits.

Permeability of terrace deposits

The permeabilities of terrace deposits measured in the boreholes B81-1 and B81-10 show rather wide range from 1.41×10^{-4} cm/sec in the former to 4.69×10^{-3} cm/sec in the latter. Although the number of stages tested are not sufficient, it appears that the above permeabilities are well correlated to the geological condition, suggesting general geological properties of the terrace deposits.

The section tested in the B81-10 was in the depth between 36.2 m to 37.6 m just above the contact with the bedrock and composed of gravely materials including cobbles and boulders. The ground water table observed in the borehole (on December 24) was at 30.0 m in depth and springs from this basal gravel layer were observed along the riverside. Then the permeability measured in the B81-10 seems to represent the permeability of aquifers composed of gravely deposits.

On the other hand the test in the B81-1 was made in the deposit which mainly consists of brownish color micaceous sandy silt with scattered pebbles, cobbles and boulders. They were rather cohesive so that the inside face of the hole drilled was well maintained even during the test. Since the lower part of the section tested seems a bit more pervious than the upper because the content of pebbles and cobbles being higher below 15 m, it appears that the permeability of 1.41×10^{-4} cm/sec as a whole section is larger than the actual, particularly for the upper section.

As a whole, the permeabilities of the terrace deposit II can be assumed around 5×10^{-3} cm/sec for rather pervious aquifers and less than 1×10^{-4} cm/sec for silty sections and there may be various intermediate permeabilities between them correlating variety of sand and gravel contents. The terrace deposit III predominantly consists of sandy silt and clay; then it may be rather impervious such as less than 1×10^{-4} cm/sec in general.

Permeability of riverbed deposits

In the borehole B81-13, the open-end pipe test was made at the depths of 5 m and 10 m, both of which are composed of gravely riverbed deposits. At the upper one (5 m in depth), the constant differential head was maintained in the standing casing pipe 2 m above the river water level; and at the lower one (10 m in depth) pumping pressures of 1, 2 and 3 kg/cm^2 were applied. Permeabilities determined are 1.5×10^{-3} cm/sec at the upper point and 5.9×10^{-3} cm/sec at the lower. It seems that the permeability ranging 1.5×10^{-3} cm/sec to 5.9×10^{-3} cm/sec indicates a deposit composed of clean sand and gravel with a little contents of silt, though only the cores of gravel were recovered by drilling.

Permeability of bedrocks

As shown in Table-E.3.9, the Lugeon value determined ranges from rather impervious (less than 1 Lugeon unit) to 159.3 Lugeon units in maximum. Most of the Lugeon value determined (67% of stages tested), however, show less than 4 Lugeon units under the maximum applied pressure shown in Table-E.3.8. The stages larger than 10 Lugeon units are scattered and correspond to 17% of all stages tested. These results consist with the rock facies observed as being massive appearance and high percentage of RQD examination in general.

As a whole, permeabilities of bedrocks have not shown any serious obstruction against a dam construction. This subject will be discussed in the section 3.5 "Test Grouting" again.

3.4.3 Water Table Observation

After core-drilled, perforated polyvinyl chloride pipes were installed in the boreholes, and the fluctuations of water table were recorded during the period from February 26 to April 7, 1982 as shown in Table-E.3.11. The result of the water table observation is briefed below.

1) The water table in the terrace deposit II on the left bank of damsite A was in the elevation of 194 m to 196 m. It appears that the water table at the elevation 195.8 m observed in the borehole B81-9 on November 3 lowered to the elevation 194.6 m observed in the borehole B81-10 on December 27.

2) The water table observed in the borehole B81-12 has lowered steadily in an average 4.25 cm per day during the period from February 26 to April 7.

3) On the terrace deposit III, there was no ground water according to the observation in the borehole B81-2 and B81-7 during this period.

4) In the terrace deposit I also, there was no ground water in the borehole B81-14 and B81-18 during this period, though a water table may be formed in the rainy season.

5) The borehole B81-8 on the damsite A left bank encountered confined ground water around 32 m in depth. The water head was 4.5 m above the collar of the hole, which was measured by means of setting a rubber packer at the depth of 30 m. This spring has considerably decreased in March and April (the late months of dry season) as seen in the water level records in the borehole lowered to the elevation 195.9 m on April 7 against the elevation 201.5 m on November 23 (the early month of dry season).

6) Other water levels observed in the boreholes are all in the fresh rocks. The water levels observed indicated a decline toward the river side. The lowest water level was observed at the elevation 185.4 m, about 1.5 m higher than the river water level, in the borehole B81-17 on the damsite B right bank.

3.5 Test Grouting

Test grouting was performed on the left bank lower terrace of the alternative damsite B as shown in the location map Fig.-E.3.

3.5.1 Geology

The bedrock test-grouted is composed of mudstones, siltstones and fine and medium sandstones of At-5 Member of which boring cores have been taken and examined in the drilled holes Nos.TG-1 to 7 as shown in "DRILLING LOG" attached at the end of this volume (Attachment II-41 to 47) and Fig.-E.7 "GEOLOGIC SECTION OF THE TEST GROUT SITE". The bedrock is overlain by a 3.5 m-thick terrace deposit and the bedding dips in the direction from the river side to the hill side by about 30 degrees. Rocks are mostly classified as C_M and C_H classes as shown in Fig.-E.8 "CROSS SECTION OF ROCK CLASSIFICATION IN THE TEST GROUT SITE".

Since rocks of the At Members are various as soft mudstone to hard calcareous shale and well bedded with laminations and cracks rather than the Ms Members and Sst Formation which show massive and uniform appearance, a test grouting site was planned to locate in At Member as well as in the conditions of flat ground surface and thin overburden. Then the test grouting site was selected as said above.

3.5.2 Grout Hole Arrangement and Test Procedure

The following seven holes were drilled in a series of test grouting of this time.

Hole No.	Depth	Sequence of Drilling and Grouting
TG-1	30 m	Primary hole
TG-2	30 m	Primary hole
TG-3	30 m	Secondary hole
TG-4	20 m	Tertiary hole
TG-5	20 m	Tertiary hole
TG-6	20 m	Check hole
TG-7	20 m	Check hole
Total	170 m	

Grout holes were laid out on a single line in split spacing method as shown in Fig.-E.9 "RECORD OF TEST GROUTING". Firstly, the primary holes TG-1 and TG-2 were drilled at 4 m of distance from each other, water-pressure-tested and grouted by 5 m stage in descending order. Then the secondary hole TG-3 was drilled, water-pressure-tested and grouted at the middle point between the two primary holes. The tertiary holes TG-4 and TG-5 were subsequently carried out in the same manner at the middle points between the primary and secondary holes. Finally the check holes TG-6 and TG-7 were drilled and water-pressure-tested at the middle points between the secondary hole and the tertiary holes.

The injected grout was neat mixture of ordinary Portland cement and water, of which mixing ratio was 1/5 at the start of grouting and changed to 1/3, 1/2 and then 1/1 in weight proportion of cement/water.

The pressures of grouting and water-pressure-testing was decided to increase in deeper stage in accordance with increase of overburden as shown in Illus.-3.2. Five meter section at the top of each hole, which was through top soil and unconsolidated terrace deposit, was ruled out, so that the test was made in the zone deeper than 5 m.

3.5.3 Test Results

Data on permeability in the Lugeon unit and grout take of each stage are summarized in Table-E.3.12 and illustrated in Figs.-E.9 and E.10. Improvement in permeability of the bedrock by grouting can be seen in the successively changed permeability and grout take in each stage from the primary hole to check hole in the spilit spacing order as shown in Fig.-E.11 and the series of sections from Fig.-E.12 to Fig.-E.14.

The original condition of permeability in the bedrocks were generally favourable for foundation of structure except a few stages. The water pressure tests in the primary holes show a large amount of leakage such as more than 100 Lugeon only in the first stage, that is, the section between 5 m and 10 m in depth. The other relatively high leakage of 27 Lugeon was encountered in the section from 15 m to 20 m of TG-1. Other stages were lower than 4 Lugeon, that could be tolerable for dam foundation even without further treatments.

In the first stage (the section from 5 m to 10 m), the grout takes were 108 kg/m (i.e. weight in kg of cement per linear-meter of borehole) in TG-1 and 50 kg/m in TG-2, and then permeability measured in the intermediate secondary hole TG-3 which was 2 m apart from both primary holes showed 5 Lugeon. In the succeeding tertiary holes TG-4 and TG-5, both permeabilities were around 4 Lugeon. Though the low permeability of 5 Lugeon measured in the secondary holes might not be due to the effect of grouting but its original condition, the results in the tertiary holes can clearly be interpreted that cement grouting is effective when injected in the holes spaced at 2 m and less.

As for the section from 15 m to 20 m in TG-1 with 27 Lugeon of leakage and 345 kg/m of grout take, the subsequent water pressure test in the secondary hole at the same stage showed 1.07 Lugeon and the intermediate tertiary hole recorded 1.71 Lugeon. The effectiveness of grouting can also be recognized from this.

3.5.4 Recommendation on the Grouting Pattern

In view of possible existence of high leakage portions which are irregularly and sporadically scattered in the bedrocks of low permeability, a safe arrangement of grouting holes for curtain grouting, i.e. two parallel lines, 1 m apart from each other, on each of which the holes will be arranged at 2 m spacings and at staggered positions against the holes on the opposite line, is recommended.

3.6 Test Adit

Two test adits have been driven in the vicinity of the damsite A; one named TA-1 on the left bank along the dam axis and the other TA-2 on the right bank at about 220 m upstream from the dam axis A as shown in Fig.-E.3 "LOCATION MAP OF INVESTIGATION". The location of TA-2 was determined in the extension of the strata At-7 and Ms-7 Members which was supposed to be traversed by the dam axis A in the riverbed. Geological inspection, seismic exploration (direct wave) and in-situ rock tests were performed in the test adits.

3.6.1 Excavation

Adits, TA-1 and TA-2, were excavated 50 m each in linear length with such dimensions as shown in Illus.-3.3 "SECTION OF TEST ADIT".

TA-1 excavation was started in portal opencut on November 15, and reached the end-face on December 15 without supporting. The average excavation progress of tunnel part was 1.9 linear meter per day. Wooden supports were installed after completion of TA-2 in the section of the portal to 4 m, which is composed of weathered rocks, and 15 m to 20.5 m where a 5 to 20 cm-thick sheared layer is intercalated (see Table-E.3.13 "EXCAVATED ADIT LENGTH"). No water spring took place through the excavation except a little water seepage at 20 m and 37 m from the entrance.

TA-2 was driven in the period from December 22 to January 29. The tunnel section of the portal to 7 m, which consists of cracky and weathered rocks dipping towards the entrance, was excavated with fore-poling and the section of 13.5 m to 18 m from the entrance, where a 30 to 40 cm-thick fractured zone with fault clay being intercalated, was excavated with supporting. The remaining sections were unsupported as shown in Table-E.3.13. The tunnel was dry except a little seepage at 46 m from the entrance.

Laborers engaged, main equipments used and main materials consumed for tunnel excavation are listed in Table-E.3.14. Blasting holes drilled were 8 to 9 holes per square meter on a face. Dynamite charged 3.1 kg per cubic meter of excavation in general and detonators used are of five steps of delay.

3.6.2 Adit Geology

Geological information obtained in the test adits is summarized in Figs.-E.15 and E.16 "GEOLOGICAL SKETCH OF THE TEST ADIT".

<u>TA-1</u> was driven and situated in the lower part of Sst Formation in which massive mudstones, medium to fine sandstones, dark gray color silt-stones and pebbly sandstones range from the entrance to the end in order of lower to upper horizons. The mudstone shows pale greenish colored, weathered and cracky appearance. Along the boundary with the overlying

medium to fine sandstone, a sheared layer filled with clayey materials by 30 cm thick is intercalated.

The medium to fine sandstones show white gray color and grading coarse in the lower and fine in the upper. The lowermost is cross-laminated and including layered pebbles and siltstone fragments (patches). The main part of this stratum includes a few cracks, some laminations and lignite fragments, though general appearance shows massive. The contact with the overlying dark gray color siltstone is gradational. The dark gray siltstone has the thickness of 1.4 m here and includes bedding slip faults.

The pebbly sandstones occupy the inner side of the adit from 20 m point and can be divided into three units. Each unit consists of medium to coarse sandstones with layered and scattered pebbles showing graded sedimentation coarser to finer from lower to upper in general. The lowermost is pebble layers and the uppermost is thin bedded and/or laminated sandstones. The contacts of units are unconformable; i.e. laminated sandstones in the uppermost of a unit are scoured and filled by pebble layers of the lowermost of the above unit.

TA-2 was driven and situated in the lower horizon of At-7 Member to the upper half of Ms-7 Member.

Rocks exposed inside in the span from entrance to 13.5 m belong to At-7 Member; they are bedded fine sandstones, massive mudstone and siltstone patched fine sandstone (breccia) from upper to lower strata.

The inner portion from 13.5 m consists of fine to medium sandstones of Ms-7 Member having rather massive appearance, though they include weak laminate and concretions. The contact of both members is a fractured zone 30 to 40 cm thick with a clay seam, which is developed along bedding (bedding ship fault).

3.6.3 Rock Classification and Speed of Direct Wave

Rock classification of boring core logging, which was discussed in the section 3.3.2, was applied to rocks in adits correspondingly. The speed of direct wave was also measured in the adits, for which detectors were laid out longitudinally at intervals of 2.5 m. The rock classification in the test adits and the measured speed of direct wave are as follows;

In the adit TA-1 (see Fig.-E.15) main parts of massive sandstones, 2.5 m to 16.5 and 19 m to the end in distance from the entrance, are classified as C_H class, and weathered rocks near entrance are as C_M to C_L class and 16.5 m to 19 m where bedding slip faults are included is as C_L class. Speeds of seismic wave show rather wide range from 1.7 km/sec to 3.6 km/sec, though it seems that sandstones are rather massive and not so various in visual condition.

Rock classification and speed of seismic wave of the adit TA-2 are shown in Fig.-E.16. Medium to fine sandstones of Ms-7 Member show rather massive and fresh appearance, so that they are mainly classified as C_H class and corresponding speed of seismic wave ranges 2.35 km/sec to 3.45 km/sec. Rocks belonging to At-7 Member which are exposed near the entrance are weathered and cracky. They are classified as C_M and C_L classes, in which speed of wave is 2.5 km/sec. The contact zone of both members is classified in D class and corresponds with a low-speed zone of 1.7 km/sec.

3.7 In-situ Rock Test

Strength of the bedrock as a mass was measured by means of in-situ plate loading test and block shear test in the test adits TA-1 and TA-2 in the damsite A, which were excavated on the left bank of the dam axis and on the right bank about 220 m upstream from the dam axis respectively. Locations of test spots are shown in Figs.-E.15 and E.16 as well as distribution of rocks.

3.7.1 Block Shear Test

Block shear test was carried out on 15 blocks, out of which 10 blocks numbered BS-1 to 10 were in the adit TA-2 and the remaining 5 blocks numbered BS-11 to 15 in the adit TA-1. In each testing spot, a concrete block with 60 cm x 60 cm square in the basal plane of contact with rock and 30 cm in height was placed upon exposed rock surface which had been cleared of loosened rocks and cleaned sufficiently. Under a certain constant vertical load applied by a hydraulic jack, each block was pushed from the upstream side by the other hydraulic jack, gradually increasing the horizontal load until the rock under the block was sheared completely. Vertical and horizontal displacements were observed with dial gauges by the minimum reading of 1/100 mm. A typical arrangement for testing in the site is illustrated in Fig.-E.17 and the load pattern, load-displacement curve and sketches of rock surface before concrete placing and after sheared are summarized in a sheet for each block and shown in Figs.-E.18 to E.32. For examining rocks tested, the irregularities of surfaces before placing concrete and after sheared were measured and displayed in contour maps and sections as shown in Figs.-E.33 to E.38.

After once tested up to failure, blocks were reloaded and retested under the same constant normal load as before in the block Nos. 12, 13, 14 and 15 as the result being shown in Fig.-E.39. This secondary test is called "friction test", because it can be assumed that the remaining strength of rocks against the secondary shearing load must be frictional force without cohesion.

Relation between normal stress (σ) and shearing stress (τ) at the failure point of each block was plotted consequently on graphs which were divided into four groups according to the kinds of rocks tested as explained below and shown in Figs.-E.40 and E.41.

Rocks tested were distinguished into the following four groups.

(1) Massive pebbly sandstone of Sst Formation classified as C_H class; Block Nos. Bs-11 to 15 in the adit TA-1 and plotted in Fig.-E.40.

(2) Laminated medium sandstone of Ms-7 Member classified as C_H class; Block Nos. Bs-5 to 7 in the adit TA-2 and plotted in Fig.-E.40.

(3) Massive medium sandstone of Ms-7 Member classified as C_H class; Block Nos. Bs-1 to 4 in the adit TA-2 and plotted in Fig.-E.41.

(4) Mudstone and weathered rocks of At-7 Member classified as C_M class; Block Nos. BS-8 to 10 in the adit TA-2 and plotted in Fig.-E.41.

The proper shear strength of each kind of rock was deduced from the normal stress - shear stress plot according to the Coulomb criterion of failure.

The shear strength is expressed by the following equation:

$$\tau = \tau_o + \sigma \tan \phi$$

where, τ = the shear resistance at failure, i.e. the shear strength (kg/cm^2),

τ_o = the shear strength at $\sigma = 0$, i.e. the cohesion (kg/cm^2),

ϕ = the internal friction angle (degree)

σ = the normal stress at failure (kg/cm^2).

The result is as follows:

Rock	Classification	τ_o (kg/cm^2)	ϕ (degree)
(1) Massive pebbly sandstone of Sst Formation	CH	9.0	55
(2) Laminated medium sandstone of Ms-7 Member	CH	9.0	50
(3) Massive medium sandstone of Ms-7 Member	CH	8.0	40
(4) Mudstone and weathered rocks of At-7 Member	CM	6.5	37

3.7.2 Plate Loading Test

Plate loading test was carried out at three spots; i.e. test No. PL-1: 42.5 m from the entrance in the adit TA-2, of which rock is massive medium sandstone of Ms-7 Member,

PL-2: 32.5 m from the entrance in the adit TA-2, of which rock is laminated medium sandstone of Ms-7 Member and

PL-3: 23.5 m from the entrance in the adit TA-1, of which rock is massive pebbly sandstone of Sst Formation.

A loading plate 30 cm in diameter was put horizontally on a thin mortar base which had been placed on clean surface of bedrock free of loose rocks. Vertical load was applied by a hydraulic jack as shown in Fig.-E.42, increasing at a rate of 2 tons every minute up to the first peak of 6 tons and then decreasing in the same rate to zero, and again incleasing in the same manner up to the second peak of 12 tons and decreasing to zero, repeating incleasingly more three times up to the final peak of 28 tons. Such repeated application and relaxation of load was again continued four times in the same peak of the final load as shown in the graph of "LOAD PATTERN" in Figs.-E.43 to E.45. Displacement of the base was measured by dial gauges pointed upon a loading plate.

From the field data obtained, the time-displacement relation and load-displacement relation curves were prepared as shown in Figs.-E.43 to E.45 for each spot. Then the moduli of elasticity on both tangential (Et) and secant (Es) slopes in each iterated peak, which are shown in Illus.3.4 as the slopes II and III respectively, were calculated by the following equation,

$$E = \frac{(1 - n^2)}{2a} \times \frac{F}{d}$$

where, E = the modulus of elasticity (kg/cm^2)

a = the radius of a loading plate (15 cm)

n = the Poisson's ratio (assumed at 0.25)

F = the change in stress (kg)

d = the change in displacement (cm)

The modulus of deformation (D) was calculated on the slope I in the Illus.3.4 by the same equation of the modulus of elasticity. The results of calculation are shown in the tables of Figs.-E.43 to E.45, of which summary is as the following:

Test No.	Rock	Modulus of Elasticity (kg/cm^2) On the Tangential Slope (Et)	Modulus of Elasticity (kg/cm^2) On the Secant Slope (Es)	Modulus of Deformation (D)(kg/cm^2)
PL-1	Massive medium sandstone	14,000	11,400	7,400
PL-2	Laminated medium sandstone	23,400	24,700	11,100
PL-3	Msssive pebbly sandstone	22,500	15,900	10,900

3.8 Laboratory Test on Rock Sample

Rock test was performed on the rock samples which had been obtained from boring core.

3.8.1 Test Items and Number of Samples

Samples were taken from 11 sections of 3 holes drilled which are listed in Table-E.3.7. Rocks sampled wered divided into the following five groups.

<u>Group</u>

I White gray color and massive medium to coarse sandstones which predominantly distribute in the damsite.

II Greenish colored massive sandy mudstones which are intercalated in places with a thickness of a few decimeters to several meters and seems to be the most soft rock facies in the damsite by visual assessment.

III Dark gray siltstones which generally show thin bedded and/or strongly laminated appearance accompanying bedding slips in places and being some calcareous.

IV Intraformational breccias which are calcareous, hard and showing a thick bedding of one meter or so.

V Grayish colored calcareous fine sandstones which show bedding appearance with various thickness of a few decimeters to a few meters and being common in the At Members.

Testing items performed and the number of samples tested are shown in Table-E.3.15.

3.8.2 Test Results

The test results are summarized in Table-E.3.16 and E.3.17.

Unconfined compressive strength of the medium sandstone tested (Group I) ranges 116.8 kg/cm^2 to 140.9 kg/cm^2 in the air-dry condition (NAT) and 96.5 kg/cm^2 to 106.6 kg/cm^2 in the saturated condition (SAT). It is found in the greenish sandy mudstone (Group II) that its strength is remarkably lowered in the saturated condition down to 53 kg/cm^2 to 95.5 kg/cm^2 notwithstanding the strength of 163.9 kg/cm^2 to 261.3 kg/cm^2 in the air-dry condition. The rock sample of greenish sandy mudstone tested seemed to be rather firm one compared to the other parts of the similar rocks, most of which tended to become fragment automatically by weathering (slaking) in the air within a few weeks after cored out. The samples tested were covered with saran-lap immediately after taking out of a borehole.

Calcareous rocks of intraformational breccia and fine sandstone (Group IV and V) show such a high strength as 687 kg/cm^2 in unconfined compression test under air-dry condition. Although calcareous siltstone (Group III) has appeared as hard as other calcareous rocks (Group IV and V), it shows 131.7 kg/cm^2 and 206.5 kg/cm^2 of unconfined compressive strength due to its anisotropism by lamination.

Other information shown in the summary tables are selfexplanatory on the physical and mechanical properties of rocks tested, but it is noted that the information does not give the properties as the actual rock mass.

3.9 Test Trench

Test trenches have been dug at four places; two trenches were in the damsite B (TR-1 and TR-2) and the other two were in the damsite C (TR-3 and TR-4) as shown in the location map Fig.-E.3. In each damsite, one was located on the left bank and the other on the right bank (see Table-E.3.18).

3.9.1 Purpose of Trenching

The purpose of test trenching was generally to reveal distribution and condition of strata which were covered by top soil. Particular purpose of each trench was as follows;

TR-1: TR-1 was excavated on the river side slope of the terrace III of which deposits had been supposed to be composed of reddish brown colored silty soil and gradationally changing into the bedrock through decomposed rock zone at the depth of 12 m in the core drilling No.B81-2. Transition of terrace deposits and decomposed rock zone was, however, unclear in the boring core. Then TR-1 was planned to inspect the composition and depth of terrace deposits as well as the condition of transitional zone and bedrock.

TR-2: TR-2 is located on the right bank slope with rather constant inclination about 30° above the elevation 220 m and supposed to be thinly covered by top soils and some screes. Since core drillings were mainly performed in the area to be thickly covered by overburden such as in riverbed and on terraces, the abutment of right bank was investigated by test trenching to check the thickness of overburden and weathering condition of bedrock as well as to confirm the dam abutment.

Purposes of the test trenches TR-3 and TR-4 to investigate the damsite C are same as those of TR-1 and TR-2 respectively.

3.9.2 Geology of Trench

<u>TR-1</u>: As shown in Fig.-E.46, the terrace deposit consists of reddish brown color silty loam with small fragmental fabric including decomposed rock fragments and pebbles in the lowermost horizon. The contact with the underlying bedrock is exposed at 12 m point from the top of trench, where is about 249 m in elevation. The bedrock exposed is composed of massive fine to coarse sandstones, conglomerates and muddy sandstones of all being weathered and yellowish colored. The conglomerate exposed at 25 m point is attributable to the lowermost conglomerate of Sst Formation, though it shows thickness is rather reduced. Below 35 m point, the bedrock is covered by thick talus deposit which consists of brown color soil with soft rock fragments and pebbles. These pebbles may be derived from pebbly sandstones and conglomerates of Sst Formation.

<u>TR-2</u>: TR-2 was excavated 42.5 m in longitudinal length along the slope of right abutment of the damsite B. Its geological profile is shown in Fig.-E.46. The slope has an angle about 28°. The bedrock was covered with weathered rock fragments and sandy soil as well as top soil a few decimeter minimum to a few meter maximum in thickness. Bedrocks are yellowish color weathered and massive medium to coarse sandstones which belong to At-3 Member. Although rocks are weathered and weak, it appears that there is no structural disturbance such as a fault but weathering from the surface. Therefore it is confirmed that there is a sufficient rock mass in the right abutment of the damsite B upto elevation 247 m at the top of the trench TR-2.

<u>TR-3</u>: As shown in Fig.-E.47, a poor basal gravel layer of Terrace III underlain by extremely weathered medium to coarse sandstones was revealed at elevation 240 m. The underlying sandstones belong to Ms-5 Member and show brownish colored and massive appearances. The bedrock has been exposed in the limited section of within 10 m span, and covered in both sides by talus deposits of which thickness appears to increase toward the river side.

TR-4: The geological profile of TR-4 is shown in Fig.-E.47. The slope of this site is as steep as 35° in average and bedrocks are covered with thin top soil and rock fragments. Though the bedrock of the right abutment of damsite C was confirmed upto elevation 270 m in such a manner similar to TR-2 site, it is found that the bedrock consists of decomposed fine sandstones, breccias and muddy sandstones of At-1 Member which are more weathered and cracky than the rocks in TR-2 site.

IV. CONCLUSION OF INVESTIGATION

4.1 General

The distribution and engineering geological features of soil and rock facies which have been revealed in the investigation and described in the preceding chapter 3 are discussed in this chapter in conclution.

4.2 Damsite Geology

The up-to-date stratigraphical study results are shown in Fig.-E.2 "GEOLOGICAL MAP AND SECTION OF DAMSITE". In general, the bedrock of the damsite consists of Bst Formation and Sst Formation, out of which Bst Formation contains 16 members of alternating At Member and Ms Member, i.e. eight At Members numbered At-0 to At-7 and the same number of Ms Members numbered Ms-1 to Ms-8 lower to upper respectively.

4.2.1 Alternative Damsite-A

A geological section along the dam-axis of the alternative damsite-A is shown in Fig.-E.2 as "Geological Section A-A'".

On the left bank, the middle terrace widely develops with a thick deposit which was penetrated by the core drilling B81-10 confirming the bedrock at 37.6 m in depth. Although the core drilling B81-1 was drilled also on the middle terrace deep to 30 m, terrace deposits of silt, sand and gravel still continued even at the bottom of the hole. This wide and thick overburden renders the alternative damsite-A little attractive compared with other alternatives. The bedrock of the left bank overlain by the middle terrace consists of Sst Formation.

The boreholes B80-1 and B80-2 in the riverbed, each of which was drilled on the dam axis at about a quarter of the width of the riverbed from either bank, reached the bedrock at 16.2 m and 17.8 m of depths respectively. In the central parts of the river channel, however, the bedrock appears to lower down to about 30 m of depth, according to the interpretation of seismic refraciton prospecting in which the boundary between velocity layers of 2.2 km/sec and 2.8 km/sec is deemed correlative to the contact of the river deposit and the underlying bedrock.

In the area from the river channel to the right bank, the bedrock consists of At-4 to Ms-8 Members of Bst Formation out of which the massive sandstones of Ms-6 and Ms-7 Members seem some weak and the rock of At-4 Member rather sound.

4.2.2 Alternative Damsite-B

A geological section along the dam-axis of the alternative damsite-B is shown in Fig.-E.2 as "Geological Section B-B'"

A geotechnically favourable feature of this site is that the middle terrace on the left bank appears to decrease in width compared with the damsite-A. On the other hand, the river gravel deposit may be thicker than in the damsite-A, as indicated by the bottom of the 2.2 km/sec velocity layer of seismic propsecting. While the bore hole B81-11 revealed the thickness of the riverbed deposit being 30.5 m at the center of the river on the dam axis, the interpretation of seismic refraction profile SL-10 (along the dam axis) shows the thickest point to be thirty plus a few meters. It seems that the thickness of the river gravel deposits tends to increase downstream as suggested by the bore hole B81-6 located about 140 m downstream which did not reach the bedrock even drilled deep to 36 m.

The bedrock of the damsite-B consists of At Members and Ms Members of Bst Formation, except the left bank rim of massive pebbly sandstones (Sst Formation). At-4 and Ms-4 Members overlain by the riverbed deposits appear hard and sound in the core drillings B81-13 and B81-15. Some fractured zones of 0.5 to 2.0 m in thickness were observed in the core drilling B81-3, which was drilled on the right bank into the rocks of At-3 Member.

4.2.3 Alternative Damsite-C

A geological section along the dam axis of the alternative damsite-C is shown in Fig.-E.2 as "Geological Section C-C'".

The lower and middle terraces on the left bank cover wider area in this site than in the alternative damsite-B; that is, 283 m as against 234 m in the site-B in width along the dam axis. The contact of middle terrace deposit and underlying bedrock appears to be at EL.200 m to 210 m as inferred by the boundary between the 0.8 km/sec and 1.8 km/sec velocity layers in the interpretation of seismic refraction profile SL-11.

It seems that the depth of the riverbed deposit in the damsite-C is not shallower than the depth in the site-B, because the damsite-C is located 400 m downstream from the damsite-B.

The bedrock consists of the strata At-1 Member to At-5 Members as shown in Fig.-E.2. On the right bank, intercalations of thin slip fault clay are commonly observed along bedding planes.

Although the damsite-C is similar to the damsite-B in the features of bedrocks, terraces and riverbed deposits, it seems not to be better than the damsite-B in all of those features from the engineering geological viewpoint.

4.3 Engineering Geology

Engineering geological condition of the damsite is discussed in this section, out of which the sub-sections 4.3.4 and 4.3.5 about physical properties, strength, permeability and grout effectiveness are essentially concerned in the bedrocks for the dam foundation.

4.3.1 Unconsolidated Deposits

(1) Top soil

This is mainly composed of silty soil and the thickness is very often 1 m or less. Organic materials are contained. This layer should be removed in foundation excavation for dam.

(2) Talus deposits

The talus deposits, mixture of unconsolidated soil and rock fragments, are locally formed on parts of the slopes. This should be also removed from the dam foundation.

(3) River deposits

Sand and gravel deposits filling the present river channel show approximately 200 m of width and 30 m or more thickness in the thickest portion. The sand is generally fine. The gravels are of biotite hornblende quartz gneisses, quartzites, schistose sandstones, slate and calcareous rocks, well rounded and hard. Their sizes range predominantly from 2 cm to 10 cm in diameter and cobbles and boulders are often included.

The river deposits can be deemed to have sufficient bearing capacity for foundation of outer shell zone of fill type dam. However, for its high permeability, impervious core zone of fill dam should be founded on the bedrock excavating the river deposits, or otherwise a cut-off wall to reach the bedrock has to be constructed through the river deposits. For foundation of concrete gravity dam, the river deposits should be excavated to the surface of the bedrock.

(4) Terrace deposits

This member develops on the left bank, forming a belt which is nearly parallel to the present river channel and covering the area of flat and partly subsided bedrock surface which was presumably an ancient river channel. The deposits consist of clay, silt, sand and gravel. The deposits are formed in the level higher than EL.180 m, with the maximum thickness more than 40 m and the width from 100 m to 450 m. Geotechnically, the terrace deposits are to be dealt in the same way as the river gravel deposits.

4.3.2 Residual Soil and Decomposed Rock

In this zone the bedrocks are intensively weathered into very soft and friable condition and further into the condition of compact soil. Intensity of alteration tends to increase from bottom to top of the zone.

This is extensively developed on the flat hill at EL.240 m to 250 m on the left bank, with several meters to more than 10 m of thickness. According to the soil tests for impervious core material of fill dam, the residual soil, reddish brown colored, contains 15 to 40 percent of clay and up to 40 percent of sand, with considerable local variation of particle size distribution presumably reflecting difference of the mother rocks. Decomposed rocks are also found under the terrace deposit, as was seen in the core dirlling B81-12.

The residual soil and decomposed rock zone are not competent for foundation of concrete gravity dam because of insufficient strength. For foundation of impervious core zone of fill dam also, this zone has to be removed because leakage paths are often likely to develop along discontinuity in intensity of decomposition to cause piping. It can be foundation for the outer shell zone of fill dam. Effect of cement grouting is dubious for this zone.

4.3.3 Weathered Bedrock

The weathered rock zone, composed of yellowish grey colored weakened rocks, develops in the surfacial parts of the bedorck and below the residual soil and decomposed rock zone. Its thickness varies locally, from 1 m to 7 m. Generally it is thicker in the higher parts of the abutment slopes. Occasionally this zone is lacked and fresh rocks are exposed or are in direct contact with unconsolidated deposits (Drilling B81-11 in the river bed, B81-9 through the terrace deposit, TG-1 and other test grouting holes).

The weathered rock zone is not competent for foudnation of concrete gravity dam, because of insufficient strength. Also in the parts where a fill type dam is high above foundation, the weathered rocks are not favoured, as the cut-off works by cement grouting would not be easy in this zone. It is good for foundation of outer shell zones of fill dams and for foundation of low fill dams.

4.3.4 Physical Properties and Strength of Bedrock

The results of laboratory rock tests are shown in Table-E.3.17, which is self-explanatory about the physical properties and strengths of rocks in solid test pieces without apparent cracks or fissures.

Strength of the bedrocks as a mass was measured by means of the in-situ plate loading test and the block shear test in the test adits TA-1 and TA-2. As discussed in the section 3.7, moduli of elasticity (Es), moduli of deformation (D), cohesions (τ_o) and angles of internal friction (ϕ) were determined as tabulated below.

Rock	Modulus of Elasticity Es (kg/cm^2)	Modulus of Deformation D (kg/cm^2)	Cohesion τ_o (kg/cm^2)	Angle of Int. Frict. ϕ (degree)
(1) Massive pebbly sandstone	1.6×10^4	1.1×10^4	9.0	55
(2) Laminated medium sandstone	2.5×10^4	1.1×10^4	9.0	50
(3) Massive medium sandstone	1.1×10^4	7.4×10^3	8.0	40
(4) Mudstone and weathered rock	--- Not tested	---	6.5	37

The bedrock on the dam axis A consists of eight strata, Ms-5, At-5, Ms-6, At-6, Ms-7, At-7 and Ms-8 Member of Bst Formation, arranged from the right bank to the left bank in order, and the massive Sst Formation on the most left side as shown in Fig.-E.2.

It appears that the strength of Sst Formation can be represented by the strength determined on the pebbly sandstone tested. Hence,

Shear strength $\tau = 9.0 + \sigma \cdot \tan 55°$ (kg/cm^2) for Sst Formation.

where, denotes the vertical stress in kg/cm^2.

The rocks in Ms-5, Ms-6, Ms-7 and Ms-8 Members can be assumed to be equivalent to the tested medium sandstone or laminated sandstone. Accordingly,

$$\text{Shear strength} \quad \tau = 9.0 + \sigma \cdot \tan 50° \quad (kg/cm^2)$$
$$\text{or} \quad \tau = 8.0 + \sigma \cdot \tan 40° \quad (kg/cm^2)$$

for the Ms Members of Bst Formation.

The members At-5, At-6 and At-7 comprise various kinds of rocks, such as calcareous breccias, sandstones, siltstones, mudstones and thin intermediates. The At Members are very often characterized by frequent beddings and laminations along which shear planes may easily develop, while they comprise fairly large portions of hard calcareous rocks of which compressive strengths are 1.3 to 5 times higher than the other rocks according to the laboratory tests. It is resulted from this condition that the resistance against shear in the At Members depends largely on the shearing strength in bedding planes or weakest strata such as mudstones. As for shearing along bedding planes, the situation is deemed similar to that of the laminated sandstone, of which test gave 9.0 kg/cm^2 of cohesion and 50° internal friction angle. Test in mudstone resulted in 6.5 kg/cm^2 of cohesion and 37° of internal friction angle, which could be some lower values than actual, considering the shearing plane in the test developed partly out of rock onto the bottom of the concrete test block which appeared to have been bound insufficiently to the underlying rock.

Accordingly, it is deemed that the shear strength of the At Member varies in the range between:-

$$\text{Shear strength} \quad \tau = 9.0 + \sigma \cdot \tan 50° \quad (kg/cm^2)$$
$$\text{and} \quad \tau = 6.5 + \sigma \cdot \tan 37° \quad (kg/cm^2)$$

From the above results, it will be appropriate to take 8.0 kg/cm^2 for cohesion (τ_0) and 40° for angle of internal friction (ϕ) for the evaluation of safety against sliding of concrete gravity dam.

The result is applicable to the alternative damsite-B and -C as well, because those damsites are situated on the same formations.

4.3.5 Permeability and Grouting Effectiveness of Bedrock

Lugeon tests (water pressure tests) for rock masses were done in 112 sections of 17 holes of which test results are shown in the Table-E.3.9. As discussed in the section 3.4, the bedrock of the damsite is generally massive and low in permeability as is indicated by the fact that 67% of the section tested shows less than 4 Lugeon units of leakage. Scattered sections of more than 10 Lugeon units cover only 17% of the tested zone.

The test grouting performed on the damsite B left bank, shows that the cement grouting in the holes arranged at two meter is able to reduce the permeability of the bedrock effectively and obtain the sufficient water-tightness of less than 4 Lugeon units. The average grout take of the whole grouting was 25.4 kg of cement in weight per borehole linear meter.

V. FUTURE GEOLOGICAL INVESTIGATION

For the detail design phase, the following geological investigation is considered necessary.

5.1 Damsite

Since the depth of riverbed gravel shows noticeable local variance ranging from 20 m to more than 36 m in the middle parts of the river channel, more accurate confirmation is deemed necessary for the detail design in the damsite selected. The surface of bedrock overlain by terrace deposits also seems to be irregular and its more accurate shape is required to be surveyed. The foundation of main appurtenant structures, such as the high retaining walls, spillways, intake structures, powerhouse, etc., should also be surveyed to reveal the depth and to assess the strength of bedrock. For the above, core drilling with determination of permeability on the riverbed, terraces and foundation of the main structures is necessary.

For performing the core drilling, it must be taken into consideration that the drilling of deep gravel bed is considerably difficult and time consuming work and that the workable period for drilling in the riverbed is limited within four months of the low river flow season, i.e. from December to the early April according to the experience in the investigation for the feasibility study. Therefore the core drilling of the detail design stage will need two dry seasons, unless several drill rigs are deployed at once.

The confirmation of the shearing strength and bearing capacity of a rock mass at the selected damsite will be required for the stability analysis of gravity type concrete structures, particularly for the spillway weir which is planned as high as about 60 m with the width of 126 m. Therefore, test adits on the selected damsite are necessary to be excavated on purpose to perform the in-situ rock tests (plate loading test and block shear test) as well as to inspect the bedrock.

It is recommendable to perform the laboratory test on rock samples, which will give supplemental data to the in-situ rock strength determination.

Test trenches and test pits are useful for examination of rock condition and distribution as a supplemental means in addition to outcrops and drilled cores.

Test grouting is also recommended to obtain more detailed data for the design of foundation grouting, although the test grouting carried out in the feasibility study has shown that the grouting is effectively done and the grout take is not so much.

The geological investigation items considered necessary are as listed in the following. Proposed locations of core drilling holes and test adits are shown in Fig.-E.48 "FUTURE INVESTIGATION PLAN".

Investigation Item	Quantity
1. Core drilling	20 holes 1,000 linear meters in total
2. Permeability test in the boreholes	20 holes 180 stages in total
3. Test adit excavation	2 adits 100 linear meter in total
4. In-situ rock test	
Plate loading test	4 points
Block shear test	16 blocks
5. Test trench excavation	about 200 linear meters in total
6. Laboratory rock test	40 pcs.
7. Test grouting	one site 10 holes 500 linear meters in total

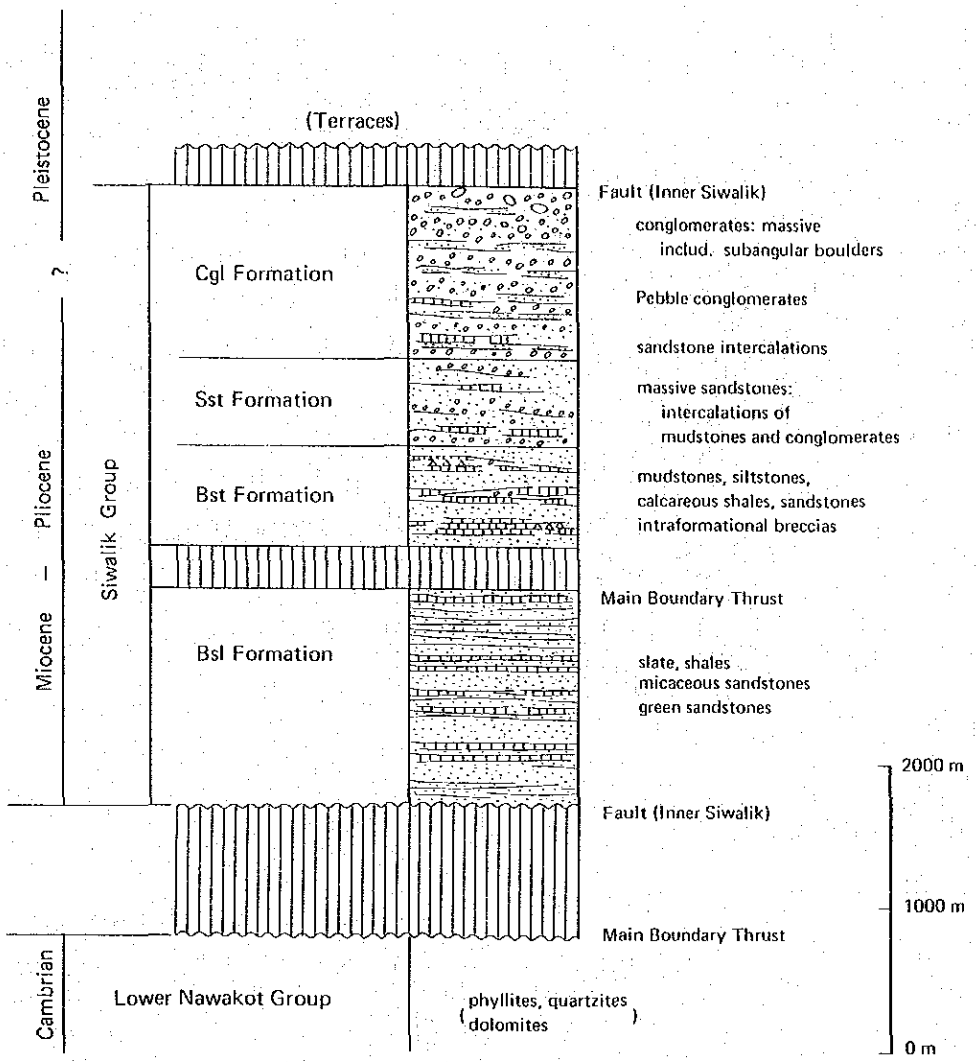
5.2 Other Sites

(1) Field reconnaissance is necessary in and along the quarry site, relocation road and transmission line.

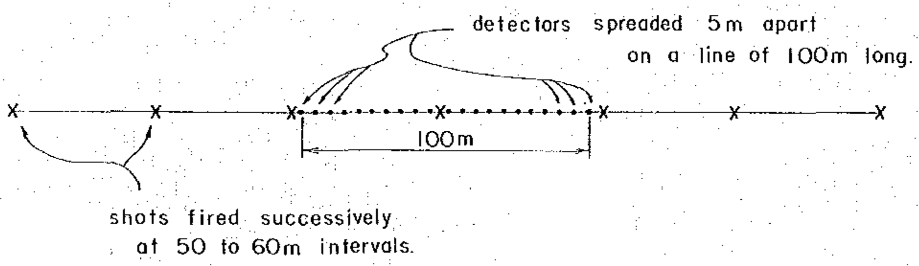
(2) Geological mapping is necessary in and along the quarry site, relocation road and transmission tower points.

(3) Core drilling and/or the Swedish weight sounding (or the Dutch double-tube cone penetration test) must be done where it is considered necessary in the field reconnaissance.

ILLUSTRATIONS



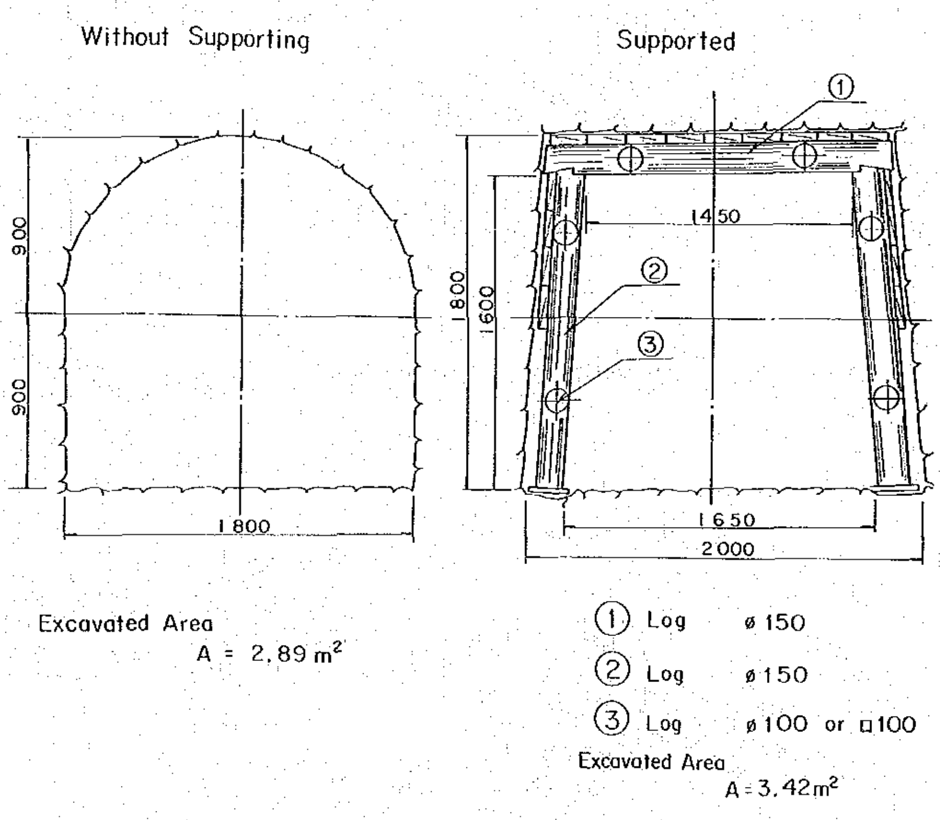
Illus.-2.1: STRATIGRAPHY OF THE RESERVOIR AREA GEOLOGY



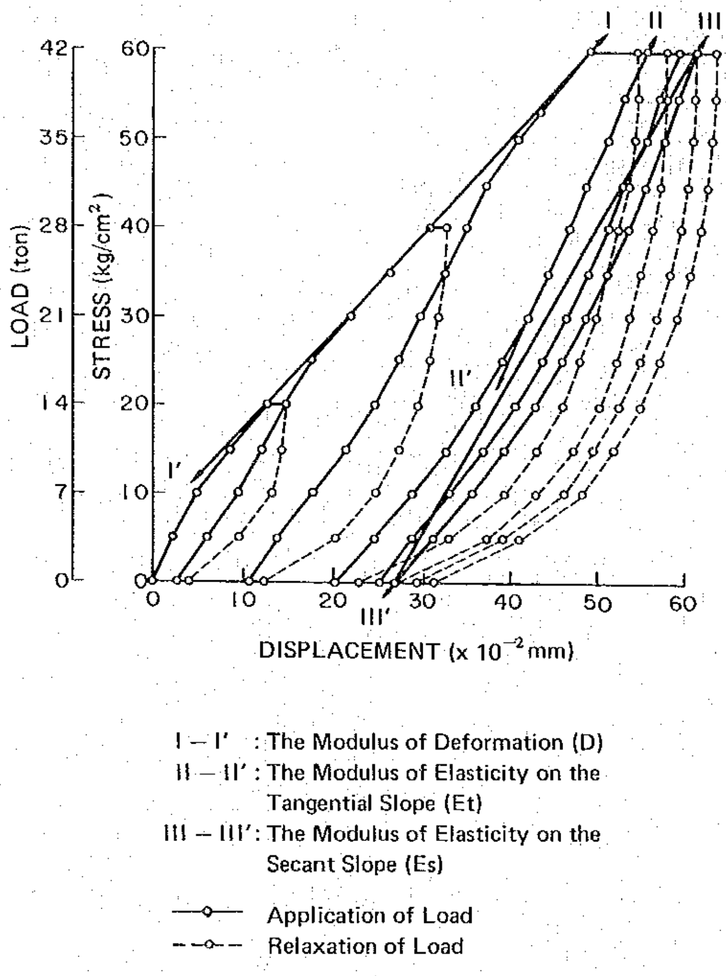
Illus.-3.1: TYPICAL REFRACTION PROFILE

DEPTH (m)	STAGE NO.	GROUTING PRESSURE (kg/cm^2)	WATER PRESSURE TEST PRESSURE (kg/cm^2)
0 -			
5 -			
10 -	1st Stage	1.25	1 → 1.75 → 1
15 -	2nd Stage	2.5	1 → 2.5 → 1
20 -	3rd Stage	3.75	1 → 2.5 → 3.75 → 2.5 → 1
25 -	4th Stage	5.0	1 → 3 → 5 → 3 → 1
30 -	5th Stage	6.25	1 → 3.5 → 6.5 → 3.5 → 1

Illus.-3.2: PRESSURE OF GROUTING AND WATER PRESSURE TEST



Illus.-3.3: SECTION OF TEST ADIT



Illus.-3.4: ILLUSTRATION OF LOAD-DISPLACEMENT CURVE