CHAPTER 6

GEOLOGY AND CONSTRUCTION MATERIALS

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6.1 Purposes of Investigation and Conclusions

6.1.1 Purposes and Particulars of Geological Investigation

The purposes of this investigation are geological survey and geological analyses necessary for further studying the safeties and carrying out technical and economical designing of civil structures regarding the Julumito Hydro-electric Power Project, while taking into account the Feasibility Study made in 1972.

With these purposes, the geological feasibilities are to be studied of the Julumito dam site, reservoir site, powerhouse site and appurtenant structures intake, headrace tunnel, surge tank, penstock, etc. and the two diversion waterway schemes Cauca and Palace-Blanco of the Julumito Hydro-electric Power Project selected in the Preliminary Study and the Feasibility Study, and also to investigate and test materials for dam construction.

In this Report are described the geological findings regarding the foundations and construction materials of the civil structures selected, including the results of the Feasibility Study carried out in 1972. The results of geological investigation works, permeability tests of foundations and tests of dam construction materials carried out this time by the Survey Team are also described. These data are given in Appendix-VII and Appendix-VIII.

This results of the geological investigations in the Feasibility Study of 1972 and the present Study are amply reflected in examination of the development scheme, layout of civil structures, and selection of designs.

6.1.2 Previous Investigations

Geological investigations regarding the Julumito Hydro-electric Power Project were started by ICEL and CEDELCA in 1969. Following this, an investigation was made by geologists of the Japanese Government in 1970. In 1971, geological surveys of sites planned for major structures were made through boring, test adit and test pit excavation, and investigations and tests of dam construction materials were carried out by ICEL and CEDELCA.

In 1972, a Feasibility Study was made by a Japanese Government survey team, and the results of these precedent surveys were confirmed by a geologist while further field reconnaissances were made. It was also recommended that still further geological surveys and tests of dam construction materials be made for the purpose of definite design.

The geological surveys based on these recommendations were carried out by the present

Survey Team and ICEL in 1979.

6.1.3 Conclusions

As a result of the various past geological investigations described in 6.1.2, and including the results of the field investigations made this time, the conclusions regarding the geology and dam construction materials of the Project are described below.

(1) It is judged that the basic development pattern Plan C of the Julumito Hydro-electric Power Project formulated in the Preliminary Study is the most superior from the results of the geological surveys also.

(2) Regarding the site for Julumito Dam, 3 locations had been considered as candidate sites at the Preliminary Study level, but taking into consideration geological survey data from surface reconnaissances and excavations such as boring, test adits and test pits, as a result of examinations of the geology, weathered condition, permeability and infiltration path lengths of the dam foundation, it was reconfirmed that the dam site selected (proposed dam site No. 2) is superior.

This Julumito Dam site is a broad one-way slope on the right-bank side, whereas the left-bank side is a scraggy ridge. Judged from the topography and the weathered condition of the dam foundation, it will be desirable for a fill dam with a inclined center core and a arch shape to be adopted on this dam site. The geology comprising the foundation of this dam is weathered andesite, and near the high water surface level, a volcanic ash deposit of low degree of consolidation, and careful engineering considerations will be required for excavation of the dam foundation and foundation treatment.

(3) The bedrock of the powerhouse site consists of tuffaceous breccia possessing compressive strength of 50 to 100 kg/cm^2 with ample bearing capacity as the foundation of a surface-type powerhouse, and it is thought there will be no problem from the standpoints of topography and geology.

(4) It is thought that leakage and sedimentation of the reservoir formed by Julumito Dam will be of degrees that can be ignored. (See 5.7, "Sedimentation")

(5) There are saddles at two locations in the reservoir area and it will be necessary to construct dikes (Dike No. 1, Dike No. 2) at these places. Regarding the site for Dike No. 1, as a result of reconnaissances this time, the height can be lowered although the length will increase at a site further upstream than the site proposed in the Feasibility Study of 1972. The results of Boreholes DDH-101 and DDH-102 show there will be no special problems with the dike site.

(6) The route of the pressure tunnel consists of andesitic lava. The penstock is to be provided chiseling a cliff of hard andesite lava. Since the andesite lava has parts where there are many cracks, in construction of the headrace tunnel, spalling and cave-ins must be prevented, while thorough preparations will be necessary against springing of water. It should be possible for anchor blocks and saddles of the penstock to be provided on bed-rock possessing ample bearing capacity, but there are portions requiring attention to be paid to slope stability after excavation.

(7) The foundations for the diversion dams of the Rio Cauca and Rio Palace diversion schemes are both unconsolidated ground strata. Although both of the dams will be of low height, the foundations should be selected at adequately consolidated strata, while suitable foundation treatment will be necessary. The Palace and Blanco waterways are planned to be tunnels, while the greater part of Cauca Waterway is to be an open canal. Although the geology of the route of this waterway consists of unconsolidated strata of river terrace deposits and volcanic ash deposits, the topography is that of a hilly area of gentle relief, and under normal conditions it is inconceivable that the waterway would be destroyed by occurrence of landslides. Tunnels will be driven through volcanic ash deposits and adequate cutting will be required in the vicinities of portals. What will be of importance is that careful placement of concrete must be done to prevent uneven settlement of open canal and tunnel foundations and sliding to the sides of waterways due to leakage from the waterways.

(8) It is anticipated that all dam construction materials can be collected from the vicinities of civil structure sites. Volcanic ash deposits will mainly be used as soil material, and these deposits are special soils which are fine-grained, and moreover, of high water contents. Consequently, the design mentioned in (3) has been proposed, but detail design and construction must be carried out on completely grasping the characteristics through further appropriate testing. It is proposed that tests be performed to obtain a core material of good workability using volcanic ash, weathered andesite and weathered residual andesitic soil on blending to adjust grain-size distribution and water content.

For rock material, filter material and concrete aggregates, the plan is to collect them from the mountain body where andesitic lava is distributed. As the candidate for the quarry site, the mountain block approximately 500 m downstream of the powerhouse site on the right-bank of the Rio Cauca was selected in consideration of transportation and other conditions. Although it is thought the weathered layer will be somewhat thick, the conditions for quarrying should be better than at the alternative (Rio Cauca left-bank).

It is thought that excavation muck from the penstock route and the pressure tunnel can also be used as a part of the dam construction materials.

6.2 General Geology of Project Area

6.2.1 Outline of Topography and Geology

(1) Topography

The Andes Mountain System which comprises the western rim of the South American continent stretches north and south with its highest point in Peru, the extension to the north decreasing in elevation as it goes through the central part of Ecuador to reach into Columbia.

The Andes Mountain System on entering Columbian territory is divided into the following three mountain ranges:

(a) Cordillera Oriental

(b) Cordillera Central

(c) Cordillera Occidental

Of these mountain ranges, the highest one is the Cordillera Central which has an average elevation close to 3,000 m, and moreover, has several peaks exceeding 5,500 m above sea level, while the length north-south is 800 km. Between the Cordillera Central and the Cordillera Occidental, the Rio Cauca with a length of more than 1,500 km flows from south to north and the catchment basin comprises one large plain with a width of 60 to 80 km.

The Rio Magdalena flows north between the Cordillera Central and the Cordillera Oriental, and merges with the Rio Cauca in the northern part of Columbia to feed the Caribbean Sea.

The plains extending to the east from the Cordillera Oriental are the sources of the Rio Amazon and the Rio Orinoco and comprise vast llanos grasslands and a wide forest area.

The Julumito Hydro-electric Power Project area is centered at a point approximately 10 km northwest of Popayan City in the Upper Rio Cauca Valley and is located where the Rio Cauca flows out to a table-like hill area from the gentle foothills of the western side of the Cordillera Central. The vicinity of Popayan is a hill area with a flat surface of elevation of about 1,750 m, this surface being gently sloped to the west, dissection not being very advanced. The river system in this area indicates a distinct pattern of parallel flows from east to west of tributaries such as the Rio Sate and Rio Palace.

The Rio Cauca comprises a broad flood plain in the vicinity of Popayan where it meanders gently and there are river terraces in the form of four stages at the west bank.

The Rio Cauca passes the northwestern part of Popayan City and the river gradient is gentle to the outskirts, but further downstream the river gradient becomes steep and there are continuations of gorges of 50 to 80 mm width and about 100 m height.

(2) Geology

Topographically, Colombia may be broadly divided into the Llanos Plain at the eastern half which comprises the fountainheads of the Rio Amazon and the Rio Orinoco, and the Andes Mountain System consisting of the three mountain ranges in the western half.

The former Llanos Plain corresponds to the northwestern fringe of the Guayana Shield which extends from Colombia to Brazil and presents a compratively simple geological structure.

The latter Andes Mountain System consists of sedimentary rocks from the Devonian Period to the Tertiary Period, and igneous rocks and metamorphic rocks. This mountain system is a fold system formed by the Andes orogenic movement of the Tertiary Period, is roughly parallel to the western fringe of the Guayana Shield, and extends more or less in a NNE-SSW direction.

The geology of the vicinity of Popayan City comprising the project area consists in most part of volcanic products called Popayan Formation and Rio Cauca Formation. The geology comprising the basement of the area is that of metamorphic rocks said to be a Mesozoic Group and basic igneous rocks. Since this Mesozoic Group is thickly covered (roughly more than 300 m) by volcanic products, it is seen only scatteredly where the Popayan Formation has been deeply eroded by the Rio Cauca and other rivers. As examples, the site of Florida II Power Station completed in 1975 on the Rio Vinagure, a tributary of the Rio Cauca, and the vicinities of diversion dams may be cited. The former is metamorphic rock consisting of green schist while the latter consists of basic igneous rocks of schalstein and diabase. The geological age periods of these strata were estimated as shown in Table 6-1 from the data which it was possible to gather.

The civil structures of this Project will all be provided at the Popayan Formation and the Rio Cauca Formation.

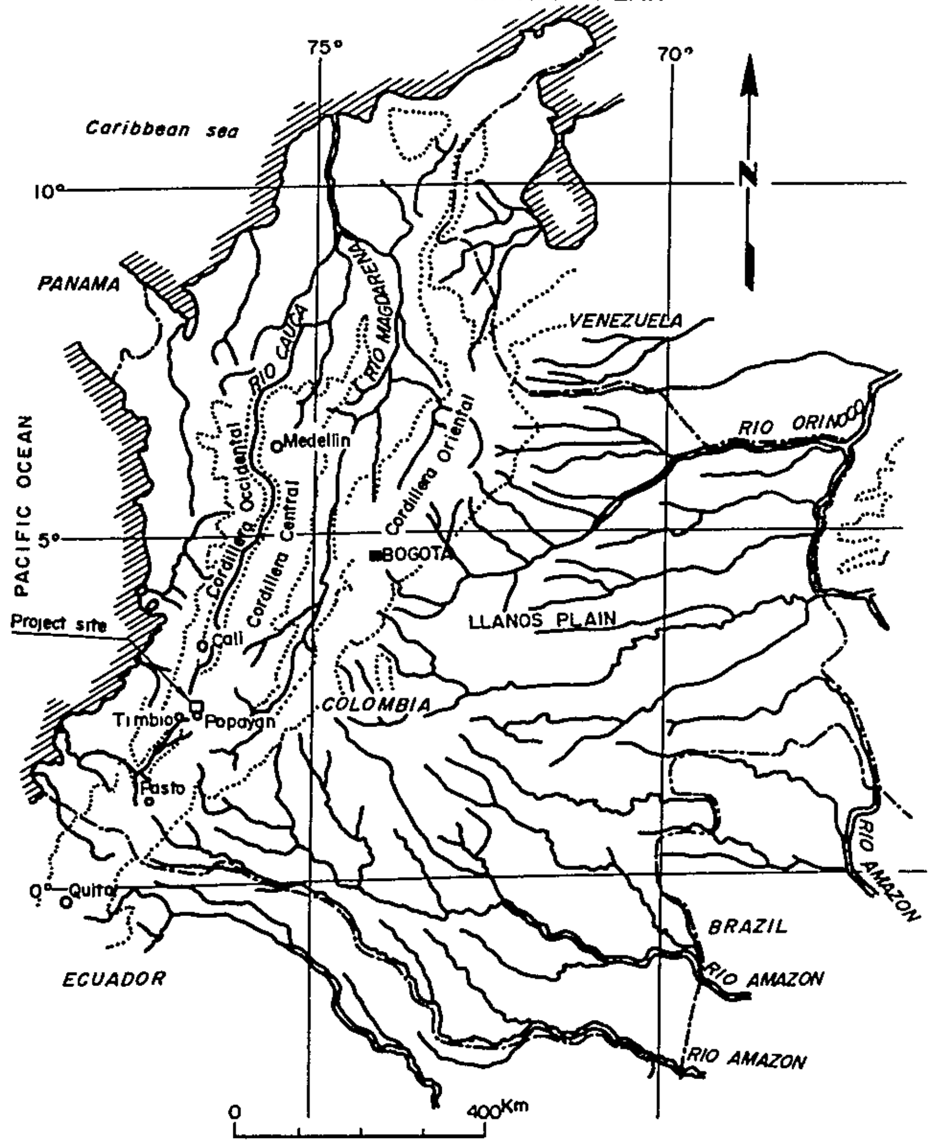
The relations between the foundations of civil structures and rock-facies, strata are roughly as shown in Table 6-2.

6.2.2 Stratigraphy and Rock Type

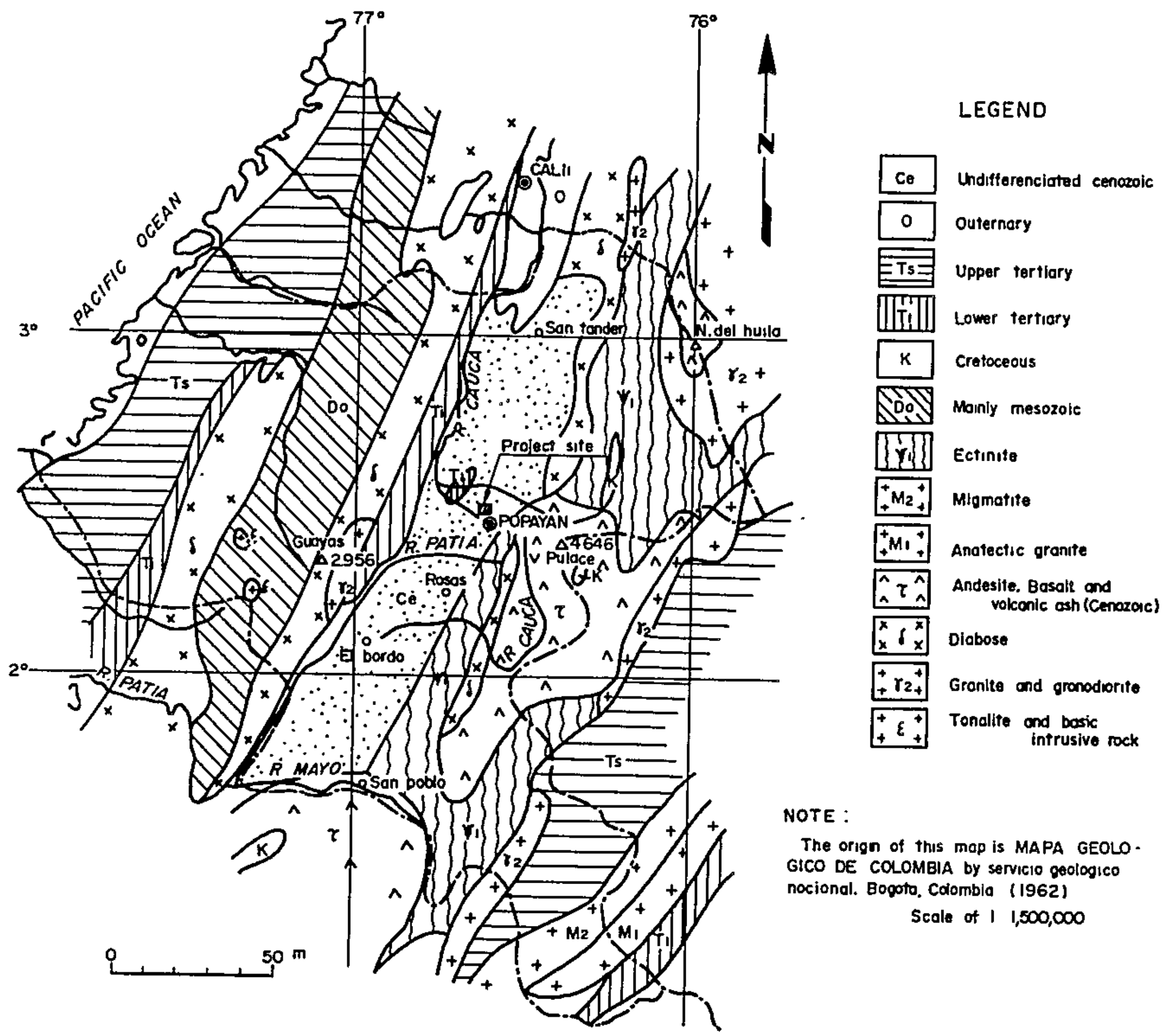
The outlines of stratigraphy and rock types comprising the Julumito Hydro-electric Power Project area are given in Table 6-2.

Fig. 6-1 General Map of Topography and Geology in Cauca District

TOPOGRAPHIC PLAN



GEOLOGIC PLAN



The characteristics of the various rock types and the evaluations as foundation rocks and materials are as described below.

(1) Tuff Breccia

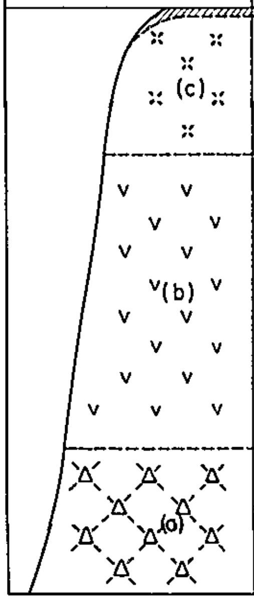
Table 6-1 Generalized Geologic Chronology in the Julumito Project Area.

Period or era	Stage	Formation	Rock
Quaternary	Pleistocene	Popayan formation	Volcanic ash. Unconformity *1
Neogene to Quaternary	Plio-pleistocene		Andesite lava Conformity *2
Neogene	Pliocene	Rio Cauca formation *3	Tuff breccia Unconformity
Mesozoic		Dagua formation	Metamorphic rocks; green schist and graphite schist. Basic igneous rocks diabase, dolerite and so on.

* 1 : Slight recognition of hiatus (time gap of sequence seen as unconformity).

* 2 : Between the tuff breccia (Rio Cauca Formation) and andesite lava (Popayan Formation) there are no facts in the survey area which parricularly indicate unconformity such as basal conglomerate.
Rather, it appears that tuff breccia is of the same geologic age as andesite lava.

Table 6-2 Outline of Stratigraphic Sequence and Rock Type of Foundation Rock in Civil Structures on Julumito Project

Diagramatic column	Rock type	Thickness※1 (m)	Distribution
	Top soil	Less than 10	Wide spread
(c)	Volcanic ash	30 to 40	Wide spread Dam site and reservoir area, etc.
(b)	Andesite lava	about 120	Dam site, headrace tunnel route, penstock line. tributary diversion waterway route, etc
(a)	Tuff breccia		Powerhouse site and vicinity of it.

※1 . data originated from boring DH-I on the left bank of the julumito powerhouse site

This is the stratum situated at the bottom of the geology comprising the Julumito Hydro-electric Power Project area.

This is exposed at the bottom of the valley where the overlying andesite lava is deeply eroded by the Rio Cauca from the outskirts of Popayan City to the downstream area. Also, according to Borehole DH-1 (vertical, length 160 m) drilled at the left-bank side of the Julumito Dam site in order to investigate the feasibility of an underground powerhouse scheme, the existence of tuff breccia was recognized at a depth of 143.0 m (El. 1,585.6 m). From the previously-mentioned outcrops and boring data the boundary line between the tuff breccia and the andesite lava is at around El. 1,585 m and is estimated to be more or less horizontal. The Julumito powerhouse construction site planned on the right-bank of the Rio Cauca consists of this rock. This tuff breccia is green-gray, includes volcanic blocks of diameters 3 to 5 cm, is dacitic, and is a soft rock easily crushed with hammer blows. However, unconfined compressive strengths are from 50 to 100 kg/cm^2 which are adequate for the foundation of a surface-type powerhouse structure.

(2) Andesite Lava Formation

This andesite lava is a stratum widely and thickly distributed in the Popayan City area and most of the foundations of the dams, headrace tunnel, penstock, etc. of the Julumito Hydro-electric Power Project are designed to be located in this andesite lava. This formation shows typical outcrops of cliffs on both banks deeply eroded by the Rio

Cauca near the proejcted powerhouse site. At the outcrops there are several series of prominent cooling joints (= columnar joints) indicating that lava flows had occurred on several occasions.

According to the data of Borehole DH-1 drilled for the purpose of investigation of an underground powerhouse scheme, the andesite lava has a thickness of approximately 120 m, and moreover, may be divided into four layers. The three layers from the bottom are of identical rock characters, but the lava at the top is of slightly different nature. In effect, the lower three strata consist of gray-purple andesite with prominent phenocrysts of feldspar, and columnar joints are developed in each of these strata. In contrast, the top stratum presents a gray-blue color, includes biotite and crystals of quartz, and indicates a rock character close to liparite.

Between the layers of andesite lava there are intercalated weathered zones of 3 to 4 m thickness and thin layers of tuff from which the differences in ejection times can be recognized. The petrographic descriptions and physical characteristics of these lava layers are described in detail in 6.3.2 (1) b) (vi).

This andesite layer is thickly covered by a volcanic ash layer, but the configuration of the surface is surprisingly flat. In other words, according to Boreholes DH-1, DH-2, DH-3 and DH-5 drilled for the purpose of geological investigation of the headrace tunnel route, the surfaces of the andesite were confirmed to be at the elevations of 1,703.2 m, 1,696.8 m, 1,702.0 m and 1,704.3 m, or roughly at 1,700 m. However, on tracing the range of distribution of volcanic ash described below, it is seen to be deposited in uniform thickness at the slopes of both banks far lower than the elevation of 1,700 m, not only on the Rio Cauca, but also tributaries such as the Rio Sate and the Rio Palace. This indicates that after ejection of the andesite lava and until deposition of volcanic ash, there was erosion occurring to present a topography extremely similar to the present one.

This geological phenomenon, as described in 6.3.2 and 6.5.2, has an important bearing on foundation treatment of the weathered andesite lava at the dam site and in selection of quarry site for rock materials.

(3) Volcanic Ash Deposit

This volcanic ash is widely distributed south from Cali City to the vicinity of Timbio City and is clearly an ash fall deposit which at the Julumito Hydro-electric Power Project area has a thickness of 30 to 40 m. The bottom part is fairly solidified and there are parts which are thought could be called tuff. As the volcanic ash was something that fell down from the air, it was deposited more or less parallel to the topography existing before deposition of the ash, and although at rivers and gullies where carrying capacities and eroding actions are prominent the thickness of the ash layer has been reduced, it has remained more or less in the same thickness at hill districts. These ash layers have been subjected to unique weathering from the ground surface and present a peculiar soil profile. This will be described in detail in the section on the volcanic ash layer at the dam site (6.3.2 (1) b) (v)). This volcanic ash layer presents a brownish-white to a milky-white color where fresh and the grain size is so fine that more than 90% passes a #200-mesh sieve. In visual observation, fine grains of quartz and scaly biotite are prominent, and the material is a liparitic volcanic ash.

6.2.3 Geological Structure

The geological structure of Colombia governed by the Andes orogenic movement shows a NNE-SWW direction to be very predominant. The Andes Mountain System also runs in the direction of the geological structure, and the Rio Cauca and the Rio Magdalena which flow north originating from the mountain system also follow the geological structure. As stated in 6.2.1 (1), "Topography," the Colombian Andes are divided into the three mountain ranges of Cordillera Oriental, Cordillera Central and Cordillera Occidental. As the ridges of the respective mountain ranges mostly correspond to anticlinal parts, old rocks and formations are distributed, while on the other hand, the valleys, since they are synclinal parts, are occupied by young formations of the Cenozoic Era.

The Colombian Andes may be broadly divided into the following two geological provinces.

(1) Cordillera Oriental Geological Province

This is mostly made up of sedimentary rocks and there is little new volcanic rock. The mountainsides of the mountain system and valleys (synclines) have distributions of Tertiary and Mesozoic formations, while the ridges (anticlines) have distributions of Paleozoic formations in bands along the axis in the NNE-SWW direction, but Cretaceous marine deposits are most widely distributed.

This geological province differs in geology from the main Andes Mountain System of Chile, Bolivia, Peru and Ecuador in which igneous rocks and metamorphic rocks are predominant.

(2) Cordillera Central and Cordillera Occidental Geological Province

This mainly consists of igneous and metamorphic rocks which are accompanied by sedimentary rocks. These rocks and formations comprise a northward extension of the main Andes Mountain System in Ecuador and to the south, with the northern end dropping down to the northern coastal lowland near El Blanco. Volcanoes are distributed along the direction of the mountain system and were quite active in the Mesozoic and Cenozoic eras, and some are still active today. Active volcanoes are particularly prominent in the southern part of the Cordillera Central.

Similarly to the Cordillera Oriental geological province, there are distributions of Paleozoic rocks such as acidic and basic plutonic rocks and gneisses at the ridges of the mountain ranges, while in the valleys there are distributions of formations and rocks of the Cenozoic Era, and in between them, distributions of Mesozoic formations in bands in the direction of the axis of folding.

The Julumito Hydro-electric Power Project area has andesite lava as its basal rock which is covered thickly by volcanic ash, the boundary appearing to be more or less flat. The results of investigations show there are almost no faults in the project area.

6.2.4 Earthquakes

Colombia is situated in part of the Circum-Pacific Earthquake Belt and active seismicity is recognizable. This seismic activity is limited mostly to the Andes Mountain System and the Pacific Ocean coast. Earthquakes are rare in the eastern llanos close to Venezuela and Brazil, and any that do occur are not severe and this part of the country may be considered as a non-earthquake zone. According to the records of past earthquakes, the locations of epicenters of earthquakes in Colombia may be broadly divided into the following four zones:

Zone-1	Southern part of Cordillera Central ... Vicinity of border with Ecuador centered at Pasto City.
Zone-2	Northern part of Cordillera Oriental ... North Departamento de Santander and near Tachira State (Venezuela).
Zone-3	Central and northern part of Cordillera Occidental and Pacific Ocean coast.
Zone-4	Scattered in northern and eastern parts of Colombia.

These earthquake records are given in Appendix IX. Of these, those marked with white circles are earthquakes which occurred between 1566 and 1900 and not measured with seismographs, and the scales and epicenters of these were estimated from various records. Regarding the records from 1900, geophysical judgments were made based on data from measurements in Columbia and at earthquake observation organs throughout the world. These earthquakes are classified according to intensities as shown in Table 6-3.

Table 6-3 Classification of Seismic Intensity in Colombia

Intensity	Degree	Correlation with Scale of Rossi-Forel	Acceleration - Part of gravity
I	Slight	I. II. III. IV.	0~0.03 g (0~30 gal)
II	Strong	V. VI. VII	0.03~0.05 g (30~50 gal)
III	Destructive	VII. IX. X	0.05g ~ (50 gal ~)

The Julumito Hydro-electric Power Project area is outside the four earthquake zones of Colombia mentioned above. In other words, it is 150 to 200 km distant from the before-mentioned Zones 1 and 2 and may be said to be an area with little earthquake occurrence for the Andes Mountain System. The occurrences of earthquakes around the project area based on past records are shown in Table 6-4.

Table 6-4 Earthquake around the Julumito Project Area

Epicenter	Intensity	Frequency	Occurred year
Within 50 Km*1	II	2	1751 and 1878
	III	2	1735 and 1736
Within 100 Km*2	I	2	1566 and 1946 (two times Mar. 29 and 30)
	II	1	1827

* 1 : Centering around popayan city

* 2 : Excluding * 1

Of these earthquakes, there is only one earthquake of Intensity I measured with seismographs, all the others having occurred prior to 1900. As shown in Appendix-IX, earthquakes actually measured have been concentrated in the four seismic zones, and around Popayan City there has been only one earthquake measured, indicating that old earthquakes not measured have been greater in number. In judging earthquakes from old documents, the records tend to be for areas of population concentrations and in many cases cannot be handled as universally applicable data, while it is extremely difficult to determine the locations of epicenters.

Consequently, in determining the probability of future earthquakes in the Julumito Hydro-electric Power Project area, whether to make judgments based on earthquakes from 1900 and after, or to also take earlier earthquakes into consideration poses a problem.

Naturally, the fact that an earthquake occurred in the past indicates the possibility of earthquakes occurring at the same site in the future. In discussing earthquakes directly affecting the Project, only those earthquakes for which there are distinct proofs of epicentral distances and intensities should be considered.

In an extremely cautions standpoint is taken here and records of all earthquakes are taken into consideration, the following may be said. From 1560 to the present, there have been two earthquakes of Intensity III in the area within 50 km of the Julumito Hydro-electric Power Project area. This is a frequency of once in 200 years. In the area within 100 km, there were three earthquakes of Intensity III, which is a rate of once in 130 years. From the above, it is thought there is a possibility of earthquake of Intensity III occurring once in 100 years at the project area.

6.3 Geologies of Major Project Sites

6.3.1 Catchment Basin and Reservoir

(1) Topography

The Julumito reservoir area presents a gently undulated topography and except for the valley bottom is uniformly covered by a thick volcanic ash layer. Only the mainstream of the Rio Sate is eroded down to the andesite lava underlying the volcanic ash layer.

The original topography before deposition of the volcanic ash appears to have had few undulations with a river system pattern of parallel flows, and to have been similar to the present topography. It is thought that after uniform settling of the volcanic ash in a thickness of 30 - 40 m over the original topography, the Rio Sate again eroded approximately the same location to produce the present topography. This can easily be surmised from the condition of deposition of volcanic ash at the upstream and downstream parts of not only the Rio Sate, but also of the Rio Cauca.

From the topographical features, the reservoir area may be broadly divided into the following two districts:

a) Upstream District

In this district, the Rio Sate and its tributaries have parallel channels which indicate very young erosion stages cutting down in the direction of gravity along the surface existing at that time after deposition of the volcanic ash. At these valleys the upstream catchment areas are small and on top of the fact that the eroding agent of the water is small, since the component geology is a soft, isotropically homogeneous formation, the gullies run in the direction of slope and only deepening action has occurred. As a result, the drainage pattern of adjacent gullies parallel to each other at intervals of roughly 500 m was formed. All of the valleys have gently sloping sides and parts of the flat initial plane remain between the adjacent gullies.

In the future, headward erosion should progress in the catchment area, but since the area does not suffer heavily concentrated rainfall, the problem of sedimentation in Julumito Reservoir cannot be thought to be significant.

(b) Middle Stream and Dam Site District

This district corresponds to the valleys of the original topography (old Rio Sate) and the widths of the valleys are far smaller than in the upstream district. The cutting-down action has eroded the weathered surface layer of the andesite lava and comparatively hard rock is exposed at the valley bottoms. The flow channels have not been subjected to much lateral erosion and there are narrow flood plains and river terraces formed in the channels.

The various tributaries of the Rio Sate which merge in this district have small discharges, and since cutting-down action is weak, the erosion has not reached down to the andesite lava and at the confluence with the Rio Sate a small-scale hanging valley is formed.

In the reservoir area the condition calling for the greatest attention is that there are two places (saddles) at part of the reservoir which are lower than the proposed high water level. The locations are to the southwest of Julumito Village where tributaries respectively of the Rio Sate and the Rio Cauca have eroded the mountain between the two rivers. The construction of dikes is necessary for this topography, details of which are given in 6.3.2.

The original ground around the reservoir other than at the above-mentioned dike sites consists of an impermeable volcanic ash layer and since the path lengths to other catchment basins are great, it is thought there is no fear of leakage from the reservoir.

(2) Geology

The greater part of the reservoir area consists of a more or less uniform ash layer, but andesite lava is distributed at the valley bottom at the dam site and upstream. Regarding the volcanic ash layer, according to the results of permeability tests conducted at the dam site and soil material borrow areas, the coefficient of permeability indicates values in the order of 10^{-4} cm/sec to 10^{-5} cm/sec on top of which there is a tendency for the permeability coefficient to be even smaller when saturated with water.

Regarding the andesite lava, according to lugeon tests carried out utilizing boreholes at the dam site, lugeon value is less than 1. Consequently, it is thought there will be no leakage from the basal rock of the reservoir.

The design high water surface level (El. 1,715 m) is in the horizon of the volcanic ash layer where the slope of a hill area of gentle relief is presented. The slope is covered with topsoil of 50 to 100 cm thickness on which shrubs and weeds thickly grow. Because of such a topography and geology it is not conceivable that large-scale landsliding will occur due to impounding of water.

6.3.2 Julumito Dam and Dikes

(1) Julumito Dam

a) Topography

The Rio Sate on which the dam is to be located flows in a roughly straight line to the northwest at the main dam site and its vicinity. The river channel comprises the form of a trough of width of 7 - 10 m and height of 3 - 5 m and the two banks extending up from the channel comprise relatively steep slopes of 35° - 40° up to around elevation of 1,700 m and are covered by tall latifoliate trees. From around El. 1,700 m and above, the slopes become gentle and continue on to the Popayan Tableland of elevation of 1,730 - 1,750 m and is covered with growths of short shrubs and grass.

On the right-bank side, there are gullies upstream and downstream of the dam axis and the mountain bulk between the gullies has few creases and is of a size that the dam will be conveniently accommodated. In contrast, on the left-bank side immediately upstream of the dam axis, there is a comparatively large gully cutting deeply in to El. 1,690 m from the confluence with the mainstream. There is a series of samll waterfalls in this section. Meanwhile at a point 180 m downstream from the dam axis, the mainstream bends sharply and the mountainside on the left-bank is eroded by the gully merging at this bend and by the gully upstream of the dam and this has become a scraggy ridge. This ridge, when overburden and weathered layers are removed, is expected will become even smaller. Therefore, it is desirable for the dam axis at the left-bank to be located at the upstream gully considering the geological conditions described below.

b) Geology

The geology of the dam site besides consisting of andesite lava and volcanic ash, also consists of talus, river terraces, deposits at the present reverbed and topsoil covering these layers.

The geology comprising the dam foundation is almost entirely andesite lava and as a whole has been subjected to strong weathering action with parts close to the ground surface having become residual soil, while underneath, weathering has extended to a considerable depth. The volcanic ash overlying the andesite lava is also weathered from the ground surface and a peculiar weathered profile is presented.

(i) Topsoil

The topsoil is humus of the vegetation growing on the ground surface, is blackish-brown in color, high in water content, and clayey. This, of course, must be removed for the dam foundation.

(ii) Talus Deposits

Distributions are seen at the gully on the right-bank at the upstream end of the dam, the dam foundation at the downstream side of the dam at the left-bank, and the left-bank side of the downstream end of the dam. These deposits are sandy silt and silty clay containinig boulders of weathered andesite here and there. The thicknesses are 2 to 3 m, and since they are unsolidified, they should be excavated and removed.

(iii) River-bed Deposits

Besides at the bottom of the Rio Sate, deposits can be seen as narrow shoals on the inside of the bent portion. There is a high content of cobbles of weathered andesite and andesite of diameters of 20 to 40 cm and the deposits are densely compacted with sandy silt. According to the data of Borehole DH-203 drilled near the riverbed at the left-bank part of the dam axis, this riverbed deposit has a thickness of approximately 5 m and the elevation of this boundary is 1,637.40 m. However, the thickness of the riverbed deposits at the center line of the stream and the right-bank, and the variations in the thickness in the upstream-downstream direction are unknown.

These deposits have high permeability for the foundation of the core embankment of the dam and since their improvement is difficult, they should be excavated and removed. Also, the surface layer portions of these deposits have some places which are insecure to some extent in regard to bearing power for the dam and there should be a need for considerations to excavate and remove these portions in accordance with the vertical load of the dam.

(iv) River Terrace Deposits

These layers from flat bands about 10 m wide at both banks near the upstream end of the dam, the left-bank at the dam axis and the right-bank at the downstream end of the dam. These terraces are located 2 to 3 m higher than the present riverbed and can also be considered as flood plains of the present river. The terrace deposits are unsolidified sand-gravel of high silt content, and as the thicknesses are extremely small (1 to 2 m), they should be removed at the dam foundation.

(v) Volcanic Ash Layer

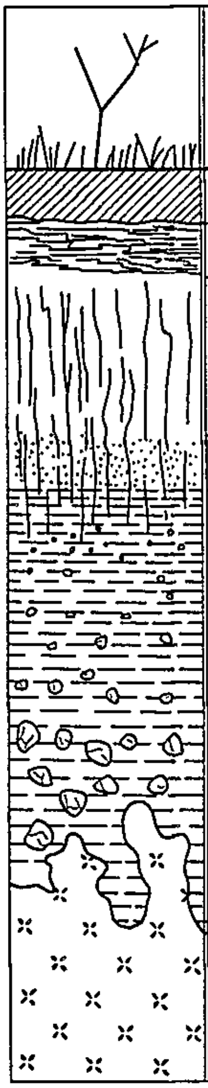
The volcanic ash layers at the dam site are distributed at El. 1,670 m and higher and become thicker the higher the elevation. The thickness in the vicinity of the crest at either bank is 15 m on the right-bank and 18 m on the left-bank. These volcanic ash layers have produced a peculiar soil profile due to weathering from the ground surface. Since elevations of the lower limits of these volcanic ash layers at the dam crest are around 1,700 m, the wings of the dam would naturally connect to these volcanic ash layers. From results of field investigations, both banks are thought to have adequate bearing capacities and watertightnesses for a dam foundation, but it was considered that these should be confirmed by consolidation tests, shear tests, jack tests and permeability tests, and these were carried out in the present Study. The results are described in 6.6.

In regard to the volcanic ash layers, CEDELCA requested the Universidad del Cauca to carry out soil tests. Of the samples, those of interest from the standpoint of pedology were again tested by INGEOMINAS*. Table 6-6 is a soil profile of the Julumito Hydro-electric Power Project area experimentally prepared referring to the field surveys and test reports. Such a weathering process can also be recognized in the weathered residual soil of the andesite lava (see 6.3.2 (1) b) (vi)) described below.

On looking at these permeability test results, it is seen that even with the same injection pressures, the permeability coefficient becomes smaller than at the start of testing with elapse of time and after approximately 5 hours becomes more or less constant. This indicates that the permeability coefficient becomes smaller as this soil layer becomes saturated with water. Since Units (II) and (III) in Table 6-5 are not necessarily judged to be satisfactory from the standpoint of bearing power, these layers should be excavated and removed, and the foundation selected at Units (IV), (V) or (VI). Units (IV), (V) and (VI) are more solidified than the upper parts and are thought to have smaller permeability coefficients, but it is desirable for permeability tests to be conducted.

*Instituto Nacional de Investigaciones Geologico Mineras

Table 6-5 Pedological Profile of Weathering Process in Volcanish Ash.



Layer	Explanation	Soil Classification ※5	Technical judgment: Degree of Compaction	Technical judgment: Permeability	Remarks
(I)	Blackish brown humus 1 m	O	loose	more	
(II)	Brown leached layer ※1	A1	↓	↑	Ct Appandix IV-6
(III)	High content of gibbsite ※2 1.5m Yellowish white to grayish white Bauxitic clay layer ※3 2 m	A2			
(IV)	Accumulated zone of colloidal material high content of silica 0.5 m Yellowish white				
(V)	Altered chiefly kaolinite and halloysite ※4 ±7 m	B1 to B2			
(VI)	Non-weathered volcanic ash	C1	compact	less	

※ 1 Leaching of sillica (SiO_2)
※ 2 Gibbsite ($Al_2O_3 3H_2O$) confirmed by X-ray diffraction and differential thermal curve analysis
※ 3 Silica-Alumina ratio (SiO_2/Al_2O_3) = 1

Chemical analysis of clay of unit (III)

Al_2O_3	SiO_2	Fe_2O_3	TiO_2	CaO	MgO	P_2O_3	Others	Total
43.00	31.66	10.00	0.64	1.12	0.92	0.09	11.40	98.83%

※ 4 Silica-Alumina ratio = 2
※ 5 U.S. Department of Agriculture (1951) ; Soil survey manual

(vi) Andesite Lava

The andesite lava distributed at this site has been strongly affected as a whole by weathering action and no fresh parts can be seen at the ground surface. This andesite lava may be divided into three layers from the degrees of weathering.

- o Non-weathered andeiste lava,
- o Weathered andesite lava (presents a bedrock appearance but the component minerals are almost all changed to secondary minerals by weathering),
- o Residual soil originating from andesite lava (weathering exceedingly advanced with rock-forming minerals changed to clay minerals such as halloysite and kaolinite. The surface layer consists of bauxitic clay).

ⓐ Non-weathered Andesite Lava

Outcrops of this type of bedrock cannot be recognized at the dam site, barely being seen in the core of Borehole DH-204 at 9 m and deeper.

This core is of bluish-gray andesite with prominent feldspar and is slightly liparitic. Microscope observations of this rock are described in Appendix IV-3 of the Feasibility Study Report of 1972. Biotite is recognizable in the lava even by visual observation and there are numerous phenocrysts of quartz, and the characteristics are more those of dacite or rhyolite. However, the greater parts of the matrix and phenocrysts are plagioclase (albite) and in this Study "andesite lava" will be used as the field name.

ⓑ Weathered Andesite

This rock is seen at the Rio Sate banks and at depths of Exploratory Adits A-2, A-3 and A-4. It is a massive rock of grayish-white to milky-white color, is weathered, and belongs to the category of soft rock, but compressive strengths are 100 kg/cm^2 or higher.

This bedrock is thought to have adequate bearing capacity for the foundation of a fill dam of 80-m class, but this should be confirmed by field jack tests in future investigations.

At Exploratory Adit A-3 excavated at the right-bank of the dam site, dropping of water can be seen from open cracks in this weathered andesite. The volume of spring water is about $3\ell/min$, the water pressure is low, and it is thought clearly to be seepage water from the ground surface. In the original rock of these weathered andesites there are distinct columnar joints seen and there is a possibility that the beforementioned open cracks are traces of these joints. The scale and condition of joint system will have a relationship with permeability of the dam foundation. In the present survey, core boring and lugeon tests were carried out over a wide area on the bedrock of the dam site, the results of which are described in 6.6.

ⓒ Residual Soil Originating from Andesite

The bottom part of this layer presents a grayish-white to milky-white color, is coarse-grained, and is slightly compacted, but the upper part is reddish-brown to grayish-brown and loose. The thickness of this layer according to data from exploratory adits and boreholes is from 3 m to 5 m, and from the degree of compaction and thickness, it should be excavated and removed for the dam foundation.

The boundary between this residual soil and weathered andesite is distinct at the side walls of trenches and test adits, but is not necessarily clear at outcrops and cores from boring. It is thought necessary for criteria for judging the strengths as a dam foundation to be provided for the residual soil and weathered andesite in carrying out construction. It is necessary, besides field jack tests and CBR tests, for direct shear tests and triaxial shear tests on undisturbed samples to be made. Most of these tests were carried out in the present Study and the results are described in 6.6.

(2) Dike No. 1 Site

This site is at a saddle where the boundary lies between the Rio Sate and the Rio Cauca at the south outskirt of Julumito Village. Both banks are gently sloped and continue on to hills of elevation about 1,725 m. The distance from this dike site to the Rio Cauca is approximately 500 m.

In the Feasibility Study of 1972, the concept was to provide the spillway at this site, but as a result of the present Study it was planned for the spillway to be appurtenant to the main dam. Consequently, only a dike is to be constructed at this saddle. On reexamining the dike location from this standpoint, it was decided to move it to a more upstream site where the height of the dike would become lower.

The geology comprising this site is mostly a volcanic ash layer. According to Borehole DDH-1 (El. 1,715.11 m) drilled at this dike site, there are alternations of sand layers and silty sand layers down to a depth of 16.00 m, while at 16.00 m - 16.30 m, there is found humus thought to be topsoil of an ancient period, below which there is a volcanic ash layer. The strata shallower than the beforementioned 16 m are clearly deposits transported by the river and have been subjected to some degree of sorting. The distribution of this layer cannot be clarified with only surface reconnaissance since the ground surface is completely covered by humus. This layer is probably distributed only at the terrace surface at El. 1,708 m, but it is necessary for confirmation to be made through investigations by trench excavations or hand auger drilling along the dike axis.

The height of this dike is to be 5.0 m, and it is considered there will be no special problem with regard to bearing power of the foundation.

(3) Dike No. 2 Site

This site is a flat hill area and the ground surface is grazing grassland covered by humus.

According to data from Borehole DDH-2 at the lowest point of the dike, there is a volcanic ash layer under the topsoil of thickness of 1 m, the thickness of this volcanic ash layer being more than 23 m. Since the embankment height of this dike is low (7.0 m), there will be no problem as a foundation for the dike.

6.3.3 Headrace Tunnel

The geology comprising the headrace tunnel route, except for the vicinity of the intake, is fresh andesite lava. Weathered andesite and its residual soil are distributed in the vicinity of the intake. The foundation of the intake is at the weathered andesite underlying the residual soil, but is thought to possess adequate bearing capacity similarly to the dam foundation.

The tunnel route passes through the top layer of the andesite lava, this layer being a grayish-blue, slightly coarse-grained, sound andesite. (See 6.5.2 for physical properties of this rock.) The distributions and properties of this andesite lava have been confirmed from Boreholes DH-1, DH-2, DH-3 and DH-4 drilled along the headrace tunnel route.

Judged by the conditions of outcrops at both banks of the Rio Cauca, this layer through which the tunnel will pass is slightly porous compared with the underlying andesite lava, but it is massive and thought to have few cooling joints. In driving of the tunnel, it is thought there will be no great problems since it appears there are few faults along the route. However, during excavation of the tunnel, there is a possibility that spalling or cave-ins may occur at side walls or spring sections due to cooling joints (columnar joints). Also, cooling joints often comprise continuous water channels. Therefore, depending on the conditions of existence of ground water along the route of the tunnel, there may be occasions when spring water would be encountered during excavation.

6.3.4 Surge Tank, Penstock and Powerhouse

In the present Study, the two alternatives below were examined regarding the route from the surge tank to the powerhouse.

(1) Surface Alternative

This is a scheme to install a surface-type penstock from the surge tank to the powerhouse along the slope on the right bank of the Rio Cauca. The vicinity of the surge tank is a flat area of about El. 1,720 m with a distribution of volcanic ash in a layer of about 30 m thickness, but the slope facing the Rio Cauca forms a gorge made by the rapid deepening action of the Rio Cauca and the gradient is steep at more than 45°.

The elevation of the bottom of the headrace tunnel at its end is 1,669.81 m and is thought will pass through andesite lava, while anchor blocks at the head of the slope can be provided in sound andesite. The volcanic ash layer at this part is unstable and should be excavated and removed. According to the present plan it will be a slope 40 m in height at a gradient of 1: 1.

Rock outcrops can be seen here and there at the slope, but the bottom part has small gullies and talus deposits are seen.

(2) Vertical Shaft Alternative

This is a scheme to provide a vertical shaft inside the original ground from the surge tank to the right-bank of the Rio Cauca and to conduct water to the powerhouse by a gently-sloped diagonal shaft of 1: 10. The geology of the vertical shaft portion consists of andesite lava as previously mentioned, and it is thought there will be no great problem since the rock is massive and hard. However, this andesite lava formation is made up of four strata with intercalations of layers of tuff and tuffaceous siltstone 1 to 3 m in thickness. These layers are impermeable to comprise localized confining beds for groundwater, and there is a possibility that groundwater stored at these layers will spring during excavation of the vertical shaft. However, so far as seen at the slope of the gorge, there is no outflow from these boundaries and the water springing at the vertical shaft is thought will be slight.

From the standpoints of design and construction, it would be desirable for the location of the vertical shaft to be directly under the surge tank. As for the location of the surge tank itself, it should be where it will be geologically stable in consideration of the existence of the thick (25 - 30 m) volcanic ash deposit.

Regarding stability of the slope and bearing capacities of the volcanic ash layer and the upper part of the andesite formation, it will be necessary for investigations to be made.

(3) Powerhouse Site

a) Topography

The powerhouse site is located at the right bank of the Rio Cauca. The powerhouse will be on a river terrace of a width of 25 m back of which is the steep cliff of

40° to 60° where the penstock is to be located. A small-scale talus deposit exists at the skirt of the cliff.

b) Geology

According to Borehole DH-4 (El. 1,568.66 m) drilled at the powerhouse site, a talus deposit of 9.5 m thickness and a terrace deposit of 5.2 m thickness underneath are passed to reach the tuff breccia to be the foundation of the powerhouse. This tuff breccia presents a greenish-gray color, is coarse-grained, and has a high content of andesite gravel. It is in the category of soft rock, but since compressive strengths are 50 to 100 kg/cm^2 and the rock is homogeneous and massive, it is considered to have adequate bearing capacity as the foundation for the proposed powerhouse.

6.4 Geologies of Diversion Scheme Sites

The diversion scheme consists of the Rio Cauca Diversion Plan drawing water from the Rio Cauca mainstream for conduction to the Rio Sate, and Rio Palace Diversion Plan and the Rio Blanco Diversion Plan drawing water from the tributary Rio Palace and Rio Blanco for conduction to the Rio Sate.

The Rio Cauca Diversion Plan consists of Rio Cauca Diversion Dam and Cauca Waterway from this dam to Quebrada La Paz which will flow into Julumito Reservoir. The Rio Palace Diversion Plan consists of Rio Palace Diversion Dam and Palace Waterway which runs to the Rio Blanco, while the Rio Blanco Diversion Plan consists of Rio Blanco Diversion Dam and Blanco Waterway from this dam to Quebrada Pambazo which will feed Julumito Reservoir.

6.4.1 Rio Cauca Diversion Plan

(1) Rio Cauca Diversion Dam Site

The dam planned for this site is an overflow-type concrete gravity dam (dam height 12.5 m, dam crest length 77.0 m) as shown in Dwg. No. 25 and Dwg. No. 26. This site is approximately 4 km northeast of downtown Popayan and is located 500 m upstream from the bridge over which the old highway to Cali crosses the Rio Cauca.

This site corresponds to the outlet for the Rio Cauca which on passing through the hills of the Popayan Plateau goes out to the flat area comprising river terraces.

The left bank, on passing the river terrace (Te_1) 200 m wide at an elevation of approximately 1,785 m continues on to the hills to the east. The water flow portion at the riverbed has a width of approximately 20 m and is shallow. Toward the right bank, 2-3 m higher than the river channel, there is a flood plain of width of approximately 50 m which continues to the downstream area. The right bank, after a terrace cliff of relative height differential of 5 - 7 m above the flood plain, continues on to the surface of the river terrace Te_2. The flat areas have come to be utilized, and in the vicinity of the right-bank side of the dam site a new girls' secondary school has been built. This flat area has a width of 250 m, west of which is a hill area of gentle relief.

The river deposits consist of round cobbles with diameters of 10 - 50 cm and sandy silt fills the interstices between the cobbles. The thickness of this river deposit is thought to be comparatively small.

The terrace deposits contain large quantities of round gravel of 5 - 20 cm with a matrix of silty sand and silty clay, and are well-compacted.

The geology of the hills spread out widely to the west consists of a volcanic ash layer.

The scales of the dam and appurtenant structures to be provided at this site are small, but the foundations of these structures will all be unconsolidated deposits. Therefore, in consideration of safety of the dam, proper foundation treatment and measures in design against uplift pressure will be required.

Core boring were carried out in the present Study in order to determine the extent of excavation of the foundation ground for the dam and intake, and to obtain design data for foundation treatment. The results are described in 6.6.

(2) Cauca Waterway

This diversion waterway is designed as indicated in Dwg. No. 29. The capacity in 40 m^3/sec. and the length 2,400 m with the greater part an open canal, but roughly at the end a tunnel of 220-m length is to be provided. The greater part of the route is located at river terraces and the topography is flat. However, at the tunnel portion and part of the outlet (Quebrada La Paz) side, volcanic ash is distributed. The terrace deposits are silty clay to sandy silt containing large quantities of cobbles of 10 to 30 cm diameters, and are well-compacted.

The volcanic ash is slightly coarse-grained, but is of identical origin to the volcanic ash in the vicinity of the Julumito dam site, and thickness and condition of weathering are completely alike. The volcanic ash layer is adequately compacted at deeper parts but is soft, while the terrace deposits are unconsolidated so that blasting will not be necessary for excavating the open canal, and it is estimated that construction can be amply achieved with ordinary excavater.

Meanwhile, since the topography of the waterway route is on the whole a gentle terrain, and since extremely high slopes will not be formed by excavation, the topographical conditions are advantageous in regard to stability of slopes. However, since the shoulders of the excavated slopes would consist of extremely weathered or very loose strata, there could be many cases when small-scale crumbling would occur. These phenomena are observed frequently at natural cliffs in the survey area, and at skirts of steep slopes loose layers at the upper parts have fallen to form talus deposits. The angle of repose of the talus deposits is around 30°. Considered from such phenomena, in order to prevent material eroded from the excavated slopes from falling directly into the waterway, berms of suitable widths should be provided between the tops of the waterway and the skirts of the slopes, or shoulders should be made to slope gently.

The volcanic ash layer and the terrace deposits are judged to have adequate bearing capacities as foundations for the open canal, except for weathered layers and loose layers close to the ground surface, but construction must be carried out carefully to prevent failure of the waterway through uneven settlement of concrete structures caused by erosion of the foundation by water leaking from construction joints in the concrete. Leakage due to defective concrete placement can cause sliding of the foundation of the waterway depending on topographical and geological conditions.

The tunnel will be driven through volcanic ash. Therefore, timbering will be required to a considerable extent, while the entire length will require concrete lining. As for leakage from the waterway tunnel, the same phenomena as at the open canal can occur, and since repairs would be far more difficult than at the open canal, even greater precautions must be taken in design and construction.

6.4.2 Rio Palace Water Diversion Scheme

(1) Rio Palace Diversion Dam

The dam planned at this site is a small overflow-type concrete gravity dam as shown in Dwg. No. 27.

This dam site is located at the outlet of the Rio Palace where it flows out from the mountainous area to the plain area, and the river flows down forming a small gorge in the bedrock andesitic lava. In short, the Rio Palace in this vicinity presents a U-shaped gorge of depth of about 10 to 15 m and width of 8 to 12 m, bends widely to the right near the dam site, where the width of the river is broadened. The river gradient is slightly steep, with well-rounded boulders of diameter 3 to 5 m scattered at the riverbed interspersed with rounded gravels of 30 to 50 cm. However, as the foundation for a concrete gravity dam of low height, it is bedrock of good condition. There is no special problem in connection with the dam foundation.

(2) Palace Diversion Waterway

Regarding the Palace Diversion Waterway, it is to be changed from a plan for an open cannal to one for a tunnel based on the results of the present investigations. This tunnel is to be a semi-circular top and square bottom type of height of 2.80 m, width of 2.80 m and length of 770 m.

Andesitic lava is distributed to heights far above the tunnel bed in the vicinity of the tunnel route, and the greater part of the tunnel will pass through andesite of good condition. There is a possibility that weathered andesite or volcanic ash will be encountered near the exit of the tunnel, and it is thought timbering and gunning of mortar or concrete will be necessary for this part.

(3) Rio Blanco Diversion Dam

The dam planned is a small overflow-type concrete gravity dam as shown in Dwg. No. 28. This site is located approximately 250 m downstream from the confluence of the Rio Blanco and a tributary, Quebrada Clarete.

Although this is a low dam, since the foundation consists of unconsolidated deposited material, the same considerations are necessary as for Rio Cauca Diversion Dam in regard to bearing capacity and permeability of the foundation, and scouring downstream of the dam.

(4) Blanco Waterway

This diversion waterway is planned as a semi-circular top, square bottom tunnel of height of 3.0 m, width of 3.0 m and length of 3,650 m.

The greater part of the tunnel route consists of volcanic ash with a part being weathered andesite. The volcanic ash layer, analogized from the conditions in an exploratory adit (width approximately 1.8 m, height 2.4 m) excavated at the dam site, is thought will be sufficiently self-supporting at the tunnel face, but will deteriorate with elapse of time to collapse. Consequently, it will be desirable for lining work to be done as soon as possible after excavation, and in case there is to be a blank period of 6 months or more, it is thought timbering and gunning of concrete or mortar will be required.

At sections of thin cover of the ground, deterioration is severe due to weathering and seepage of rainwater, and timbering will be necessary.

At present, the relation between the elevations of excavation of the tunnel and the groundwater level is unknown, and it is thought that at the time a detailed topographical map is completed, boring investigations including geological investigations should be made on the tunnel route.

Regarding springing of water in the tunnel during excavation, this can be estimated from the relation between the tunnel elevation and the groundwater level, and from the coefficient of permeability and void ratio of the constituent geology, but it is inconceivable that water springing will be particularly severe according to the results of investigations up to this point. In the event there were to be special springing, a drainage tunnel should be provided leading to a nearby gully, and allowing water to flow freely inside the tunnel to the portal should be avoided.

6.5 Construction Materials

6.5.1 Impervious Materials

Volcanic ash and weathered residual soil of andesite lava may be considered as impervious materials at this site. Volcanic ash is widely distributed at the project area in a thickness of 20 m to 40 m and may easily be borrowed. In the Preliminary Study Report, there were two areas selected as possible areas at the left-bank side of the Rio Sate downstream from the Julumito dam site, and it was recommended for investigations to be made of these areas. Based on these recommendations, 8 test pits were excavated and 2 boreholes drilled in 1971 by ICEL and CEDELCA. Soil tests were performed on samples collected from these points. As a result of soil tests and surface reconnaissances, there is no superiority in particular of one over the other between Borrow Area No. 1 and Borrow Area No. 2. If pressed to find a difference, Borrow Area No. 2 may be said to be superior considered from the standpoints of the abundance of the material existing (requirement: 177,000 m^3) and hauling distance.

The above tests were carried out upon request at the Universidad del Cauca, while triaxial compression tests were performed at the Universidad del Mexico. The comprehensive tests results are given in Table 6-6 and Table 6-7. As for the peculiar weathering action on this volcanic ash deposit, a report has been produced by INGEOMINAS from a pedological standpoint.

Based on these data and observations in the field, the following comments may be made on the impervious materials:

(1) Watertightness as an impervious core material is thought to be adequate from examination of the gradation analysis curve. However, in calculating optimum water contents and maximum dry densities in compaction tests, the coefficients of permeability at the respective water contents should be measured and compaction-permeability curves prepared. The permeability tests by the Universidad del Cauca were limited to the occasions of consolidation tests. Considered from these data, this volcanic ash indicates coefficients

of permeability of the orders of 10^{-5} cm/sec to 10^{-7} cm/sec in compacted samples in the vicinity of the optimum water content and is thought to have adequate watertightness as a core material. Further, in order to clarify the relations between weathering stages and suitability as core material of this volcanic ash, in the present Study, test pits were excavated and various tests were performed on samples collected at every 1 m of depth. The results are described in 6.6.

Still further, regarding the results of in-situ permeability tests performed utilizing test pits and boreholes (DH-6 and DH-7) at the borrow areas, the coefficients of permeability were of the order of 10^{-4} cm/sec to 10^{-5} cm/sec.

(2) In case this volcanic ash is used as core material, the problems will be the strength as a fill material and trafficability of construction equipment because of the extreme fineness, added to which it is a cohesive soil of high natural water content.

a) According to gradation tests, the percentage passing the #200 sieve (= ϕ0.074 mm) is close to a remarkable 90%, and going by the Unified Soil Classification System, the soil corresponds roughly to MH and ML. Also, according to soil consistency tests, the liquid limit (LL) is low at around 60% and the natural water contents are in many cases close to this value. Based on these test results, the plastic limits (PL) of the majority of the samples are abnormally high and the plasticity index (PI = LL - PL) is 10% or lower. It is difficult to understand why not only the optimum water contents but also even the natural water contents based on compaction tests indicate value smaller than the plastic limits in the data furnished. In conducting tests, gradation analyses should be made on sizes passing the #200 sieve for use as references in consistency tests.

b) With respect to tests regarding strengths of soil, unconfined compression tests, triaxial compression tests and direct shear tests have been conducted and results are given in Table 6-6. According to these results, the angle of internal friction (ϕ) indicates an abnormally high value (35° to 55°), and cohesion (C) a low value. This characteristic, at first glance, would suggest a sandy material, whereas in reality the material is a cohesive soil of more than 85% passing #200, and values greatly different from volcanic ashes in general are indicated. Further, the axial pressures ($\bar{\sigma}_1$) during the triaxial compression tests indicate very high values of 40 to 70 kg/cm^2, and these values are thought to be far above the normal ultimate bearing capacity of volcanic ash. However, in the results of direct shear tests also, values of "ϕ" and "C" close to those in the triaxial compression tests have been obtained. It may be that in this volcanic ash, even with more than 90% passing #200, silt is predominant in the sizes under #200 and the volcanic ash thus indicates inactivity as a cohesive soil. The activity of clay, based on the results of gradation analyses of sizes under #200, may be given by the following formula.

$$\text{Activity of clay} = \frac{\text{Plastic index}}{\text{Clay content under } 2\mu \text{ size}}$$

Judging from the situations in work execution on volcanic ash deposits at the regulating pond of Florida II Power Station located in the suburbs of Popayan City, it is thought that with regard to trafficability, construction with ordinary civil engineering machinery (including crawler-mounted swamp bulldozers) is not impossible. With respect to this point, it is desirable for confirmation to be made by CBR tests and cone penetration tests on undisturbed samples of the volcanic ash and on material compacted at the water content anticipated during construction.

(3) Tests and construction control in case volcanic ash is to be used as core material require special consideration. On performing compaction tests with samples air-dried from natural water content conditions, a fluid state is often reached before the specified number of rammer blows have been applied so that subsequent compaction becomes impossible. It is thought this indicates the Proctor compaction energy (E_c) to be excessive for this volcanic ash. In case the natural water content of this volcanic ash is very high and the saturation rate (S_r) is a minimum of 75%, when compacting this, the S_r will reach close to 100% at the slightest compactive effort. Consequently, for the compaction test, the relation between "E_c," or the number of rammer blows, and "S_r" should be clarified and a method of test devised applying the number of rammer blows and a rammer weight appropriate for this relation. It is thought necessary to calculate optimum water content and maximum dry density by such a method at the same time carrying out permeability tests and unconfined compression tests under the conditions of compaction at various water contents. Studies of the types of compaction machinery placement thickness, number of compaction passes, etc. should be made based on the results of such tests. Also, with this type of volcanic cohesive soil, it is often the case that at the same water content, the results of compaction on addition of water to a thoroughly dried sample and the result of compaction on drying from a state of natural water content indicate completely different dry densities. Therefore, it is not desirable to perform tests adding water after thorough oven-drying, and it should be necessary to test under drying conditions considering conditions on the job. When the properties of such a volcanic cohesive soil is considered, in embankment of the core, it is felt more desirable for embankment control based on saturation rate, "S_r," to be carried out rather than on compaction rate (percentage of compaction) based on dry density. (Table 6-6, Table 6-7)

Table 6-6 Result of Soil Test (Sheet 1 of 2)

		Borrow area No. 1																					TP101 TP204
Name of test pit		TP-101	TP-101	TP-102	TP-102	TP-102	TP-103	TP-103	TP-104	TP-104	Average	TP-201	TP-201	TP-201	TP-202	TP-202	TP-202	TP-203	TP-203	TP-204	TP-204	Average	Average
Date		Oct. 26/70	Sep. 30/70	Nov. 7/70	-	Sep. 30/70	Oct. 28/70	Sep. 30/70	Oct. 26/70	Sept. 30/70	-	Nov. 9/70	Feb. /70	Sep. 30/70	Sep. 30/70	-	Oct. 27/70	Nov. 4/70	Sep. 30/70	Sep. 30/70	Oct. 31/70	-	-
Depth of sampling (m)		-	3.00	-	-	3.00	-	3.00	-	3.00	-	3.00	2.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	-	-
Classification U.S.C.E		no plastic	MH	MH	MH	MH	SMu	SM	MH	MH	-	MH	ML	ML	ML	MH	MH	SMu	SM	SM	SMu	-	-
Gradation analysis	4.76 mm > (%)	100	-	100	100	-	99.93	-	100	-	99.99	97.82	100	-	-	100	99.92	99.88	-	-	100	99.60	99.78
	2.0 mm > (%)	99.22	99.99	100	100	99.37	98.06	99.66	99.98	99.82	99.57	97.65	99.11	99.09	99.44	99.86	99.62	97.20	99.96	98.66	99.98	99.60	99.78
	0.42 mm > (%)	94.49	99.00	99.81	98.36	97.96	88.19	92.07	99.72	98.17	96.42	96.24	96.53	96.70	96.98	98.49	97.25	76.20	98.67	90.98	79.37	92.74	99.30
	0.149 mm > (%)	89.02	-	97.65	97.90	-	76.51	-	97.94	-	91.80	92.46	91.38	-	-	96.45	91.97	50.23	-	-	59.25	93.74	94.48
	0.074 mm > (%)	83.03	92.16	94.85	94.41	89.17	57.26	79.00	94.26	91.52	86.18	85.71	86.42	84.25	88.53	94.56	87.79	34.65	90.47	67.84	51.54	80.29	85.52
Atterberg's limits	LL (%)	-	61.0	61.0	67.00	60.0	-	-	67.0	61.0	62.83	58.3	46.30	40.0	47.5	78.75	50.0	-	-	-	-	77.18	81.44
	PL (%)	-	45.4	47.5	60.83	41.0	-	-	52.6	15.9	48.87	39.9	39.28	30.0	37.4	64.50	37.4	-	-	-	-	53.48	58.15
	Ip (%)	-	15.6	13.5	6.17	19.0	-	-	14.4	15.1	13.96	18.4	7.02	10.0	10.1	14.25	12.6	-	-	-	-	41.41	45.14
	I_L	-	2.42	0.72	-2.08	0.74	-	-	0.97	1.42	0.70	0.16	1.14	1.08	0.35	-0.65	0.87	-	-	-	-	12.07	13.01
	I_c	-	-1.42	0.28	3.08	0.26	-	-	0.03	0.42	0.30	0.84	-0.14	-0.08	0.65	1.65	0.13	-	-	-	-	0.49	0.60
	G	2.69	2.69	2.60	2.71	2.72	2.36	-	2.58	2.58	2.62	2.42	2.53	2.47	2.45	2.37	2.53	2.46	2.66	2.58	-	0.51	0.40
ω natural (%)		49.13	83.1	57.2	48.0	55.0	102.0	76.2	66.51	67.3	67.16	42.9	47.3	40.8	40.8	55.3	48.3	87.8	58.4	107.2	2.59	2.52	2.56
e natural																					178.0	70.69	69.02
Sr. natural (%)																							
Compaction test	Standard *1)	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H	P / H
	ω optimum (%)	- / -	- / 42.0	- / 45.0	43.5 / 41.5	- / 38.0	- / -	- / -	- / 46.3	- / 41.9	43.5 / 42.4	- / 38.1	33.0 / 32.0	- / 26.9	- / 30.3	39.0 / 44.0	- / -	- / -	- / -	- / -	- / -	36.0 / 34.3	38.5 / 38.7
	γ d. max (g/cm³)	- / -	- / 1.20	- / 1.19	1.18 / 1.20	- / 1.25	- / -	- / -	- / 1.14	- / 1.22	1.18 / 1.20	- / 1.30	1.30 / 1.31	- / 1.42	- / 1.3	1.22 / 1.25	- / -	- / -	- / -	- / -	- / -	1.26 / 1.32	1.23 / 1.26
	e optimum	- / -	- / 1.24	- / 1.18	1.30 / 1.26	- / 1.18	- / -	- / -	- / 1.26	- / 1.11	1.22 / 1.18	- / 0.86	0.86 / 0.93	- / 0.74	- / 0.8	0.94 / 0.90	- / -	- / -	- / -	- / -	- / -	1.00 / 0.91	1.08 / 1.03
	Sr. optimum (%)	- / -	- / 91.1	- / 99.1	90.7 / 89.3	- / 87.6	- / -	- / -	- / 94.8	- / 97.4	93.4 / 94.1	- / 95	87.9 / 87.1	- / 89.8	- / 88.4	98.3 / 95	- / -	- / -	- / -	- / -	- / -	90.7 / 95.0	91.3 / 96.2
Consolidation test	inicial	1.51	-	1.55	1.39	-	2.91	-	1.40	-	1.75	1.00	0.90	-	-	1.05	1.01	2.34	-	-	5.19	1.92	1.84
	final	1.30	-	1.36	1.25	-	2.18	-	1.24	-	1.47	0.88	0.88	-	-	0.98	0.83	1.97	-	-	4.56	1.67	1.58
	a_v	0.037	-	0.025	0.028	-	0.150	-	0.025	-	0.053	0.012	0.01	-	-	0.023	0.02	0.044	-	-	0.055	0.027	0.039
	c_v	0.0096	-	0.0341	0.0119	-	0.0103	-	0.0056	-	0.0143	0.0132	0.008	-	-	0.0129	0.0066	0.0131	-	-	0.0049	0.0098	0.0118
	m_v	0.015	-	0.0098	0.012	-	0.038	-	0.0104	-	0.0170	0.006	0.0053	-	-	0.01122	0.0011	0.0132	-	-	0.0089	0.0093	0.0128
	K	1.42×10^{-4}	-	3.3×10^{-4}	1.4×10^{-7}	-	4×10^{-4}	-	4.7×10^{-5}	-	1.8×10^{-4}	7.9×10^{-5}	4.2×10^{-8}	-	-	1.4×10-7	6.6×10^{-5}	1.7×10^{-4}	-	-	4.4×10^{-5}	0.6×10^{-4}	1.2×10^{-4}
Unconfined compression Strength qu (Kg/cm²)		-	2.06	1.40	1.52	3.25	-	-	2.26	2.22	2.12	2.41	2.02	2.92	4.5	1.87	-	-	-	-	-	2.74	2.40
Triaxial compression test	C (Kg/cm²)	-	-	0.10	0.25	-	-	-	0.75	-	0.37	0.5	1.0	-	-	1.5	-	-	-	-	-	1.0	0.68
	ϕ (°)	-	-	40	55	-	-	-	22	-	39	51	49	-	-	33	-	-	-	-	-	44	42
	tan ϕ	-	-	0.839	1.43	-	-	-	0.404	-	0.810	1.23	1.15	-	-	0.649	-	-	-	-	-	0.966	0.900
Direct shear test	C (kg/cm²)	0.1	-	0.00	0.20	-	0.18	-	0.12	-	0.12	0.08	0.22	-	-	0.19	-	0.0	-	-	0.50	0.20	0.16
	ϕ (°)	40	-	40	36	-	30	-	32	-	36	34	43	-	-	39	-	40	-	-	23	36	36
	tan ϕ	0.839	-	0.839	0.726	-	0.577	-	0.625	-	0.726	0.674	0.932	-	-	0.810	-	0.839	-	-	0.424	0.726	0.726

Note: 1) P ---- PROCTOR Standard, H ---- HARVARD Standard

2) This soil test was conducted by Universidad Del Cauca responding to the request of CEDELCA.

Table 6-7 Result of Soil Test (Sheet 2 of 2)

	Headrace tunnel				Dam site No. 2	Borrow area No. 1			Borrow area No. 2			Dike sites: Dike No. 1	Dike sites: Dike No. 2			DH2 D.DH2
Name of drill hole	DH-2	DH-2	DH-2	DH-2	DH-203	DH-6	DH-6	DH-6	DH-7	DH-7	DH-7	D.DH-1	D.DH-2	D.DH-2	D.DH-2	Average
Date	Nov. 25/70	Nov. 25/70	Nov. 25/70	Nov. 25/70	Nov. 25/70	Oct. /70	Oct. /70	Oct. /70	-	-	-	May 5/71	May 5/71	May 5/71	May 5/71	-
Depth of sampling (m)	5.00 to 6.00	10.00 to 11.00	15.00 to 16.00	20.00 to 21.00	6.00 to 7.00	6.00 to 7.00	10.00 to 11.00	15.00 to 16.00	5.00 to 6.00	10.00 to 11.00	16.00 to 17.00	20.00	5.00	10.00	20.00	-
Classification U.S.C.E	MH	MH	ML	-	-	MH	MH	ML	MH	ML	-	ML and gravel	ML	ML	ML	-
Gradation analysis: 4.76 mm > (%)	-	-	100	99.88	97.31	-	-	-	-	99.94	99.63	64.75	100	100	99.97	95.72
Gradation analysis: 2.0 mm > (%)	99.92	99.97	99.98	99.86	95.11	99.90	-	99.86	99.95	99.87	99.17	55.61	99.72	99.55	98.46	96.21
Gradation analysis: 0.42 mm > (%)	99.15	99.40	96.31	94.34	81.39	99.34	99.13	98.40	99.17	99.24	97.07	45.83	98.73	98.13	87.01	92.84
Gradation analysis: 0.149 mm > (%)	93.97	96.20	86.00	80.08	68.58	96.82	94.33	92.68	94.84	93.30	85.87	35.74	93.17	92.75	73.92	85.22
Gradation analysis: 0.074 mm > (%)	89.00	94.25	82.80	74.37	65.86	92.42	90.12	88.15	89.81	86.92	74.24	34.98	81.61	84.86	63.44	79.52
Atterberg's limits: LL (%)	56.0	67.0	35.0	32.8	-	66.5	56.5	42.5	62.0	47.5	-	38.50	63.00	58.00	58.00	52.56
Atterberg's limits: PL (%)	40.3	52.6	26.9	28.8	-	51.6	44.3	32.8	44.4	31.5	-	37.97	56.28	50.61	50.26	42.18
Atterberg's limits: Ip (%)	15.7	14.4	8.1	4.0	-	14.9	12.2	9.7	17.6	16.0	-	0.53	6.72	7.39	7.74	10.38
Atterberg's limits: I_L	1.55	1.68	1.25	2.33	-	1.68	1 16	1.01	0.80	0.98	-	17.88	1.98	0.85	3.38	1.26
Atterberg's limits: Ic	-0.55	-0.68	-0.25	-1.33	-	-0.68	-0.16	-0.01	0.20	0.02	-	-16.88	-0.98	0.15	-2.38	-0.26
Atterberg's limits: G	2.58	2.43	2.49	2.5	2.60	-	-	-	-	-	-	2.59	2.62	2.74	2.66	2.58
ω natural (%)	64.7	76.8	37.0	38.1	37.4	76.7	58.5	42.6	58.4	47.1	42.3	47.45	69.60	56.9	76.39	55.33
γ natural (g/cm^3)	1.72	1.65	1.89	1.85	-	-	-	-	-	-	-	-	-	-	-	1.78
e natural	1.47	1.60	0.80	0.87	-	-	-	-	-	-	-	-	-	-	-	1.25
Sr. natural (%)	95<	95 <	95<	95 <	-	-	-	-	-	-	-	-	-	-	-	95 <
Unconfined compression strength q_u (kg/cm^2)	1.42	0.71	-	-	-	1.63	1.34	4.04	1.17	0.70	0.87	0.72	1.93	2.7	0.49	1.48

Note 1) This soil test was conducted by Universidard Del Cauca responding to the request of CEDELCA.

6.5.2 Rock Material and Filter Material

(1) Rock Material

Andesite lava is conceivable to be used as rock material. According to the results of boring done at the candidate rock quarry site selected in the Preliminary Study, volcanic ash is deposited to a depth of 39 m with fresh andesite at 51 m and below. With the volcanic and weathered andesite layers thick in this manner it is economically impracticable to excavate and remove these upper layers.

The upstream and downstream areas of the Rio Sate were reinvestigated to select a quarry site, but as described in detail in 6.3.1, "Catchment Basin and Reservoir," weathering has reached deep into the rock and there was not a single location suitable as a quarry site. The hill bounded by the Rio Sate and the Rio Cauca, as described in 6.3.3, "Headrace Tunnel," is covered thickly and uniformly by a volcanic ash deposit, and is unsuitable as an area for quarrying rock.

Where the volcanic ash deposit has been removed by erosion is limited to the surroundings of the powerhouse site forming a gorge due to the rapid cutting-down action of the Rio Cauca (see 6.3.4).

In the Feasibility Study of 1972, the ridge on the left bank of the Rio Cauca, the opposite bank from the powerhouse, was selected as a site where fresh andesite could be quarried without fail, but in the present Study, as a result of examination of quarrying methods, transportation road plans and related penstock and powerhouse construction plans, it was decided to change to a site on the right bank of the Rio Cauca and approximately 500 m downstream from the powerhouse.

This rock quarry area consists of thick andesite lava and is divided into four layers, but all of the layers are considered to be of suitable quality as rock materials.

Although the locations differ, physical tests of rock were carried out with cores from Boreholes DH-1, DH-2, DH-3, DH-5 and DH-204 drilled in the same andesite formation. The test items were specific gravity, absorption, ultrasonic velocities (P-waves and S-waves) and unconfined compressive strength. The results are as shown in Table 6-8.

Table 6-8 Result of Rock Test

Sample: Specimen No.	Sample: Locality	Sample: Depth (m)	Rock name	Length (cm)	Diameter (cm)	Area of cross section (cm^2): Upper	Area of cross section (cm^2): Bottom	Area of cross section (cm^2): Average	Weight (g): Natural	Weight (g): Dry	Weight (g): In Water	Weight (g): Wet	Specific gravity: Dry	Specific gravity: Wet	Ratio of absorption (%)	Ultra sonic velocity (m/sec.) P-wave: Dry	Ultra sonic velocity (m/sec.) P-wave: Wet	Ultra sonic velocity (m/sec.) S-wave: Dry	Ultra sonic velocity (m/sec.) S-wave: Wet	Poisson's ratio: Dry	Poisson's ratio: Wet	Compressive strength: Max. load (kg)	Compressive strength: Max. strain ($\times10^{-3}$)	Compressive strength: Strength (kg/cm^2)	Compressive strength: Future of breaking	Modulus of elasticity E (kg/cm^2): Static	Modulus of elasticity E (kg/cm^2): Dynamic	Remarks
1	DH- 1	38.00	Andesite	5.52	4.19	13.8	14.1	14.0	175.84	175.30	103.50	178.74	2.33	2.37	1.96	3410	3430	2100	2170	0.192	0.168	7300	2.50	521.4	Large pieces	2.11×10^5	2.60×10^5	
2	DH- 1	65.00	Andesite	4.36	4.20	14.0	14.1	14.1	140.68	140.33	83.02	143.02	2.34	2.38	1.92	3350	3600	2180	2190	0.136	0.206	7300	3.50	517.7	Large pieces	1.48×10^5	2.75×10^5	
3	DH- 1	83.00	Andesite	6.21	4.19	13.7	12.2	13.0	182.01	181.27	107.11	188.25	2.23	2.32	3.85	2770	2890	1670	1730	0.180	0.221	2400	2.55	184.6	Large pieces	6.52×10^4	1.70×10^5	
4	DH- 1	98.00	Andesite	4.63	4.19	10.1	11.3	10.7	106.38	106.05	60.73	110.50	2.13	2.22	4.20	3260	3480	2020	2060	0.186	0.230	5000	1.65	467.3	Something into pieces	2.86×10^5	2.32×10^5	
5	DH- 1	105.00	Andesite	5.31	4.19	13.5	14.1	13.8	168.97	166.66	97.32	168.63	2.34	2.36	1.18	3880	4180	2380	2410	0.198	0.251	3800	3.00	275.4	Large pieces	9.52×10^4	3.43×10^5	
6	DH- 1	135.00	Andesite	5.72	4.20	14.0	14.2	14.1	187.05	185.94	108.96	187.76	2.36	2.38	0.98	2860	3030	1740	1820	0.204	0.218	7000	1.60	496.6	Large pieces	2.86×10^5	1.92×10^5	
7	DH- 1	150.00	Silty tuff	8.99	3.91	11.9	12.8	12.4	113.70	99.82	56.33	169.16	0.88	1.50	69.47	814	-	490	-	0.221	-	-	-	-	Broken in water	-	(D) 5.14×10^3	
8	DH- 1	158.00	Tuff breccia	6.78	4.12	13.6	13.6	13.6	134.33	132.40	69.40	155.64	1.54	1.80	17.55	1600	1650	860	880	0.297	0.301	420	5.10	30.9	Large pieces	5.71×10^3	3.63×10^4	
9	DH- 2	40.00	Andesite	7.61	4.18	14.0	14.0	14.0	235.70	235.10	139.80	243.00	2.28	2.36	3.36	2550	2800	1620	1710	0.159	0.203	7800	6.15	557.1	Small pieces	9.09×10^4	1.66×10^5	
10	DH- 3	15.00	Andesite	6.07	4.16	13.0	12.0	12.5	173.83	173.26	102.38	177.25	2.31	2.37	2.30	3490	3680	2090	2130	0.221	0.248	6000	3.75	480.0	Small pieces	1.25×10^5	2.68×10^5	
11	DH- 3	25.00	Andesite	6.53	4.19	14.1	13.9	14.0	208.00	207.00	122.64	211.50	2.33	2.38	2.17	3800	4110	2280	2420	0.221	0.235	11000	6.10	785.7	Something into pieces	1.25×10^5	3.44×10^5	
12	DH- 4	10.00	River gravel	4.34	4.18	13.8	13.3	13.6	125.96	125.44	73.91	131.27	2.19	2.29	4.65	2080	2420	1250	1360	0.215	0.269	3000	6.40	220.6	Small pieces	3.39×10^4	1.07×10^5	
13	DH- 4	19.00	Tuff breccia	4.70	3.99	12.2	12.7	12.5	125.34	119.35	73.53	136.46	1.90	2.17	14.34	1640	1830	890	-	0.294	-	-	-	-	Broken after measure P wave	-	(D) 3.69×10^4	
14	DH- 5	50.00	Weathered andesite	4.62	4.19	13.7	13.9	13.8	113.35	112.31	65.37	126.64	1.83	2.07	12.76	1230	1340	720	750	0.235	0.272	350	3.60	25.4	Something into pieces	6.90×10^3	2.96×10^4	
15	DH- 5	57.00	Andesite	5.80	4.19	14.0	13.8	13.9	173.75	172.87	102.61	181.65	2.19	2.30	5.08	2040	2220	1270	1310	0.180	0.233	3200	6.30	230.2	Large pieces	3.57×10^4	9.73×10^4	
16	DH-204	6.00	Weathered andesite	5.26	4.19	13.8	13.7	13.8	150.89	149.46	88.67	159.97	2.10	2.24	7.03	2050	2240	1180	1250	0.249	0.274	2070	7.20	150.0	Small pieces	2.08×10^4	8.92×10^4	
17	DH-204	12.00	Weathered andesite	5.81	4.18	13.7	13.5	13.6	169.75	169.20	98.32	175.68	2.19	2.27	3.83	2110	2370	1260	1310	0.221	0.280	2400	2.40	176.5	Small pieces	7.14×10^4	9.97×10^4	
18	Power House	-	Dacitic andesite	4.78	3.05×3.04	9.3	9.4	9.4	100.26	100.16	58.36	101.27	2.33	2.36	1.11	2100	2330	1190	1250	0.262	0.298	1650	2.90	175.5	Small pieces	5.88×10^4	9.57×10^4	
19	3-6-1 Rio Cauca	-	Andesite	4.81	3.05×3.05	9.6	9.4	9.5	103.21	102.77	60.23	104.23	2.34	2.37	1.42	4010	4040	2200	2100	0.329	0.315	5400	4.60	568.4	Small pieces	1.29×10^5	2.75×10^5	
20	La Titella	-	Meta Dolerite	4.54	3.05×3.03	9.7	9.5	9.6	123.36	123.08	81.76	123.45	2.95	2.96	0.30	5540	5670	3070	3100	0.280	0.287	13800	4.10	1437.5	Small pieces	3.64×10^5	7.32×10^5	
21	DH- 5	57.00	Andesite	5.09	4.19	13.7	14.0	13.9	151.63	150.81	89.64	158.63	2.19	2.30	5.19	2140	2230	1330	1410	0.186	0.167	4400	11.50	316.5	Small pieces	2.78×10^4	1.07×10^5	

According to these physical tests of rock, this andesite lava is considered to possess adequate conditions as rock material.

In order to confirm the degree of weathering of the andesite, X-ray diffraction was performed on cores from Boreholes DH-204 and DH-1. Hardly any clay minerals could be found in the diffraction chart with only traces of existence of halloysite and montmorillonite recognized. Otherwise, cristobalite is prominent, but this is due to crystallization as an igneous rock, and it is not a mineral produced by weathering. The results of X-ray diffraction indicate that the deep interior of this andesite lava is on the whole fresh.

In the present Study, geological investigations through boring were carried out at the above rock quarry area. The results are described in 6.6.

(2) Filter Material

The requirements for this material are 118,000 m^3. There are no large quantities of river deposits with suitable particle sizes in the vicinity of the dam site.

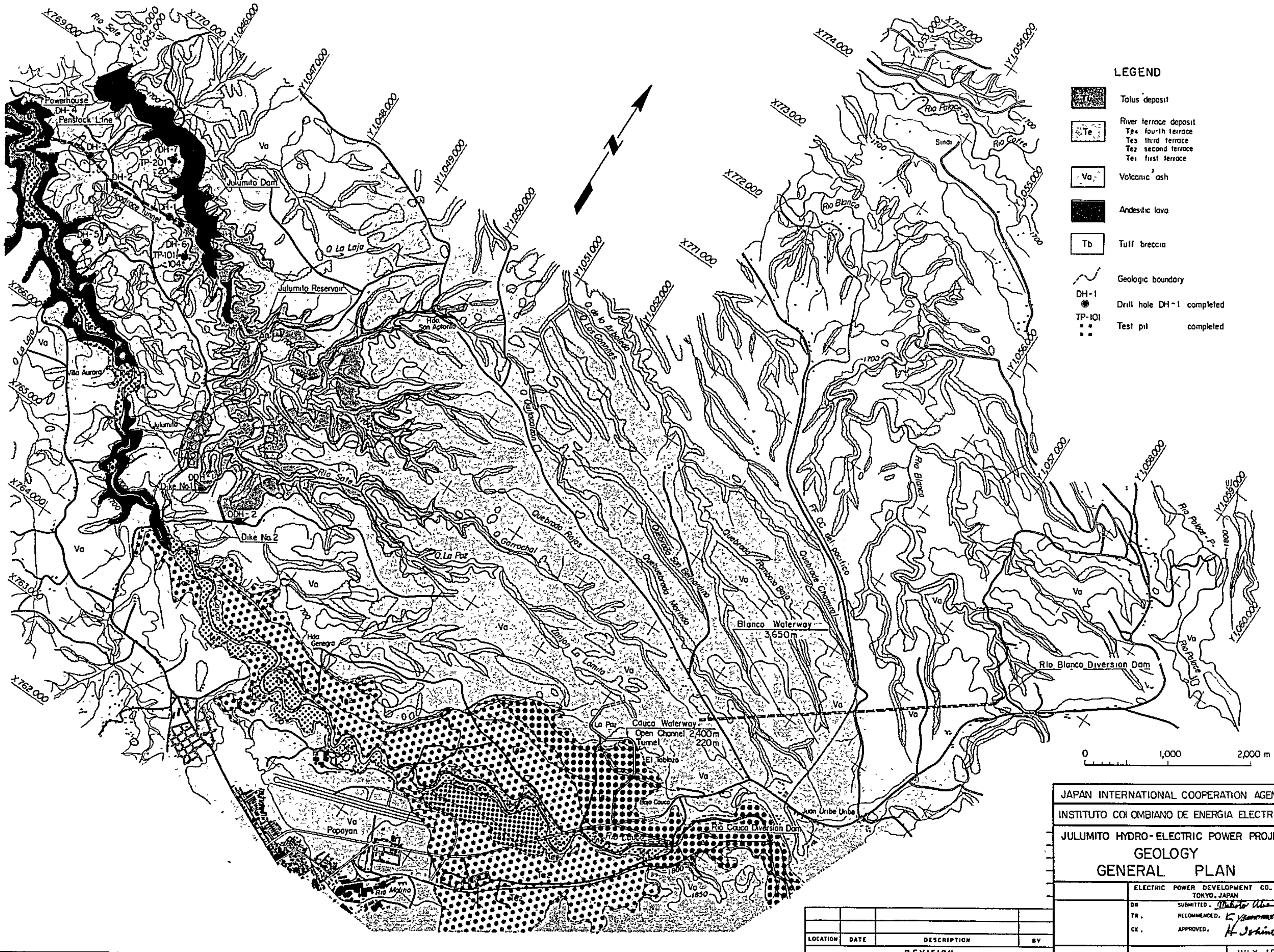
Meanwhile, since very fine-grained volcanic ash is to be used as core material, it will be desirable for the filter material to have a particle-size distribution exactly as designed, and the use of artificially manufactured aggregate is advantageous from the standpoint of quality control. It is thought the crude rock can be the same as the rock from the quarry area.

6.5.3 Concrete Aggretages

Approximately 80,000 m^3 of concrete aggregates will be required for the Julumito Hydroelectric Power Project. The plan is for aggregates to be secured through artificial manufacture mainly from the rock quarry area, and excavation much from the penstock and pressure tunnel.

According to the results of rock tests of the andesite lava at this site, the specific gravity is 2.35, the absorption is 3 - 5%, and compressive strength 500 kg/cm^2 to 700 kg/cm^2, and it is thought that with careful selection of the collection sites, coarse aggregate can be artifically manufactured from this andesite. As for fine aggregate, a quantity approximately half that of coarse aggregate will be required and it is thought this, too, can be manufactured from the andesite. However, since the crude rock contains a large amount of feldspar, the quality of fine aggregate is thought will be slightly inferior.

Meanwhile, it is planned also to supplementarily use river deposits of the Rio Timbio and the Rio Ondo from which aggregates were taken for the Florida II Power Station work. In carrying out detailed design, it will be necessary to have a quantitative grasp of these deposits.



JAPAN INTERNATIONAL COOPERATION AGENCY

INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA

JULUMITO HYDRO-ELECTRIC POWER PROJECT

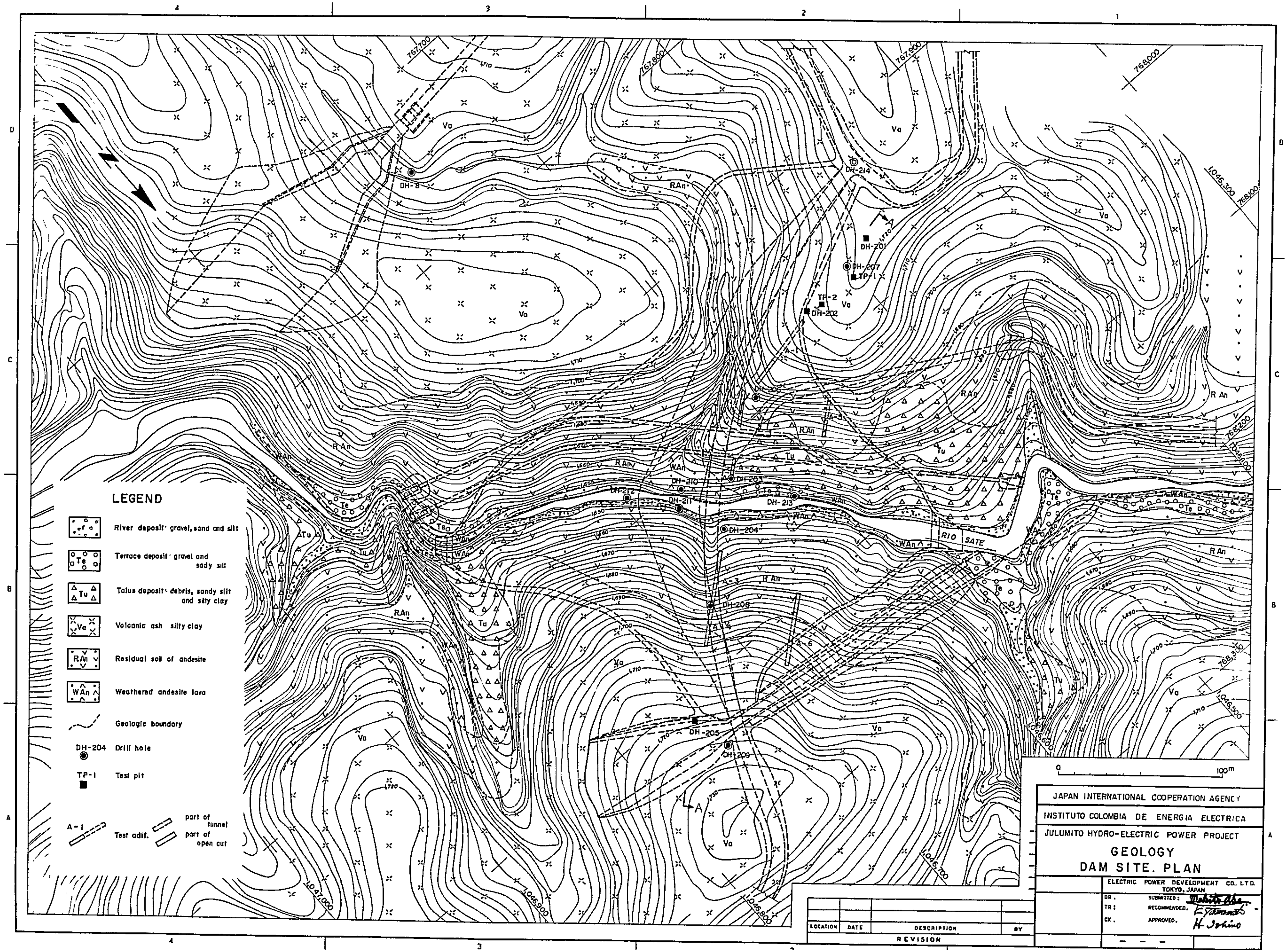
GEOLOGY GENERAL PLAN

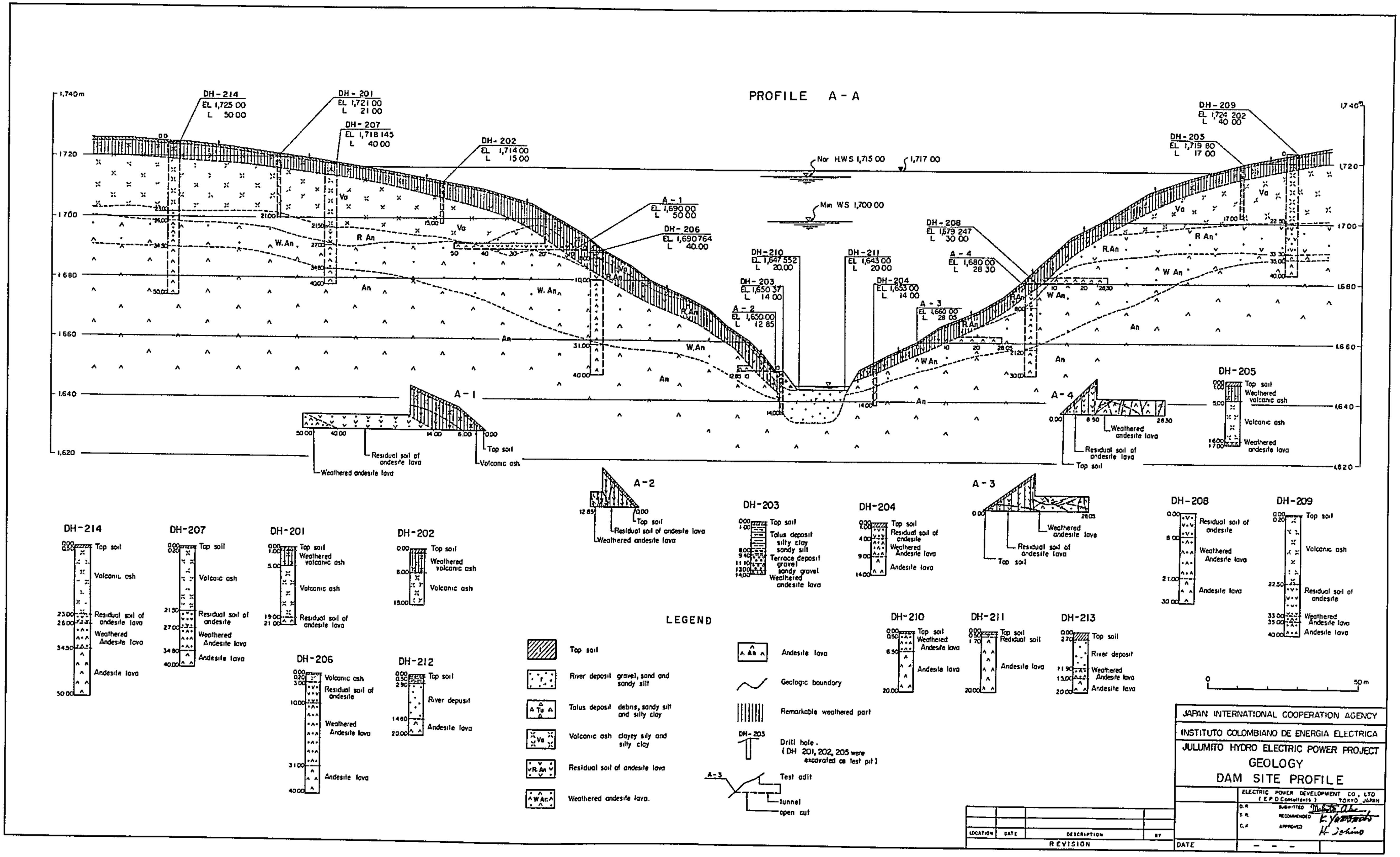
ELECTRIC POWER DEVELOPMENT CO., LTD. TOKYO, JAPAN

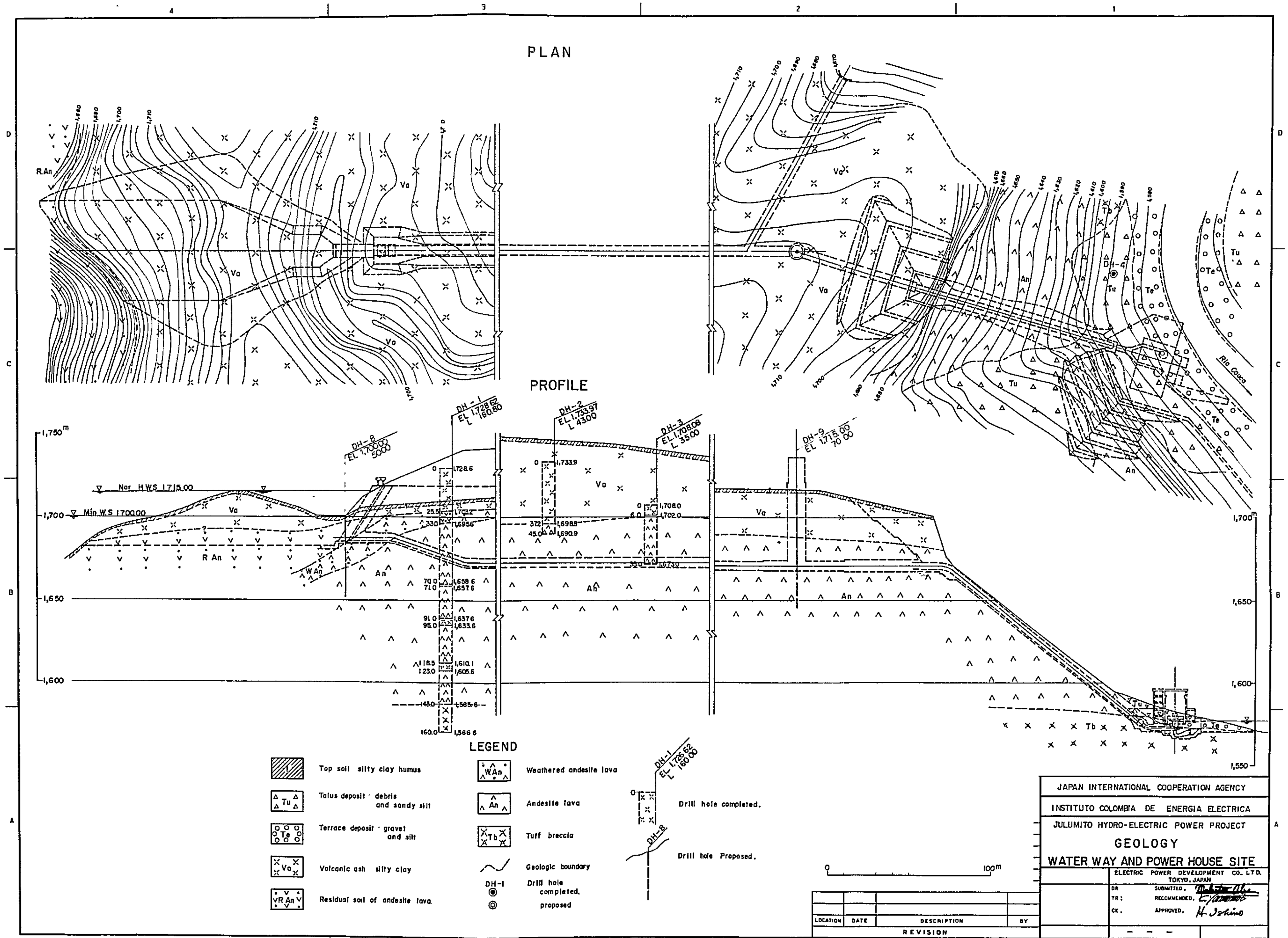
DR.	SUBMITTED. Makoto Abe
TR.	RECOMMENDED. K. Yamamoto
CK.	APPROVED. H. Ishino
– – –	JULY 1979

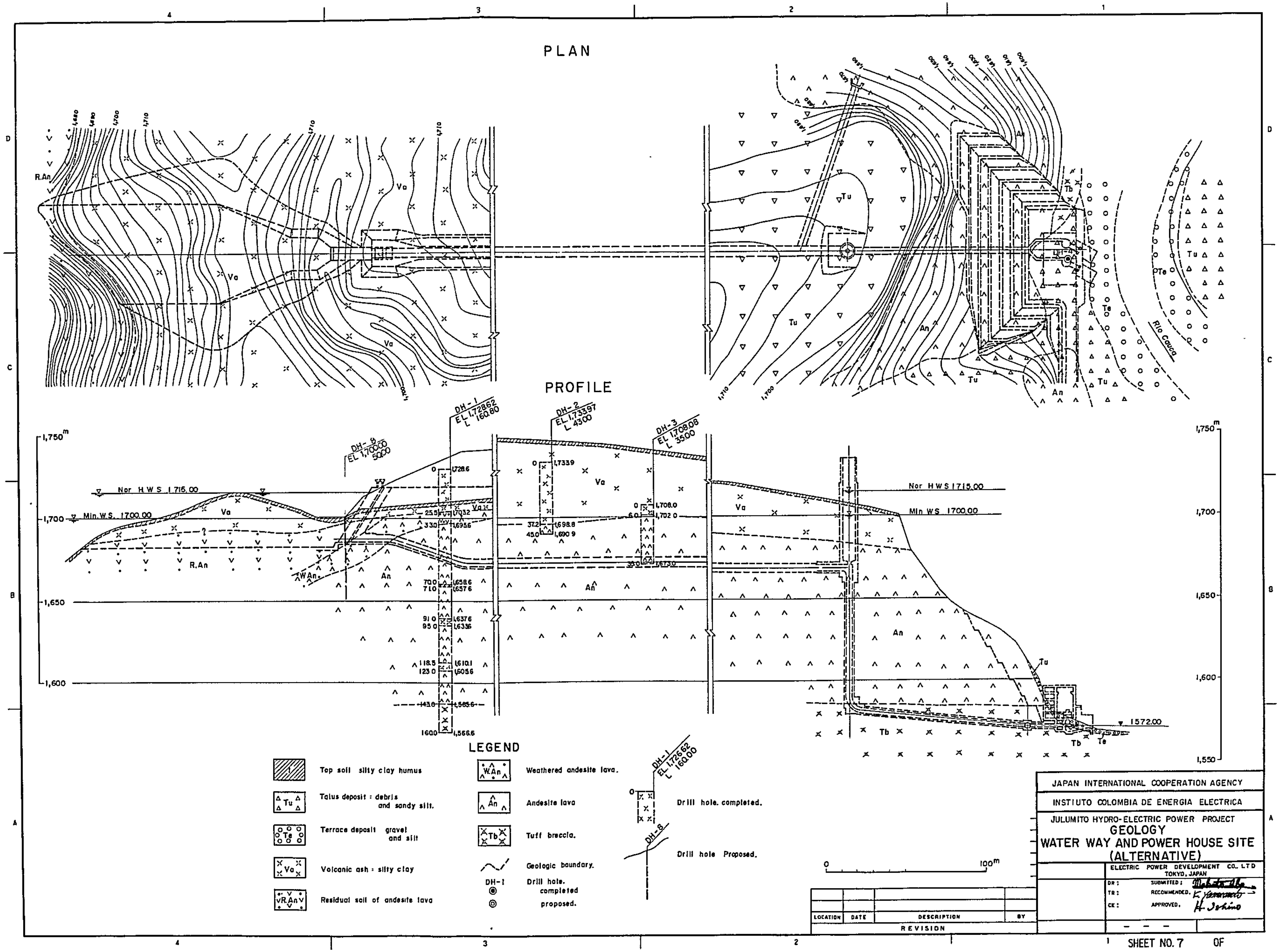
LOCATION	DATE	DESCRIPTION	BY
REVISION			

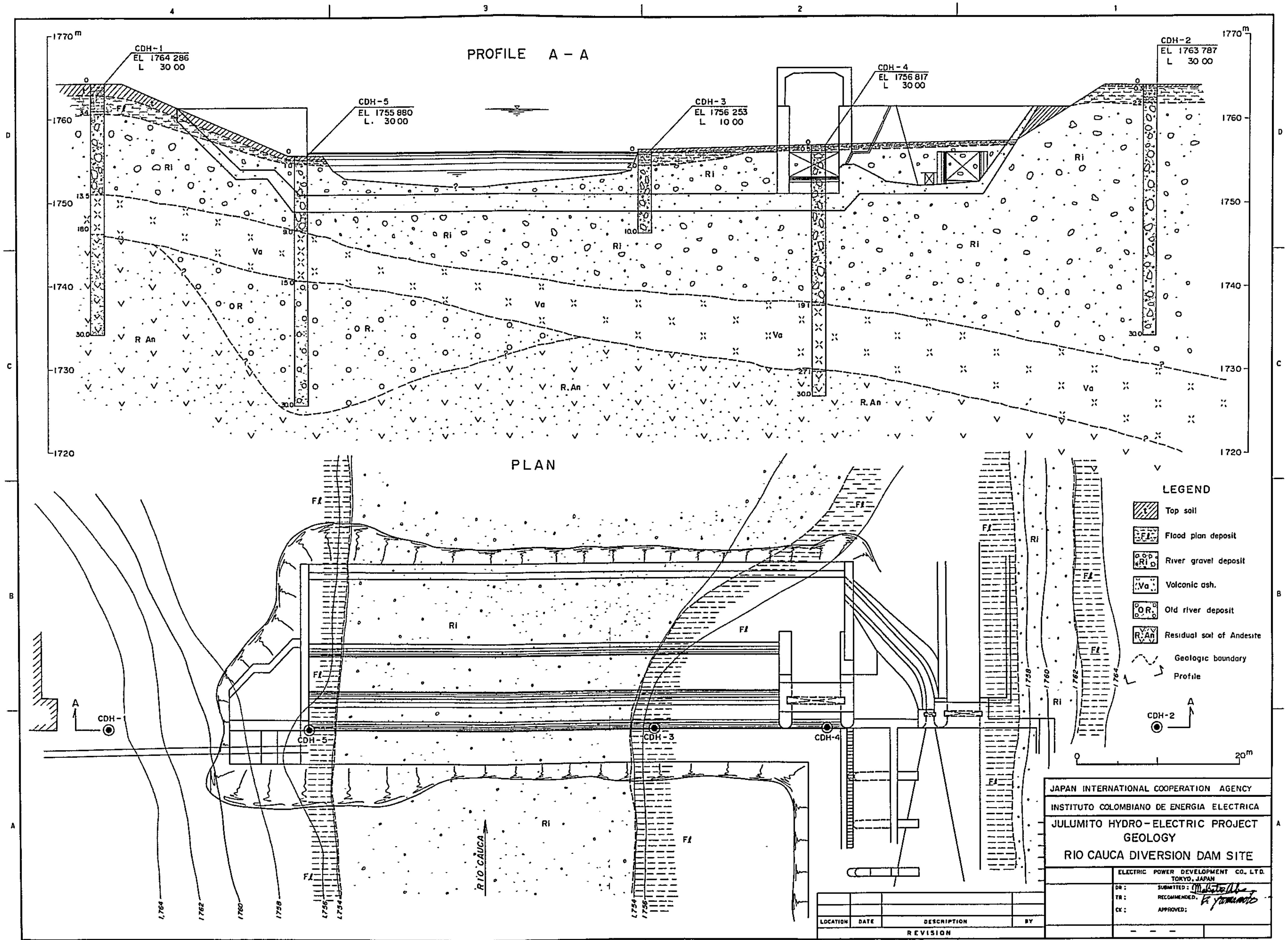
SHEET NO. 3 OF


LEGEND
River deposit: gravel, sand and silt
Terrace deposit: gravel and sady silt
Talus deposit: debris, sandy silt and silty clay
Volcanic ash silty clay
Residual soil of andesite
Weathered andesite lava
Geologic boundary
DH-204 Drill hole
TP-1 Test pit
A-1 Test adit. part of tunnel part of open cut
RIO SATE
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIA DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
GEOLOGY
DAM SITE. PLAN
ELECTRIC POWER DEVELOPMENT CO. LTD. TOKYO, JAPAN
SUBMITTED:
RECOMMENDED.
APPROVED.
LOCATION DATE DESCRIPTION BY
REVISION
SHEET NO. 4 OF


PROFILE A - A
DH-214 EL 1,725.00 L 50.00
DH-201 EL 1,721.00 L 21.00
DH-207 EL 1,718.145 L 40.00
DH-202 EL 1,714.00 L 15.00
A-1 EL 1,690.00 L 50.00
DH-206 EL 1,690.764 L 40.00
DH-210 EL 1,647.552 L 20.00
DH-203 EL 1,650.37 L 14.00
A-2 EL 1,650.00 L 12.85
DH-211 EL 1,643.00 L 20.00
DH-204 EL 1,653.00 L 14.00
A-3 EL 1,660.00 L 28.05
DH-208 EL 1,679.247 L 30.00
A-4 EL 1,680.00 L 28.30
DH-205 EL 1,719.80 L 17.00
DH-209 EL 1,724.202 L 40.00
Nor. H.W.S 1,715.00
1,717.00
Min. W.S 1,700.00
Va
R.An
W.An
An
Top soil
Volcanic ash
Residual soil of andesite lava
Weathered andesite lava
Weathered volcanic ash
Andesite lava
Talus deposit silty clay sandy silt
Terrace deposit gravel sandy gravel
River deposit
LEGEND
Top soil
River deposit gravel, sand and sandy silt
Talus deposit debris, sandy silt and silty clay
Volcanic ash clayey sily and silty clay
Residual soil of andesite lava
Weathered andesite lava.
Andesite lava
Geologic boundary
Remarkable weathered part
Drill hole. (DH 201, 202, 205 were excavated as test pit)
Test adit
tunnel
open cut
0 50m
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO ELECTRIC POWER PROJECT
GEOLOGY
DAM SITE PROFILE
ELECTRIC POWER DEVELOPMENT CO., LTD (E.P.D.Consultants) TOKYO JAPAN
SUBMITTED
RECOMMENDED
APPROVED
LOCATION DATE DESCRIPTION BY
REVISION
DATE
SHEET NO 5 OF


PLAN
PROFILE
Rio Cauca
DH-4
DH-1 EL 1,728.62 L 160.00
DH-2 EL 1,733.97 L 43.00
DH-3 EL 1,708.08 L 35.00
DH-8 EL 1,700.00 50.00
DH-9 EL 1,715.00 70.00
Nor HWS 1,715.00
Min W.S 1,700.00
LEGEND
Top soil silty clay humus
Talus deposit · debris and sandy silt
Terrace deposit · gravel and silt
Volcanic ash silty clay
Residual soil of andesite lava
Weathered andesite lava
Andesite lava
Tuff breccia
Geologic boundary
DH-1 Drill hole completed.
proposed
Drill hole completed.
Drill hole Proposed.
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIA DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
GEOLOGY
WATER WAY AND POWER HOUSE SITE
ELECTRIC POWER DEVELOPMENT CO. LTD. TOKYO, JAPAN
SUBMITTED.
RECOMMENDED.
APPROVED.
LOCATION
DATE
DESCRIPTION
BY
REVISION
SHEET NO. 6 OF


PLAN
PROFILE
Rio Cauca
Nor. H.W.S. 1,715.00
Min. W.S. 1700.00
Nor. HWS 1715.00
Min. WS 1700.00
DH-1 EL 1,728.62 L 160.80
DH-2 EL 1,733.97 L 43.00
DH-3 EL 1,708.08 L 35.00
DH-6 EL 1,700.00 50.00
1,750m
1,700
1,650
1,600
1,550
1572.00
LEGEND
Top soil silty clay humus
Talus deposit : debris and sandy silt.
Terrace deposit gravel and silt
Volcanic ash : silty clay
Residual soil of andesite lava
Weathered andesite lava.
Andesite lava
Tuff breccia.
Geologic boundary.
DH-1 Drill hole. completed proposed.
Drill hole. completed.
Drill hole Proposed.
0 100m
LOCATION DATE DESCRIPTION BY
REVISION
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTIUTO COLOMBIA DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC POWER PROJECT
GEOLOGY
WATER WAY AND POWER HOUSE SITE
(ALTERNATIVE)
ELECTRIC POWER DEVELOPMENT CO., LTD TOKYO, JAPAN
DR: SUBMITTED:
TR: RECOMMENDED.
CK: APPROVED.
SHEET NO. 7 OF


PROFILE A - A
CDH-1
EL 1764 286
L 30 00
CDH-5
EL 1755 880
L. 30 00
CDH-3
EL 1756 253
L 10 00
CDH-4
EL 1756 817
L 30 00
CDH-2
EL 1763 787
L 30 00
1770m
1760
1750
1740
1730
1720
PLAN
LEGEND
Top soil
Flood plan deposit
River gravel deposit
Volcanic ash.
Old river deposit
Residual soil of Andesite
Geologic boundary
Profile
RIO CAUCA
JAPAN INTERNATIONAL COOPERATION AGENCY
INSTITUTO COLOMBIANO DE ENERGIA ELECTRICA
JULUMITO HYDRO-ELECTRIC PROJECT
GEOLOGY
RIO CAUCA DIVERSION DAM SITE
ELECTRIC POWER DEVELOPMENT CO., LTD.
TOKYO, JAPAN
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
SHEET NO. 8 OF

6.6 Results of Geological Investigation Works

6.6.1 Quantities, Details and Locations of Investigation Works

Investigation works were carried out with drilling as main, while permeability tests were also performed at dam sites. At candidate sites for soil borrow areas, test pits were excavated in addition to drilling, soil samples were collected, and various soil tests were carried out. The details will be described elsewhere. The geological investigation works executed are indicated in the tables below.

Drilling

Location	Hole Name	Length (m)	Elevation	Coordinates X	Coordinates Y	Final Size of hole	Permiability test
Dam	DH-206	40	1690,764	1,046,595,330	767,991,770	NX	O
	DH-207	40	1718,145	1,046,501,832	767,970,031	NX	O
	DH-208	30	1679,247	1,046,698,464	768,068,340	NX	O
	DH-209	40	1724,202	1,046,746,121	768,137,284	NX	O
	DH-210	20	1647,552	1,046,666,218	768,004,564	NX	O
	DH-211	20				NX	O
	DH-212	20	1647,249	1,046,694,303	767,986,028	NX	O
	DH-213	20	1646,584	1,046,616,730	768,051,461	NX	O
	DH-214	50				NX	O
Dike	DDH-101	25	1715,173			NX	-
	DDH-201	25	1714,995			NX	-
	DDH-202	25	1712,161			NX	-
Intake	DH-8	50	1698,674	1,046,666,876	767,750,214	NX	-
Surgetank	DH-9	70	1718,399			NX	-
Cauca Diversion Dam	CDH-1	30	1764,286			NX	-
	CDH-2	30	1763,787			NX	-
	CDH-3	10	1756,253			NX	-
	CDH-4	30	1756,817			NX	-
	CDH-5	30	1755,880			NX	-
Borrow Area	BDH-1	40	1738,059	1,046,099,641	767,350,251	NX	-
Quarry Site	QDH-1	100	1730,197			BX	-
Total		745m	21 holes				

Test Pit

Location	Test Pit Name	Depth(m)	Elevation	Coordinates X	Y	Size of Pit
Borrow Area	BTP-102	5	1738,241	1,046,126,919	768,395,539	1.5 x 1.5m
	BTP-102	5	1735,965	1,046,111,559	768,375,152	1.5 x 1.5m
	BTP-103	5	1735,754	1,046,138,073	768,350,429	1.5 x 1.5m
	BTP-104	5	1713,441	1,046,196,636	768,374,800	1.5 x 1.5m
Total		20m	4 Test Pits			

6.6.2 Results of Dam Site Investigation Works

At the dam site, drilling was done near high water level, at mid-height and at the river bed on both banks. Drilling was performed at the river bed upstream and downstream of the dam axis and permeability tests were carried out at the boreholes.

(1) Rock Character and Degree of Weathering

According to Boreholes DH-214, DH-207 and DH-209 made in the vicinity of high water level, the volcanic ash layer has a thickness of approximately 20 to 22 m, and the elevation of its boundary on the left bank is about El. 1,690 m to 1,700 m. At the right bank, the boundary is at around El. 1,700 m. These boundaries are more or less symmetrical at both banks, and it is thought they exist roughly horizontally with some amount of irregularities from El. 1,690 m to 1,700 m. Under the boundaries there are strongly weathered layers of andesitic lava which existed as the ground surface before ejection of the volcanic ash. Weathered residual soil layers are distributed at thickness of around 5 m at the left bank and around 10 m at the right bank. The slopes facing the Rio Sate have been weathered even after distribution of volcanic ash, and show weathered residual soil layers of thickness of approximately 5 m with the exception of parts near the river bed.

Under these layers there is weathered andesite and cores successfully recovered are in rod form, indicating the andesite comprises bedrock, but are soft. Vertical joints exist, but are mostly closed by deposition of materials such as limonite. Although this weathered andesite layer is soft as bedrock, it is thought to possess sufficient bearing capacity and shear strength as the foundation for this dam. It has been clarified that this layer gradually changes to fresh andesite, which is found at depths of about 35 m near high water level at both banks, at around 20 m near mid-height, while at the river bed fresh andesite is directly exposed.

According to results from Boring DH-210, DH-211, DH-212 and DH-213 at the river bed, fresh andesite at the river bed was eroded in a U-shape through deepening of the Rio Sate to form a small gorge after which it was buried by a sand-gravel deposit. The depth is

14 to 15 m from the elevation of the present river bed.

(2) Results of Permeability (Lugeon) Test

Permeability tests were carried out every 2 m utilizing boreholes of NX size. Although there were some restrictions encountered in fixing packers and with pumping capacities, the tests were completed in more or less satisfactory condition.

The results of these tests are as shown in APPENDIX VII. The outlines are as described below.

The parts of fresh andesite and weathered andesite, in spite of the fact that there are vertical cooling joints in places, indicate 1 to 3 lugeons against pressure of 10 kg/cm^2 and may be said to be good impermeable bedrock. At the river bed portion there are some parts which are slightly high in permeability, but they can all be readily grouted by normal methods and will present no problem.

The parts of weathered residual soil indicate an average of about 25 lugeons and can withstand pressure application of 5 to 7 kg/cm^2. Although there are parts in places indicating around 50 lugeons, it is thought these can be treated by zone grouting consisting of 5 to 7 rows of set grouting.

Next, regarding the volcanic ash layer, a thickness 5 to 7 m of the surface layer indicates high permeability of 60 to 110 lugeons against pressure of 1 to 3 kg/cm^2. Further, a critical pressure clearly exists for parts deeper, and when a certain pressure is exceeded, the constitution of the soil is destroyed, and the original permeability is not restored even when the pressure is lowered. This critical pressure differs depending on depth, and although it cannot be distinctly established, failure does not occur in most cases at pressure of 3 kg/cm^2, but does at 5 kg/cm^2.

At the vicinity of El. 1,720 m, a thickness of about 20 to 25 m is made up by this volcanic ash, and the boundary continues on into the mountain in a plane close to horizontal. It is thought necessary for the core of the dam to be extended as much as possible into the mountain, and at the same time, for zone grouting to be done in the volcanic ash layer at low pressure.

It will be necessary for a grouting method to be concretely established and field grouting tests to be performed at the stage of definite design.

6.6.3 Results of Dike Site Investigation Works

The three boreholes of DDH-101, DDH-102 and DDH-201 were drilled at the No. 1 and No. 2 dike sites. All three were to a depth of 25 m and terminated on confirming the volcanic ash layer. The material in 1.5 to 2.0 m from the ground surface shows extreme soil formation and is soft. It is desirable for the dikes to be low as possible, but if excavation of foundations is deep, the dikes will become much higher. It should be aimed for excavation to be held to about 3 m, and stability of the dikes secured through counterweight fills upstream and downstream of the dikes in accordance with the shear strengths of the foundations. At the stage of detail design, it will be necessary for strength tests such as triaxial and uniaxial shearing tests with undisturbed samples to be performed excavating test pits of 5- to 8-m depths.

Further, from the ground surface to approximately 1.7 m, there are thin layers of old topsoil buried in the volcanic soil. This is not limited to the dike sites, but is a phenomenon widely seen in the surroundings of Popayan, and probably, there were repetitions of vegetation covering the ground surface to bring about stabilization, after which eruptions again occurred to cover the surface which was again covered by vegetation, and it is thought there were 3 or 4 violent eruptions producing volcanic ash falls at cycles of 2,000 to 3,000 years. These parts are especially soft and it will be necessary for a minimum of 2 m to be replaced with other soil materials.

6.6.4 Results of Intake Vicinity Investigation Works

Boreholes DH-8 (length 50 m) was provided at the intake site. This hole was drilled at the bottom of a ravine with down to 7.5 m consisting of 2 m of humus soil and 5.5 m of a volcanic ash layer under which was fresh andesitic lava, and it was clarified that there would be no problem with the foundation of the intake and the part passed by the headrace.

6.6.5 Results of Surge Tank Site Investigations Works

According to Borehole DH-9, the volcanic ash layer is to a depth of 32.7 m (El. 1,685 m), and the weathered andesite layer to 42 m, below which there is a fresh andesite formation. The riser portion of the surge tank is at around El. 1,670 m, and is in an adequately sound rock. However, the upper 33 m will be in the volcanic ash layer and it is thought sublining or extra lining will be necessary.

Further, with regard to DH-9, in spite of the fact that it was located fairly close to the cliff of the Rio Cauca, the groundwater level was stable between 22 and 27 m from the top during drilling to 70 m depth. This is felt to be strange as it is due either to the mountain being full of water with no place for drainage, or the coefficient of permeability is low with the hydraulic gradient high, whereas' when seen from the geological conditions of the cliff at the right bank of the Rio Cauca where in spite of columnal jointing being fairly well-developed and there being considerable creep phenomena, the groundwater level is stable at a high elevation. In the case of the former it is expected there will be considerable springing of water during excavation, with this possibility being very strong.

6.6.6 Results of Cauca Diversion Dam Site Investigation Works

The Rio Cauca flows from a mountainous area to a plain around Popayan, with 4 steps of river terraces formed, while further, a gorge produced by deepening reaches the vicinity of the projected powerhouse site.

The Cauca Diversion Dam site is located where the Rio Cauca flows out into the plain and the river basin begins to spread out, with the first terraces at both banks and part of the fourth near the river bed.

In the present investigation, 5 boreholes were drilled at the dam axis. Boreholes CDH-1 and CDH-2 were provided on the first terraces at both banks. At CDH-1 on the left bank, down to 3.4 m from the ground surface is a flood-plain deposit consisting of fine-grained material, underlying which a sand-gravel layer has been confirmed down to 12.4 m, followed by a volcanic ash layer, and from 18 m, a light purplish-gray residual soil thought to belong to weathered andesite.

The sand-gravel layer is thick at the right bank and at CDH-2 drilling was done to a depth of 30 m, but it was still sand-gravel. Boreholes CDH-3, CDH-4 and CDH-5 were drilled at the left and right banks on either side of the present river bed, and the sand-gravel layer was passed to reach the foundation volcanic ash layer and weathered andesite at 9.0 m in CDH-3 and at 19.5 m in CDH-4. However, in CDH-5, a porous volcanic lapilli layer thought to be a pyroclastic flow is reached at 9 m after which sand-gravel is found again at 15 m. There is much that is not clear about this condition. In other words, if the upper volcanic lapilli layer is not a deposit of the present river bed, this underlying sand-gravel layer can be considered to be the deposit of an old river bed of an ancient age. Deposits of like this not connecting anywhere are often seen along the Rio Cauca. In any event, the dam is to be a low, floating type and although it would not be necessary to perfectly treat all of these permeable layers, it will be desirable for further boring to be added on both sides of CDH-5 at the stage of definite design to follow.

On looking at boreholes such as CDH-4 and CDH-3, the sand-gravel layers are well-interlocked and they are considered to be fairly well-compacted. In effect, while drilling with diamond bits, cores were recovered without movement of gravels, with gravels mutually interlocked, and it is thought interstices are filled with sand. It is considered there will be no special problem as the foundation for the dam.

6.6.7 Results of Soil Borrow Area Investigation Works

The projected soil borrow area is located at the left bank of the Rio Sate 600 m downstream of the dam site, selected where the volcanic ash layer close to the ground surface is eroded and the underlying weathered residual soil of andesite and weathered andesite can be borrowed simultaneously.

The investigation works consisted of one borehole 40 m in length and 4 test pits of depth of 5 m.

Three of the test pits were excavated in the volcanic ash layer, with one in the part of weathered residual soil. Soil samples were collected from all of the pits, and the results of soil tests are as described in detail in 6.7.

As the result of boring, the thickness of the volcanic ash layer is found to go down to 26.5 m, underlying which is weathered residual soil to 34.00 m, after which is weathered andesite. So far as seen from this boring the weathered residual soil is very fine-grained, and it is thought to be unsuitable as core material to be used mixed with volcanic ash. Consequently, it is considered advisable to look elsewhere for material for mixing.

6.6.8 Results of Quarry Site Investigation Works

The quarry site is located 400 m downstream of the scheduled powerhouse site at the right bank of the Rio Cauca. Drilling of length of 100 m was done at this location at NX size from the ground surface to 75 m, and at BX size further below.

The volcanic ash layer is relatively shallow extending down to 26.0 m, while the underlying andesite is greatly weathered and core recovery was extremely poor. The core was recovered in rod form from a depth of 54 m and the rock is then sound. Good bedrock continues further down, but from the point where the borehole diameter was changed from 75 mm to BX, in spite of the fact that the rock character should be the same, the core recovery rate became extremely poor. This is thought to be due to various causes such as excessive pressure application and falling of rock fragments from the borehole wall, but because of the poor recovery rate, even the constituent rock character is unclear.

At this quarry site, although the volcanic ash is relatively shallow at 26 m, weathering goes down to approximately 54 m and it cannot be said to be very good as a quarry. Based on the results of the present investigations, it is thought it will be of significance to carry out boring at the alternative site on the opposite bank of the powerhouse, and in addition, to examine methods of quarrying.

6.7 Results of Soil Tests

Soil tests were performed on material collected at the 4 test pits in the candidated borrow area indicated in Table 6-6-2. The various test results are shown in APPENDIX VIII. The following observations may be made from these test results.

The natural water contents of the materials are between 40 to 60%, and they are same or slightly lower than the plastic limit, while degrees of saturation are 80% or higher. However, the natural water contents of the material from BTP 103 are abnormally high at 100 to 200%, and the liquid limits and plastic limits are also extremely high. The specific gravities of soil grains are from 2.7 to 2.9, but the natural void ratio is high at approximately 1.9, and consequently, the unit weights are also low at 1.1 to 1.6 g/cm^3.

Mechanical analysis of soil was performed by the sieve method and the sedimentation method. According to these tests, practically all of the samples had particles smaller than 74μ as much as 90% or more by weight, and had particles smaller than 2μ about 50%. The material from BTP 103 has a gap grading at particle size of around 20μ, and has particles smaller than 2μ about 30% by weight. So far as seen from the grading of this material, it is thought the material possesses adequate impermeability.

Compaction tests were performed on representative samples by the two methods of dry and non-dry. (Dry method pertains to a method where samples are dried at the preparatory stage, while non-dry method pertains to a method where samples are not dried and are tested in conditions of natural water content.) In compaction tests of such fine-grained volcanic ash soils, in general, the more that drying treatment is done in sample preparation, that is, the more that the water content ωo is lowered, the maximum dry density γdmax becomes higher with the same work quantity, and the optimum water content ωopt becomes lower. However, according to the test results of this material, the extent is that γd of the material which is dried more is slightly higher, and ωopt is lower, with there being not much difference, which is unbelievable. It is considered that the proportion of confined water in the moisture of soils is small and this material possesses a sandy soil-type property, and it is necessary that further tests should be performed and examinations should be made.

According to consistency tests, approximately, liquid limits are between 60 to 80%, plastic limits between 45 to 60% and plasticity indices between 15 to 20%, and the soil belongs to MH going by the Classification of U. S. C. E. Compared with the previous test results, both liquid limits and plasticity indices are higher, and are reasonable considering the gradation. However, as previously described, with regard to the material of BTP 103, abnormal values are indicated for both liquid limit and plasticity index.

The triaxial tests performed were $\overline{CU}$ tests, CD tests and UU tests. The ϕ' values according to $\overline{CU}$ tests were approximately 35° and are slightly higher than ϕ_{CD} (31° - 34°) according to CD tests, while C' is approximately 0.3 kg/cm^2 and slightly lower than C_{CD} (approximately 0.6 kg/cm^2) according to CD tests. The ϕ_u value according to the UU test is 11°, while C_u is 0.8 kg/cm^2. So far as seen from the results, though the material consists of cohesive soil having particles smaller than #200 90% or more, high ϕ values and low C values are indicated. It is considered that this material has an inactive property as a cohesive soil. On investigation of the degree of activity of this sample, the result is

$$\text{Degree of Activity of Cohesive Soil} = \frac{\text{Plasticity Index}}{\text{Soil Particles smaller than } 2\mu} \doteqdot \frac{20}{50} = 0.4$$

and the activity is small.

When this material is to be used as core material, the following two methods will be considered.

a. Case of Using this Material Only as Core Material

This material is of high natural water content, while moreover, the void ratio is high, and it is conceivable that there will be occurrence of high pore pressure during embankment and settlement due to consolidation. Because of this, it is necessary that the dissipation of pore pressure should be expedited, and the settlement due to consolidation after completion should be controlled by making core thickness smaller and by restricting the speed of work execution. Further, since cohesion of this material is comparatively small while the core is to be thin, it will be necessary for special care to be exercised with regard to piping. In such case, since the grain sizes of the core materials are very fine, it is considered a single filter zone will be insufficient. It is necessary that more than one filter zone should be provided under strict quality control.

b. Case of Using this Material Mixed with Other Coarse-Grained Material

The quality of the material is to be improved by mixing with another coarse-grained material. Through adjustments of the mix proportions, material of suitable gradation, density, impermeability, and strength, with little production of pore-water pressure and having good workability is to be made. For this purpose, it is necessary for a suitable coarse-grained material to be found, and new tests to be performed on the mixed materials.

CHAPTER 7

STUDIES OF DEVELOPMENT SCALE AND POWER GENERATION PLAN

CHAPTER 7 STUDIES OF DEVELOPMENT SCALE AND POWER GENERATION PLAN

CONTENTS

FIGURE LIST

TABLE LIST

CHAPTER 7 STUDIES OF DEVELOPMENT SCALE AND POWER GENERATION PLAN

7.1. Basic Principles and Conditions

7.1.1 Basic Principles

The basic development concept of the Julumito Hydro-electric Power Project is that of diverting the water of the Rio Cauca mainstream and the tributaries, Rio Palace and Rio Blanco, and conducting the water by waterways (Cauca Waterway, Palace Waterway and Blanco Waterway) to Julumito Reservoir provided on the Rio Sate, another tributary of the Rio Cauca. The water conducted, after being regulated at the reservoir, is to be led through a headrace tunnel to Julumito Power Station for generation of electric power.

Consequently, in order to determine the scale of development of the Project, it is necessary for the following studies to be made.

a) Study of basic development plan.
b) Studies of location and type of dam.
c) Studies of location and type of powerhouse.
d) Studies of types and capacities of waterways.
e) Study of reservoir scale (reservoir high water surface level and effective storage capacity).
f) Studies of available discharge of power station (pressure tunnel capacity) and optimum scale of equipment installed.

Of the above, studies have already been made regarding a), b) and c) in "Preliminary Study on Julumito Hydro-electric Power Project, June 1970," and they will be omitted from the present Report. Accordingly, studies of d), e) and f) will be made in this chapter to determine the optimum scale for the Julumito Hydro-electric Power Project.

7.1.2 Basic Conditions

The study of the above optimum scale is to be made in accordance with the following conditions:

(1) Basic Development Plan

Plan C selected as the optimum development pattern in above-mentioned Preliminary Study is to be adopted.

(2) Location and Type of Dam

As the location and type of the dam, the No. 2 dam site and rockfill dam as concluded in the Preliminary Study are to be adopted.

(3) Location and Type of Powerhouse

As the location and type of the powerhouse, the Rio Cauca right bank downstream plan and a surface type as concluded in the Preliminary Study are to be adopted.

(4) Evaluation Standards for Determining Optimum Scale

The respective costs of the alternative thermal Power Station and Julumito Hydro-electric Power Station in consideration of the conformity with the economic evaluation criteria described in Chapter 11 were calculated according to the following:

a) kW and kWh Costs of Alternative Power Station

The cost of the alternative thermal power station calculated in Chapter 11 is the total cost including construction cost, operation and maintenance cost, fuel cost, and replacement cost corresponding to the service life of hydro-electric power station of 50 years, and cost escalation during the service life is taken into consideration. And the kW and kWh benefits at the beginning of 1985 when Julumito Hydro-electric Power Station will be started up were calculated taking above factors into account as follows:

(i) The fuel cost in 1979 at the transmitting end of the No. 3 unit of Paipa Thermal is expressed as the following:

US¢0.185/1000 kcal x 860 kcal/kWh x 1/0.313 = US¢0.508/kWh

where, thermal efficiency is 31.3%. Therefore, the kWh benefit at New Popayan Substation of Julumito Hydro-electric Power Station as of the beginning of 1985 may be calculated as follows:

US¢0.508 $(1 + 0.57)^6$ x 1/(1 - 0.003) (1 - 0.01) = US¢0.826 (kWh)

where, station service loss is 0.3% and transmission line loss is 1.0%.

(ii) The average fuel cost $_1$ (kWh benefit) during the service life of Paipa Thermal Power Station is expressed as follows (see Appendix IV):

$$\alpha_1 = \frac{a(1+i)[(1+i)^n - (1+e)^n]}{(1+i)^n(i-e)} \times \frac{i}{1 - \frac{1}{(1+i)^n}}$$

$$= \frac{0.826(1+0.108)[(1+0.108)^{25} - (1+0.07)^{25}}{(1+0.108)^{25} \times (0.108 - 0.07)} \times \frac{0.108}{1 - \frac{1}{(1+0.108)^{25}}}$$

= US¢1.652/kWh

where, a : fuel cost in 1985

i : 10.8% (average interest rate during service life)

e : 7.0% (escalation rate of fuel cost)

(iii) Benefit-Cost Ratio Julumito Hydro-electric Power Station to Alternative Thermal Power Station

The benefit-cost ratio determined from the total costs of Julumito Hydro-electric Power Station and the alternative thermal Power Station converted to present values as of the beginning of 1979 is 1.567. Meanwhile, since the annual cost of Julumito Hydro-electric Power Station averaged for the service life is US$10,935,000, the kW benefit will be the following:

$$\alpha_2 = \frac{1.567C \cdot f - G \cdot \alpha_1}{p}$$

$$= \frac{1.567 \times 103,200 \times 10^3 \times 0.106 - 300 \times 10^6 \times 0.01652}{53,000}$$

$$= \text{US\$}229.91/\text{kW}$$

where, C : construction cost of Julumito Hydro-electric Power Station

f : annual cost of Julumito Hydro-electric Power Station

G : energy production of Julumito Hydro-electric Power Station

: fuel cost of alternative thermal Power Station

Based on the above, the kW and kWh benefits of the alternative thermal Power Station will be decided as follows:

kWh benefit (α_1) : US$0.01652/kWh

kW benefit (α_2) : US$229.91/kW

b) Annual Cost and Generating Cost of Julumito Hydro-electric Power Station

In estimating the annual cost of Julumito Hydro-electric Power Station, the interest, depreciation cost, repair expenses, personnel expenses and share of general administrative costs for the service life of the power station must be calculated.

Without consideration of the loan conditiones described in Chapter 12, Financial Analysis, the interest and depreciation on the construction cost of Julumito Hydro-electric Power Station were calculated as follows:

The interests 8% for the foreign currency of the construction cost and 10% for the domestic currency are considered for the service life of 50 years. As a result, the interest and depreciation cost according to the sinking fund method are 9.1%.

Personnel expenses, repair expenses and general administrative costs are generally expressed in percentages to construction cost, and internationally, there are not much differences in these rates. The following percentages to construction cost were assumed for the Julumito Hydro-electric Power Project:

Personnel cost, repair cost	:	1.0%
General administrative cost, others	:	0.5%

Based on the above, the cost factor of Julumito Hydro-electric Power Station is estimated as follows:

Interest and depreciation	:	0.091
Personnel cost and repair cost	:	0.010
General administrative cost, others	:	0.005
Total	:	0.106

Consequently, the annual cost of Julumito Hydro-electric Power Station and the generating cost at New Popayan Substation will be the following:

$$\text{Annual Cost} = \text{Construction Cost} \times \text{Annual Cost Factor}$$
$$= \text{US\$}103{,}200 \times 10^3 \times 0.106$$
$$= \text{US\$}10{,}939 \times 10^3$$

$$\text{Generating Cost} = \frac{\text{Annual Cost}}{\text{Available Energy Production}}$$
$$= \frac{10{,}939 \times 10^3}{300 \times 10^6} = \text{US\$}0.0365/\text{kWh}$$

(5) Method of Evaluating Diversion Waterway and Reservoir Scales

Based on the fundamental conditions described above and considering the capacities of the Palace, Blanco and Cauca waterways and the dam height as parameters, the benefits (B) of the vari ous scales were determined from the kW and kWh values of the alternative thermal power station, while the costs (C) of the respective scales were determined from the construction costs and cost factor (cost factor 10.6% for all development schemes), and the scheme resulting in maximum benefit-cost ratio B/C was taken to be the optimum scale.

As for operation of the reservoir, 7.4, "Operation of Reservoir" was applied as a result to all schemes, and the available energy production figures were calculated.

7.2 Studies of Diversion Waterways

7.2.1 Studies of Diversion Waterway Routes and Types

The direct catchment area of Julumito Reservoir located in the Rio Sate is 31 km^2, which is only 3% of the total catchment area of 1,124 km^2 of the Project. The remaining 1,093 km^2 are made up by the catchment areas of the Rio Cauca, the Rio Palace and the Rio Blanco, and the waters of these catchment areas are to be conducted by waterways to Julumito Reservoir.

The water collection catchment areas at the diversion dam sites of the rivers which are to be diverted to Julumito Reservoir are as indicated below.

Diversion Waterway	Own Catchment Area (km^2)	Cumulative Catchment Area (km^2)
Palace	197	197
Blanco	39	236
Cauca	857	1,093

The diversion waterways to be constructed for collection of water from these catchment areas will be comparatively long, in addition to which the capacities will be large, so that their construction costs will affect the development scale and the economics of the Julumito Project as a whole. Also, the types of the waterways must be selected considering the aspect of maintenance after completion.

(1) Cauca Diversion Waterway

This is the most important of the diversion waterways. The route of Cauca Diversion Waterway will pass a flat terrain at the right bank of the Rio Cauca so that there will be no fear of inflow of sediment. Therefore, there will be no problem of maintenance so that an open canal plan of the lowest construction cost is to be adopted, and moreover, a route of the shortest waterway length is selected.

(2) Palace and Blanco Diversion Waterways

Approximately 26% of the entire diversion quantity is to be collected by these two waterways. The routes of these waterways are in a hilly area of complex topography where there are many undulations such as hills and valleys. Consequently, tunnels and open canals (partly tunnle) are conceivable for the waterways. Comparisons were made of the open canal (partially tunnel) and the tunnel proposals regarding the routes and types of the Palace and Blanco diversion waterways. The results are as shown below.

	Open Canal	Tunnel
Palace Waterway		
Maximum capacity (m^3/sec)	12.0	12.0
Length (m) Open canal	3,970	-
Tunnel	420	770
Cross section (m) Open canal	4.5 x 1.9 x 2.2	-
Tunnel	2.8 x 2.8	2.8 x 2.8
	(Semi-circular top, square bottom)	(Semi-circular top, square bottom)
Blanco Waterway		
Maximum capacity (m^3/sec)	13.8	13.8
Length (m) Open canal	3,950	-
Tunnel	850	3,650
Cross section (m) Open canal	4.8 x 2.0 x 2.3	-
Tunnel	3.0 x 3.0	3.0 x 3.0
	(Semi-circular top, square bottom)	(Semi-circular top, square bottom)
Construction cost *1 (US$)	5,840,000	7,470,000
Maintenance and repair cost *2 (US$)	1,860,000	470,000
Construction cost plus maintenance and repair cost (US$)	7,900,000	7,940,000

*1. Construction cost consists of direct civil construction cost only. Contingency and interest costs are not included.

*2. Amounts of maintenance and repair costs for the service life of 50 years converted to present worth.
However, interest rate 10%, and escalation of commodity price and personnel costs taken to be about 10% annually.
In case of the open canal alternative, the cost of preventing inflow of sediment from the mountain-side is included in addition to normal maintenance costs.

In the cases of the tunnel proposals, both tunnels would be provided mostly in the volcanic ash layer (partly weathered andesite), but unless there are problems in particular such as leakage, it is thought there will be no special trouble in construction.

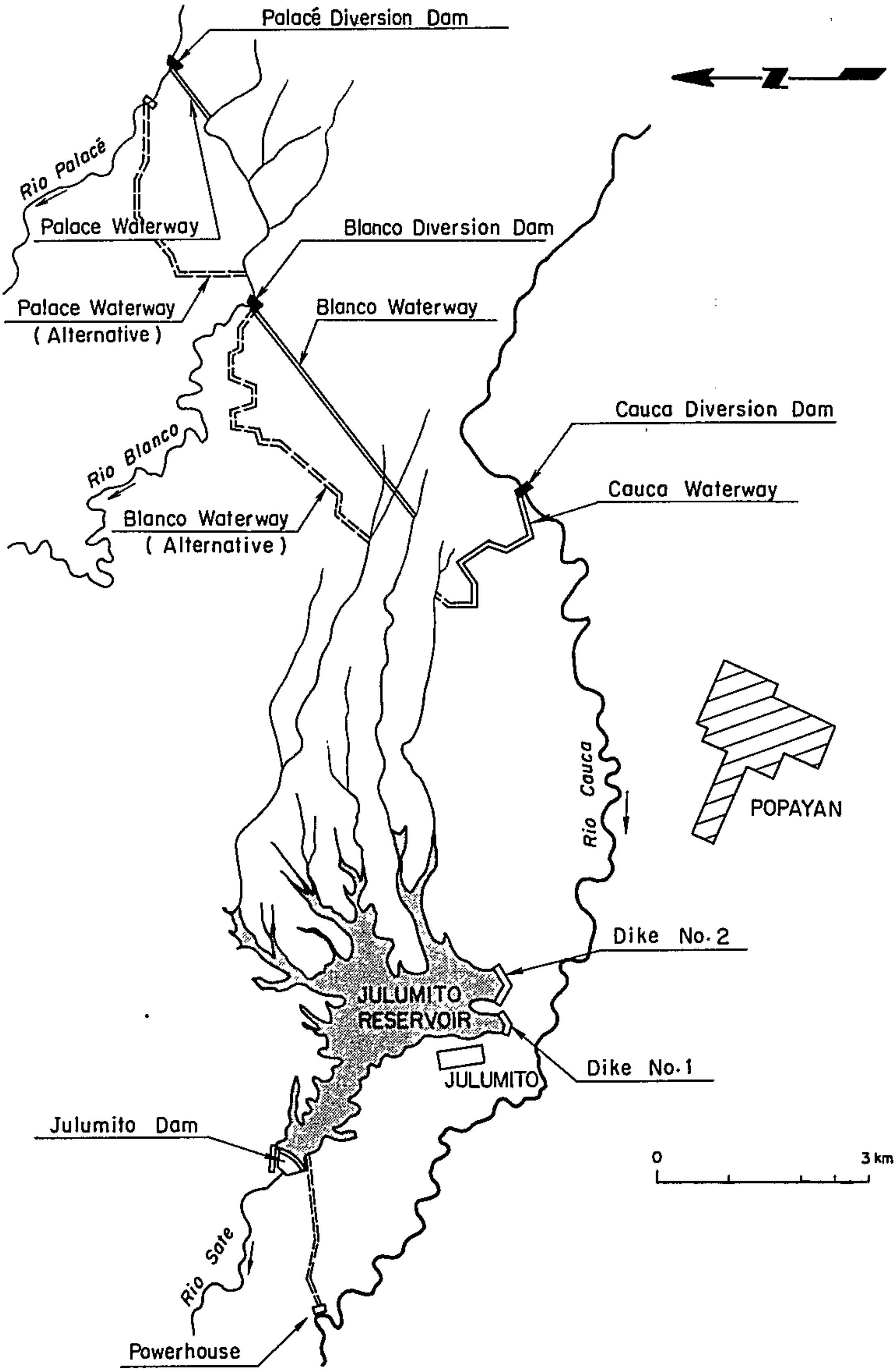
Further, as seen from the administrative aspects of maintenance and repair in the future, the tunnel proposals will involve hardly any problems and the maintenance costs will be extremely low.

On the other hand, since an open canal would be surface work, construction work would be easy, in addition to which the initial construction cost would be low. However, seen from the topographical and geological conditions of the waterway routes, it must be considered that soil will slide from the mountainsides into these long waterways when being operated after completion. It is expected that the maintenance cost would be considerable and if the construction cost and the maintenance cost were to be evaluated converted to present worth they would be practically equal.

Further, there would be reduction in power generation due to discontinuation of water intake in case soil should slide in.

Therefore, with regard to the two diversion waterways of Palace and Blanco, future maintenance are taken into consideration and the tunnel proposals are adopted although the initial construction cost may be somewhat higher.

Fig. 7-1 Route of Diversion Waterway



7.2.2 Study of Diversion Waterway Capacities

The optimum capacities of the diversion waterways are to be studied according to the following method. The intakes from the diversion dam sites on the Rio Cauca, Rio Palace and Rio Blanco are to be as follows:

Maximum Intake Discharge at Rio Cauca Diversion Dam Site (Maximum Capacity of Cauca Diversion Waterway)

30.0 m^3/sec, 35.0 m^3/sec, 40.0 m^3/sec, 45.0 m^3/sec

Maximum Intake Discharge at Rio Palace Diversion Dam Site (Maximum Capacity of Palace Diversion Waterway)

8.0 m^3/sec, 10.0 m^3/sec, 12.0 m^3/sec, 14.0 m^3/sec

Maximum Intake Discharge at Rio Blanco Diversion Dam Site (Maximum Capacity of Blanco Diversion Waterway)

1.4 m^3/sec, 1.6 m^3/sec, 1.8 m^3/sec, 2.0 m^3/sec

Power generation costs, benefits, annual expenses, etc. for combinations of the above maximum intake quantities are to be studied and the most economical intake quantities (maximum capacities of waterways) are to be determined.

Calculations are to be made according to the following conditions:

(1) The high water level of Julumito Reservoir is to be at 1,715.00 m and the effective storage capacity is to be 50.37 x 10^6 m^3.

(2) The dependable discharge is obtained operating the reservoir according to the operation rules and the power station output enabling 12-hour operation is to be set.

(3) Other general conditions are to be according to 7.1.2.

The results of studies according to the above are shown in Table 7-1, Fig. 7-2 and Fig. 7.3. As is clearly seen in the table and figures, the most economical scale is when the intake from Rio Cauca Diversion Dam is 40.0 m^3/sec and that from Rio Palace Diversion Dam 12.0 m^3/sec. Consequently, the maximum capacities of the waterways are to be the following:

Cauca Diversion Waterway	40.0 m^3/sec
Palace Diversion Waterway	12.0 m^3/sec
Blanco Diversion Waterway	13.8 m^3/sec*

(Table 7-1 (1)-(2), Fig. 7-2, Fig. 7-3)

* As a result of similar study of the optimum intake at Rio Blanco Dam, it was found to be 1.8 m^3/sec.

Table 7-1 (1) Study on Optimum Capacity of Cauca and Palace Diversion Waterway

(Unit: m^3/sec.-day)

Case	Mark	Capacity of Waterway		Rio Cauca				Rio Palace'				Rio Blanco				Rio Sate	Reservoir Inflow
		Cauca (m^3/sec.)	Palace' (m^3/sec.)	Run-off	Intake-flow	Over-flow	Intake-Ratio	Run-off	Intake-flow	Over-flow	Intake-Ratio	Run-off	Intake-flow	Over-flow	Intake-Ratio		
1	30 - 8		8.0 (9.4)						2,212.4	1,121.8	66.4						10,830.0
2	30 - 10	30.0	10.0 (11.4)	9,019.6	7,920.4	1,099.2	87.8	3,334.2	2,444.7	889.5	73.3	410.5	363.2	47.3	88.5	334.0	11,062.3
3	30 - 12		12.0 (13.4)						2,616.8	717.4	78.5						11,234.4
4	30 - 14		14.0 (15.4)						2,744.9	589.4	82.3						11,362.5
5	35 - 8		8.0 (9.6)						2,212.4	1,121.8	66.4						11,173.4
6	35 - 10	35.0	10.0 (11.6)	9,019.6	8,251.1	768.5	91.5	3,334.2	2,444.7	889.5	73.3	410.5	375.9	34.6	91.6	334.0	11,405.7
7	35 - 12		12.0 (13.6)						2,616.8	717.4	78.5						11,577.8
8	35 - 14		14.0 (15.6)						2,744.9	589.4	82.3						11,705.9
9	40 - 8		8.0 (9.8)						2,212.4	1,121.8	66.4						11,408.0
10	40 - 10	40.0	10.0 (11.8)	9,019.6	8,476.6	543.5	94.0	3,334.2	2,444.7	889.5	73.3	410.5	385.0	25.5	93.8	334.0	11,640.3
11	40 - 12		12.0 (13.8)						2,616.8	717.4	78.5						11,812.4
12	40 - 14		14.0 (15.8)						2,744.9	589.4	82.3						11,940.5
13	45 - 8		8.0 (10.0)						2,212.4	1,121.8	66.4						11,562.2
14	45 - 10	45.0	10.0 (12.0)	9,019.6	8,624.5	395.1	95.6	3,334.2	2,444.7	889.5	73.3	410.5	391.3	19.2	95.3	334.0	11,794.5
15	45 - 12		12.0 (14.0)						2,616.8	717.4	78.5						11,966.6
16	45 - 14		14.0 (16.0)						2,744.9	589.4	82.3						12,094.7

Table 7-1 (2) Study on Optimum Capacity of Cauca and Palace Diversion Waterway

Case	Mark	Firm discharge (m^3/sec.)	Installed Capacity (MW)	Annual Energy Production (10^6 kWh)	Construction Cost (10^3U.S.$)	Annual Cost (10^3U.S.$)	Cost of Energy (U.S.¢/kWh)	Surplus Benefit (10^3U.S.$)	B/C
1	30 - 8	25.0	53.0	276.2	101,509	10,767	3.898	5,975	1.555
2	30 - 10			282.6	101,991	10,821	3.829	6,027	1.557
3	30 - 12			287.3	102,387	10,849	3.776	6,076	1.559
4	30 - 14			290.8	103,802	10,999	3.782	5,984	1.544
5	35 - 8	25.0	53.0	285.5	102,255	10,838	3.796	6,058	1.559
6	35 - 10			291.7	102,774	10,889	3.733	6,109	1.561
7	35 - 12			296.2	103,104	10,923	3.688	6,149	1.563
8	35 - 14			299.0	104,255	11,045	3.694	6,074	1.550
9	40 - 8	25.0	53.0	291.0	102,566	10,868	3.735	6,119	1.563
10	40 - 10			296.3	102,981	10,910	3.682	6,164	1.565
11	40 - 12			300.0	103,200	10,935	3.645	6,200	1.567
12	40 - 14			302.4	104,406	11,059	3.657	6,116	1.553
13	45 - 8	25.0	53.0	292.8	103,377	10,957	3.742	6,059	1.553
14	45 - 10			297.9	103,783	10,997	3.692	6,103	1.555
15	45 - 12			301.4	104,009	10,020	3.656	6,138	1.557
16	45 - 14			303.8	105,274	11,146	3.669	6,052	1.543

Fig. 7-2 Result of Study on Optimum Capacity of Cauca and Palace Diversion Waterway (Cost of Energy)

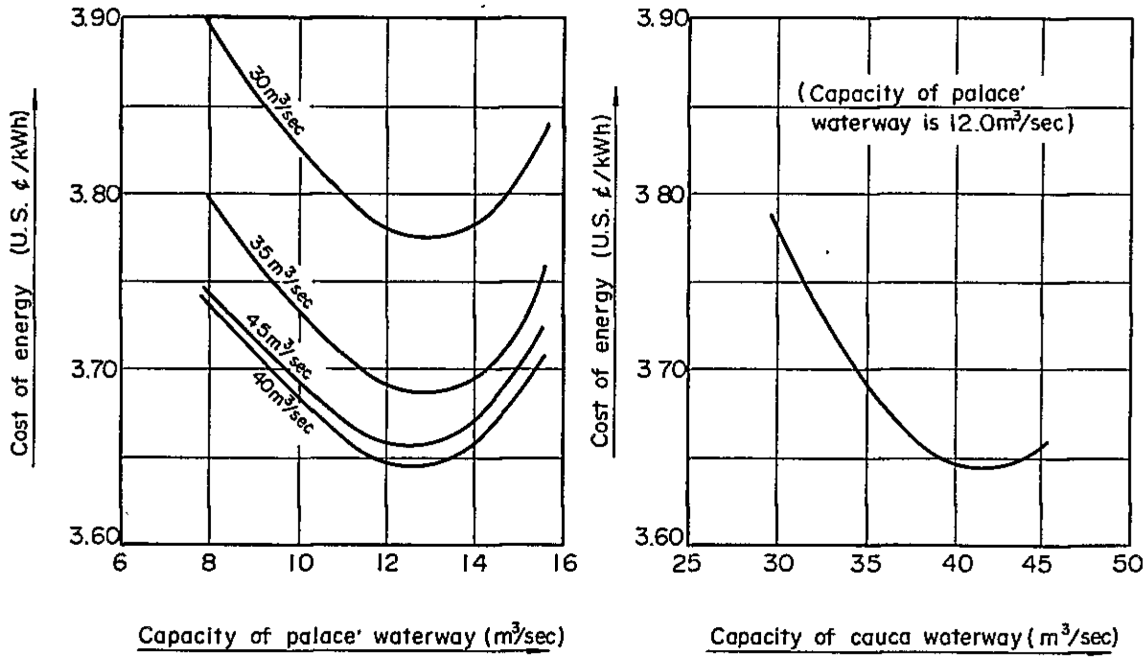
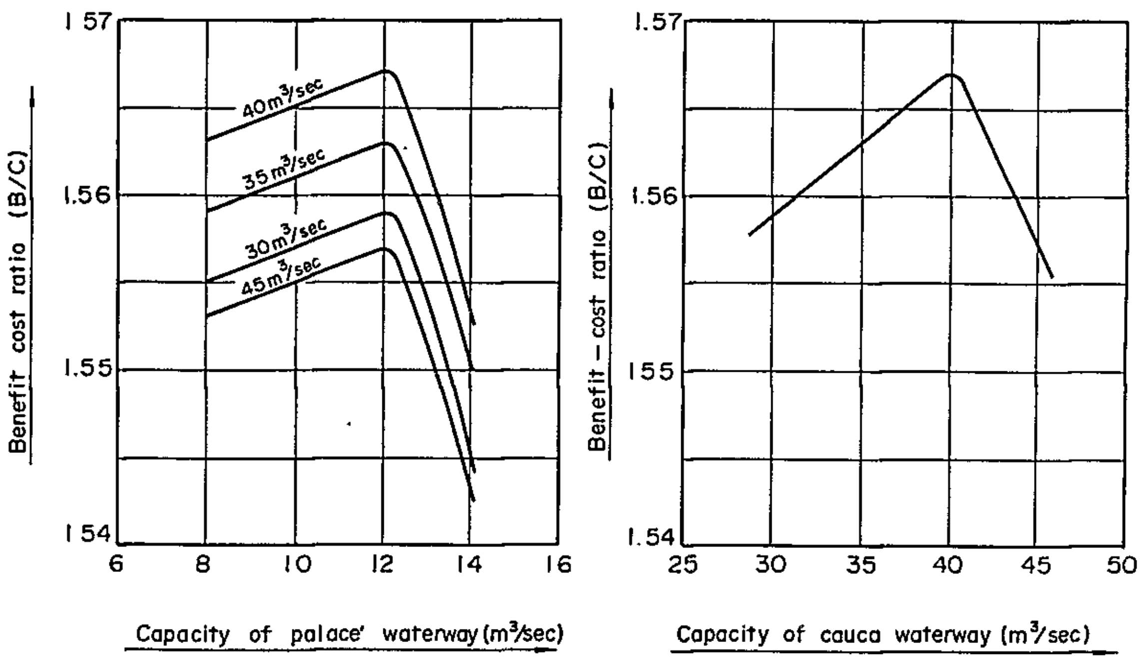


Fig. 7-3 Result of Study on Optimum Capacity of Cauca and Palace Diversion Waterway (Benefit Cost Ratio)



7.3 Study of Reservoir Scale

The run-offs of the Rio Cauca, Rio Palace and Rio Blanco are taken in the respective diversion dam sites and collected in Julumito Reservoir through diversion waterways. The seasonal and annual variations in the inflow to the reservoir are as shown by mass curves in Appendix V.

It is necessary to secure a reservoir storage capacity for effective regulation of such seasonal and annual variations for long-term stability of power station output supplementing water in dry seasons and dry years. The optimum high water surface level of the reservoir must be decided to obtain the above-mentioned storage capacity and it must be most advantage ous in the economics of power generation.

In this section, with regard to the high water surface level of Julumito Reservoir based on the above viewpoint, the cases below in the range of El. 1,710 m to 1,718 m considered to be technically feasible judging from the results of field investigations are compared and studied calculating the energy costs, benefits and annual expenses.

	High Water Surface Level (Elev. m)	Effective Storage Capacity (10^6 m^3)		
Case 1	1,712	20	30	40
Case 2	1,715	40	50	55
Case 3	1,718	40	50	60

(1) The maximum capacities of the Cauca and Palace diversion waterways are to be 40.0 m^3/sec and 12.0 m^3/sec, respectively. (Blanco Diversion Waterway 13.8 m^3/sec).

(2) The dependable discharge is to be that which can be guaranteed throughout the 15-year period operating the reservoir based on the reservoir operation rules described in 7.4.

(3) The maximum installed capacity of the power station is determined by 12-hour operation based on this dependable discharge.

(4) The construction costs of the principal structures are to be determined upon preliminary designing.

(5) Otherwise, the conditions given in 7.1.2 are to be observed.

Fig. 7-4 Result of Study on Optimum Water Surface and Effective Storage Capacity of Reservoir

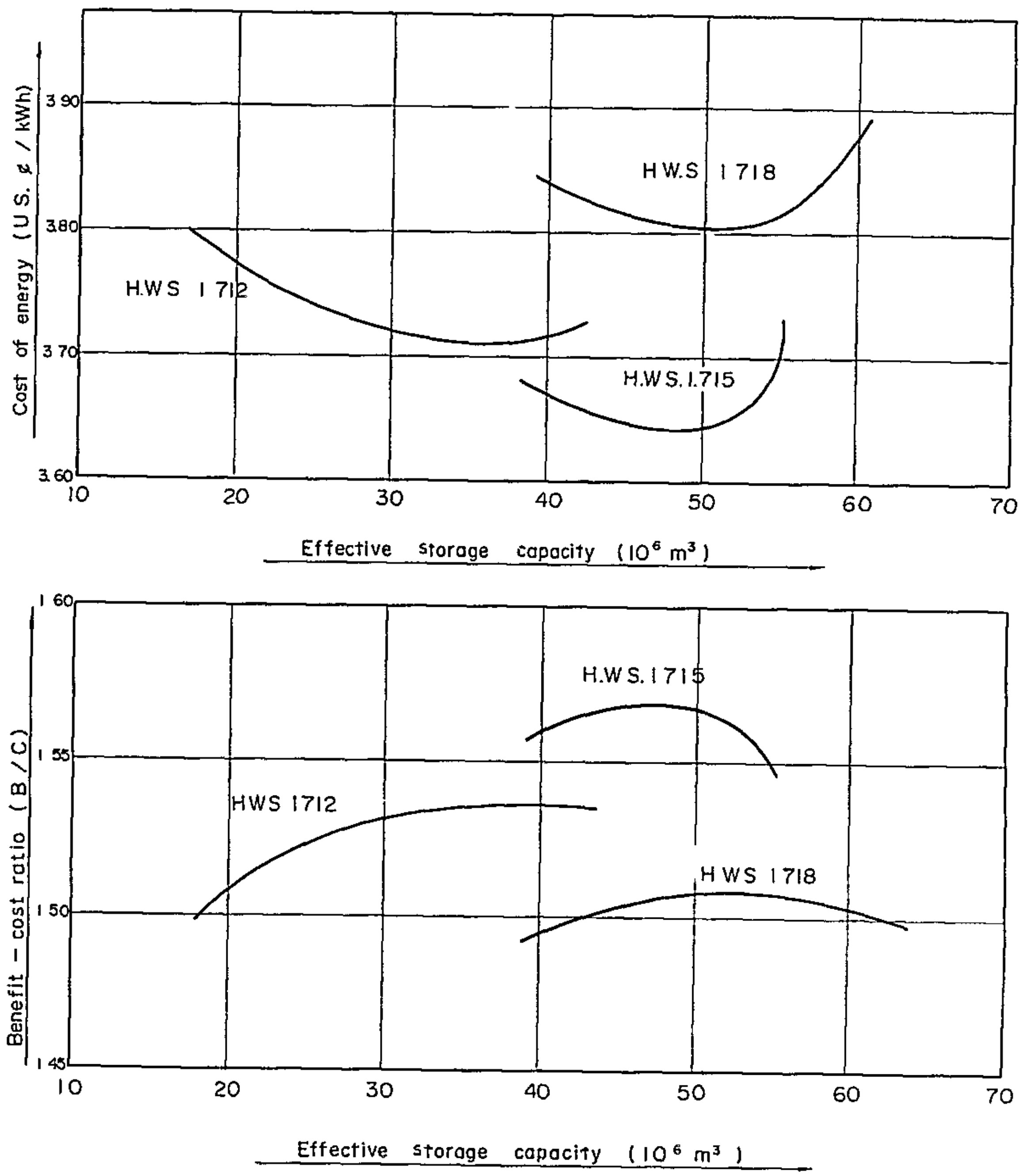


Table 7-2 Study on Optimum High Water Surface and Effective Storage Capacity

Case	H.W.S. (m)	Storage Capacity ($10^6 m^3$)		Firm Discharge (m^3/sec.)	Max. Discharge (m^3/sec.)	Effective Head (m)	Installed Capacity (MW)	Annual Energy Production (10^6 kWh)	Construction Cost (10^3U.S.$)	Annual Cost (10^3U.S.$)	Cost of Energy (U.S.¢/kWh)	B/C
		Gross	Effective									
1	1,712	47.6	20	23.1	46.2	126.3	49.2	286.4 (279.9)	99,725	10,564	3.774	1.508
2			30	23.7	47.4	125.2	50.0	290.8 (284.2)	99,817	10,572	3.720	1.531
3			40	24.4	48.8	123.5	50.8	294.5 (287.8)	100,926	10,695	3.716	1.536
4	1,715	60.8	40	24.4	48.8	127.6	52.5	303.2 (296.3)	103, 261	10,871	3.669	1.560
5			50	25.0	50.0	126.0	53.1	307.0 (300.0)	103,200	10,935	3.645	1.567
6			55	25.3	50.6	124.9	53.3	305.4 (298.4)	104,745	11,097	3.719	1.548
7	1,718	77.5	40	24.4	48.8	131.2	54.0	311.0 (303.9)	111,051	11,658	3.836	1.495
8			50	25.0	50.0	130.3	54.9	316.2 (309.0)	110,935	11,751	3.803	1.508
9			60	25.7	51.4	129.1	55.9	314.5 (307.3)	112,510	11,924	3.880	1.503

(the number in parenthesis is energy at new substation)

The results of studies according to the above are shown in Table 7-2 and Fig. 7-4. Seen from these results, it would be the most economical and optimum scale for the high water surface level of the reservoir to be at El. 1,715.0 m with effective storage capacity of the reservoir 50 x 60^6 m^3.

Therefore, the scale of the reservoir in this Project is to be high water surface level at El. 1,715.0 m, available drawdown of 15.0 m, and effective storage capacity of 50.0 x 10^6 m^3.

7.4 Operation of Reservoir

The operation rules for Julumito Reservoir are to be decided on considering the following points.

(1) Within any one year, the inflow of the high-water season is to be stored for discharge in the low-water season, it is so-called dry-season supplementation.

(2) Floods of wet years are to be stored for supplementation in dry years, operating in manner to secure dependable discharge as much as possible.

(3) Operation is to be done to minimize waste overflow of the reservoir.

(4) Operation is to be done in a manner that stable output can be secured over a long term, in addition to which energy production will be high.

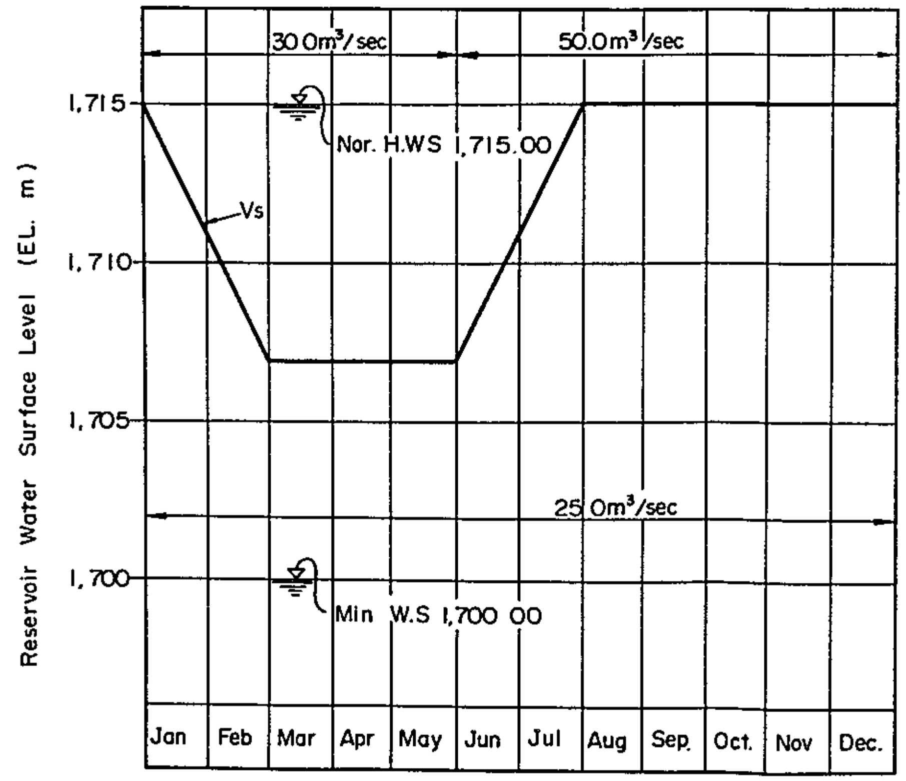
Fig. 7-5 shows the operation rules for Julumito Reservoir determined based on the above. These operation rules are provisional ones for study of the Julumito Hydro-electric Power Project, and when Julumito Power Station has been completed and actual operation is to be done more precise operation rules must be set forth.

In case of operation of Julumito Reservoir for the 15-year period from 1962 through 1976 based on the reservoir operation rules shown in Fig. 7-5, the available discharge, storage and supplementation, reservoir water level, etc. will be as indicated in Fig. 7-6.

The energy production at Julumito Power Station according to the above reservoir operation will be as shown in Table 7-5 and Fig. 7-9 .

Fig. 7-5 Operation Rule of Reservoir

Month	Vs		
	m	$10^6 m^3$	m^3/sec-day
Jan	1,710.8	31.30	362.3
Feb	1,707.0	17.38	201.2
Mar	1,707.0	17.38	201.2
Apr.	1,707.0	17.38	201.2
May	1,707.0	17.38	201.2
Jun	1,710.9	31.71	367.0
Jul	1,715.0	50.37	583.0
Aug	1,715.0	50.37	583.0
Sep.	1,715.0	50.37	583.0
Oct.	1,715.0	50.37	583.0
Nov	1,715.0	50.37	583.0
Dec	1,715.0	50.37	583.0



Symbols (Unit . m^3/sec-month)

V_{n-1}	.	Storage at the end of previous month
V_n		Storage at the end of current month
V_s	.	Standard middle limit of storage
V_{max}		Maximum storage
V_{min}	:	Minimum storage
f_n		Overflow in current month
Q_u	:	Standard upper limit of discharge for power
Q_L		Standard lower limit of discharge for power
q_n	:	Inflow in current month
Q_n		Discharge for power in current month

Constants (Unit : m^3/sec - month)

Q_u = 30.0 m^3/sec Jan to May
50.0 m^3/sec Jun to Dec

Q_L = 25.0 m^3/sec Jan to Dec

Basic Formulas

$V_{max} \geqq V_{n-1} + q_n - Q_n \longrightarrow V_n = V_{n-1} + q_n - Q_n$

$V_{max} < V_{n-1} + q_n - Q_n \longrightarrow \begin{cases} V_n = V_{n-1} + q_n - Q_n - f_n \\ f_n = V_{n-1} + q_n - Q_n - V_{max} \end{cases}$

Operation rule

1 $V_{n-1} + q_n > V_s$

(1) $Q_u \leqq V_{n-1} + q_n - V_s \longrightarrow Q_n = Q_u$
(2) $Q_u > V_{n-1} + q_n - V_s \longrightarrow Q_n = V_{n-1} + q_n - V_s$

2. $V_{n-1} + q_n < V_s$

(1) $Q_L \leqq V_{n-1} + q_n - V_{min} \longrightarrow Q_n = Q_L$
(2) $Q_L > V_{n-1} + q_n - V_{min} \longrightarrow Q_n = V_{n-1} + q_n - V_{min}$

Fig. 7-6 Reservoir Operation (1962 to 1976)