DIRECTORATE GENERAL OF
WATER RESOURCES DEVELOPMENT,
MINISTRY OF PUBLIC WORKS

JAPAN INTERNATIONAL
COOPERATION AGENCY
(JICA)

THE FEASIBILITY STUDY
ON
THE GILIRANG IRRIGATION PROJECT
IN
THE REPUBLIC OF INDONESIA

FINAL REPORT

ANNEX VOLUME I

Annex 1 Geology and Soil Mechanics
Annex 2 Meteorology, Hydrology and Water Balance Study
Annex 3 Dam and Weir
Annex 4 Irrigation and Drainage

JUNE 1995

NIPPON KOEI CO., LTD.

DIRECTORATE GENERAL OF
WATER RESOURCES DEVELOPMENT,
MINISTRY OF PUBLIC WORKS

JAPAN INTERNATIONAL
COOPERATION AGENCY
(JICA)

THE FEASIBILITY STUDY ON THE GILIRANG IRRIGATION PROJECT IN THE REPUBLIC OF INDONESIA

FINAL REPORT

ANNEX VOLUME I

Annex 1	**Geology and Soil Mechanics**
Annex 2	**Meteorology, Hydrology and Water Balance Study**
Annex 3	**Dam and Weir**
Annex 4	**Irrigation and Drainage**

JUNE 1995

NIPPON KOEI CO., LTD.

ANNEX 1

GEOLOGY AND SOIL MECHANICS

ANNEX 1 GEOLOGY AND SOIL MECHANICS

Table of Contents

List of Table

List of Figure

ANNEX 1 GEOLOGY AND SOIL MECHANICS

1. GEOLOGY

1.1 Introduction

The objective of this geological investigation was to provide geotechnical information required for the Feasibility Study of the Project.

Reference was made to the following two (2) reports:

a) Master Plan for the Central South Sulawesi Water Resource Development, March 1980 by Japan International Cooperation Agency (JICA).

b) Geological Investigation by Directorate Penyelidikan Masalah Air (DPMA) in 1981.

The major activities implemented for the geological investigation were as follows:

1) Data collection

In advance of field surveys, the Team collected the above mentioned two reports.

2) Field reconnaissance

Field conditions at the proposed dam site (two alternative dam axes identified: Alternative-1 and Alternative-2), proposed weir sites and main structure sites of irrigation structures were investigated through field reconnaissances. The alternative-1 dam axis was selected by the master plan of 1980 and the alternative-2 dam axis was newly selected by this Feasibility Study.

3) Survey on subcontract basis

The Team sublet the following geological investigations to a local contractor. The local contractor smoothly executed all the required investigations on schedule under the supervision of the Geologist of the Team and completed all the works by the end of the Field Works of the First Stage Study.

a) Core Drilling; Core drilling of 340 m in total in parallel with standard penetration tests and Lugeon tests were made at two alternative dam axis (seven (7) holes, 260 m in total), the narrow ridge site of the right bank of reservoir (one (1) hole, 25 m in total), and the quarry site (two (2) holes, 55 m) respectively, as given in Table A.1.1 and Figure A.1.1.

b) Geophysical Exploration; As shown in Table A.1.1 and Figure A.1.1, seismic prospecting of 1,000 m in total was carried out along two alternative dam axes; 600 m along the axis of Alternative-1 and 400 m along that of Alternative-2 respectively.

c) Test Pitting; Four (4) test pits were provided as given in Table A.1.1 and Figure A.1.1 to observe the subsurface geological conditions and collect soil samples for laboratory tests; two (2) pits of 1.80 m x 1.80 m x 2.50 m deep at the upstream of the right bank and two(2) holes of 1.80 m x 1.80 m x 2.00 to 3.00 m deep at the quarry site. All the sampling materials obtained at the pits were sent to a laboratory for soil mechanical tests.

This report also dealt with the geotechnical investigations under the items a) and b) mentioned above.

1.2 Regional Geology

The project area is located on both sides of the lower Gilirang river basin. The Gilirang river originates from the mountainous zone between Bila and Awo river basins. The river meanders from the north to the south near the junction of the provincial road and turns its flow course to the east, and then flows directly into the Bay of Bone.

The regional geological map covering whole Kabupaten Wajo and its surrounding area is illustrated in Figure A.1.2 which was prepared on the basis of the regional geological map for Majene and western part of Palopo, Sulawesi Selatan compiled by Juri and Sudjiotmiko, 1974.

The upper Gilirang river basin is predominantly underlain by a conglomerate layer of about 100 m to 400 m thick, intercalated by glauconitic sandstone layers and mudstone layers. The layers bear coquina, molluscs and foraminifers which indicate the geological age ranging from "late middle of Miocene to Pliocene." Meanwhile, the lower Gilirang river basin is mostly underlain by Quaternary Alluvium deposits about 100 m thick, consisting of clay, silt, sand, gravel, and relief limestone.

The proposed dam site is underlain by an alternation of the Miocene-Pliocene sedimentary layers predominantly consisting of conglomerate layers intercalated by sandstone land mudstone layers. The Miocene-Pliocene layers strike N60°E-N55°W and dip 5°-25°W and are covered with the Quaternary deposits consisting of un-consolidated silt, sand, and gravels.

1.3 Geology of Proposed Dam Site

The proposed dam site where the two alternative dam axes, Alternative-1 and Alternative-2, have been identified, is located on the middle reach of the Gilirang river . The geological map of the proposed dam site is shown in Figure A.1.3.

1.3.1 General Geology of Proposed Dam Site

(1) Topography

Low hills with gentle slopes are observed in the reservoir area. Topography of the dam site is characterized by fine erosion of the soft bedrock which forms small gullies and thin ridges with general orientation trend of northwest to southeast. The topographic trends appears to accord roughly with strikes of the bed rock. The proposed dam axis has also the similar orientation connecting a couple of thin ridges on both sides of the Gilirang river. The ridge shows Cuesta topography on the left bank that shows steep slope at upstream and gentle slope at downstream. Terrace and flood plains are widely observed at the valley.

(2) Geology

The dam site is underlain by semi-consolidated sedimentary rocks of a geological age ranging from middle of Miocene to Pliocene, the sedimentary semi-consolidated rock is composed of an alteration of conglomerate layers, sandstone layers and mudstone layers and is covered with Quaternary deposits such as terraces deposit, talus and river bed deposit at locations Figure A.1.4 and A.1.5).

The mudstone is bluish to dark gray colored, well-consolidated. The rock is however cracked and susceptible to quick slaking where rock is decomposed under repeated dry and wet conditions.

The sandstone is medium - coarse grained, gray and massive; softer and less-consolidated than the mudstone; and is susceptible to slaking though not quicker than the mudstone.

The conglomerate is massive; composed of medium-coarse grained sand matrix, with plenty of cobbles and pebbles of diameter less than 20 cm. As the pebbles and cobbles consist of harder andesite, sandstone, limestone and etc. of the age before Miocene, the conglomerate is more resistible against erosion than the mudstone and the sandstone.

The Quaternary deposit is brown to gray colored, mainly composed of un-consolidated sand, gravels, clay and silt. At the dam site its width is about 40 m and thickness ranges from 0 to 5 m. N-value is more than ten (10) at silt layers and sandy clay layers.

(3) Geological Structure

The Tertiary sedimentary layers (the conglomerate, sandstone and mudstone layers) gently dip 5° to 20°W, strike N60°E - N80°W. As the orientation of the upstream dam axis is similar to the strike of the bed rock, the geological structure on a cross section along the axis looks apparently horizontal.

(4) Faults

No major fault was identified around the dam site. Although two normal faults of 1 - 20 cm thick, 5 - 60 cm displacement, were identified at a tributary and in the reservoir respectively; the faults will not adversely affect on this Project feasibility because the faults are located at remote areas more than 900 m from the dam site.

(5) Weathering

A superficial portion of the bedrock is weathered which made the rock decomposed and discolored to brown-reddish brown. Ridge areas are more deeply weathered than the portion of slopes and close to the riverbed. At the Dam site, the ridges are weathered to 10 m at the maximum. As weathering intensity is largely dependent upon rock type; a ridge underlain by the sandstone which is pervious and susceptible to weathering, has been weathered to a deeper portion.

(6) Others

Calcite of less than 5 cm thick is observed on outcrops in a tributary. This calcite has been formed by precipitation of dissolved calcium in the river water under a certain condition combining various conditions such as river flow, water temperature, exposure to air, and etc.

1.3.2 Geotechnical Conditions

(1) Rock quality of the foundation

1) General

The foundation rock consists of semi-consolidated sedimentary soft rock which an axial compression strength is estimated to be less than 200 kgf/cm^2 for fresh rock. The strength varies depending on rock types such that the sandstone inherits the least strength, the conglomerate does generally highest strength due to the inclusion of harder pebbles and cobbles although the strength depends on the strength of the matrix.

A superficial part of the bedrock is severely weathered which resulted in softening of the rock and opening cracks. The brown colored, severely weathered zone shows N-value ranging from 34 to more than 50, indicating heterogeneous geological conditions. On the other hand, gray-dark gray colored fresh rock is homogeneous showing N-Value more than 50 (refer to Table A.1.2).

2) Seismic refraction survey

Seismic refraction survey was carried out on two lines, i.e. A-line for the downstream axis (Alternative-2) and B-line for the upstream axis (Alternative-2), and the results are shown in Figure A.1.4 and Figure A.1.5 respectively.

i) The foundation was divided into five (5) layers in terms of seismic velocity (the Primary wave) as follows:

The first velocity layer	:	Vp= 0.2 - 0.3 km/sec
The second velocity layer	:	Vp= 0.5 - 0.7 km/sec
The third velocity layer	:	Vp= 0.8 - 0.9 km/sec
The fourth velocity layer	:	Vp= 1.3 - 1.7 km/sec
The fifth velocity layer	:	Vp= 2.5 - 2.7 km/sec

The first and second layer roughly correspond to the severely weathered rock where seismic velocity is small due to the heterogeneous rock conditions with decomposition and open cracks. The fourth velocity layer corresponds to the fresh rock foundation. However, the seismic velocity even in the fresh rock zone is not great, reflecting the semi-consolidated rock conditions.

ii) The seismic velocities on each alternative axis are characterized as follows:

- On A-line (the downstream axis ,Alternative-2), the depth of the fifth velocity layer is deeper in the river bed.
- On B-line (the upstream axis, Alternative-1), the depth of the fifth velocity layer is deep in the left bank.

However, the rock conditions of the fourth and fifth velocity layer is similar according to an investigation hole JD-1 located on the left bank of the upstream axis. Therefore, the implication of this result will not be significant.

From the results of the investigations, the bearing capacity of the fresh rock is considered to be sufficient for a fill type dam of 40 m high once the severely weathered rock is removed at the impervious core foundation. However, an additional investigation is recommended to obtain further information on rock strength of each rock type, deformation factor of the foundation and etc. for the detailed design.

(2) Permeability

The results of the Lugeon tests carried out in this study are summarized as follows (Table A.1.3, Figure A.1.6 and A.1.7).

- Although Lugeon-value shows more than 10 -20 in places in the superficial area, the deeper the depth is, the smaller the Lugeon value becomes to less than 5. This is because the permeability of this site largely depends on the weathering profile which proceeded from the surface.

- In the downstream axis, the Lugeon values are less than 5 below 20m deep, whereas Lugeon value is more than 20 in places even below 35 m deep.

- Critical pressure of the foundation is 5 - 7 kgf/sec in places even in fresh rock zone.

From the results of the Lugeon tests, it is considered that the foundation can be treated with the ordinary cement grouting once the severely weathered rock zone has been removed. However, attention has to be drawn to a grouting plan so as not to damage the foundation because the critical pressures of the foundation are rather low in places.

In addition, detail information on the Lugeon testing carried out in the upstream axis are not available in the existing report. It might be probable that the Lugeon values were determined without consideration of the Critical Pressures. In this case, the Lugeon values might have been taken as larger values. Therefore, it is recommended that a confirmation investigation should be carried out along the upstream dam axis for the Detail Design of the curtain grouting.

(3) Ground water

The ground water table line rises roughly conforming with the slope gradient on both side as shown in Figure A.1.4 to A.1.5 and Table A1.4. However the groundwater level is lower than the normal high water level of reservoir at the both abutments. Although the condition is considered treatable by rim grouting, the groundwater level has to be traced for the Detail Design because the rim grouting should generally be reached the area where the ground water level is higher than the normal high water level.

1.3.3 Quarry Site and Other Material

(1) Quarry site

An area on the left bank of Gilirang river, approximately 500 meters north west of the dam site is proposed as a source area of embankment materials for the transition zone. This area is situated on a hill of 25 to 70 meters in elevation, and composed of Miocene-Pliocene sedimentary rocks. The rock is poorly to moderately cemented soft (semi-consolidated) rock, of which a superficial 5 to 6.7 meter zone is highly to moderately weathered and the zone below this is slightly weathered (refer to Figure A.1.8).

The slightly weathered rock zone can be a source of the materials for the transition zone which will be the major part of the dam embankment, and partially can be a source of the materials for the rock zone. It is however not suitable for the rip-rap and concrete aggregates that can be supplied from the riverbed or an other project site.

(2) Other material

The random materials could be taken from the thin ridge on the left bank and the hills on the right bank, underlain by the Tertiary sedimentary rock which predominates conglomerate. Although rock quality varies among the available rocks, it is considered that the sedimentary rocks can be utilized as the random material.

However, as physical property will vary depending upon mix proportion of the sedimentary rocks, it is recommended to carry out a further investigation for the Detail Design.

1.4 Geology of Proposed Weir Sites

Two alternative sites were proposed for the weir site. Both alternatives sites are located in neighborhood about 11 km downstream of the proposed dam site. These sites are underlain by Miocene-Pliocene semi-consolidated sedimentary rocks covered with Quaternary unconsolidated deposits.

The Miocene-Pliocene sedimentary rocks consist of alternating beds of sandstone, conglomerate layers intercalated by mudstone layers. Those are mostly grayish colored, massive and soft rocks. The bedrock are generally covered with surface soil, thin talus deposit, terrace deposit and/or river bed deposit (refer to Figure A.1.9 and A.1.10).

1.4.1 Geology of Alternative-1 Weir site (Upstream alternative)

The weir is planned to be constructed by Coupure method on Alluvium plan which is underlain by thick terrace and river bed deposits, consisting of un-consolidated fine-grained sediments such as silty clay, silt and fine-grained sand.

As no outcrop of the bedrock is observed in this weir site, a test pit was provided and Dutch cone penetration test was carried out to clarify the foundation conditions (Table A.1.5).

N-values converted from the Dutch cone tests increase according to the tested depth: i.e.: less than 10 by 4 m to 6.6 m deep, less than 20 by 7 m to 9.6 m deep and more than 30 by 8.8 m to 10 m deep.

The results of the penetration tests indicate that the thickness of the terrace and river deposits is about 9 m and the elevation of the bedrock surface is estimated to be EL. 9.0 m. As the bedrock is located on a deeper level, a floating type concrete weir would be suitable for this site.

1.4.2 Geology of Alternative-2 Weir Site (Downstream alternative)

The weir site is located about 200 m downstream of the Alternative-1 weir site. The weir is planned to be constructed directly on the present river course. The outcrops of the bedrock are observed at the riverbed and the right abutment, and the bearing capacity of this bedrock is estimated to be more than 100 kgf/cm^2. Hence a fixed type concrete gravity weir can be constructed on this foundation.

1.4.3 Geology at Closure Dike

There is no outcrop of bedrock at the closure dike sites of the both alternatives. As the dike is mostly placed on the terrace portion of the banks, a core trench cut might be required in the terrace deposit down to an required depth.

1.5 Narrow Ridges on Right Bank of Reservoir

A line of narrow ridge is located at a right bank area about 2 km from the dam site. The surface elevation of narrow ridge is less than EL. 60 m with the lowest point of EL. 56.7 m, and about 1 km long (refer to Figure A.1.2). From the results of an investigation of drilling hole carried out for this study, the rock is composed of an alteration of the conglomerate, sandstone and mudstone; severely weathered to 9 m deep; without surface overburden: the groundwater level was 3.52 m deep i.e. EL. 53.1 m (refer to Figure A.1.11).

Although a significant leakage could not take place from this narrow ridge zone, it is advisable to further investigate the geological conditions for the Detail Design.

1.6 Geology along Proposed Canal Routes

Two main irrigation canals are designed; one about 26.5 km long on the right bank and the other about 21.0 km on the left bank of the Gilirang river. The areas are underlain by semi-consolidated Tertiary sedimentary rocks and un-consolidated Quaternary deposits (refer to Figure A.1.12).

On the right bank, a section about 1.5 km from the intake is underlined by Tertiary semi-consolidated sedimentary rock, and the whole area of the remaining sections of the canal be underlain by the Quaternary un-consolidated deposit.

On the left bank, two sections about 2.0 km from the intake weir site and about 6.0 km from the middle portion of the canal are underlain by the Tertiary semi-consolidated sedimentary rock and the remaining sections are underlain by the Quaternary deposit.

1.7 Recommendations for Design and Construction

1.7.1 Excavation Line of Dam Foundation

(1) Core trench excavation

The impervious earth core zone of the main dam should generally be placed on a sufficiently stable bedrock foundation of fresh to slightly weathered rock and may be placed on a moderately weathered rock foundation at the dam wings. The rock zone of permeability over 20 Lugeon should be removed because such permeability is generally difficult to improve.

Taking into consideration the above, the 10 m or less of the overburden and unsuitable rock should be removed for the core trench on the both abutments; and 16 m at the valley bed.

(2) Excavation for shell zone

The river alluvial deposit and flood terrace deposits are not reliable for foundation of the shell zones in terms of bearing capacity and potential settlement. It is therefore recommended to remove all the river alluvial deposit as practically as possible for the entire dam foundation.

1.7.2 Foundation Treatment

(1) Curtain grouting

To reduce the seepage, foundation treatment will be required. Two possible alternatives would be the countermeasure against the seepage for this dam site; one is blanketing in the reservoir side, and the other is grouting. However the upstream blanket is not considered to be suitable due to a rather steep slope in the upstream side of the thin ridge on the left bank. Hence, seepage water through the dam foundation should be cut off by grouting. Ordinary cement grouting is considered to be preferable.

As the critical pressures of bedrock (Pc) are rather low in places. The pressure of the grouting, therefore, should be carefully planned and the grouting should also be carefully performed so as not o damage the foundation.

Grouting may not be effective and essential in a low permeable foundation. Nevertheless, it is recommendable to extend the curtain grouting to a depth of low permeability zone, because grout holes on lines can detect every isolated seepage path in the bedrock, and seal it effectively. Hence, depths of curtain grouting holes to be recommended is 15 m to 20 m from the impervious core foundation. Hole layout for the curtain grouting, suitable to this site, is 1 to 1.5 m intervals on two parallel lines laid out along the dam axis at a distance of 1 or 2 m from each other.

In addition, a confirmation investigation is advisable for the upstream axis because the Lugeon-values obtained along this axis did not confirm to the data obtained in the downstream axis.

(2) Blanket Grouting

Blanket grouting should be made for the entire area of the impervious core foundation, with grouting holes spaced at 2 m to 3 m intervals, on parallel lines of 1 m to 2 m intervals and grouting depth of 5 m.

The above recommendations for cement grouting should be reviewed by grouting tests at the site for the detail design. Cored pilot holes and check holes should be provided for the grouting tests to investigate rock conditions and grouting results respectively.

Though no major fault was identified through the present geological investigation, faults might be encountered in the foundation during the construction. However, such faults would be small, since the foundation at the site is semi-consolidated soft rock. Replacement with concrete or cement grouting will be effective for treatment of such faults.

(3) Rim Grouting

Rim grouting will be necessary for water tightness for both right and left bank. The area to be grouted is tentatively estimated as 710 m from the Dam for the right bank and 200 m for the left bank. It is recommended to carry out an additional investigation to trace the groundwater level in both banks.

1.7.3 Spillway

The bedrocks at the right bank site are fresh rock, varying from semi-consolidated to rather soft (Figure A.1.13). It is considered their bearing capacity and shear strength are enough for the proposed spillway structure. However, consolidation grouting shall be required for improvement of the foundation which might have been damaged by excavation works.

1.7.4 Diversion Tunnel

A couple of diversion tunnel will be constructed across the thin ridge of the left bank. The length of each tunnel will be around 300 m. Conglomerate-dominant layers intercalated by sand and mudstone layers will be encountered in the tunnels. Careful attention should be drawn to the excavation works due to slaking, particularly in mudstone zones. It is recommended that the tunnels should be lined or covered immediately after each excavation so that exposed rock should not be weathered/slaked.

In the upstream side along the tunnels, the overburden might not be sufficiently thick. Supporting measures should be placed immediately after each excavation to avoid lessening of the excavated section.

The intake structure should be constructed on the foundation of moderately to slightly weathered rock to obtain a required bearing capacity. The protection measures after construction of the structure should be provided against erosion of the excavated and loosened rock ; shotcreting might be suitable as the protection measures.

1.7.5 Excavation Works

(1) Bedrock

Since the bedrock at the dam site are Miocene-Pliocene semi-consolidated rock, the bedrock can be excavated with the slope of approximately 1: 1.0. The left bank slope of the dam axis should be carefully excavated, because the rock layers dip south with 5 to 25 degrees.

The bedrock at the site are semi-consolidated and susceptible to weathering/slaking, which will result in severe erosion. Accordingly shotcrete or re-planting method will be required for protection of excavated slope, especially for mudstone layers.

(2) Other layers

Un-consolidated Quaternary deposits such as terrace, talus and residual deposit are mostly composed of un-consolidated sand and silt. The slopes to be excavated in such un-consolidated materials should therefore be between 1 : 1.0 and 1 : 1.5 and a height of slope be limited within 5 m. The excavation works in the un-consolidated deposits should be carefully carried out, because the deposits are more than 10 m thick under wet conditions.

The open channels upstream and downstream of the diversion tunnels and the beginning point of the canals will be constructed in the loose deposits. Protection works for the slops and invert against erosion will be required.

1.7.6 Reservoir Area

The reservoir rim is composed of soft rocks of weathered Upper Tertiary sedimentary rock. The slopes are generally of low angle covered with grass and only partially by woods. No sign of land-sliding of large scale is observed (Figure A1.14). Although possibility of sliding on reservoir impounding can not be cleared, it seems improbable that those sliding would be of such rapid movement as to cause any substantial threat on safety of the dam and reservoir.

No specific seepage passage are conceivable through bedrock from the reservoir, although a further investigation is advisable for the narrow ridge on the right bank.

The thin ridge on the left bank will be practically water-tight once the rim grouting has been placed.

2. SOIL MECHANICS

2.1 General

The soil mechanical investigations were carried out in order to clarify the soil mechanical characteristic of materials to be used for the construction of dam, canal and road embankment and other facilities, such as rock materials, soil materials and filter materials.

Field investigation and laboratory test were carried out to clarify the soil mechanical characteristics of construction materials.

The first stage study were mainly carry out by laboratory test. The second stage study was mainly carry out by field survey for surface geology, distribution and properties of construction materials.

2.2 Materials for Dam Construction

2.2.1 Field Investigation

The field survey for fill materials of the dam was conducted at the following locations:

1) Rock materials : Conglomerate and sand stone in/around the proposed dam site
2) Soil materials : Highly weathered rock at surface of the ridge and terrace deposits
3) Filter materials : River bed deposits (sand and gravel)

Four (4) test pit borings were carried out and then samples were taken from those test pits and outcrops for physical and mechanical tests in laboratory. Location and quantities of the field works are given in Figure A.1.1 and Table A.1.6 respectively. Schmidt hammer test was carried out in addition to the field reconnaissance survey. The result of the Schmidt hammer test is shown in Table A.1.7.

Through the field survey, conditions of fill materials available in/around the proposed dam site are considered as follows:

(1) Distribution and properties of construction materials

1) Rock in/around the dam site is mainly composed of conglomerate and sand stone of geological age ranging from middle of Miocene to Pliocene.

2) Mudstone exists at river bed about 2 km upstream of the dam site.

3) Fresh rock of conglomerate and sand stone can be used as pervious material, because 100 kgf/cm^2 to 300 kgf/cm^2 of compressive strength has been obtained by Schmidt hammer test. Also, it is considered that the fresh rock of conglomerate and sand stone has medium durability judging from condition of the outcrop.

4) Compressive strength of mudstone is inferred about 100 kgf/cm^2 in fresh condition. However, it is very poor durability and easily turns into soil at outcrop in 10 days.

5) Right ridge at the dam site is configured by alternation of conglomerate and sandstone, and mudstone widely exists at surface of north to west slope with thickness of about 5 m. Since the mudstone is highly weathered at the surface layer with about 2 m depth, it can be used for impervious materials. Also the mudstone at the surface layer at about 2 km upstream of the proposed dam site can be used for impervious materials.

6) Left ridge at the dam site is also configured by alternation of conglomerate and sand stone. Highly weathered rock exists at the surface. However, those are still in coarse condition and unsuitable for impervious materials.

7) Terrace deposits are distributed at the left bank side about 0.5 to 1 km upstream of the dam site, consisting of gravelly clay. It can be used for impervious materials, but quantity of deposits is rather small.

8) Sand and gravel are distributed at river bed, having best quality as filter materials at dam site, however, the volume of deposits is very small.

(2) Estimate of available volume of construction materials

Available volume of construction materials are estimated as follows:

1) Borrow area

Borrow areas are picked out from north to west slope of right ridge at the proposed dam site and right bank side at about 2 km upstream of the dam site, as shown in Figure A.1.15 and A.1.16. Surface layer which is shallower than 2 m can be used for impervious materials (refer to Figure A.1.17).

Available volume are estimated as follows:

Right ridge of the dam site	:	60,000 m^3	(available area x 2.0 m)
Right bank side at upstream	:	360,000 m^3	(available area x 2.0 m)

2) Quarry site

Quarry site are picked out from right and left ridges at upstream of the dam site as shown in Figure A.1.15.

Available volume are estimated as follows from topographic map of 1/5,000 scale.

Right ridge	:	3,700,000 m^3
Left ridge	:	1,200,000 m^3

3) Sand and gravel

Available volume of sand and gravel from the river bed between the dam site and 10 km upstream are estimated at 40,000 m^3.

(3) Purchase materials

Purchased materials would be used for riprap and filter due to less availability of hard rock or sand and gravel around the dam site. According to the reconnaissance survey, the river bed deposits of Bila river can be picked out as the quarry site of filter material. Enough quality and quantity are expected in this area. Andesite of mountainous area about 50 km west of the dam site can be also used as the riprap material (refer to Figure A.1.18).

2.2.2 Laboratory Test

In order to clarify physical and mechanical properties of fill materials distributed in and around the dam site, laboratory tests were made for samples taken from the test pits and outcrops, comprising of seven (7) samples from four (4) test pits and four (4) samples from spots on the outcrops. Figure A.1.1 shows the location of those samplings.

The test samples including classification of materials are summarized below:

Test pit sample

Classification of Material	Sample No.	Pit No.	Depth (m)	Remarks
Soil Material	1	TP-1	0.5 - 1.0	Highly weathered mud stone
	2	TP-2	0.5 - 1.0	Highly weathered mud stone
	3	TP-2	1.5 - 2.0	Highly weathered mud stone
	4	TP-3	1.5 - 2.0	Sandy clay
	5	TP-3	2.5 - 3.0	Gravelly clay
	6	TP-4	1.0 - 1.5	Weathered conglomerate
Rock Material	7	TP-1	1.5 - 2.0	Fresh mudstone

Outcrop sample

Classification of Material	Sample No.	Spot	Remarks
Rock Material	8	Left bank side	Conglomerate
	9	Left bank side	Sand stone
Filter Material	10A	River bed	Sand
	10B	River bed	Sand and gravel

Test items and quantities conducted in laboratory are listed in Table A.1.8.

(1) Soil materials

The results of laboratory test for soil materials are shown in Table A.1.9, A.1.10 and Figure A.1.19.

It is noted that compaction energy (Ec) = 100 % is applied to the density of specimen for the mechanical tests as follows:

$$Ec = \frac{W \cdot H \cdot n \cdot L}{V}$$

where Ec : Compaction energy
(Ec of 100 % = 5.625 kg•cm/cm^3)
w: Weight of rammer
H: Fall height of rammer
n: Number of compaction per layer
L: Number of layer
V: Volume of mould

(2) Rock and filter materials

The results of laboratory test are shown in Table A.1.11, A.1.12 and Figure A.1.20 to A.1.21.

It is noted that one fifth of mould diameter (Ø 15 cm) is applied to maximum grain size for the mechanical test. Grain size curve of mechanical test is shown in Figure A.1.20. Compaction energy (Ec) = 100 % is also applied to the density of specimen.

2.2.3 Evaluation of Fill Dam Materials

According to the results of the field survey and laboratory tests, fill materials available in and around the proposed dam site are evaluated as follows:

(1) Highly weathered mudstone

Natural moisture content of highly weathered mudstone is considerably high at surface layer up to 1 m depth, because samplings have been made in the rainy season. However, that of deeper layer below about 2 m is less than D95-wet as shown below:

Depth (m)	Moisture Content (%)	Moisture Content of D95wet (%)
0.5 - 1.0	33 - 57 (Average 35)	24
1.0 - 1.5	17 - 27 (Average 22)	

The cone bearing capacity in condition of less than D95-wet is more than 20 kgf/cm^2. Good workability is expected for the construction of this material, because necessary cone bearing capacity for dump truck is more than 12 kgf/cm^2.

Design coefficient of permeability for impervious zone is less than 1 x 10^{-5} cm/sec, and that of laboratory test is about one ten of in-situ permeability test, in general. Since result of laboratory test shows a value of less than 1 x 10^{-6} cm/sec for this material, it can be used as impervious material.

(2) Highly weathered conglomerate

Highly weathered conglomerate can not be used for impervious material due to its coarse condition.

(3) Terrace deposits

Terrace deposits can be used as impervious material because they are fine-grained and impervious. However, the available volume is rather small in comparison with the requirement.

(4) Fresh mudstone

Fresh mud stone is weaker than sand stone or conglomerate, and so easily fractured. Therefore, this material is judged impervious to semi-pervious material. Mixed material with the highly weathered rock and the fresh rock can be used as impervious material.

(5) Conglomerate and sandstone

Result of laboratory permeability test shows a value of 1 x 10^{-4} cm/sec to 6 x 10^{-4} cm/sec. But, this value is considered less than that of in-situ permeability test because surface of compaction layer is easily fractured. Internal friction angle of 43 to 45 degree obtained by the test means that shear strength of these materials would be enough for rock material. However, they can not be used for riprap material due to its high slaking characteristics under repetition test (10 times) of dry and wet as follows:

a) Loss after percentage of conglomerate = 15 %

b) Loss after percentage of sand stone = 36 %

(6) Sand and gravel at river bed

Sand and gravel at river bed can be used as filter or concrete aggregate because of their enough durability. However, available volume is not sufficient.

2.3 Materials for Embankments of Canals and Other Structures

The embankment of canals and other structures would be executed by maximum use of in-site materials excavated in the section of canals and other structures. The materials available along the canal routes are mainly classified into following three (3) groups:

(1) River terrace material

This is a fine soil and mainly consists of clayey sand and clay, and partly a mixture with sand and gravel. It has average plasticity and medium dry strength. Its colour in moist condition is dark gray, and partly a mixture with dark brown. The soil includes gravels in small quantity.

(2) Alluvial material

This is a fine soil and mainly consists of silt and clay. This material is usually distributed over the alluvial flat plain in the project area. Most of paddy cultivation are extended over this layer. It has average plasticity, but high dry strength. Its colour in moist condition is dark gray or dark brown. Cracks due to drying shrinkage are usually observed in the deposit layer of this material. This soil partly has gravels in small quantity. The effective soil depth is more than 50 cm and the water table is comparatively high in soils of flat low land but it does not act as hazard in the area.

(3) Hilly soil material

Two kinds of soils derived from the geological stones cover the hilly area or higher terrace of old alluvium. One is the soil from conglomerate contains a plenty of gravel and is not suitable for agricultural purpose. Another hill soil on sand and mudstone are reclaimed as terrace field on the hilly slope and are used mainly for paddy culture. In accordance with the result of analysis, the soil texture of this muddy soil is fine as silty clay, the soil texture is almost the same as that of alluvial soil; sticky and plastic.

Judging from the above descriptions and field reconnaissance survey, the soil materials in good workability and stability for embankment of canals and structures would be available along the canal routes.

Table A.1.1 Item and Quantity of Geological Investigation

Hole No.	Depth (m)	Location	Commenced Date	Completed Date
Core Drilling				
JD-01	50.00	Upstream Damsite	18.Apr. 1994	26.Apr. 1994
JD-02	30.00	Quarry Site	06.Apr. 1994	10.Apr. 1994
JD-03	25.00	Quarry Site	04.Apr. 1994	11.Apr. 1994
JD-04	25.00	Auxiary Damsite	08.May.1994	13.May.1994
JD-05	50.00	Downstream Damsite	29.Apr. 1994	15.May.1994
JD-06	30.00	Downstream Damsite	05.May.1994	11.May.1994
JD-07	30.00	Downstream Damsite	29.Apr. 1994	10.May.1994
JD-08	50.00	Downstream Damsite	18.Apr. 1994	27.Apr. 1994
JD-09	20.00	Spillway	07.Apr. 1994	11.Apr. 1994
JD-10	30.00	Spillway	20.Apr. 1994	27.Apr. 1994
Previous core dlilling				
GB-1	30.00	Upstream Damsite	18.NOV.1981	21.NOV.1981
GB-2	40.00	Upstream Damsite	22.NOV.1981	28.NOV.1981
GB-3	60.00	Upstream Damsite	01.DEC.1981	07.DEC.1981
GB-4	35.00	Upstream Damsite	09.DEC.1981	11.DEC.1981
GB-5	35.00	Upstream Damsite	14.DEC.1981	17.DEC.1981
Test Pit				
TP-1	2.50	Borrow-Pit	26.Mar.1994	28.Mar.1994
TP-2	2.50	Borrow-Pit	26.Mar.1994	28.Mar.1994
TP-3	3.00	Quarry Site	29.Mar.1994	02.Apr.1994
TP-4	2.00	Quarry Site	29.Mar.1994	02.Apr.1994
Seismic Exploration Line				
A.Line	400.00	Downstream Damsite	20.Apr. 1994	07.May.1994
B.Line	600.00	Upstream Damsite	20.Apr. 1994	08.May.1994

Table A.1.2 Result of Standard Penetration Test

Hole No.	Depth (m)	N-Varue
JD-1	1.5-2.0	N=43
	4.0-4.3	more than 50
JD-2	1.0-1.5	N=25
	2.5-3.0	N=15
	4.0-4.5	more than 50
JD-3	1.0-1.5	N=11
	2.0-2.5	N=15
	4.0-4.5	N=16
	5.0-5.2	more than 50
JD-4	1.0-1.5	N=50
JD-5	1.0-1.5	N=40
	2.0-2.5	N=42
	3.0-3.1	more than 50
JD-6	1.0-1.5	N=43
	2.0-2.5	more than 50
JD-7	1.0-1.5	N=5
	2.0-2.5	N=5
	3.5-4.0	N=10
	5.0-5.2	more than 50
JD-8	1.0-1.5	N=44
	2.5-3.0	more than 50
JD-9	1.0-1.5	more than 50
	2.0-2.2	more than 50

Hole No.	Depth (m)	N-Varue
JD-10	0.5-1.0	N=12
	1.5-1.7	more than 50
GB-1	1.0-1.5	N=55
	2.0-2.5	more than 50
	3.0-3.5	more than 50
	4.0-4.5	N=34
	5.0-5.2	more than 50
GB-2	1.0-1.5	N=48
	2.0-2.1	more than 50
GB-3	1.0-1.5	N=28
	2.0-2.5	N=41
	3.0-3.1	more than 50
GB-4	1.0-1.5	N=14
	2.0-2.5	N=22
	3.0-3.5	N=49
	4.0-4.0	more than 50
GB-5	1.0-1.5	N=37
	2.0-2.5	more than 50

Table A.1.3 Result of Water Pressure Test (1/2)

Hole No.	Depth (m)	Section length (m)	Hole radius (mm)	Permeability coefficient	Lugeon unit
JD-1	0.5- 5.0	4.50	66	2.26E-05	1.70
	5.0-10.0	5.00	66	7.45E-05	5.60
	10.0-15.0	5.00	66	8.51E-05	6.40
	15.0-20.0	5.00	66	8.38E-05	6.30
	20.0-25.0	5.00	66	9.04E-05	6.80
	25.0-30.0	5.00	66	5.99E-05	4.50
	30.0-35.0	5.00	66	5.85E-05	4.40
	35.0-40.0	5.00	66	5.32E-05	4.00
	40.0-45.0	5.00	66	6.38E-05	4.80
	45.0-50.0	5.00	66	8.11E-05	6.10
JD-4	0.2- 5.0	4.80	66	1.21E-04	9.07
	5.0-10.0	5.00	66	1.60E-04	12.04
	10.0-15.0	5.00	66	6.37E-05	4.79
	15.0-20.0	5.00	66	4.08E-05	3.07
	20.0-25.0	5.00	66	2.27E-05	1.71
JD-5	0.5- 5.0	4.50	66	2.47E-05	1.86
	5.0-10.0	5.00	66	3.37E-04	25.36
	10.0-15.0	5.00	66	1.36E-04	10.24
	15.0-20.0	5.00	66	1.24E-04	9.31
	20.0-25.0	5.00	66	9.34E-05	7.02
	25.0-30.0	5.00	66	6.40E-05	4.81
	30.0-35.0	5.00	66	7.66E-05	5.76
	35.0-40.0	5.00	66	3.76E-05	2.83
	40.0-45.0	5.00	66	3.11E-05	2.34
	45.0-50.0	5.00	66	2.38E-05	1.79
JD-6	0.5- 5.0	4.50	66	3.68E-04	27.67
	5.0-10.0	5.00	66	2.27E-04	17.09
	10.0-15.0	5.00	66	1.30E-04	9.76
	15.0-20.0	5.00	66	8.94E-05	6.72
	20.0-25.0	5.00	66	6.42E-05	4.83
	25.0-30.0	5.00	66	3.58E-05	2.69

Table A.1.3 Result of Water Pressure Test (2/2)

Hole No.	Depth (m)	Section length (m)	Hole radius (mm)	Permeability coefficient	Lugeon unit
	0.5- 5.0	4.50	66	9.12E-05	6.86
	5.0-10.0	5.00	66	1.10E-04	8.29
	10.0-15.0	5.00	66	6.82E-05	5.13
JD-7	15.0-20.0	5.00	66	6.81E-05	5.12
	20.0-25.0	5.00	66	4.43E-05	3.33
	25.0-30.0	5.00	66	4.84E-05	3.64
	0.5- 5.0	4.50	66	2.94E-04	22.14
	5.0-10.0	5.00	66	1.95E-04	14.64
	10.0-15.0	5.00	66	1.19E-04	8.95
JD-8	15.0-20.0	5.00	66	7.04E-05	5.29
	20.0-25.0	5.00	66	7.05E-05	5.30
	25.0-30.0	5.00	66	4.99E-05	3.75
	30.0-35.0	5.00	66	3.29E-05	2.47
	35.0-40.0	5.00	66	3.34E-05	2.51
	40.0-45.0	5.00	66	3.35E-05	2.52
	45.0-50.0	5.00	66	2.57E-05	1.93
	0.5- 5.0	4.50	66	3.67E-04	27.56
JD-9	5.0-10.0	5.00	66	1.94E-04	14.56
	10.0-15.0	5.00	66	1.30E-04	9.80
	15.0-20.0	5.00	66	7.53E-05	5.66
	0.5- 5.0	4.50	66	4.12E-04	30.97
	5.0-10.0	5.00	66	1.87E-04	14.05
JD-10	10.0-15.0	5.00	66	7.95E-05	5.98
	15.0-20.0	5.00	66	5.51E-05	4.14
	20.0-25.0	5.00	66	3.90E-05	2.93
	25.0-30.0	5.00	66	2.79E-05	2.10

Table A.1.4 Result of Groundwater Level Observation

			Date of Observation									
			21 Nov - 11 Dec 1981		4 Apr - 27 May 1994		7 Sep - 28 Sep 1994		13 Oct 1994		11 Nov 1994	
Hole No.	Depth (m)	Elevation (m)	Water Level (m)	Water Level EL. (m)	Water Level (m)	Water Level EL. (m)	Water Level (m)	Water Level EL. (m)	Water Level (m)	Water Level EL. (m)	Water Level (m)	Water Level EL. (m)
JD-1	50.00	62.513			12.900	49.613	15.320	47.193				
JD-2	30.00	48.413			8.600	39.813	8.150	40.263			10.010	38.403
JD-3	25.00	38.322			6.100	32.222	3.920	34.402	4.280	34.042	4.540	33.782
JD-4	25.00	57.030			3.390	53.640	2.700	54.330	3.020	54.010	3.520	53.510
JD-5	50.00	71.805			12.150	59.655	22.850	48.955	22.970	48.835	23.140	48.665
JD-6	30.00	43.295			10.300	32.995	8.530	34.765	8.490	34.805	8.540	34.755
JD-7	30.00	27.508			3.930	23.578	2.190	25.318	2.370	25.138	2.730	24.778
JD-8	50.00	44.605			8.430	36.175	9.750	34.855	9.980	34.625	10.400	34.205
JD-9	20.00	31.089			4.770	26.319	4.820	26.269				
JD-10	30.00	57.380			15.750	41.630	11.720	45.660	13.350	44.030	13.830	43.550
GB-1	30.00	59.077	23.200	35.877								
GB-2	40.00	55.161	20.900	34.261								
GB-3	60.00	30.259	5.500	24.759								
GB-4	35.00	50.205	16.200	34.005								
GB-5	35.00	57.610	19.900	37.710								

Table A.1.5 **Result of Dutch Cone Penetration Test (Alternative-1)**

Penetrated Depth (m)	Location : WU - 1 Elevation (m)	Cone Bearing Capacity, qc (kg/cm2)	N Value	Skin Friction Capacity, fc (kg/cm2)	Location : WU - 2 Elevation (m)	Cone Bearing Capacity, qc (kg/cm2)	N Value	Skin Friction Capacity, fc (kg/cm2)	Location : WU - 3 Elevation (m)	Cone Bearing Capacity, qc (kg/cm2)	N Value	Skin Friction Capacity, fc (kg/cm2)
0.0	18.0	0	0.0	0.0	18.5	0	0.0	0.0	18.0	0	0.0	0.0
0.2	17.8	0	0.0	0.0	18.3	0	0.0	0.0	17.8	0	0.0	0.0
0.4	17.6	0	0.0	0.0	18.1	0	0.0	0.5	17.6	0	0.0	0.5
0.6	17.4	0	0.0	0.1	17.9	0	0.0	0.5	17.4	0	0.0	0.5
0.8	17.2	5	1.3	0.1	17.7	10	2.5	2.0	17.2	5	1.3	0.5
1.0	17.0	5	1.3	0.1	17.5	40	10.0	1.0	17.0	5	1.3	0.5
1.2	16.8	25	6.3	0.1	17.3	30	7.5	2.0	16.8	10	2.5	1.0
1.4	16.6	15	3.8	0.1	17.1	30	7.5	1.0	16.6	10	2.5	0.5
1.6	16.4	15	3.8	0.2	16.9	30	7.5	1.0	16.4	15	3.8	2.5
1.8	16.2	15	3.8	0.2	16.7	30	7.5	1.0	16.2	15	3.8	2.5
2.0	16.0	20	5.0	0.2	16.5	35	8.8	1.5	16.0	15	3.8	2.5
2.2	15.8	20	5.0	0.2	16.3	30	7.5	2.0	15.8	15	3.8	2.5
2.4	15.6	30	7.5	0.2	16.1	25	6.3	2.5	15.6	15	3.8	1.5
2.6	15.4	30	7.5	0.3	15.9	25	6.3	2.5	15.4	25	6.3	0.5
2.8	15.2	20	5.0	0.3	15.7	25	6.3	3.0	15.2	25	6.3	1.0
3.0	15.0	30	7.5	0.3	15.5	25	6.3	3.0	15.0	25	6.3	1.0
3.2	14.8	15	3.8	0.3	15.3	30	7.5	2.0	14.8	30	7.5	1.0
3.4	14.6	15	3.8	0.3	15.1	30	7.5	2.0	14.6	30	7.5	1.5
3.6	14.4	20	5.0	0.4	14.9	35	8.8	2.0	14.4	30	7.5	0.5
3.8	14.2	15	3.8	0.4	14.7	40	10.0	2.0	14.2	35	8.8	4.0
4.0	14.0	15	3.8	0.4	14.5	40	10.0	2.0	14.0	35	8.8	4.0
4.2	13.8	20	5.0	0.4	14.3	45	11.3	1.5	13.8	35	8.8	3.5
4.4	13.6	15	3.8	0.4	14.1	45	11.3	1.5	13.6	35	8.8	3.5
4.6	13.4	25	6.3	0.5	13.9	50	12.5	1.0	13.4	40	10.0	4.0
4.8	13.2	20	5.0	0.5	13.7	50	12.5	1.0	13.2	50	12.5	3.0
5.0	13.0	20	5.0	0.5	13.5	50	12.5	1.0	13.0	50	12.5	3.0
5.2	12.8	20	5.0	0.5	13.3	30	7.5	1.0	12.8	50	12.5	2.0
5.4	12.6	25	6.3	0.5	13.1	35	8.8	1.0	12.6	50	12.5	2.0
5.6	12.4	25	6.3	0.6	12.9	35	8.8	1.0	12.4	60	15.0	2.0
5.8	12.2	20	5.0	0.6	12.7	35	8.8	1.5	12.2	60	15.0	2.0
6.0	12.0	30	7.5	0.6	12.5	50	12.5	1.0	12.0	60	15.0	4.0
6.2	11.8	35	8.8	0.6	12.3	60	15.0	1.0	11.8	75	18.8	2.5
6.4	11.6	30	7.5	0.6	12.1	60	15.0	1.5	11.6	80	20.0	2.0
6.6	11.4	50	12.5	0.7	11.9	60	15.0	1.5	11.4	90	22.5	2.0
6.8	11.2	35	8.8	0.7	11.7	60	15.0	2.0	11.2	90	22.5	2.0
7.0	11.0	40	10.0	0.7	11.5	70	17.5	3.0	11.0	90	22.5	5.0
7.2	10.8	110	27.5	0.7	11.3	60	15.0	3.0	10.8	60	15.0	5.0
7.4	10.6	70	17.5	0.7	11.1	60	15.0	3.0	10.6	60	15.0	3.0
7.6	10.4	30	7.5	0.8	10.9	60	15.0	1.0	10.4	60	15.0	3.5
7.8	10.2	15	3.8	0.8	10.7	50	12.5	3.0	10.2	50	12.5	2.5
8.0	10.0	55	13.8	0.8	10.5	50	12.5	3.0	10.0	50	12.5	2.5
8.2	9.8	60	15.0	0.8	10.3	40	10.0	3.0	9.8	80	20.0	2.0
8.4	9.6	40	10.0	0.8	10.1	40	10.0	3.0	9.6	80	20.0	4.0
8.6	9.4	70	17.5	0.9	9.9	50	12.5	3.0	9.4	100	25.0	3.0
8.8	9.2	150	37.5	0.9	9.7	50	12.5	3.0	9.2	140	35.0	4.0
9.0	9.0	150	37.5	0.9	9.5	50	12.5	3.0	9.0	150	37.5	5.0
9.2					9.3	60	15.0	3.0				
9.4					9.1	70	17.5	2.5				
9.6					8.9	75	18.8	6.5				
9.8					8.7	80	20.0	6.0				
10.0					8.5	95	23.8	5.5				
10.2					8.3	150	37.5	5.0				

Table A.1.6 Work Quantity of Field Test

Item	Unit	Q'ty	Remarks
1. Test Pit	spots	4	2m x 2m x 2.5m
2. Sampling			
2.1 Disturbed samples	samples	6	Test pit (Impervious for physical properties test
2.2 Disturbed samples	samples	6	Test pit (Impervious for mechanical test material)
2.3 Disturbed samples	samples	3	Out crop 2 samples for rock test pit 1 sample
2.4 Disturbed samples	samples	2	Out crop for Filter test

Table A.1.7 Result of Schmidt Hammer Test at Dam Site

Sample Name	Condition	Rebound load	Modulos of Deformation (1,000 kgf/cm2)	Compressing strength (kgf/cm2)
Mud stone	fresh	18 to 22 (Average 20)	20 to 27 (Average 23)	90 to 120 (Average 100)
Sand stone	moderatoly Weathered	16 to 22 (Average 18)	17 to 27 (Average 20)	80 to 120 (Average 90)
Conglomerate	moderatoly Weathered	14 to 20 (Average 16)	15 to 23 (Average 17)	20 to 100 (Average 80)
	fresh	22 to 46 (Average 30)	27 to 168 (Average 50)	120 to 650 (Average 210)

Table A.1.8 Work Quantity of Laboratory Test

Item	Q'ty	Remarks
Test Pit		
1. Specific Gravity	6	3 samples x 2 test pits
2. Moisture Content	6	3 samples x 2 test pits
3. Grain Size analysis	6	3 samples x 2 test pits
4. Atterberg limits	6	3 samples x 2 test pits
5. Compaction test	1	Typical sample
6. Permeability test	2	Typical sample x 2 moisture content
7. Toriaxail Shear (uu)	2	Typical sample x 2 moisture content
8. Toriaxial Shear (cu)	2	Typical sample x 2 moisture content
9. Consolidation	2	Typical sample x 1 moisture content
Outcrop		
10. Grain Size analysis	3	1 samples x 2 spot
11. Specific Gravity and water absorpion of gravel	3	1 samples x 2 spot
12. Slaking test	2	1 samples x 2 spot
13. Compaction test	3	1 samples x (2 spots and mix material)
14. Permeability test	3	1 samples x (2 spots and mix material)
15. Toriaxial shear (c-d)	3	1 samples x (2 spots and mix material) River Bed
16. Grain size analysis	2	Dam site
17. Specific gravity and water absorption of gravel	2	Dam site
18 Abrasion resistance	1	Dam site

Table A.1.9 Summary of Laboratory Test for Physical Properties (Borrow Area of Gilirang Dam)

SAMPLE NO.	SPECIFIC GRAVITY	NATURAL MOISTURE CONTENT (%)	ATTERBERG LIMIT PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	GRAIN SIZE ANALYSIS MAXIMUM GRAIN SIZE (mm)	GRAVEL +4.76MM (%)	SAND 4.76 ~ 0.074mm (mm)	SILT AND CLAY - 0.074mm (mm)	UNIFIED SOIL GLASIFICATION SISTEM	REMARKS
1	2.613	37.67	29.135	59.62	30.485	>4.75	1.44	32.34	66.22	CH	TP.1 0.5-1
2	2.544	33.535	41.650	61.63	19.65	>4.75	4.42	36.99	58.59	M1	TP.2 05-2
3	2.767	27.695	34.245	71.79	37.545	>4.75	9.05	46.78	44.17	SM	TO.2 1.5-2
4	2.557	34.66	30.030	53.05	23.02	>4.75	0.59	15.38	84.03	OH	TO.3 2.5-3
5	2.727	17.315	29.565	46.48	16.915	>4.75	46.99	23.18	18.28	GM	TP.3 2.5-3
6	2.589	8.705	22.680	30.40	7.72	>4.75	31.5	60.75	7.75	SP - SC	TP.4 1-1.5

Table A.1.10 Summary of Laboratory Test for Mechanical Properties (Borrow Area of Gilirang Dam)

SAMPLE NO.	COMPACTION TEST (Ec=100%)		INITIAL SPECIMENT CONDITION						TRIAL COMPRESSTION TEST				CONSOLIDATION TEST			COEFICIENT OF PERMEABILITY
									U-U(TOTAL STRESS)		C-U(EFETIVE STRESS)					
	MAXIMUM DRY DENCITY (g/cm3)	OPTIMUM MOINSTURE CONTENT (%)	MOINSTURE CONTENT	DRY DENCITY	WET DENCITY	VOID RATIO	DEGREE OF SATURATION	D VALUE (%)	COHESION (kgf/cm2)	INTERNAL FRICTIAN ANGEL (degre)	COHESION (kgf/cm2)	INTERNAL FRICTIAN ANGEL (degree)	COEFICIENT OF VALUME COMPRESSI-BILITY (cm2/day)	COEFICIENT OF CONSOLI-DATION (cm2/day)	COMPRESSION INDEX	(cm/sec)
1+2	1.62	20.50	20.70	14.90	1.838	0.703	76.90	100	2.539	18.4	-	-	-	-	-	-
			24.03	14.40	1.82	0.767	84.50	95	2.386	26.4	-	-	-	-	-	-
			26.97	14.30	1.85	0.777	89.20	95	-	-	0.509	24.3	-	-	-	-
			26.89	1.300	1.60	1.078	64.58	95	-	-	-	-	0.0176	4.4525	0.343	-
			22.04	1.475	1.80	0.756	75.51	100	-	-	-	-	-	-	-	1.40E-06
			26.80	1.341	1.70	0.931	74.55	95	-	-	-	-	-	-	-	1.74E-07
1+3	1.66	21.00	17.87	L.50	1.80	0.759	63.13	100	1.438	29.8	-	-	-	-	-	-
			21.50	L.50	1.86	0.759	70.07	95	2.355	12.8	-	-	-	-	-	-
			28.83	L.39	1.83	0.893	86.97	95	-	-	0.622	11.5	-	-	-	-
			19.00	L.60	1.90	0.68	75.16	100	-	-	-	-	-	-	-	3.61E-07
			23.42	L.34	1.65	1.01	62.37	95	-	-	-	-	-	-	-	9.79E-07

It is noted that compaction energy (Ec)=100% is applied to the density of specimen for the mechanical test as follow ;

$$Ec = \frac{W.H.n.L}{V}$$

Where Ec : Compaction energy
(Ec of 100%=5.625 kg.cm/cm3)
w: Weight of rammer
H: Fall height of rammer
n: Number of compaction per layer
L: Number of layer
V: Volume of mould

Table A.1.11 Summary of Laboratory Test for Physical Properties of Rock (Quarry of Gilirang Dam)

SAMPLRE No.	SPECIFIC GRAVITY OF SURFACE DRY BASIS	SPECIFIC GRAFITY OF DRY BASIS	SPECIFIC GRAFITY	ABSORPTION	ABRASION LOSS AFTER (%)	SLAKING LOSS AFTER (%)	GRAIN SIZE ANALYSIS				REMARKS
							MAXIMUM GRAIN SIZE (mm)	GRAVEL +4.76mm (mm)	SAND 4.76 - 0.074mm (mm)	SILT AND CLAY - 0.074mm (mm)	
7	2.016	1.727	2.43	16.719	-	-	>4.75	0.09	66.57	33.34	MUD STONE
8	2.251	2.053	2.65	9.694	-	15.42	>4.75	48.7	49.24	2.06	CONCLOMERAT
9	2.291	2.051	2.787	13.717	-	36.23	>4.75	30.04	64.41	5.55	SAND STONE
10.A	2.307	2.058	2.742	12.114	-	-	<4.75	-	88.56	11.44	SAND
10.B	2.647	2.598	2.731	1.385	24.268	-	>4.75	19.62	74.9	5.48	GRAVEL

Table A.1.12 Summary of Laboratory Test for Mechanical Properties of Rock (Quarry of Gilirang Dam)

SAMPLE No.	INITIAL SPECIMENT CONDITION			TRIAXIAL COMPRESSION TEST 15cm (CD)		COEFFICIENT OF PERMEABILITY	REMARKS
	MOINSTURE CONTENT (%)	DRY DENSITT (g/cm3)	VOID RATIO DRY BASIS	CHESION (kg/cm2)	INTERNAL FRICTION ANGLE (DEGREE)	(cm/sec)	
8	5,998	1,821	0.334	0.38	43.8	1.17E-04	CONCLOMERAT
9	4,746	1,676	0.527	0.38	45.2	3.52E-04	SAND STONE
8+9	3,679	1,729.	0.612	0.53	43.9	6.15E-04	CONGLOMERAT AND SAND STONE (MIX MATERIAL)

It is noted that compaction energy (Ec) = 100% is applied to the density of speciment for the mechanical tests as follows;

$$Ec = \frac{W.H.n.L}{V}$$

Where Ec : Compaction energy
(Ec of 100% = 5.625 kg.cm/cm3)
w : Weight of rammer
H : Fall height of rammer
n : Number of compaction per layer
L : Volume of layer
V : Volume of mould

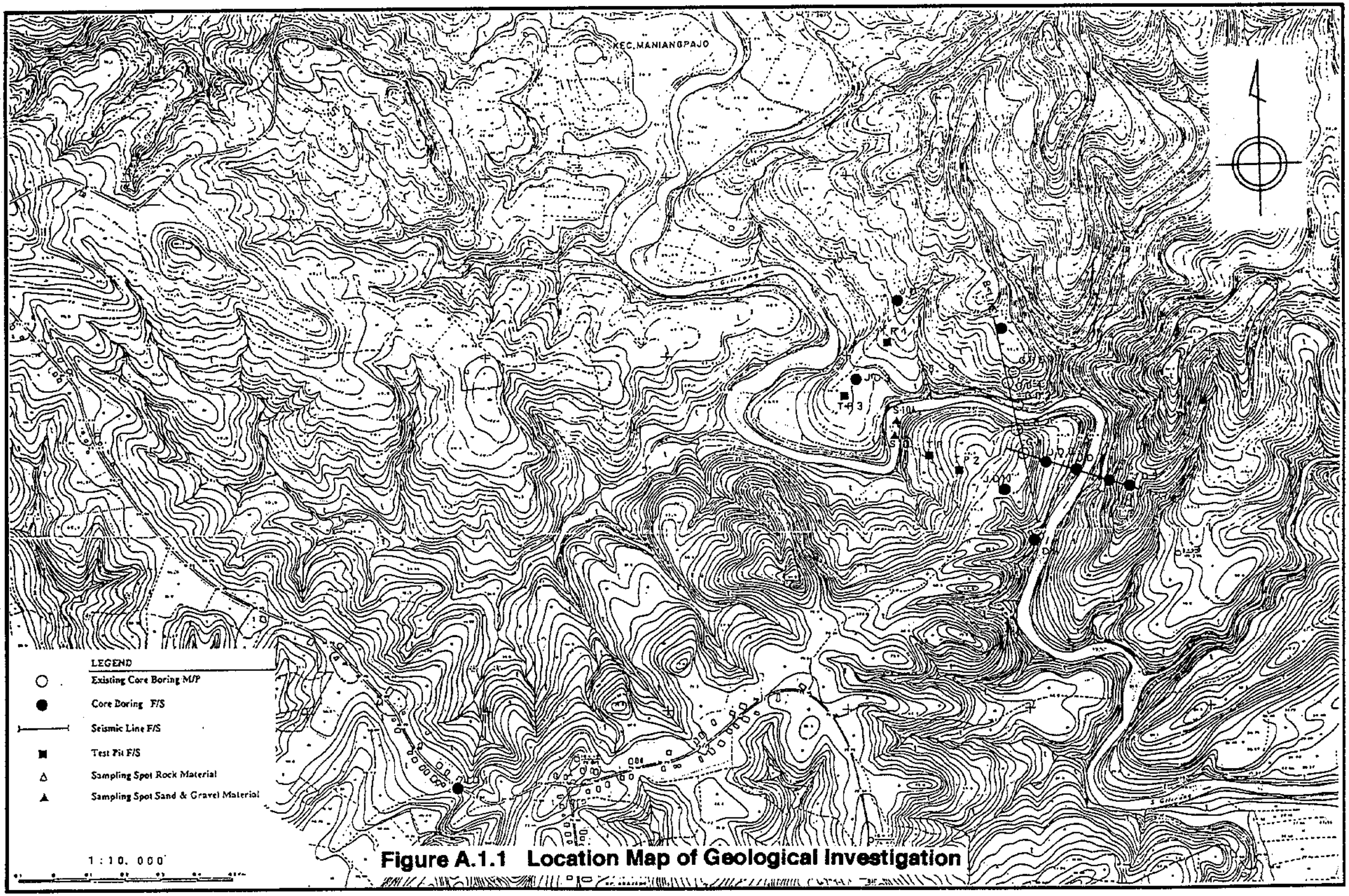
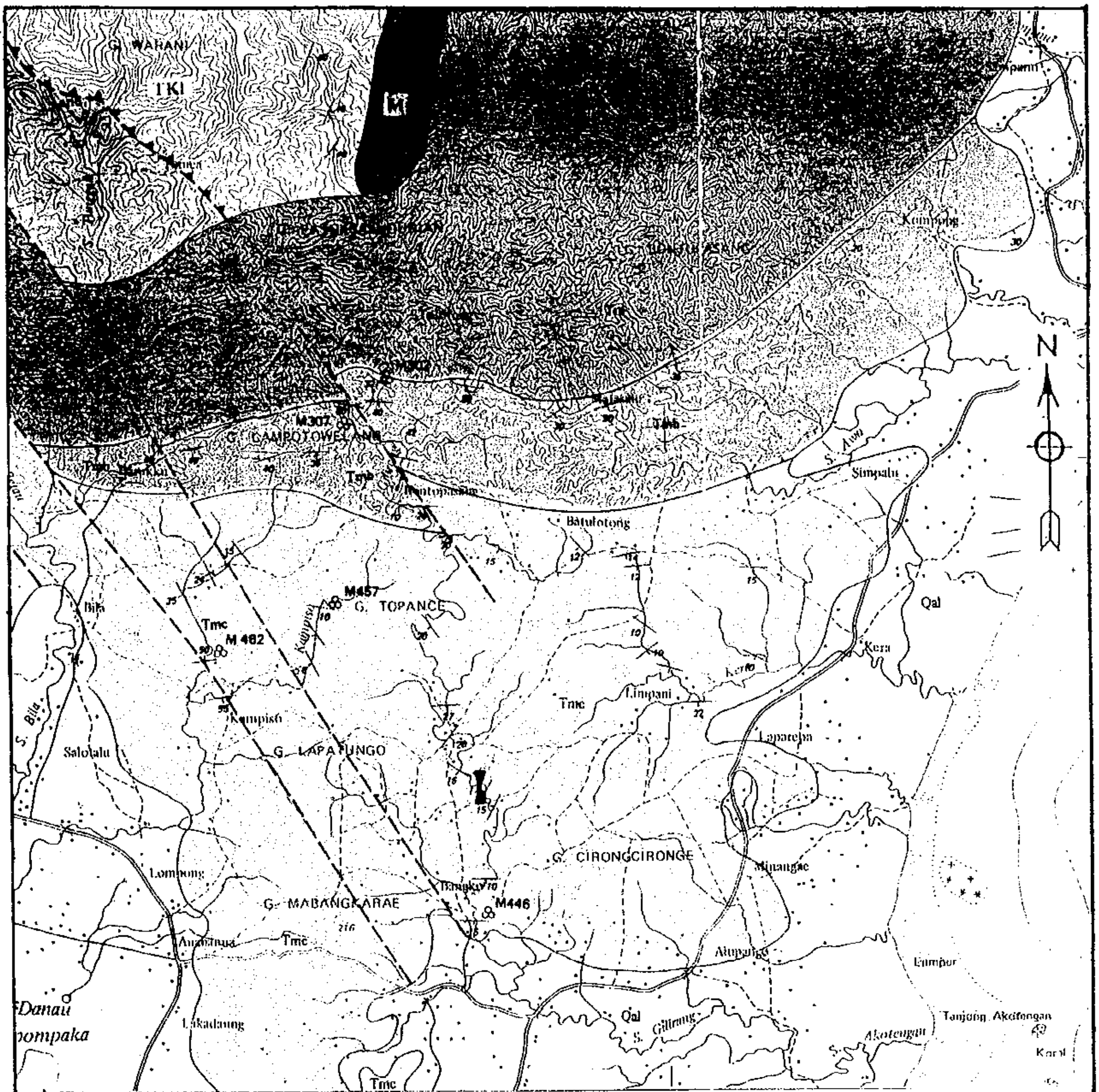


Figure A.1.1 Location Map of Geological Investigation



Qal	Allvium, Clay, silt,sand,gravel and reef limestone;tickness about 100m
Tmc	Conglomerate, includes a little glauconitic sandstone with shale, coquina and molluscs; unit forms undulating topography. Foraminifers range in age from late middle Miocene to Pliocene about 100m to 400m thick.
Tbm	Marl and intercalated limestone,locally contains bluish grey to block calcareous sandstone, conglomerate and breccia. Foraminifers range in age from late early Miocene to latest middle Miocene
Tol	Lava flows,basaltic,to andesitic,volcanic breccia volcanic sandstone and siltstone;locally contain feldpathoids, Most rocks chloritized and silicified. No fossil found.
Tet	Shale,raddish brown and grey marly shale and limestone,quartzose sandstone, quartz conglomerate,and, locally, coal
TKl	LATIMOJONG FORMATION-Moderatelly metamorphozed rocks, slate,phyllite, chert,marble,quatzite and silicified breccia, some intermediate to basic intrusions. The formation is named from exposures in the Latimojong Montains. These are the oldest rocks exposed in the region,and because of pervasive folding and metamorphism,little is known about their thicness. No fossils have been found; nor top is exposed.
Mt	Quartz monzonite and adamellite.

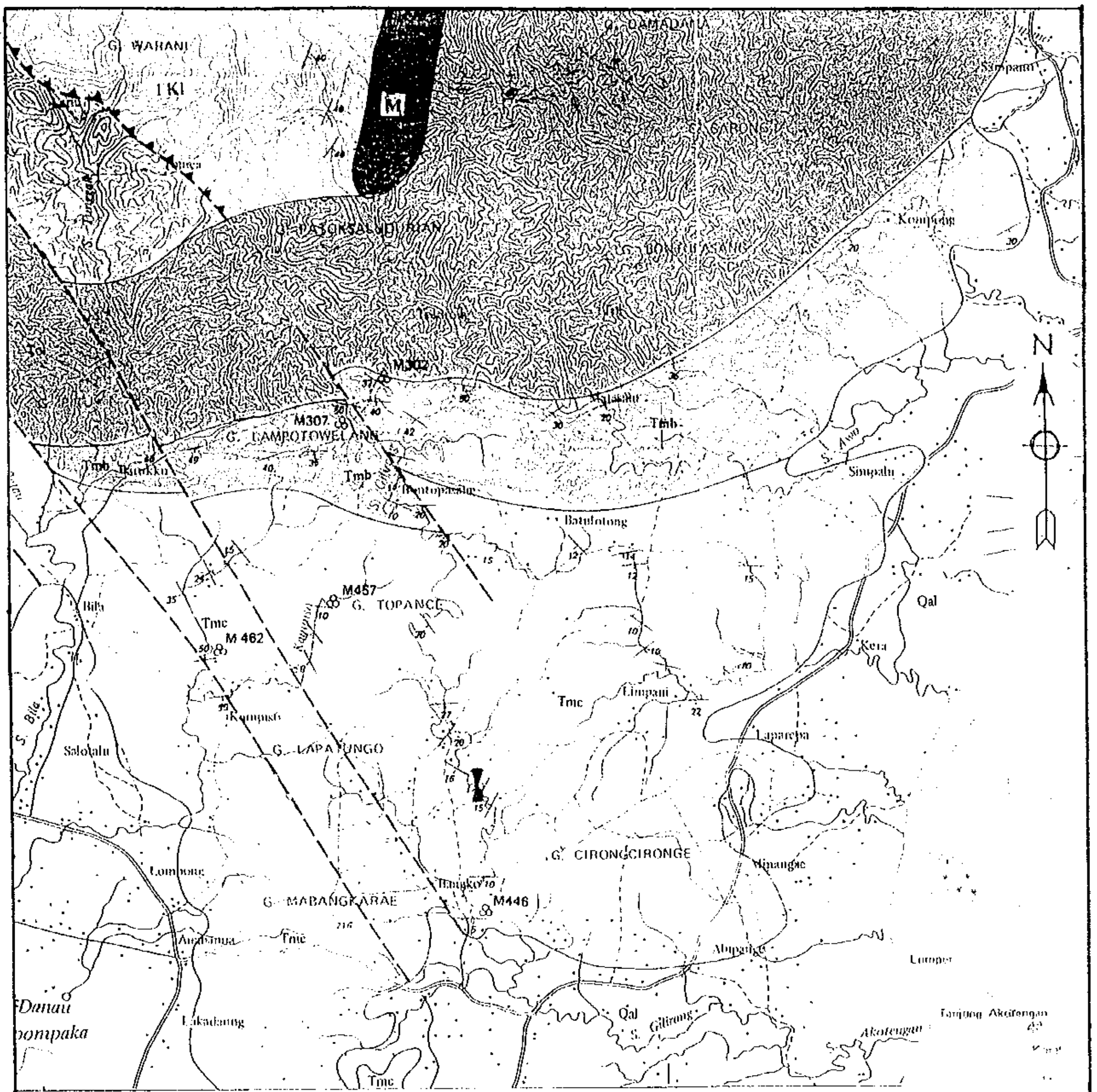
Paseloreng Dam Site

— — — Fault

Scale : 1 : 250.000

——— Geological boundary

Figure A.1.2 Regional Geological Map



Qal	Allvium, Clay, silt,sand,gravel and reef limestone;tickness about 100m
Tet	Shale,raddish brown and grey marly shale and limestone,quartzose sandstone, quartz conglomerate,and, locally, coal
Tmc	Conglomerate, includes a little glauconitic sandstone with shale, coquina and molluscs; unit forms undulating topography. Foraminifers range in age from late middle Miocene to Pliocene about 100m to 400m thick.
TKl	LATIMOJONG FORMATION-Moderatelly metamorphozed rocks, slate,phyllite, chert,marble,quatzite and silicified breccia, some intermediate to basic intrusions. The formation is named from exposures in the Latimojong Montains. These are the oldest rocks exposed in the region,and because of pervasive folding and metamorphism,little is known about their thicness. No fossils have been found; nor top is exposed.
Tbm	Marl and intercalated limestone,locally contains bluish grey to block calcareous sandstone, conglomerate and breccia. Foraminifers range in age from late early Miocene to latest middle Miocene
Tol	Lava flows,basaltic,to andesitic,volcanic breccia volcanic sandstone and siltstone;locally contain feldpathoids, Most rocks chloritized and silicified. No fossil found.
M	Quartz monzonite and adamellite.

Paseloreng Dam Site

— — — Fault

Scale : 1 : 250.000

——— Geological boundary

Figure A.1.2 Regional Geological Map

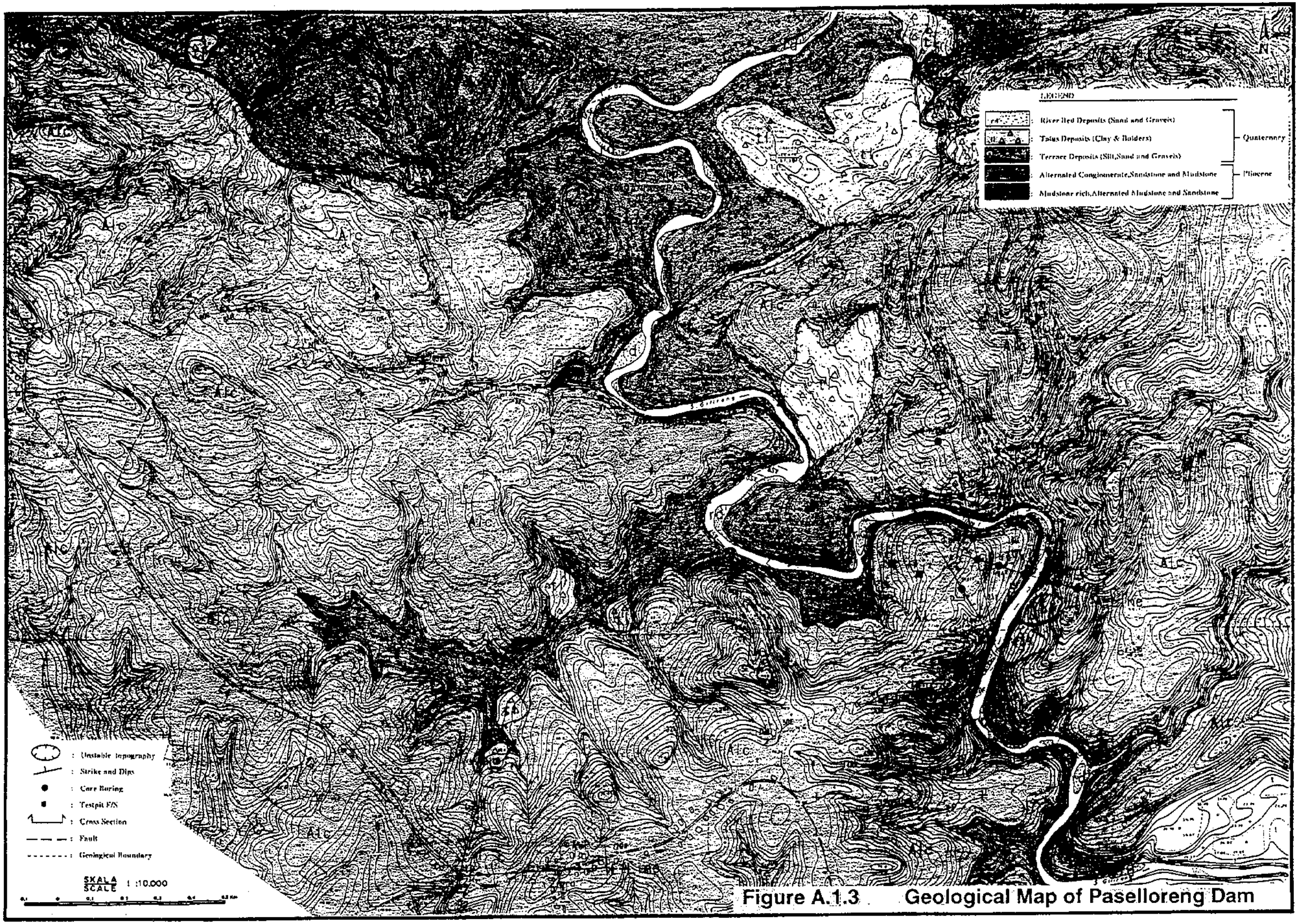


Figure A.1.3 Geological Map of Pasellorenq Dam

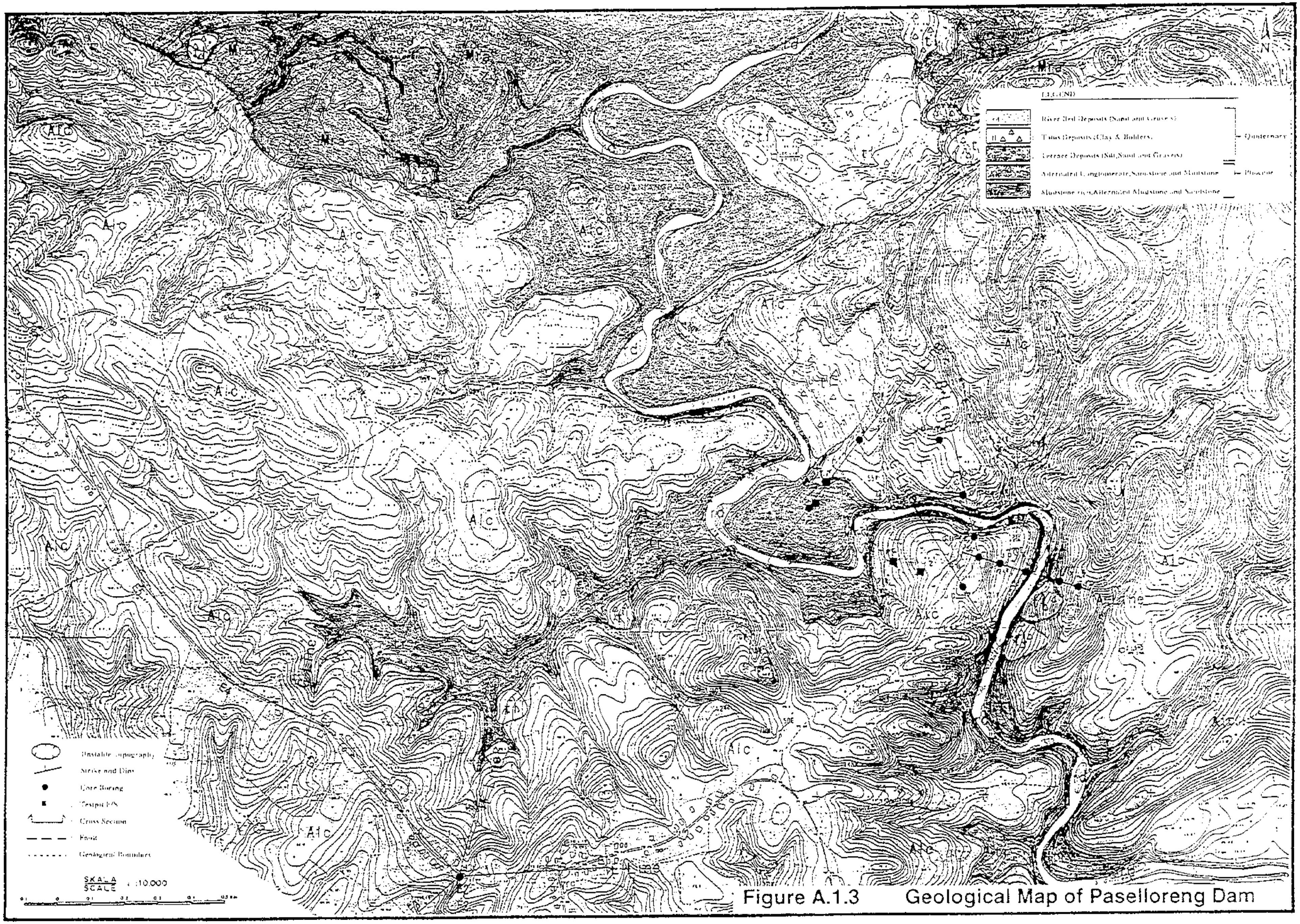


Figure A.1.3 Geological Map of Paselloreng Dam

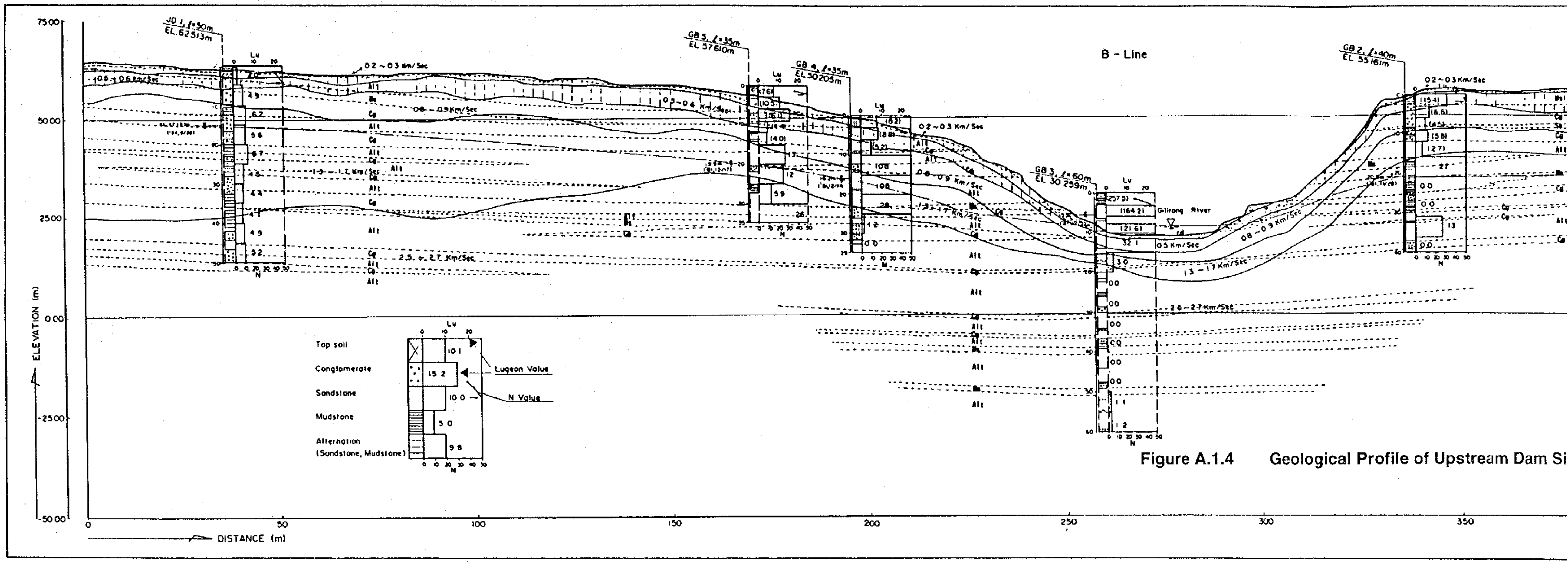


Figure A.1.4 Geological Profile of Upstream Dam Si

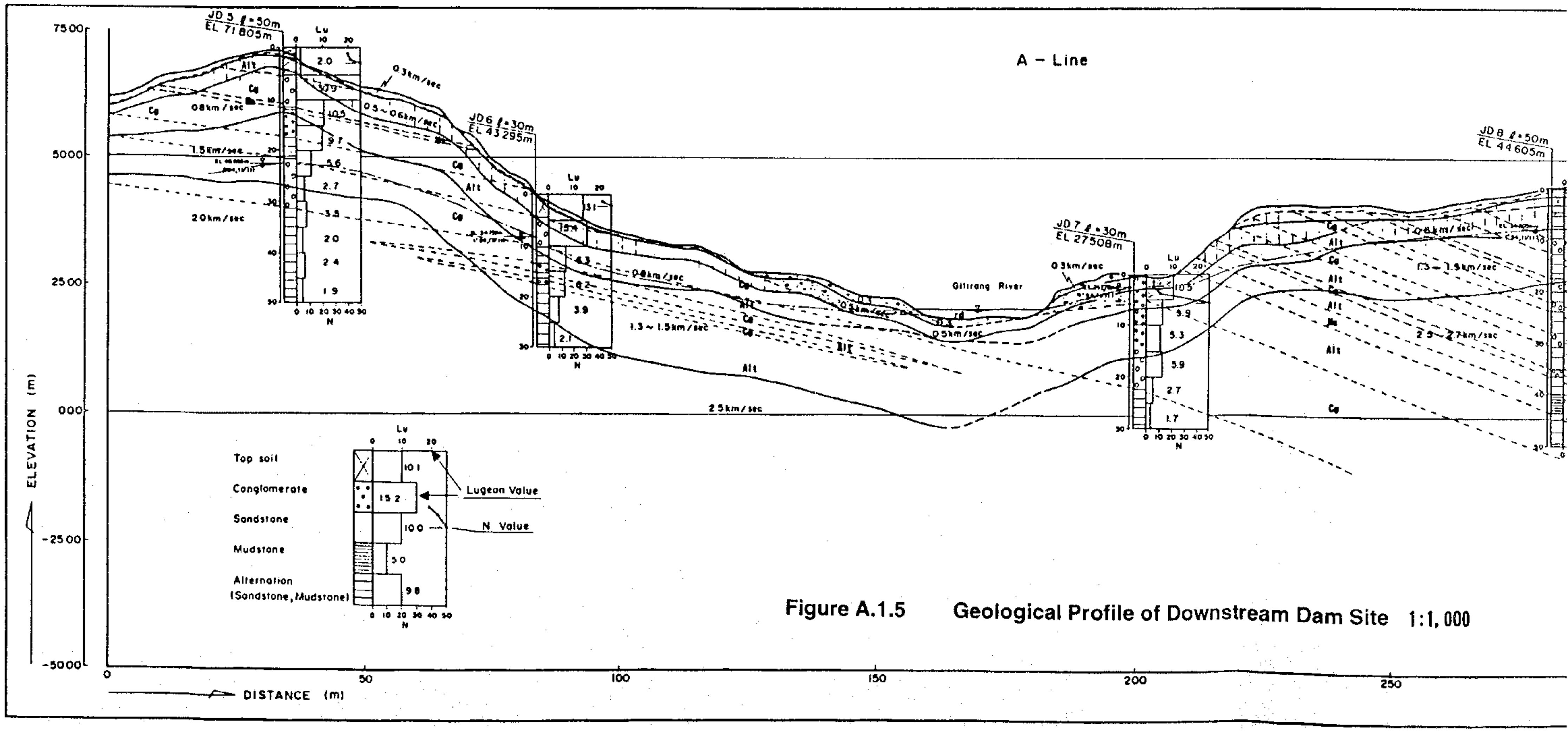


Figure A.1.5 Geological Profile of Downstream Dam Site 1:1,000

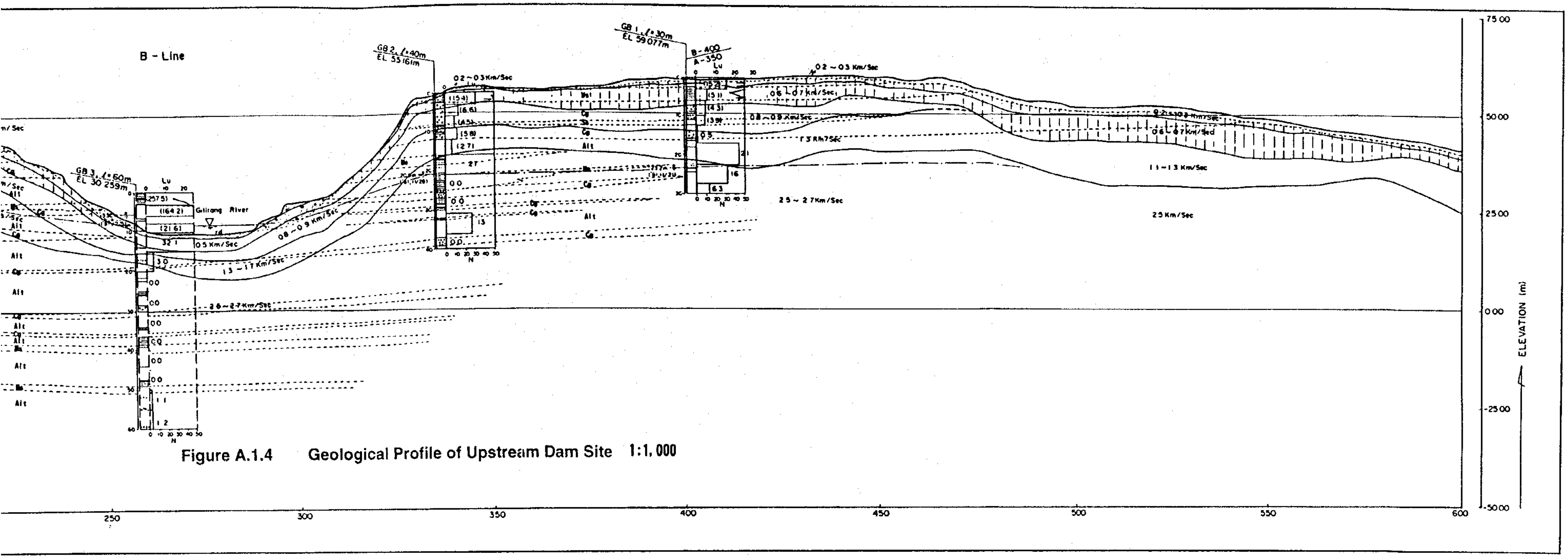


Figure A.1.4 Geological Profile of Upstream Dam Site 1:1,000

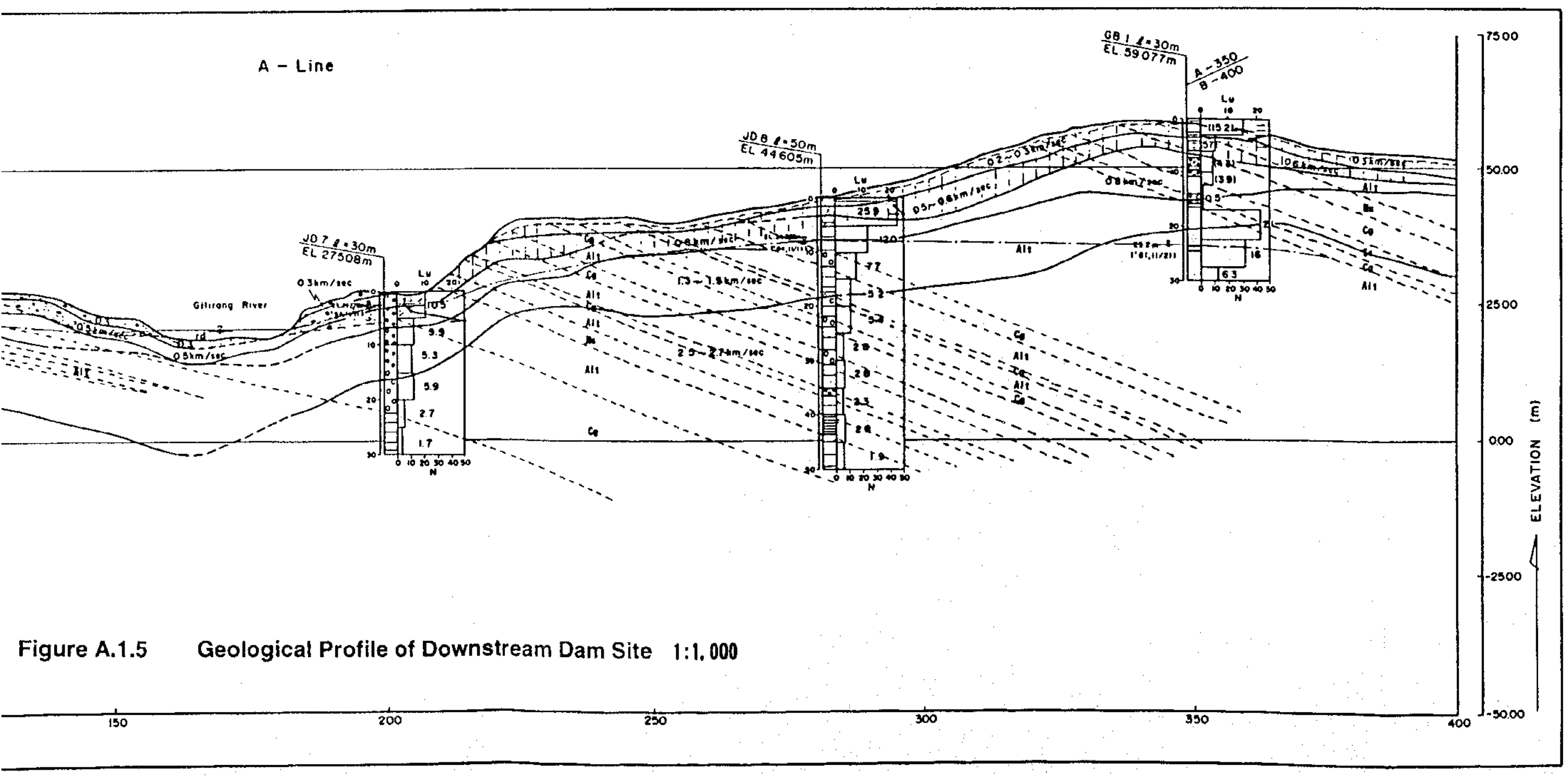


Figure A.1.5 Geological Profile of Downstream Dam Site 1:1,000

LEGEND

Quaternary			Top Soil
		rd	River Bed Deposits (Sand and Gravels)
		tl	Talus Deposits (Clay & Bolders)
		tr	Terrace Deposits (Silt,Sand & Gravels)
Pliocene	Alc	Alt	Alternating beds (Sandstone & Mudstone)
		Cg	Conglomerate
		Ms	Mudstone (includ Sandstone)
		Ss	Sandstone (includ Mudstone)

: Highly Weathered Rock

: Ground Water Level

: Geological Boundary

2.5km/sec : Seismic Velosity Profile

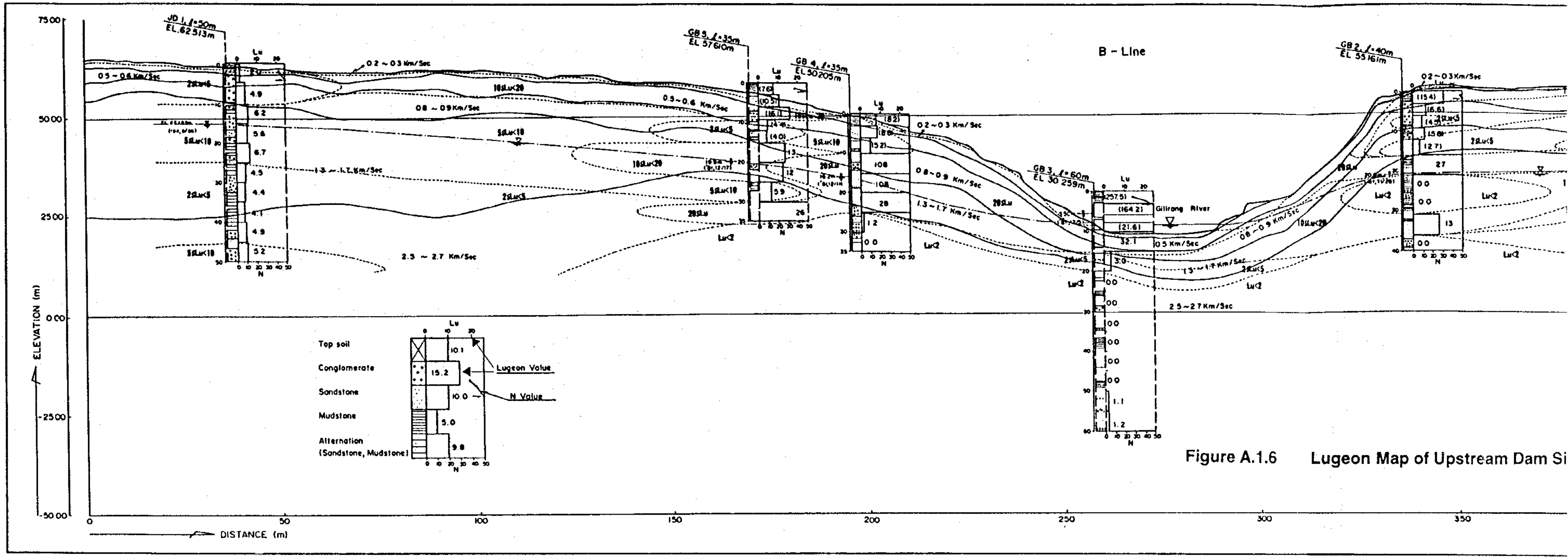


Figure A.1.6 Lugeon Map of Upstream Dam Si

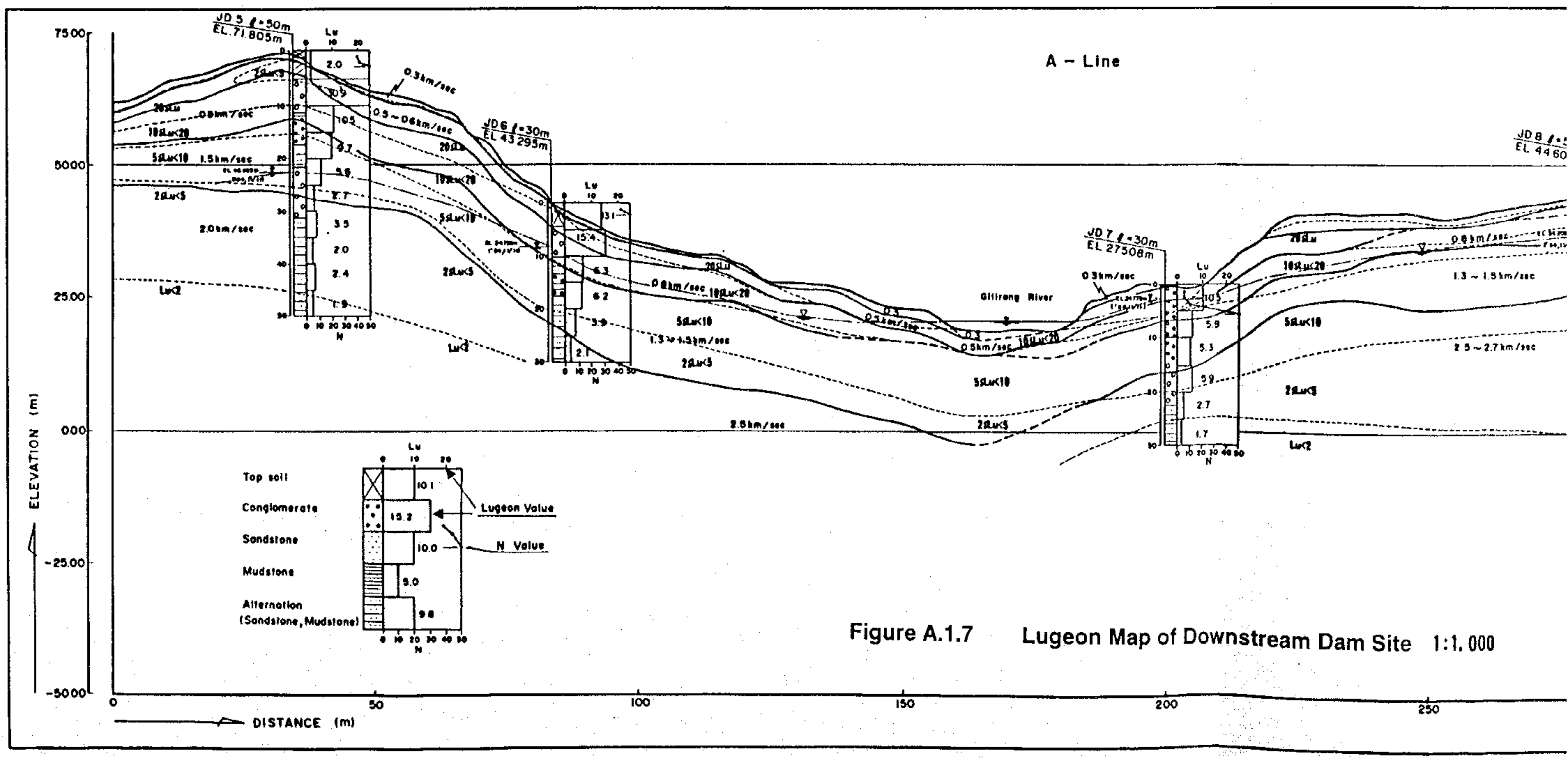


Figure A.1.7 Lugeon Map of Downstream Dam Site 1:1,000

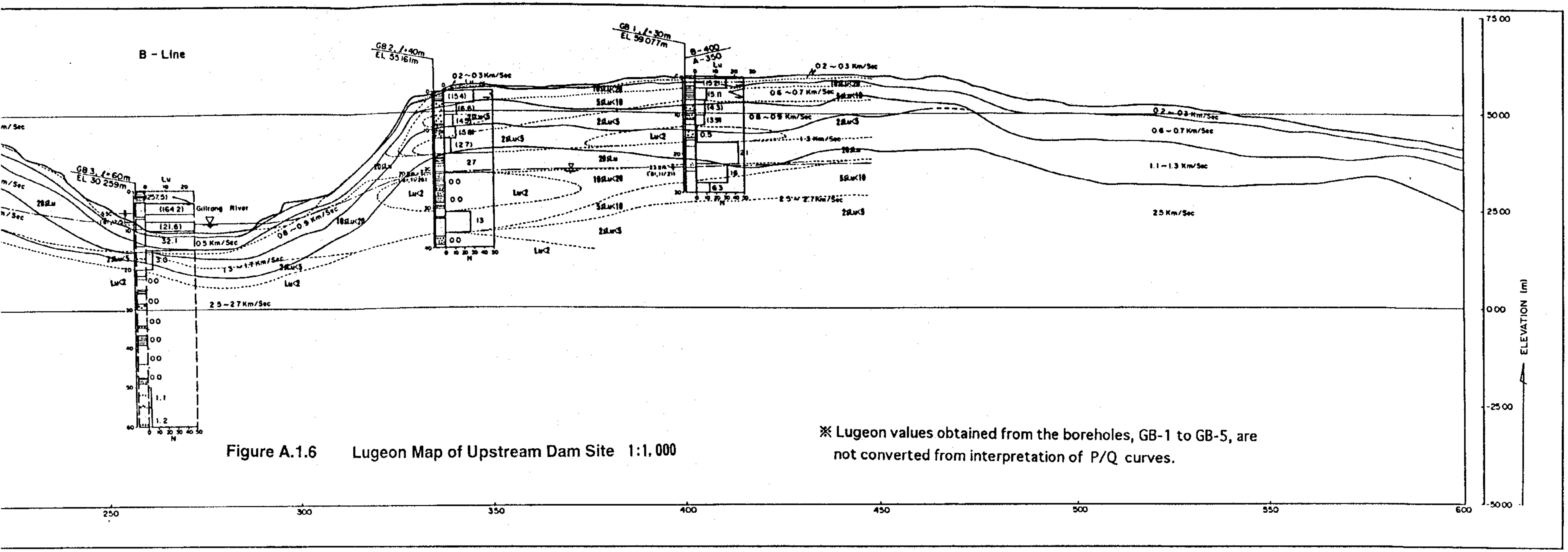


Figure A.1.6 Lugeon Map of Upstream Dam Site 1:1, 000

※ Lugeon values obtained from the boreholes, GB-1 to GB-5, are not converted from interpretation of P/Q curves.

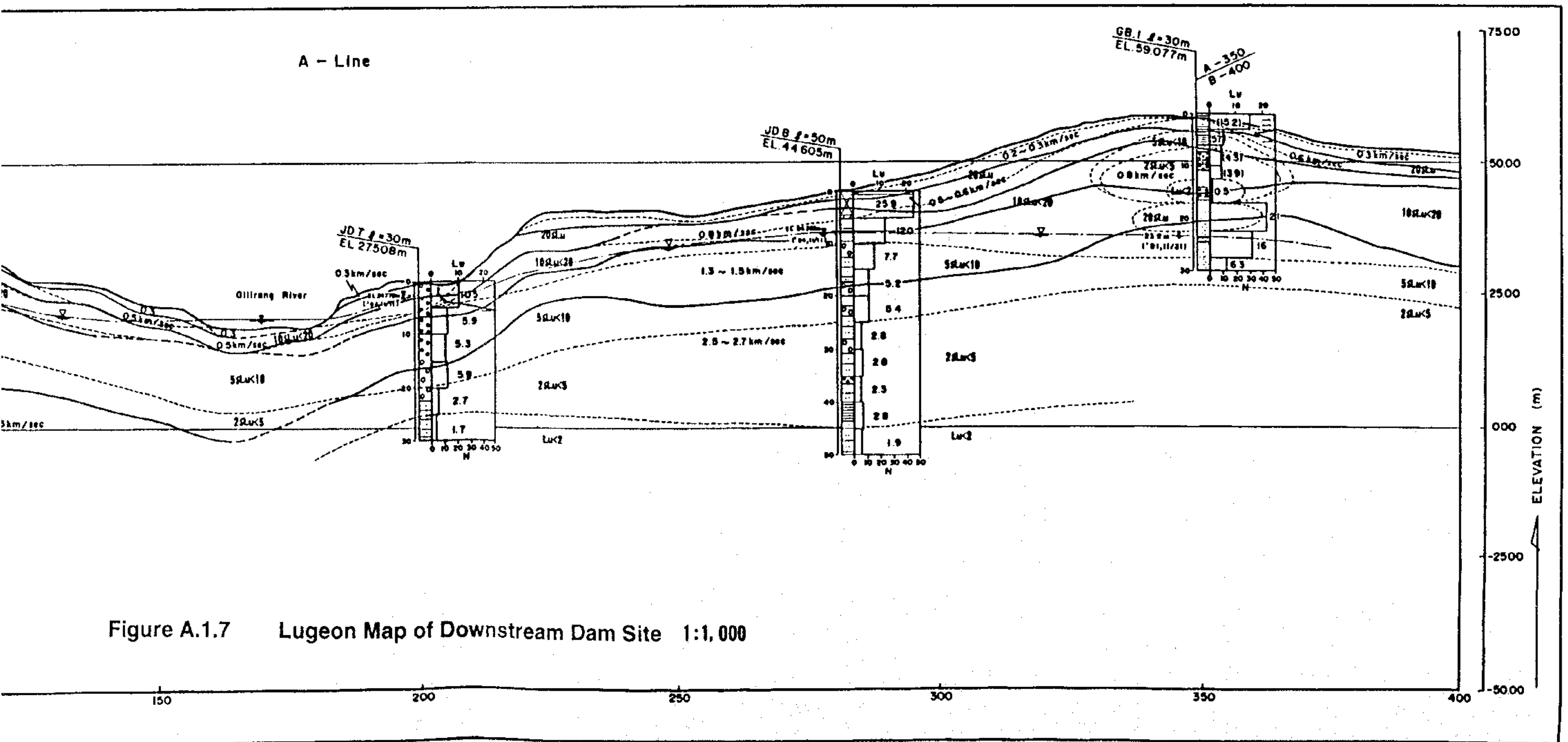
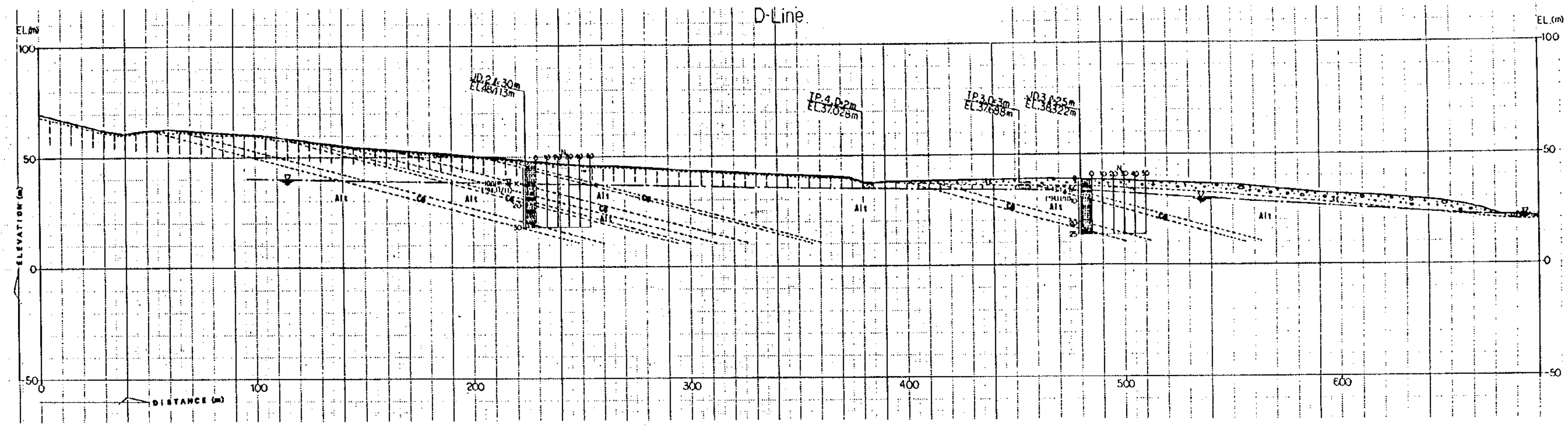


Figure A.1.7 Lugeon Map of Downstream Dam Site 1:1, 000



LEGEND

Quate-enary			Top Soil
	rd		River Bed Deposits (Sand and Gravels)
	tl		Talus Deposits (Clay & Bolders)
	tr		Terrace Deposits (Silt,Sand & Gravels)
Plioc-ene	Alc	Alt	Alternating beds (Sandstone & Mudstone)
		Cg	Conglomerate
		Ms	Mudstone (includ Sandstone)
		Ss	Sandstone (includ Mudstone)

: Highly Weathered Rock

: Ground Water Level

- - - - - : Geological Boundary

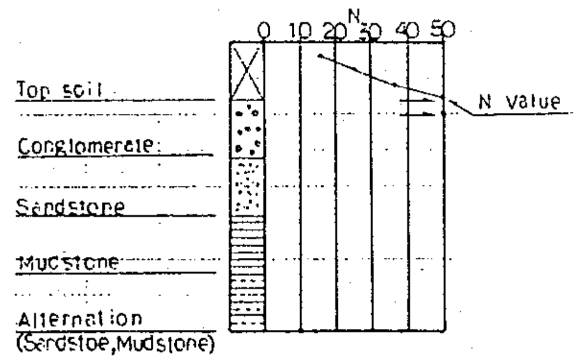


Figure A.1.8 Geological Profile of Quarry Site

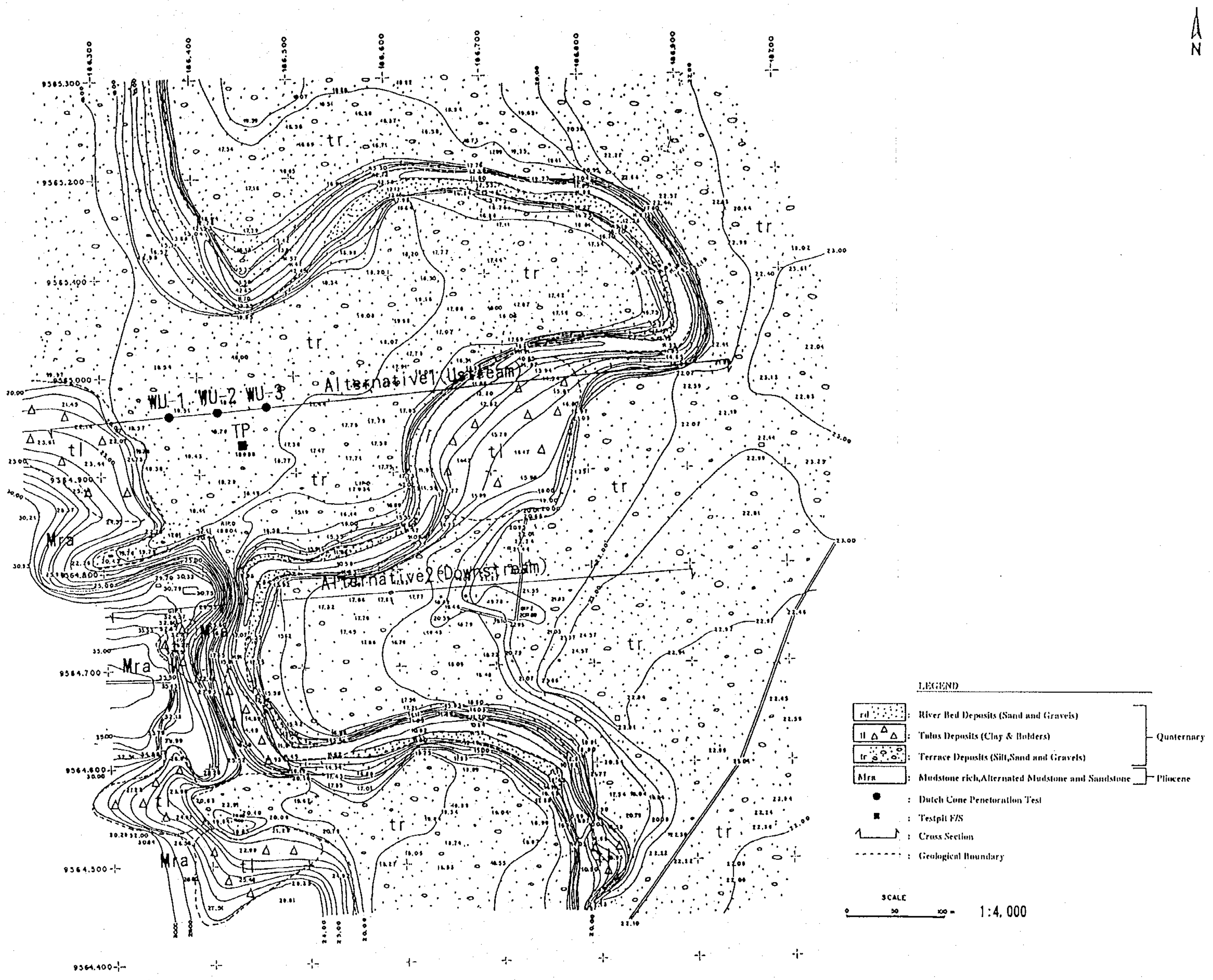
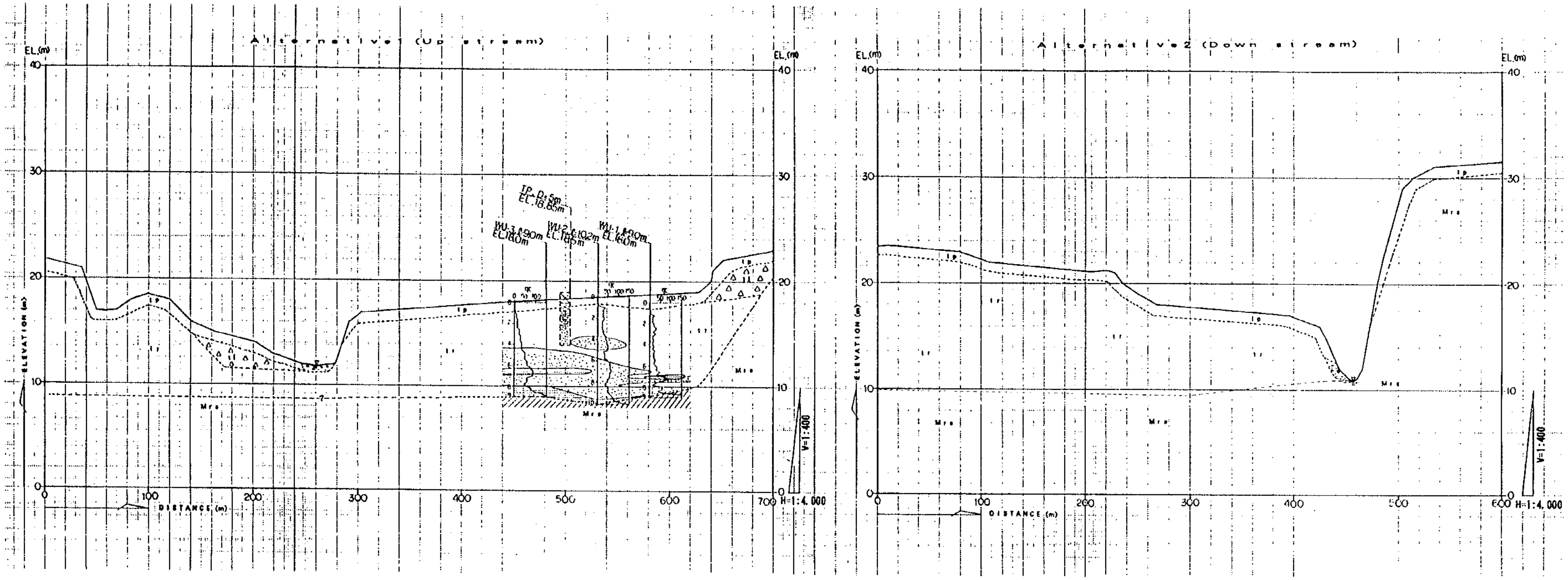


Figure A.1.9 Geological Map of Gilirang Intake Weir Site



LEGEND OF Dutch Cone Penetration Test

Symbol	qc (kgf/cm2)	Conversion N-Value
	qc < 40	N < 10
	40 ≦ qc < 80	10 ≦ N < 20
	80 ≦ qc < 120	20 ≦ N < 30
	120 ≦ qc	30 ≦ N

※: qc=4N (Meyerhof)

LEGEND

	: Top soil	Quaternary
rd	: River Bed Deposits (Sand and Gravels)	Quaternary
tl	: Talus Deposits (Clay & Bolders)	Quaternary
tr	: Terrace Deposits (Silt,Sand and Gruvels)	Quaternary
Mra	: Mudstone rich,Alternated Mudstone and Sandstone	Pliocene
- - - -	: Geological Boundary	

Figure A.1.10 Geological Profile of Gilirang Intake Weir Site

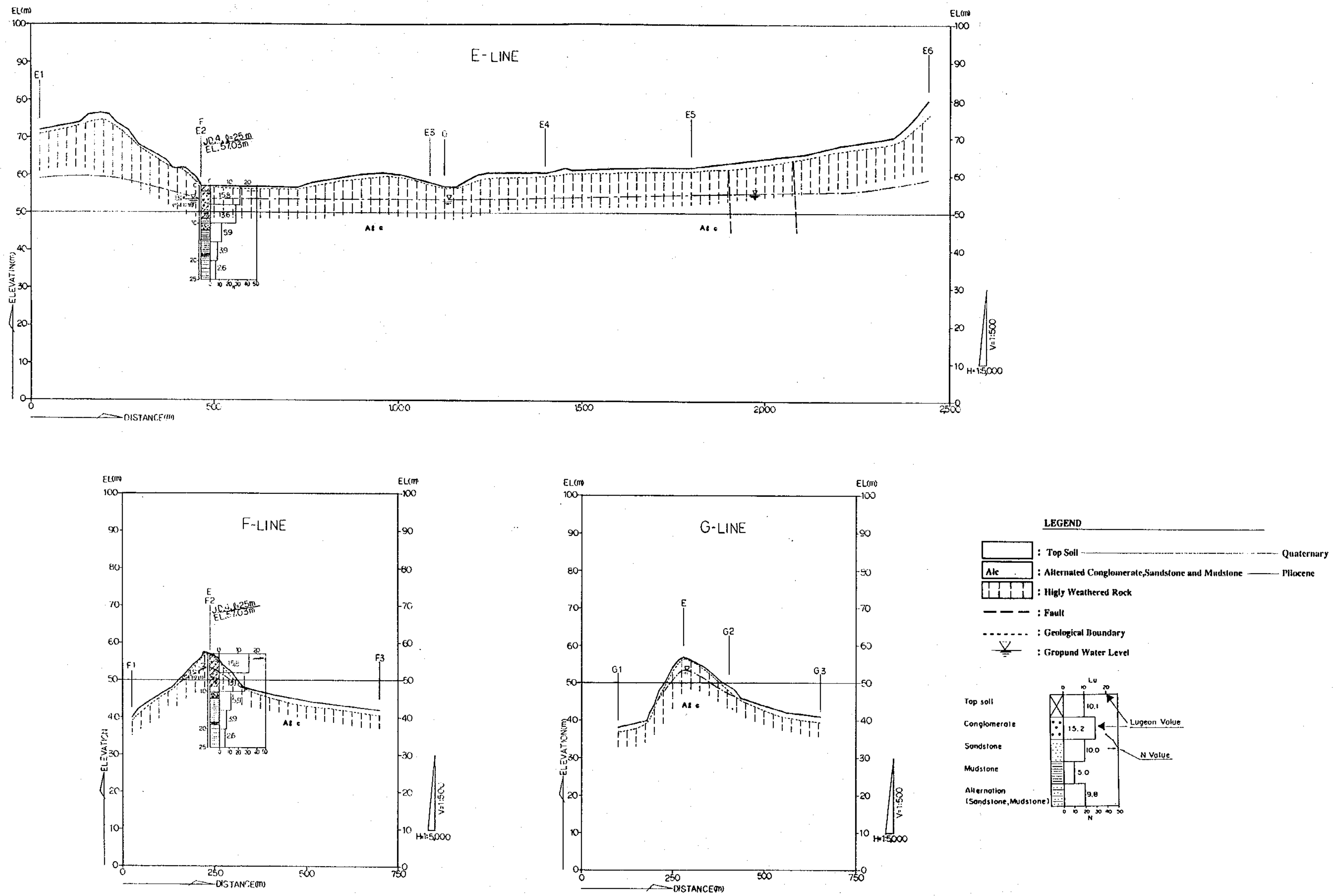


Figure A.1.11 Geological Profile of Narrow Ridge

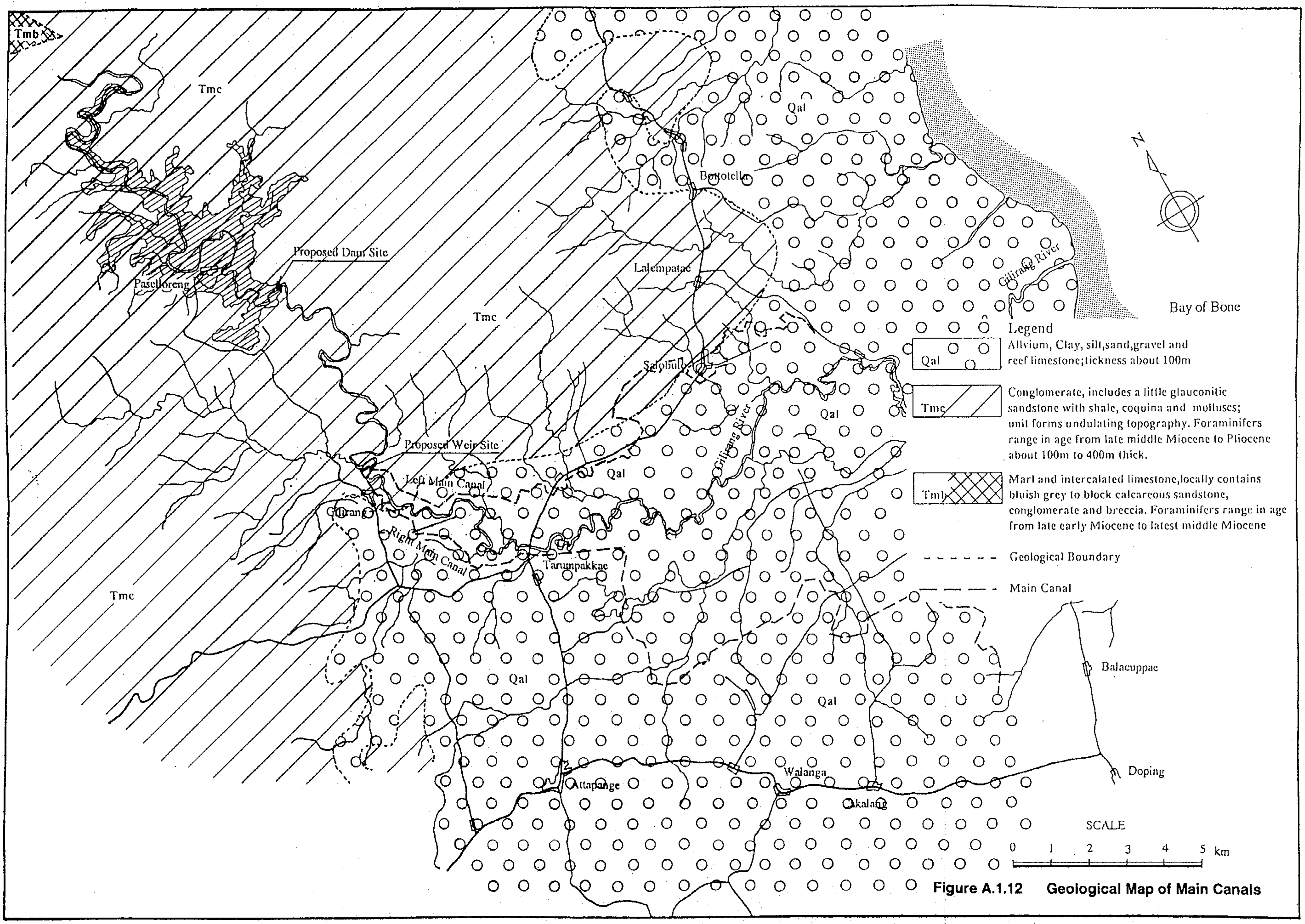


Figure A.1.12 Geological Map of Main Canals

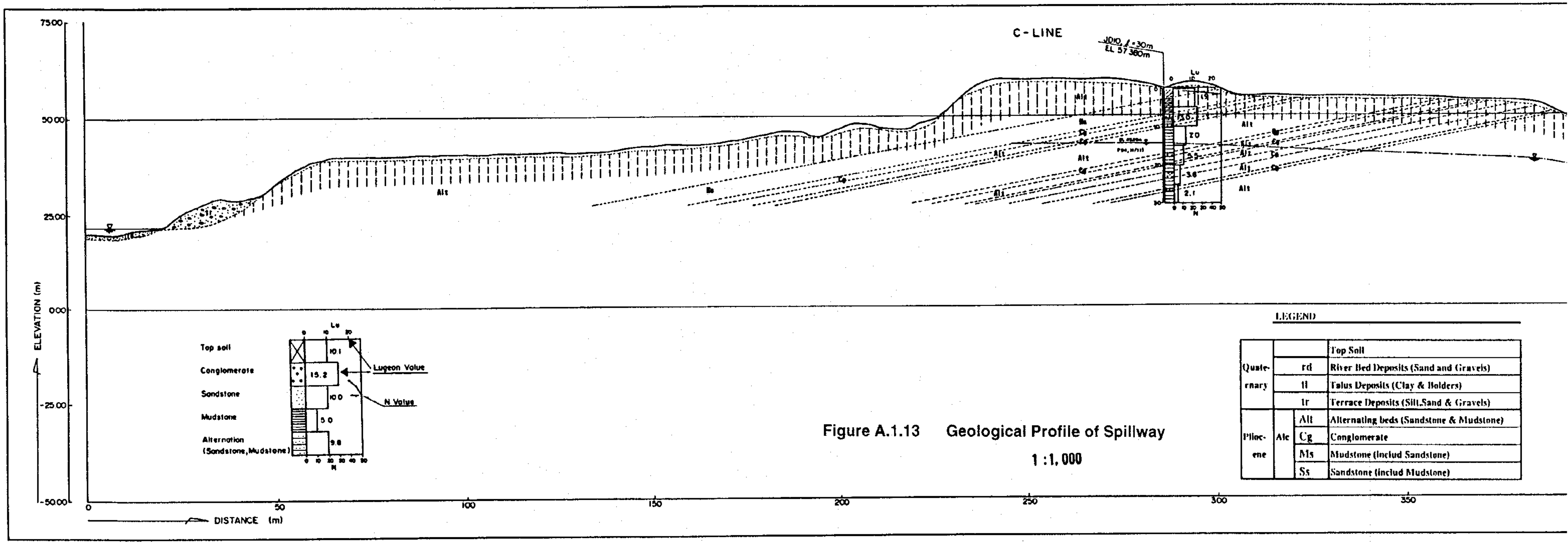


Figure A.1.13 Geological Profile of Spillway

1 :1, 000

LEGEND

			Top Soil
Quaternary		rd	River Bed Deposits (Sand and Gravels)
		tl	Talus Deposits (Clay & Bolders)
		tr	Terrace Deposits (Silt, Sand & Gravels)
Pliocene	Alc	Alt	Alternating beds (Sandstone & Mudstone)
		Cg	Conglomerate
		Ms	Mudstone (includ Sandstone)
		Ss	Sandstone (includ Mudstone)

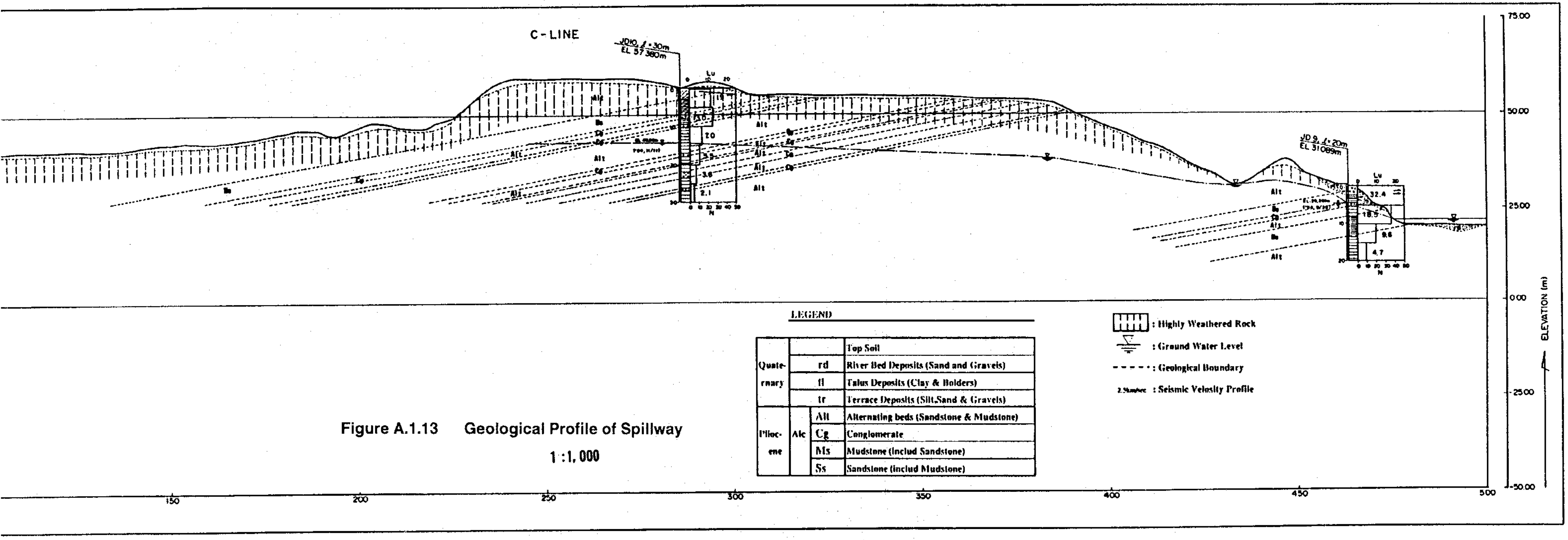


Figure A.1.13 Geological Profile of Spillway

1 : 1, 000

LEGEND

Quaternary			Top Soil
		rd	River Bed Deposits (Sand and Gravels)
		tl	Talus Deposits (Clay & Bolders)
		tr	Terrace Deposits (Silt,Sand & Gravels)
Pliocene	Alc	Alt	Alternating beds (Sandstone & Mudstone)
		Cg	Conglomerate
		Ms	Mudstone (includ Sandstone)
		Ss	Sandstone (includ Mudstone)

: Highly Weathered Rock

: Ground Water Level

: Geological Boundary

: Seismic Velocity Profile

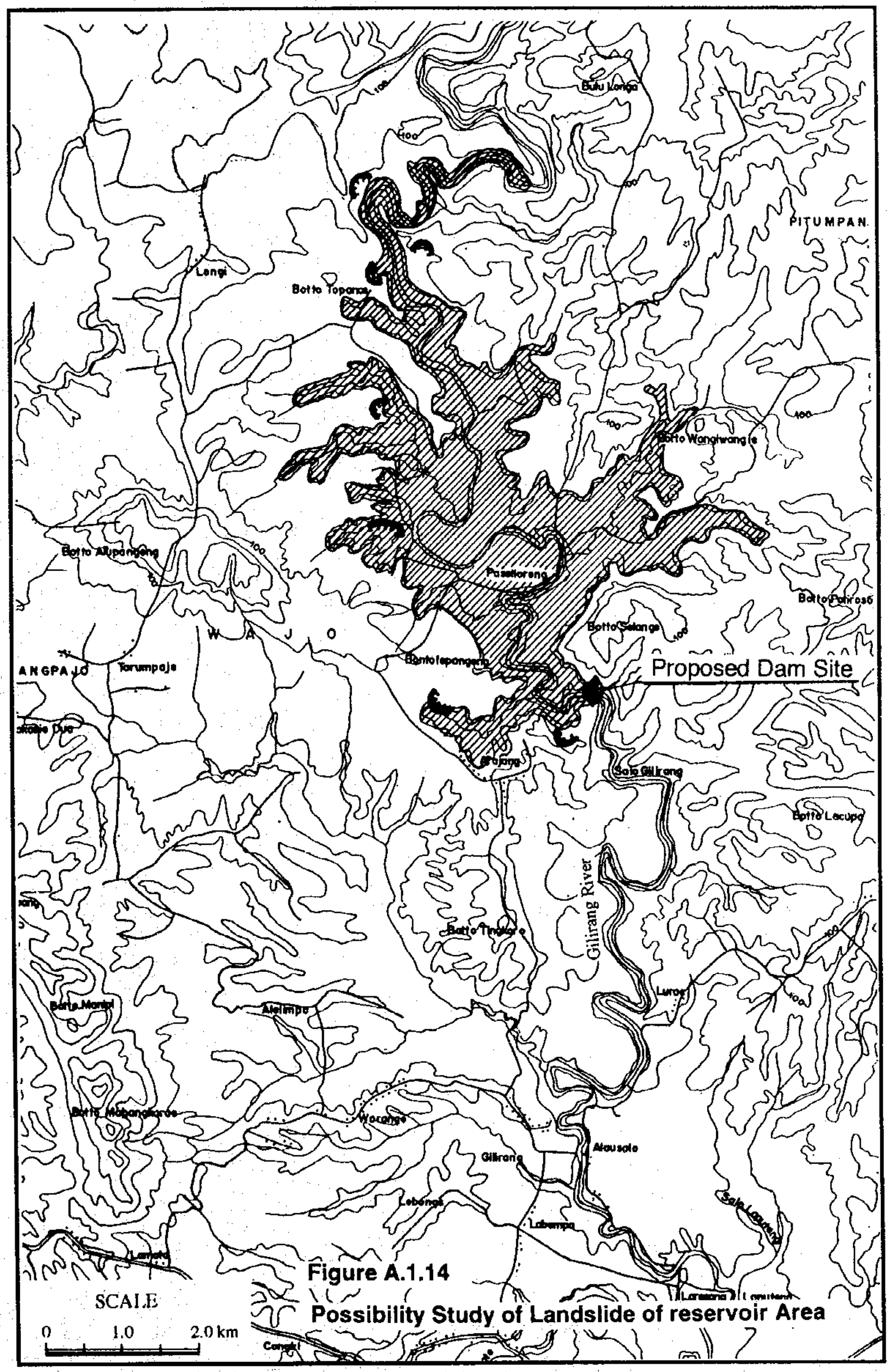


Figure A.1.14

Possibility Study of Landslide of reservoir Area

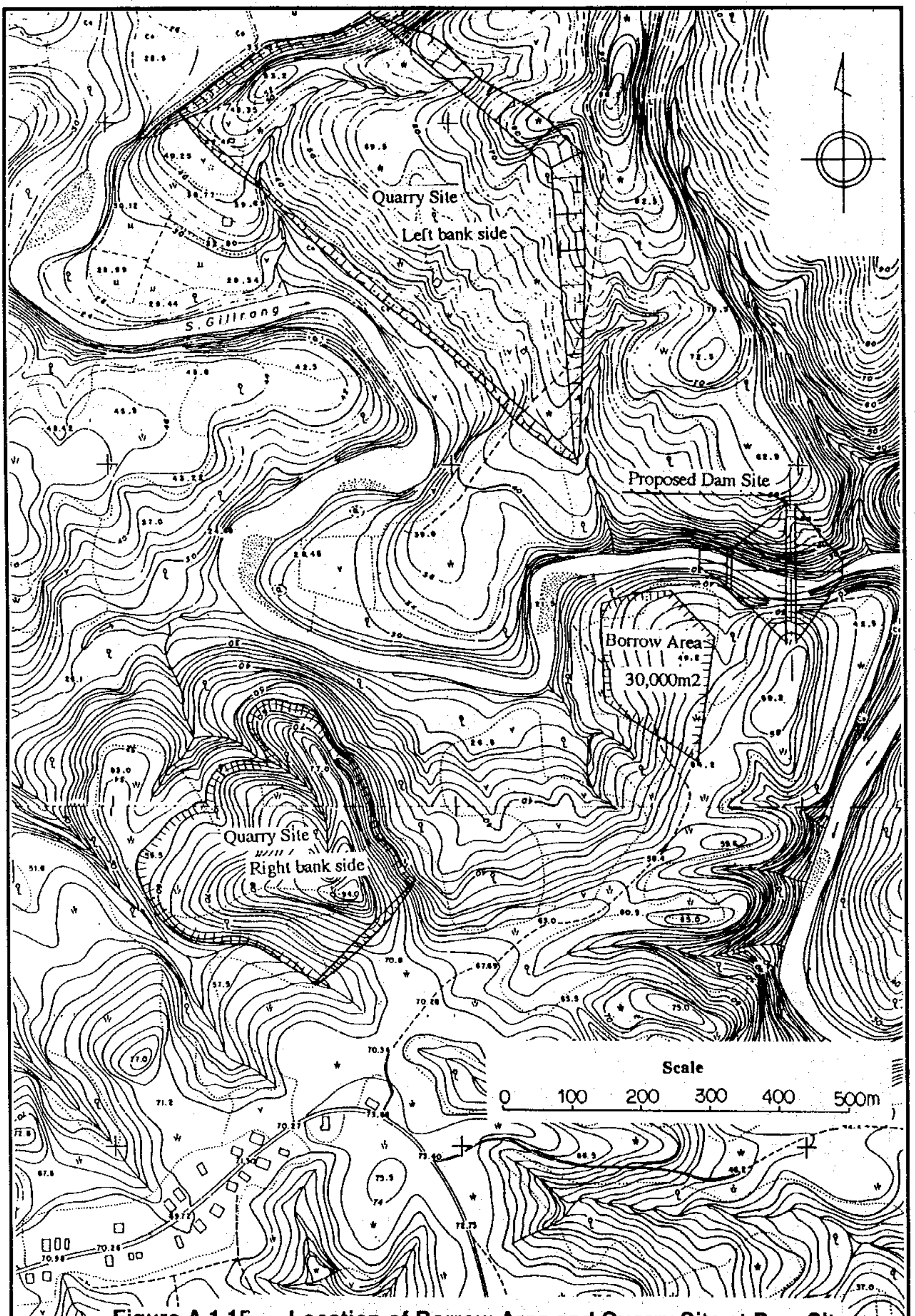


Figure A.1.15 Location of Borrow Area and Quarry Site at Dam Site