6.4 Diversion Scheme

The diversion scheme consists of the Rio Cauca Diversion System and the Rio Palacé Diversion System. The Rio Cauca Diversion System is comprised of Rio Cauca Diversion Dam and Rio Cauca Diversion Waterway (from this dam to Quebrada La Paz which will flow into Julumito Reservoir). The Rio Palacé Diversion System includes Rio Palacé Diversion Dam, Rio Palacé No. 1 Diversion Waterway (from the dam to Rio Blanco Diversion Dam), Rio Blanco Diverskon Dam and Rio Palacé No. 2 Diversion Waterway (from the dam to Quebrada Morinda which will feed Julumito Reservoir).

6.4.1 Rio Cauca Diversion System

A. Rio Cauca Diversion Dam Site

The dam planned for this site is an overflow type concrete gravity dam (height 10.0 m, crest length 69.5 m) as indicated in DWG. No. 16 and 17. This site is approximately 4 km northeast of downtown Popayan City 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 comprised of river terraces.

The left bank on passing the river terrace 60 m wide at EL. 1,770 m continues on to the hills to the east. There are isolated, slightly elevated hills between the river terraces and the hill area. The river bed has a width of approximately 40 m, and at the left bank there are narrow shoals where river deposits are seen. The right bank comprises a gentle slope of about 15^{o} which continues to the hill district to the west and at the skirt of the slope there is a narrow terrace of elevation of 1,765 m which comprises a low cliff of several meters facing the Rio Cauca.

The geology comprising the hills spread out widely to the west is a volcanic ash. The terrace deposits contain large quantities of round gravel of 5 to 20 cm with a matrix of silty sand and silty clay and are well compacted. The river deposits are comprised of round cobbles with diameters of 10 to 50 cm and sandy silt fills the interstices between the cobbles or boulders. Although any subsurface investigations have not been carried out in this site, the thickness of this river deposit is thought to be comparatively.

The scales of the dam and appurtenant atructures to be provided at this site are small, but the foundations of these structures will all be unconsolidated deposits. Of these deposits, the river deposits should be loose, but it is expected that the deep parts of terrace deposits and volcanic ash will be extremely well compacted. Nevertheless, proper foundation treatment should be carried out considering the safety of the dam, and for this purpose, adequate geological investigations such as core boring accompained with permeablity test are necessary.

B. Rio Cauca Diversion Waterway

This diversion waterway is designed as indicated in DWG. No. 20. The capacity is 40 m^3/sec and the length 2,400 m with the greater part an open channel but near the end of the route a tunnel of 220 m length is provided.

The greater part of the route is located in 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 and boulders of 10 to 30 cm diameters and are adequately 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 very similar.

The volcanic ash is adequately compacted at deeper parts but is soft while the terrace deposits are unconsolidated so that blasting is not necessary for excavating the open channel and it is estimated that construction can be amply achieved with normal earthmoving excavation equipment.

Meanwhile, since the topography of the waterway route is on the whole a gentle terrain, there should be no sliding of the original ground. Again, as extremely high slopes will not be formed by excavation. However, since the shoulders of the excavated slopes would be comprised of extremely weathered or very loose sediments, there would 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 materials eroded from the shoulders of excavated slopes from falling directly into the waterway, berms of suitable widths should be provided at the shoulders of the channel.

The volcanic ash and the terrace deposits are judged to have adequate bearing power as foundations for the open channel except weathered 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. Also, leakage due to defective concrete placement can cause sliding of the foundation of waterways depending on topographic and geologic conditions.

The tunnel would be driven through volcanic ash. Therefore, supporting would be required for the considerable length and the entire length would naturally be lined with concrete. In construction of the tunnel, ample coverage of earth would be required and open excavation would be increased at portals due to the gentle topographic feature. Therefore, the length and the height of the excavated slopes will not be small. Consideration must be given to slope stabilization of this wide excavated surface. As for leakage from the waterway tunnel, the same phenomena as at the open channels can occur, and as repairs are far more difficult than with open channels, even greater precautions must be taken in design and construction.

6.4.2 Rio Palacé Diversion Plan

A. Rio Palacé Diversion Dam

The dam planned for this site is a small overflow type concrete gravity dam (height 7.7 m, crest length 22.0 m) as indicated in DWG. No. 18.

At both banks of this site there are low river terraces continuing from flood plains. The width of the terrace at the left bank is approximately 40 m but is narrow at the right bank. The Rio Palacé of a width of approximately 10 m flows close to the right bank. On the left bank terraces continue to be uniform slope of approximately 15^{0} while at the right bank a slope of approximately 30^{0} comes down close to the river bed. The geology comprising the site has slightly coarse grained volcanic ash as its basement, and from the river bed to the left bank there are distributions of river terrace deposits, while at the river channel and the flood plains there are river deposits. The thickness of these river and terrace deposits together is estimated to be 10 m or more. As the deeper parts of the river and terrace deposits and volcanic ash are comparatively well compacted, a foundation good enough for this low dam should be obtainable if the loose parts near the ground surface are excavated and removed. However, since these sediments are unconsolidated, considerations must be taken for supplementing bearing capacity of the foundation, preventing leakage through the foundation and preventing scouring of the ground downstream of the dam.

The investigation methods described in 6.4.1.A, "Rio Cauca Diversion Dam" should be followed in this site.

B. Rio Palacé No. 1 Diversion Waterway

This diversion waterway is an open channel (capacity 12 m^3/sec) as indicated in DWG. No. 21 which connects Rio Palacé Diversion Dam and Rio Blanco Diversion Dam. The length is 4,000 m and the route is planned along the 1,800 m contour line as shown in DWG. No. 2.

The waterway cuts across numerous small ravines along its route, but all of these ravines are crossed close to their most upstream parts and the valley are shallow. The mountainside slopes along the route are gentle so that there will be no great increase in excavation quantities and the conditions are favorable in regard to stabilities of excavated slopes.

The geology of the waterway route is comprised of volcanic ash. The cross section of the channel is smaller than that of the Rio Cauca Diversion Waterway but the considerations to be given in regard to design and construction may be thought to be nearly similar.

C. Rio Blanco Diversion Dam

The dam planned is a small overflow type concrete gravity dam (height 5.5 m, crest length 21 m) as indicated in DWG. No. 19.

The site is located immediately downstream from the junction of the Rio Blanco and one of its tributaries, Quebrada Clarete, and there are very flat and wide river terraces spread out on both banks. The surfaces of these river terraces are 4 to 5 m higher than the present river bed. These deposits of the terraces and the river bed are mainly weathered andesite boulder of 10 cm to 70 cm diameters and the deeper parts of the deposits appear to be adequately compacted.

Although this is a low dam, since the foundation is unconsolidated deposited material, the same considerations in investigations, design and construction are necessary as for the Rio Cauca and Rio Palacé diversion dams in regard to bearing capacity and permeability of the foundation and scouring downstream of the dam.

D. Rio Palacé No. 2 Diversion Waterway

This diversion waterway is designed as indicated in DWG. No. 21 and is mostly an open channel (capacity 13.8 m^3/sec) but there are three tunnels along the way. The diversion waterway, as indicated in DWG. No. 3, connects Rio Blanco Diversion Dam with the outlet at Quebrada Morinda and the length is 3,660 m. The lengths of the three tunnels are, in the order from the one closest to the intake, 310 m, 280 m and 180 m, the total of which is 770 m.

Besides tributaries of the Rio Blanco, the waterway crosses numerous valleys such as Quebrada Chamiral and Quebrada Pambazo Bajo.

As the geology of the route is comprised of a soft, unconsolidated volcanic ash, there is a necessity for concrete lining to be provided after excavation of the tunnel. The characteristics of the geology of the route are the same as for the beforementioned Rio Palacé No. 1 Diversion Waterway and similar considerations of geology and engineering must be given.

6.5 Construction Materials

6.5.1 Impervious Material

Volcanic ash and weathered residual soil of andesite are considered as impervious materials available at this site.

Volcanic ash is widely distributed at the project area in a thickness of 20 to 40 m and is easily borrowed. In the Report of Preliminary Studies[1], there were two areas selected as possible berrow areas at the left bank side of the Julumito dam site, and it was recommended for investigations to be made of the area. Based on the recommendation, 8 test pits and 2 core borings were made for these areas by CEDELCA (cf. DWG. No. 3). Soil tests have been performed on samples collected from these points (see Appendix IV-5). As a result of soil tests and field investigations, there is no superiority in particular of one over the other between Borrow Area No. 1 and Borrow Area No. 2. If forced to find a difference, Borrow Area No. 2 may be said to be superior considered from the standpoints of the abundance of material existing (requirement: 120,000 m^3) and hauling distance.

The tests were carried out at the Department of Civil Engineering of Universidad del Cauca at Popayan City at the request of CEDELCA, of which the triaxial compression test was performed at Universidad del Mexico. The comprehensive results of the tests are compiled in Appendix IV-5-5. As for the peculiar weathering action on this volcanic ash, a report has been produced by INGEOMINAS from a pedological standpoint (A-2-(e) of 6.3.2).

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 inspection of the gradation analysis curve (Appendix IV-5-3).

As for the permeability test by Universidad del Cauca, it was only limited to occasion of consolidation test. Considered from the result of this test this volcanic ash is assumed to have 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 core material. 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. Further, in order to clarify the relations between weathering stages and suitability as core material of this volcanic ash, it is desirable for test pits of depths of 10 m or more to be excavated and various tests performed on samples collected every 1 m deep. Regarding the results of field permeability tests performed utilizing test pits and core borings at the borrow areas, they are as indicated in Appendix IV-2. The coefficients of permeability in these tests also indicated orders of 10^{-4} 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 fine grain added to which is a cohesive soil of high natural water-content.

(2)-1 According to gradation analysis, the percentage passing the No. 200 sieve (0.074 mm) is indeed close to 90% and according to the Unified Soil Classification System*[1] the soil corresponds roughly to MH and ML. Also according to soil consistency test the liquid limit (LL) is high at around 60% and the natural water content is in many cases close to this value. Based on these test results, the plastic limits (PL) of the majority of the samples are high and the plasticity index (PI=LL-PL) is 10% which is abnormally low (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 values smaller than the plastic limit in the information furnished. In conducting tests, gradation analyses should be made on sizes passing the No. 200 sieve for use as references in consistency tests.

(2)-2 In regard to tests for strengths of soil, unconfined compression test, triaxial compression test and direct shear test have been conducted and the reulsts are compiled in Appendix IV-5-2. According to these results, the angle of internal friction (ϕ) indicates an abnormally high value (35° to 55°) and cohesion (C) a low value. The axial pressures ($\overline{\sigma}_1$) 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 No. 200 sieve, silt is predominant in the sizes under No. 200 sieve and the volcanic ash thus indicates inactivity as a cohesive soil. Accordingly it is desirable to examine the activities of clay, one of which is shown by the following formula:

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

(2)-3 Judged by the condition of construction in volcanic ash at the regulating pond of Florida II Power Station Project in the suburbs of Popayan City, it is not thought trafficability is of a degree that construction with normal earthmoving equipment is impossible. Regarding this point, it is desirable for confirmations to be made by CBR test and cone penetration test on undisturbed samples of the volcanic ash and samples compacted at water contents 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 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 degree of saturation (Sr) is 75% or more, when compacting this the S_r reaches 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, times 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 that at the same water content, the result 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 same drying conditions as actual dam construction. 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 degree of saturation, "S_r", to be carried out rather than on compaction rate (percent compaction) based on dry density.

6.5.2 Rock Material and Filter Material

A. Rock Material

Andesite lava is conceivable to be used as rock material.

The results of Core Boring DH-5 drilled at the quarry site selected in the Preliminary Study[1] are given in Appensix IV-2 . Briefly described, the volcanic ash changes to andesite lava at a depth of 39 m, but fresh rock is at 51 m and below. It is economically impracticable to excavate and remove the volcanic ash and extremely weathered andesite. The geologic condition is the same at the alternative quarry site on the right bank of the Rio Sate.

In the present investigation the upstream and downstream areas of the Rio Sate were again reconnoitered for selection of a quarry site, but as described in detail under "Geology of Reservoir Area" (6.3.1), weathering has penetrated deeply and there are no places suited for a quarry. As for the hill area comprising the boundary between the Rio Sate and the Rio Cauca, as stated in "Geology of Diversion Waterway" (6.3.3), the volcanic ash is thick, so that this area is unsuitable as a quarry site. Where the volcanic ash has been removed by erosion is limited to the proposed powerhouse site and its immediate vicinity (see 6.3.4). The ridge proposed for the penstock route is a favorable site for a quarry, but since there are no alternative sites for the powerhouse, this ridge cannot be used as a quarry. Therefore, besides selecting the ridge on the opposite bank of the powerhouse for the quarry site as indicated in Apx. I. Fig. I-9, it was decided to apply excavation muck from the penstock as part of the rock material.

This quarry site is comprised of thick andesite lava and is divided into four flames are suitable for rock material. The microscopic description of this rock is as given in Appendis IV-3 Also, laboratory tests of rock were carried out with cores from Bore Holes DH-1, DH-2, DH-3, DH-5 and DH-204 as samples. The test items were specific gravity, absorption, ultrasonic velocity (P- and S-waves) and compressive strength. The results are as shown in Appendix IV-6-2.

According to the physical tests, this andesite lava is considered to possess adequate quality as rock material.

Although, the rock of the site selected is good with respect to quality, the fact that the terrain is extremely rugged added to which the necessity for a transport road with a drop in elevation of approximately 100 m to the skirt of the quarry comprises a draw-back. However, it will not be possible to find a location with better conditions than this site in the project area.

In order to confirm the degree of weathering of the andesite, X-ray diffraction was performed on cores from Bore Holes 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. The results of the X-ray diffraction indicate that the interior of this andesitic lava is on the whole fresh.

B. Filter Material

The requirements for this material are 80,000 m^3.

There are no large quantities of river deposits with suitable grain sizes in the vicinity of the dam site. However, it is thought possible to artificially manufacture filter material from the same quarry as for the rock material. The outlook is that material satisfying the gradation requirements for filter can be obtained by selecting excavated faces at the rock quarry as the densities and feature of cracks differ at the various flows and parts of the andesite lava.

6.5.3 Concrete Aggregate

Approximately 80,000 m^3 of concrete aggregates are necessary for the Julumito Project.

The plan is for aggregates to be secured through artificial manufacture from materials of the quarry on the opposite bank of the powerhouse site and of excavated muck from the penstock.

According to the results of rock tests of the andesite lava at this site, the specific gravity is 2.35, the absorption 3 to 5% and compressive strength 500 to 700 kg/cm^2, and it is thought that with careful selection of the working face, coarse aggregate can be artificially manufactured from this andesite. As for fine aggregate, a quantity approximately half that of coarse aggregate is required and it is thought this too can be manufactured from the andesite.

Meanwhile, it is planned also to supplementarily use river deposits of the Rio Timbio and the Rio Ondo from which aggregates are presently being taken for the Florida II Power Station Project. In carrying out definite study, it will be necessary to have a quantitative grasp of these deposits.

References

1/ Overseas Technical Cooperation Agency, Government of Japan, (1970), Report of preliminary studies on Julumito hydro-electric Project. – prepared for Centrales Electricas Del Cauca S.A., Republic of Clombia.

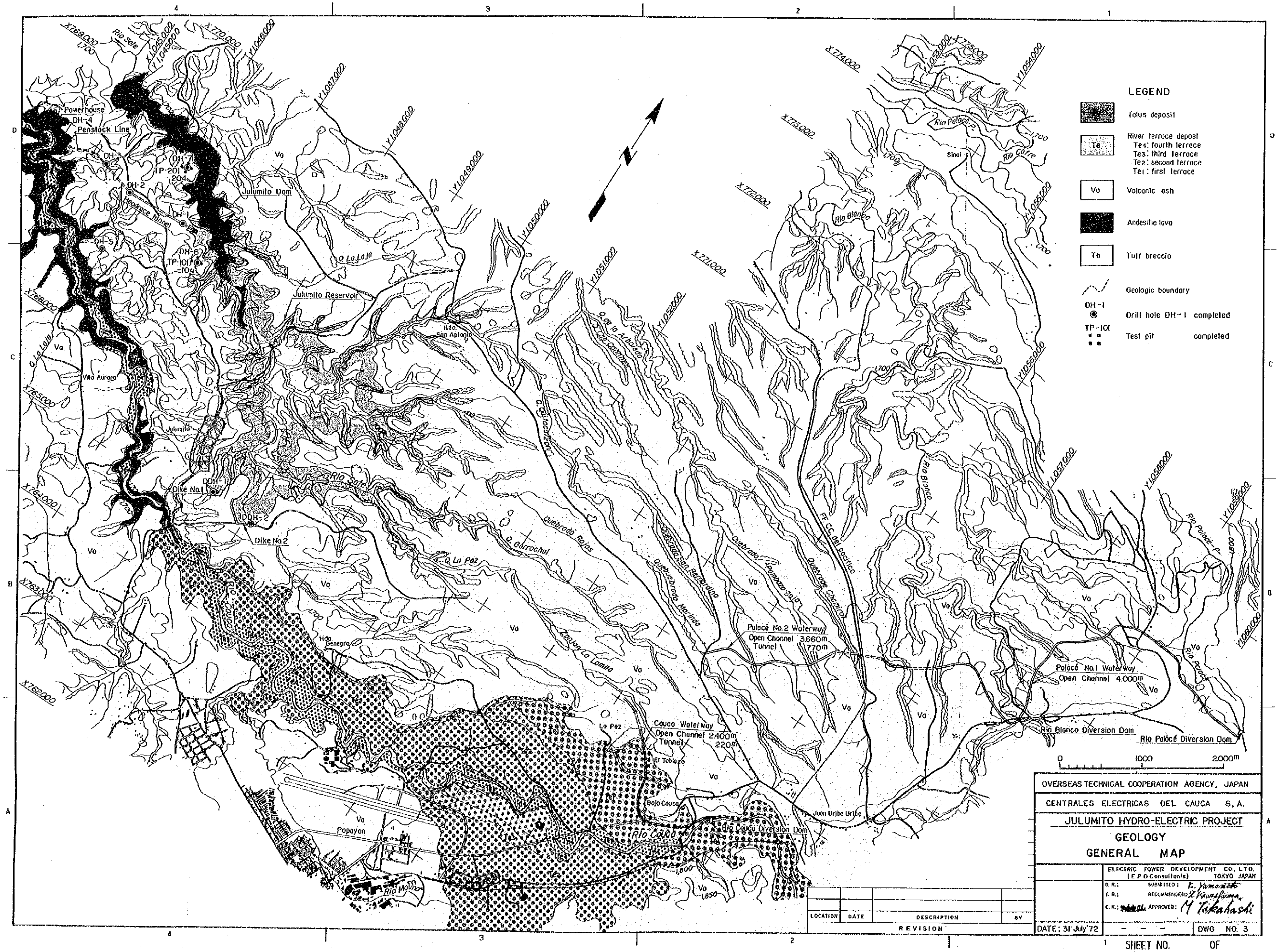
2/ Servicio Geologico Nacional, Bogota, Colombia (1962), Mapa Geologico de Colombia, scale of 1:1,500,000.

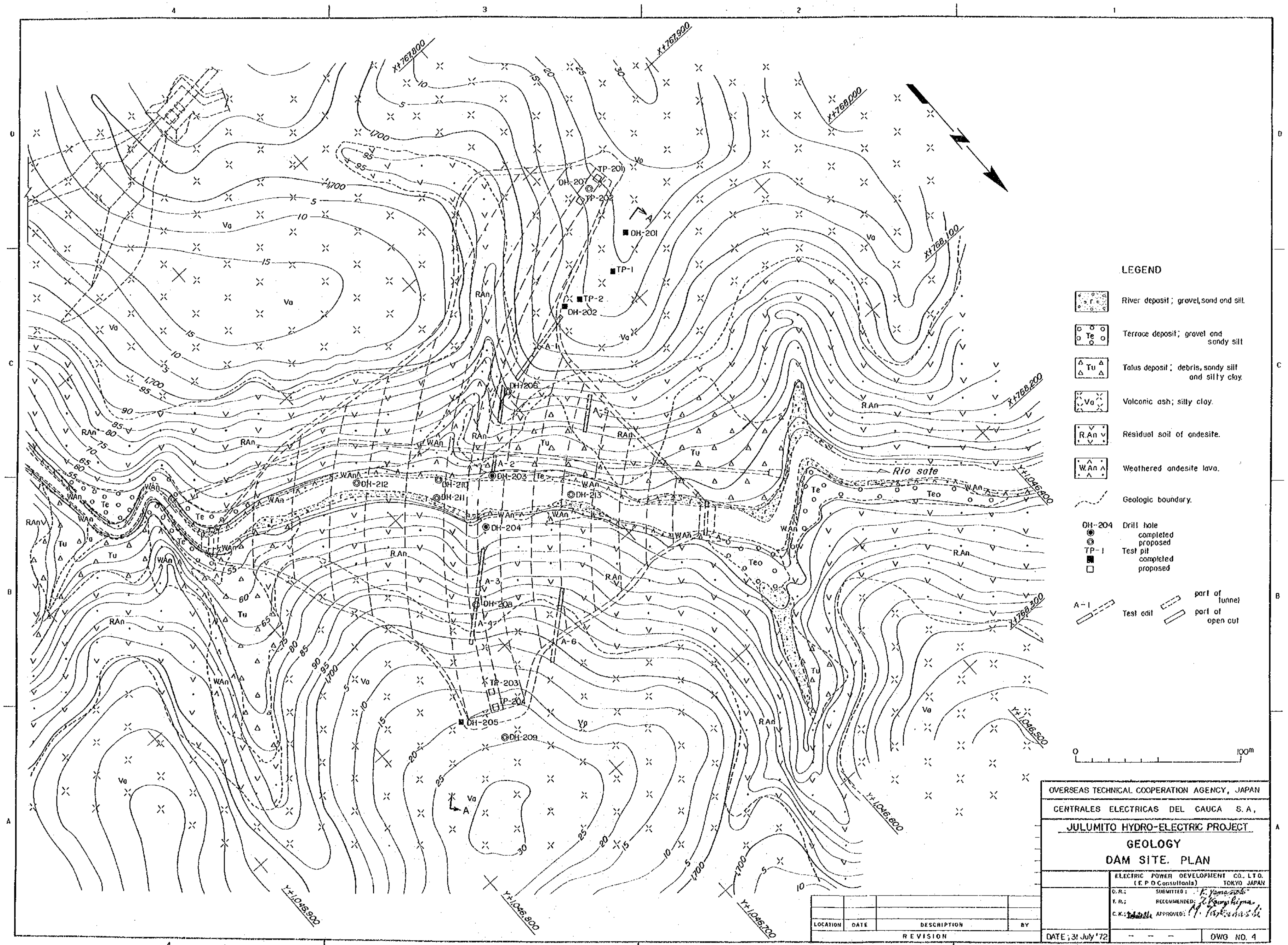
3/ Benjamin Alvarado B. and Enrique HUBACH (1932), Estudios Geologicos Alnededon de Popayan; Ministerio de Minas y Petroleos – Servicio Nal.

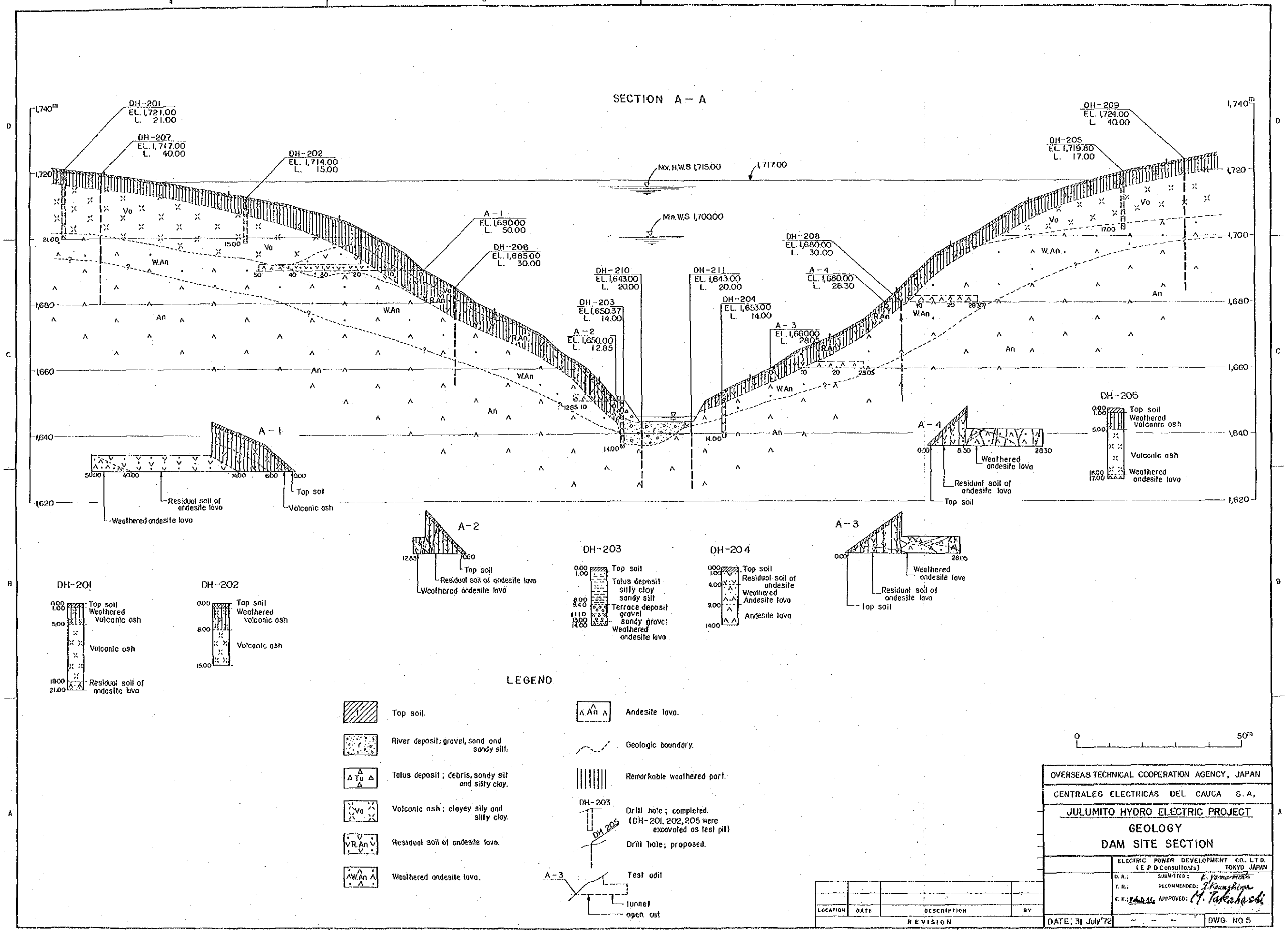
4/ E. Hubach and B. Alvarado (1932), Estudio Geologico en la Ruta Popayan – Bogota; Depto. de Minas y Petróleos, Ministerio de Indastrias.

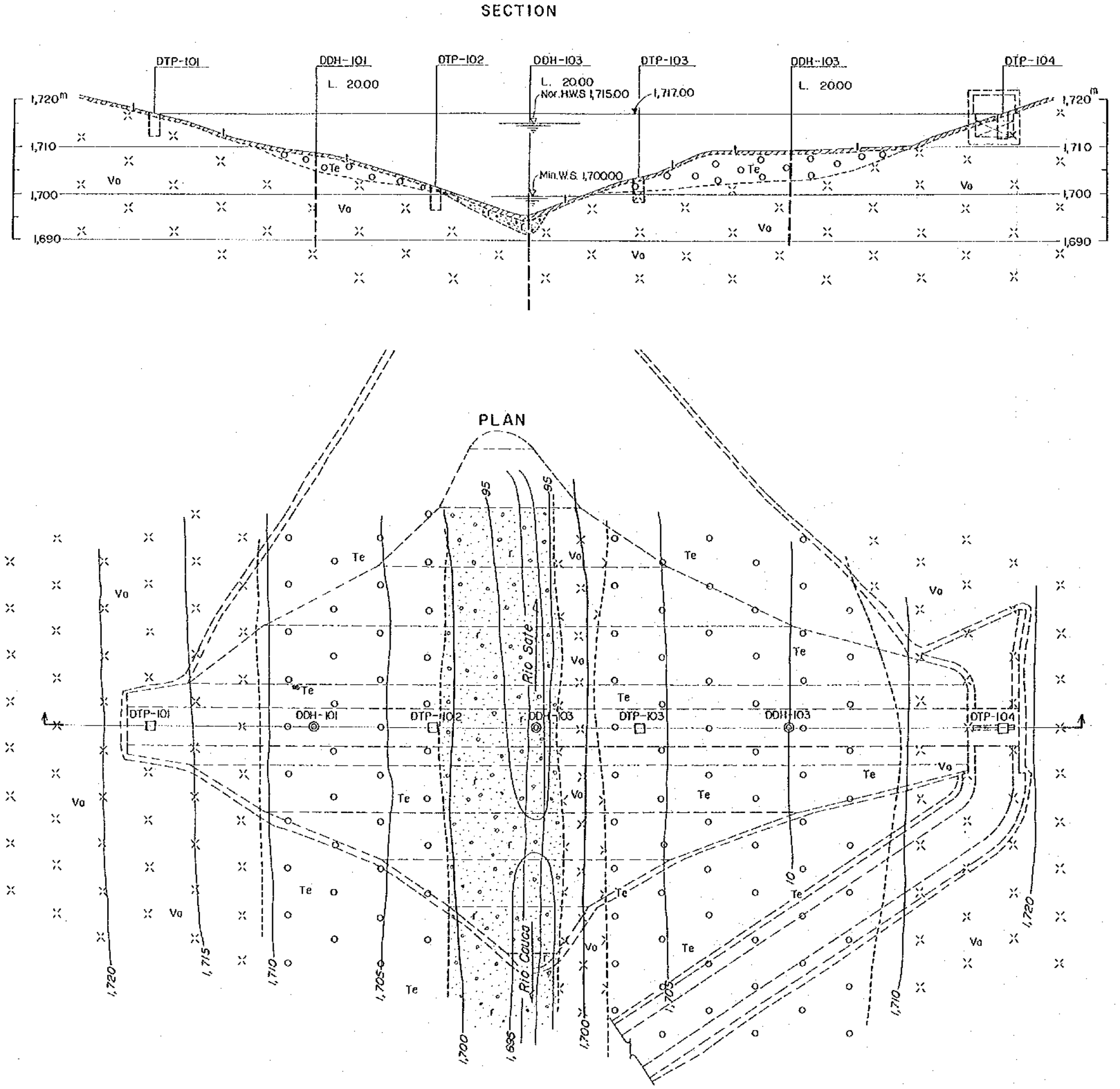
5/ Humberto Rosas Carcia (1971), Observacions Sobre Suetos Bauxiticos, como Contribución al Estudio de Materiales para la Presa de Tierra del Proyecto Hidro-electrico "JULUMITO"; Instituto Nacional de Investigacion Geologico Mineras (INGEOMINAS), Ministerio de Minas y Petroleos.

6/ Instituto Geofisico de los Andes Colombianos (1957), El Mapa Sismico y Tectonico de Colombia; Sene "A" Sismoglogia (Segunda Epoca) Publicacion No. ...


LEGEND
Talus deposit
Te
River terrace deposit
Te4: fourth terrace
Te3: third terrace
Te2: second terrace
Te1: first terrace
Va
Volcanic ash
Andesitic lava
Tb
Tuff breccia
Geologic boundary
DH-1
Drill hole DH-1 completed
TP-101
Test pit completed
Powerhouse
DH-4
Penstock Line
DH-3
DH-2
DH-1
DH-5
DH-6
TP-101~104
TP-201~204
Julumito Dam
Julumito Reservoir
Q. La Laja
Villa Aurora
Julumito
Dike No.1
DDH-1
DDH-2
Dike No.2
Rio Sate
Quebrada Rojas
Q. Garrachal
Q. La Paz
Zanjon La Lomita
Hda San Antonio
Hda Genegro
Rio Cauca
Popayan
Rio Molino
La Paz
El Tablazo
Bajo Cauca
Rio Cauca Diversion Dam
Cauca Waterway
Open Channel 2,400m
Tunnel 220m
Palacé No.2 Waterway
Open Channel 3,660m
Tunnel 770m
Palacé No.1 Waterway
Open Channel 4,000m
Rio Blanco Diversion Dam
Rio Palacé Diversion Dam
Juan Uribe Urice
Rio Blanco
Rio Palacé
Rio Cofre
Sinai
Quebrada Chamizal
Quebrada San Bernardino
Quebrada Marinda
Q. de la Arboleda
0
1000
2,000m
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
GEOLOGY
GENERAL MAP
ELECTRIC POWER DEVELOPMENT CO., LTD.
(E P D Consultants) TOKYO JAPAN
SUBMITTED: E. Yamamoto
RECOMMENDED: Z. Kawashima
APPROVED: M. Takahashi
LOCATION DATE DESCRIPTION BY
REVISION
DATE: 31 July '72
DWG NO. 3
SHEET NO. OF


LEGEND
River deposit; gravel, sand and silt.
Terrace deposit; gravel and sandy 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 completed proposed
TP-1 Test pit completed proposed
A-1 Test adit
part of tunnel
part of open cut
Rio sate
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
GEOLOGY
DAM SITE. PLAN
ELECTRIC POWER DEVELOPMENT CO., LTD. (E.P.D.Consultants) TOKYO JAPAN
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE ; 31 July '72
DWG NO. 4
SHEET NO.
OF


SECTION A – A
DH-201 EL. 1,721.00 L. 21.00
DH-207 EL. 1,717.00 L. 40.00
DH-202 EL. 1,714.00 L. 15.00
Nor. H.W.S 1,715.00
1,717.00
Min. W.S 1,700.00
A-1 EL. 1,690.00 L. 50.00
DH-206 EL. 1,685.00 L. 30.00
DH-210 EL. 1,643.00 L. 20.00
DH-211 EL. 1,643.00 L. 20.00
DH-203 EL. 1,650.37 L. 14.00
A-2 EL. 1,650.00 L. 12.85
DH-204 EL. 1,653.00 L. 14.00
A-3 EL. 1,660.00 L. 28.05
A-4 EL. 1,680.00 L. 28.30
DH-208 EL. 1,680.00 L. 30.00
DH-205 EL. 1,719.80 L. 17.00
DH-209 EL. 1,724.00 L. 40.00
1,740m
1,720
1,700
1,680
1,660
1,640
1,620
Va
W.An
An
R.An
A-1
Top soil
Volcanic ash
Residual soil of andesite lava
Weathered andesite lava
A-2
A-3
A-4
DH-201
DH-202
DH-203
DH-204
DH-205
Top soil
Weathered volcanic ash
Volcanic ash
Residual soil of andesite lava
Talus deposit silty clay sandy silt
Terrace deposit gravel sandy gravel
Weathered andesite lava
Andesite lava
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; completed. (DH-201, 202, 205 were excavated as test pit)
Drill hole; proposed.
Test adit
tunnel
open cut
0
50m
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO ELECTRIC PROJECT
GEOLOGY
DAM SITE SECTION
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE; 31 July '72
DWG NO 5
SHEET NO.
OF



LEGEND

- Top soil
- River deposit; sandy silt and silty clay.
- Terrace deposit; sand, silty sand and gravel.
- Volcanic ash; clayey silt.
- Geologic boundary.
- DDH-101 Drill hole. proposed
- DTP-101 Test pit. proposed

0 — 50m

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S. A.

JULUMITO HYDRO-ELECTRIC PROJECT

GEOLOGY

DIKE NO.1 SITE

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

D. R.: SUBMITTED: K. Yamamoto

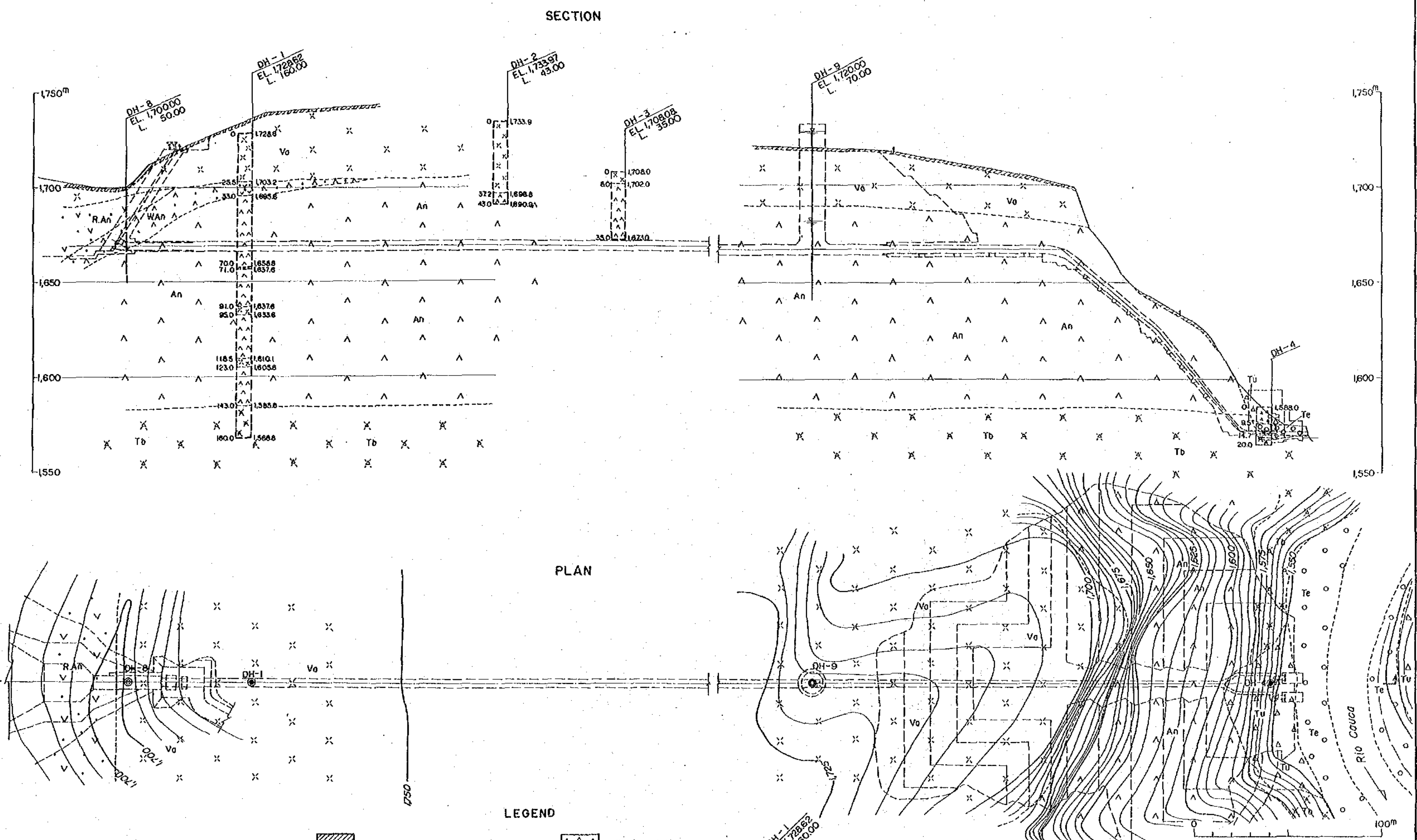
T. R.: RECOMMENDED: S. Kawashima

C. K.: APPROVED: M. Takahashi

LOCATION	DATE	DESCRIPTION	BY
		REVISION	

DATE; 31 July '72 — — — DWG NO. 6

SHEET NO. OF



LEGEND

T	Top soil; silty clay humus.	W.An	Weathered andesite lava
Tu	Talus deposit; debris and sandy silt.	An	Andesite lava
Te	Terrace deposit; gravel and silt.	Tb	Tuff breccia
Va	Volcanic ash; silty clay.		Geologic boundary
R.An	Residual soil of andesite lava.	DH-1	Drill hole completed / proposed

DH-1 EL. 1,728.62 L. 160.00 — Drill hole, completed
DH-8 — Drill hole, proposed

LOCATION	DATE	DESCRIPTION	BY
REVISION			

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S. A.

JULUMITO HYDRO-ELECTRIC PROJECT

GEOLOGY

WATERWAY AND POWERHOUSE SITE

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

D. R.: SUBMITTED: K. Yamamoto
T. R.: RECOMMENDED: T. Kawashima
C. K.: APPROVED: M. Takahashi

DATE; 31 July '72 — — — DWG NO. 7

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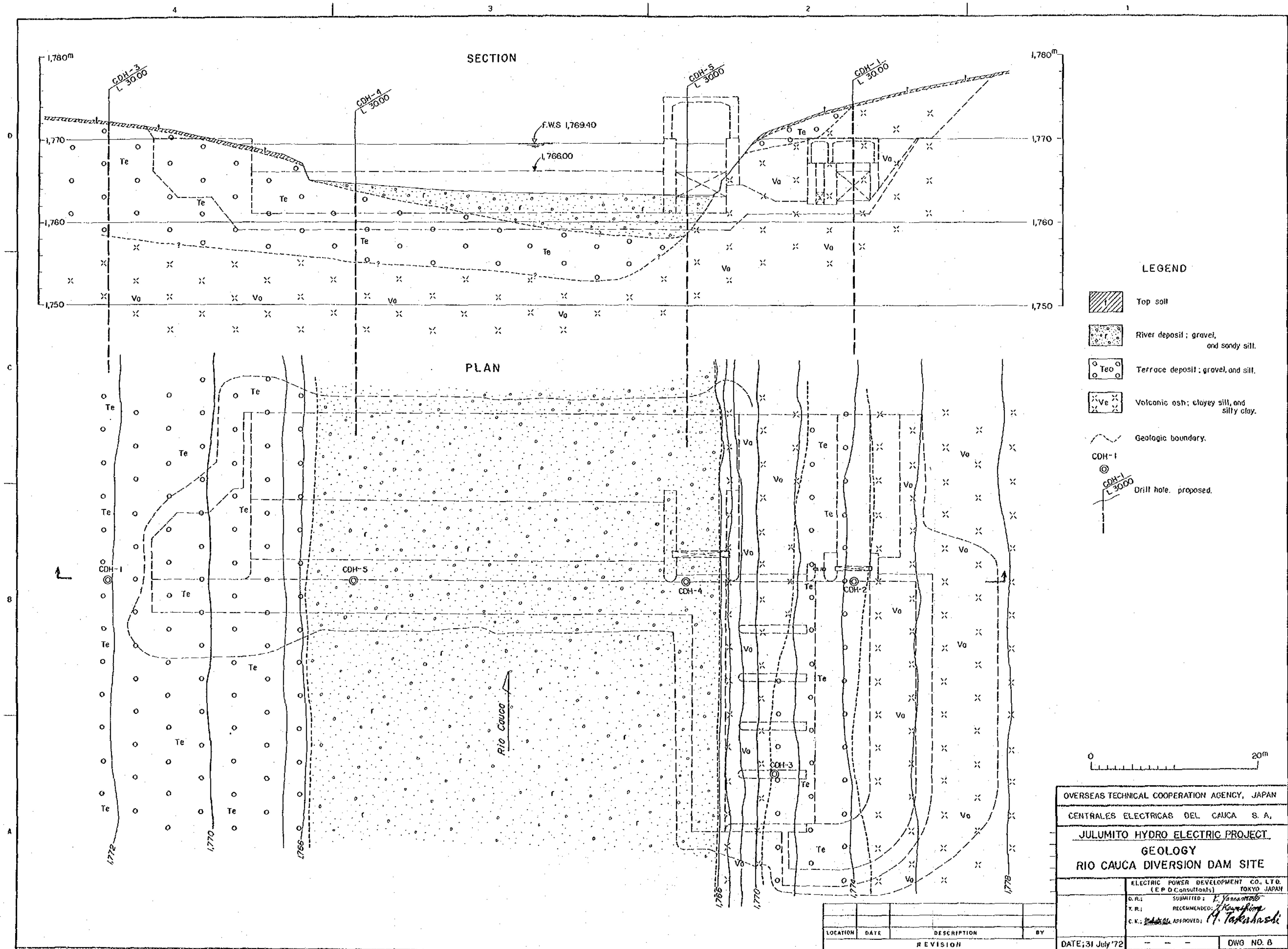
SECTION
PLAN
F.W.S 1,769.40
1,766.00
1,780m
1,770
1,760
1,750
CDH-1
CDH-2
CDH-3
CDH-4
CDH-5
L 30.00
Rio Cauca
LEGEND
Top soil
River deposit ; gravel, and sandy silt.
Terrace deposit ; gravel, and silt.
Volcanic ash ; clayey silt, and silty clay.
Geologic boundary.
Drill hole. proposed.
0
20m
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S. A.
JULUMITO HYDRO ELECTRIC PROJECT
GEOLOGY
RIO CAUCA DIVERSION DAM SITE
ELECTRIC POWER DEVELOPMENT CO., LTD.
(E P D Consultants) TOKYO JAPAN
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE; 31 July '72
DWG NO. 8

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CHAPTER 7

ENERGY PRODUCTION

CONTENTS

FIGURE LIST

TABLE LIST

CHAPTER 7 ENERGY PRODUCTION

7.1 Study of Diversion Waterway Capacity

The basic development pattern of the Julumito Hydroelectric Power Project, as stated previously, is as follows. Namely, run-offs of the mainstream Rio Cauca and its tributaries, the Rio Palacé and Rio Blanco, are diverted to Julumito Reservoir to be provided on the Rio Sate, which is also a tributary, by diversion waterways (called Cauca Diversion Waterway and Palacé Diversion Waterway). The water conducted to the reservoir is to be regulated in the most effective manner utilizing the reservoir storage capacity after which it is to be conducted to Julumito Power Station via a headrace tunnel for power generation.

Therefore, the diversion quantities from the Rio Cauca and Rio Palace will have a great influence on the development scale and economy of the Project.

In this paragraph, a study of the optimum diversion quantities (capacities of diversion waterways) from the abovemention two rivers is made by the method described below.

In other words, the intake quantities from the diversion dam sites on the Rio Cauca, Rio Palace and Rio Blanco are taken to be the followings:

Maximum intake quantity at Rio Cauca Diversion Dam site
(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 quantity at Rio Palace Diversion Dam site
(capacity of Palace No. 1 Diversion Waterway): 8.0 m^3/sec, 10.0 m^3/sec, 12.0 m^3/sec, 14.0 m^3/sec

Maximum intake quantity at Rio Blanco Diversion Dam site
(capacity of Palace No. 2 Diversion Waterway): 1.4 m^3/sec, 1.6 m^3/sec, 1.8 m^3/sec, 2.0 m^3/sec

Based on combinations of the above diversion capacities, the energy costs and the ratios between benefits and annual costs are studied for the various cases and the most economical diversion capacities are determined.

The calculations are made based on the following conditions;

(1) The high water surface level of Julumito Reservoir is to be at 1,715.00 m and the effective storage capacity is to be 50 x 10^6 m^3.

(2) The reservoir is to be operated in accordance with the operation rules described in 7.3, and the discharge of powerhouses and annual energy productions are to be calculated for the 10-year period from 1962 through 1971.

(3) The installed capacity is to be decided by determining the dependable discharge based on the abovementioned reservoir operation with plant factor of 50% and normal intake water level.

(4) The benefit is to be calculated with the annual costs per kW and kWh of the alternative thermal power station described in 10.3.2 as criteria (see 10.3.2).

(5) The annual costs are calculated multiplying the construction cost by the annual cost ratio. The construction costs of major structures are to be obtained through preliminary designing.

The results of comparison studies of the intake quantities at the various diversion dam sites carried out in accordance with the above are indicated in Table 7-1, Fig. 7-1 and Fig. 7-2. As is clear from the table and figures, the optimum an intake quantity is 40.0 m^3/sec at Rio Cauca Diversion Dam and 12.0 m^3/sec at Rio Palacé Diversion Dam.

Consequently, the capacities of the two diversion waterways would be as follows:

Cauca Diversion Waterway		40.0 m^3/sec
Palace Diversion Waterway	No. 1	12.0 m^3/sec
	No. 2	*13.8 m^3/sec

* The optimum diversion quantity from Rio Blanco Dam studied in a similar manner resulted in 1.8 m^3/sec.

7.2 Study of Reservoir Scale

The run-offs of the Rio Cauca, the Rio Palace and the Rio Blanco are to be diverted at their respective diversion dams and conducted in Julumito Reservoir through diversion waterways. The seasonal fluctuation and yearly fluctuation in the reservoir inflow are as indicated on the mass curve of Fig. 7-3. This mass curve was prepared by compiling by month the daily quantity of water possible to divert at each diversion site based on the 10-year run-off data from 1962 through 1971 described in Chapter 5, "HYDROLOGY".

According to this mass curve, inflow is generally small in the first half of a year with a trend of slightly larger inflow in the last half. In effect, the average annual inflow for the 10-year period is 951 x 10^6 m^3 (30.2 m^3/sec) of which 448 x 10^6 m^3 (28.7 m^3/sec)

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

(Unit: m^3/sec-day)

Case	Mark	Capacity of Waterway		Rio Cauca				Rio Palacé				Rio Blanco				Rio Sate	Reservoir Inflow
		Cauca (m^3/sec)	Palacé (m^3/s)	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	30.0	8.0 (9.4)	8,424.3	7,533.4	890.9	89.4	3,133.0	2,044.6	1,088.4	65.3	385.0	341.8	43.2	88.8	305.8	10,225.6
2	30-10		10.0 (11.4)						2,230.0	903.0	71.2						10,411.0
3	30-12		12.0 (13.4)						2,389.2	743.8	76.3						10,570.2
4	30-14		14.0 (15.4)						2,516.1	616.9	80.3						10,697.1
5	35- 8	35.0	8.0 (9.6)	8,424.3	7,797.0	627.3	92.6	3,133.0	2,044.6	1,088.4	65.3	385.0	354.7	30.3	92.1	305.8	10,502.1
6	35-10		10.0 (11.6)						2,230.0	903.0	71.2						10,687.5
7	35-12		12.0 (13.6)						2,389.2	743.8	76.3						10,846.7
8	35-14		14.0 (15.6)						2,516.1	616.9	80.3						10,973.6
9	40- 8	40.0	8.0 (9.8)	8,424.3	7,965.8	458.5	94.6	3,133.0	2,044.6	1,088.4	65.3	385.0	362.5	22.5	94.2	305.8	10,678.7
10	40-10		10.0 (11.8)						2,230.0	903.0	71.2						10,864.1
11	40-12		12.0 (13.8)						2,389.2	743.8	76.3						11,023.3
12	40-14		14.0 (15.8)						2,516.1	616.9	80.3						11,150.2
13	45- 8	45.0	8.0 (10.0)	8,424.3	8,003.1	421.2	95.0	3,133.0	2,044.6	1,088.4	65.3	385.0	365.8	19.2	95.0	305.8	10,719.3
14	45-10		10.0 (12.0)						2,230.0	903.0	71.2						10,904.7
15	45-12		12.0 (14.0)						2,389.2	743.8	76.3						11,063.9
16	45-14		14.0 (16.0)						2,516.1	616.9	80.3						11,190.8

Note: ( ) Shows the capacity of Palacé No. 2 waterway

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

Case	Mark	Firm discharge (m^3/sec)	Installed Capacity (MW)	Annual Energy Production (10^6 kWh)	Construction Cost (10^6 $)	Annual Cost (10^6 $)	Cost of Energy (¢/kWh)	Surplus Benefit (10^6 $)	B/C
1	30- 8	25.0	53.0	262.2	347.1	36.55	13.94	24.89	1.631
2	30-10			266.9	349.5	36.80	13.79	25.19	1.635
3	30-12			271.0	351.9	37.06	13.68	25.40	1.636
4	30-14			274.3	356.8	37.57	13.70	25.25	1.623
5	35- 8	25.0	53.0	269.3	347.9	36.63	13.60	25.66	1.649
6	35-10			274.0	350.4	36.90	13.47	25.93	1.651
7	35-12			278.1	352.7	37.14	13.35	26.16	1.653
8	35-14			281.4	357.6	37.66	13.38	26.01	1.640
9	40- 8	25.0	53.0	273.8	348.8	36.73	13.41	26.10	1.659
10	40-10			278.6	351.2	36.98	13.27	26.39	1.661
11	40-12			282.6	353.6	37.23	13.17	26.60	1.662
12	40-14			285.9	358.5	37.75	13.20	26.44	1.649
13	45- 8	25.0	53.0	274.8	351.5	37.01	13.47	25.91	1.650
14	45-10			279.6	353.9	37.27	13.33	26.21	1.652
15	45-12			283.7	356.3	37.52	13.23	26.42	1.654
16	45-14			286.9	361.2	38.03	13.26	26.26	1.641

Note: (   ) Shows the capacity of Palacé No. 2 Waterway

Fig. 7-1 Result of Study on Optimum Capacity of Cauca and PaLace Diversion Waterway (Cost of Energy)

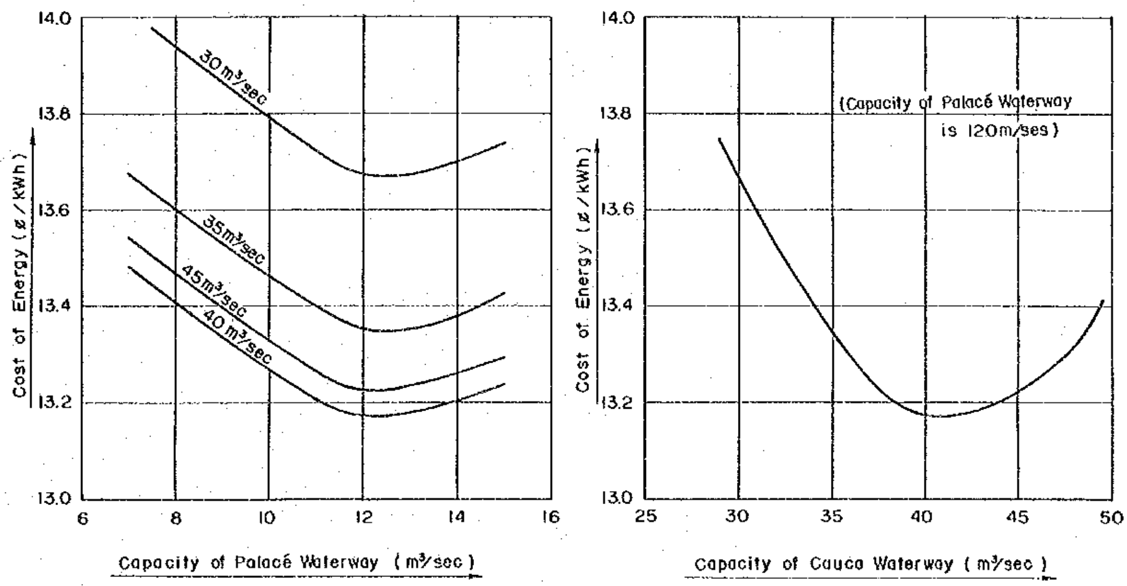
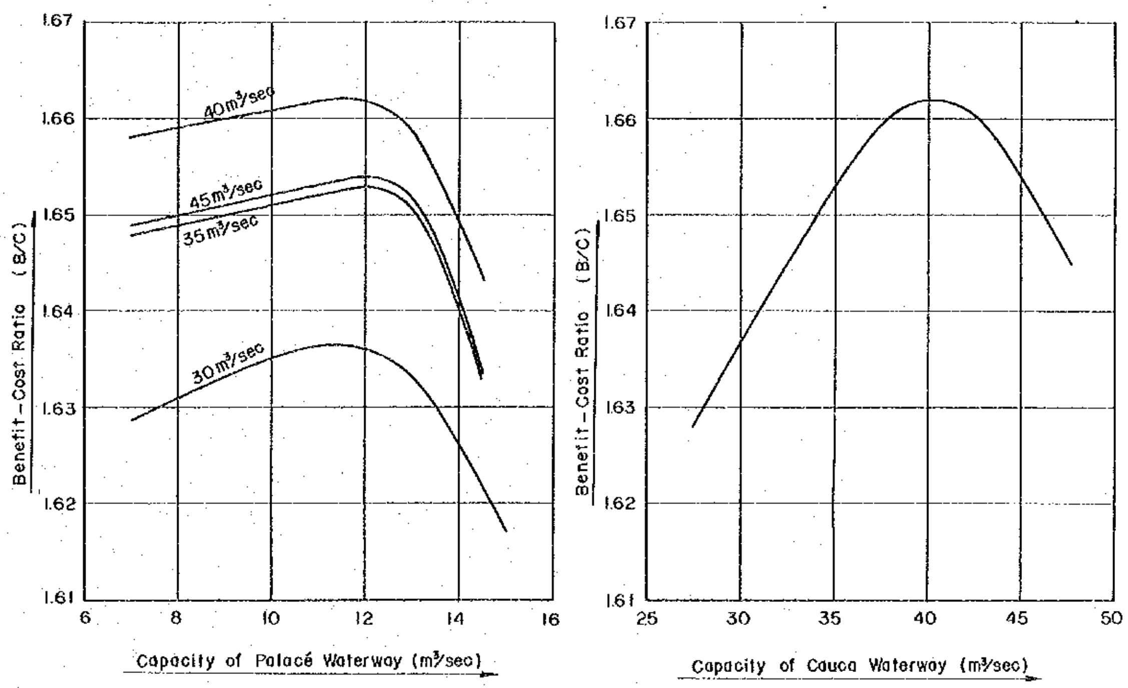


Fig. 7-2 Result of Study on Optimum Capacity of Cauca and PaLace Diversion Waterway (Benefit-Cost Ratio)



is in January ~ June and 503 x 10^6 m^3 (31.7 m^3/sec) in July - December, the inflow for the latter half being approximately 1.1 times that of the first half.

The yearly inflow for the latest 10 years shows a fairly great fluctuation. Namely, of the 10 years from 1962 to 1971, relative drought continued for the initial 3 years while the last 3 years are high water years. The driest year in the 10-year period was 1966, the reservoir inflow in this year being 857 x 10^6 m^3 (27.2 m^3/sec) while the wettest year was 1970 with inflow of as muc as 1,072 x 10^6 m^3 (34.0 m^3/sec).

In order to develop the water resources in the most effective and economical manner by adjusting such seasonal and yearly flactuations in inflow it is necessary to secure a reservoir storage capacity allowing the inflow to the reservoir to be effectively regulated for supplementation in dry seasons and dry years to aim for stabilization of the power supply over lnng periods of time.

The high water surface level of the reservoir must be at an elevation making it possible for the abovementioned storage capacity to be secured while maintaining the economy of power generation.

From the above viewpoint, comparison studies will be made in this paragraph of three elevations within the range of 1,712 m and 1,718 m, which are considered to be technichally feasible judged from the results of field surveys in regard to the high water surface level of Julumito Reservoir. They are El. 1,712 m, 1,715 m and 1,718 m.

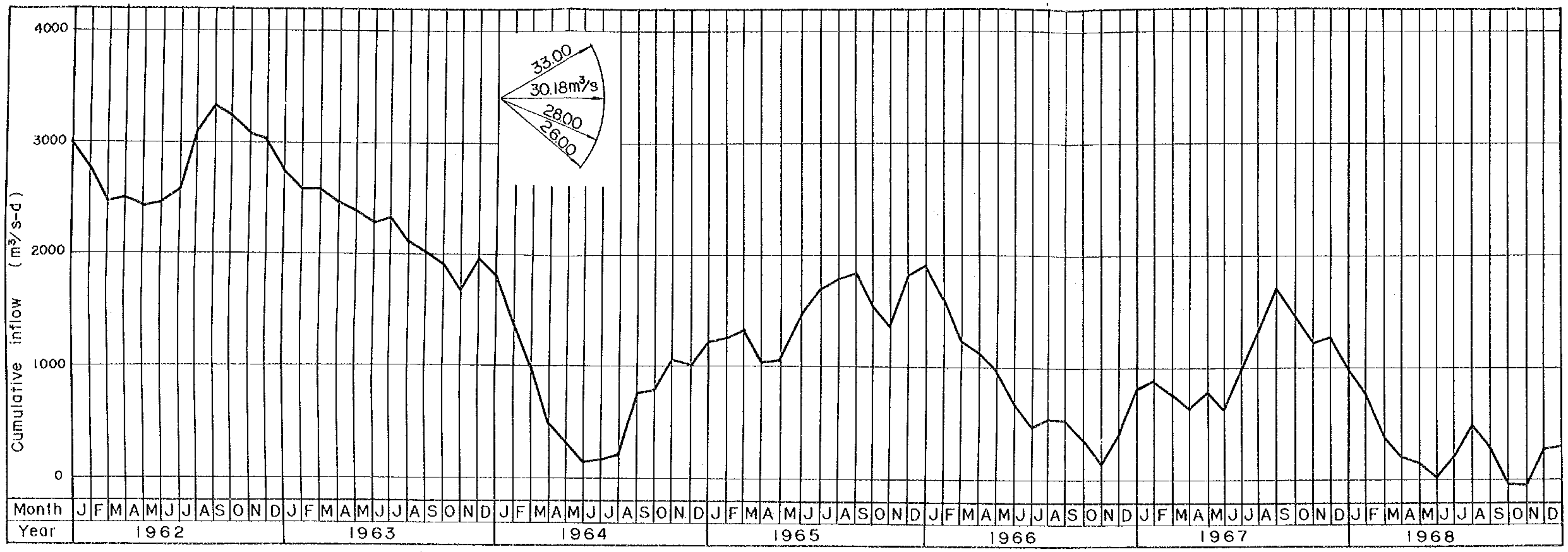
In regard to the effective storage capacity of the reservoir, the cases below are selected in the range of 20.0 x 10^6 m^3 ~ 60.0 x 10^6 m^3 for the various abovementioned high water surface levels, and the respective energy costs and benefit cost ratios are calculated and compared.

	High Water surface Level (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

The conditions for the calculations are as follows

(1) The capacities of the Cauca and Palacé diversion waterways are to be 40.0 m^3/sec and 12.0 m^3/sec (Palacé No. 2, diversion waterway is 13.8 m^3/sec) respectively.

Fig. 7-3 Mass Curve of Julumito Reservoir



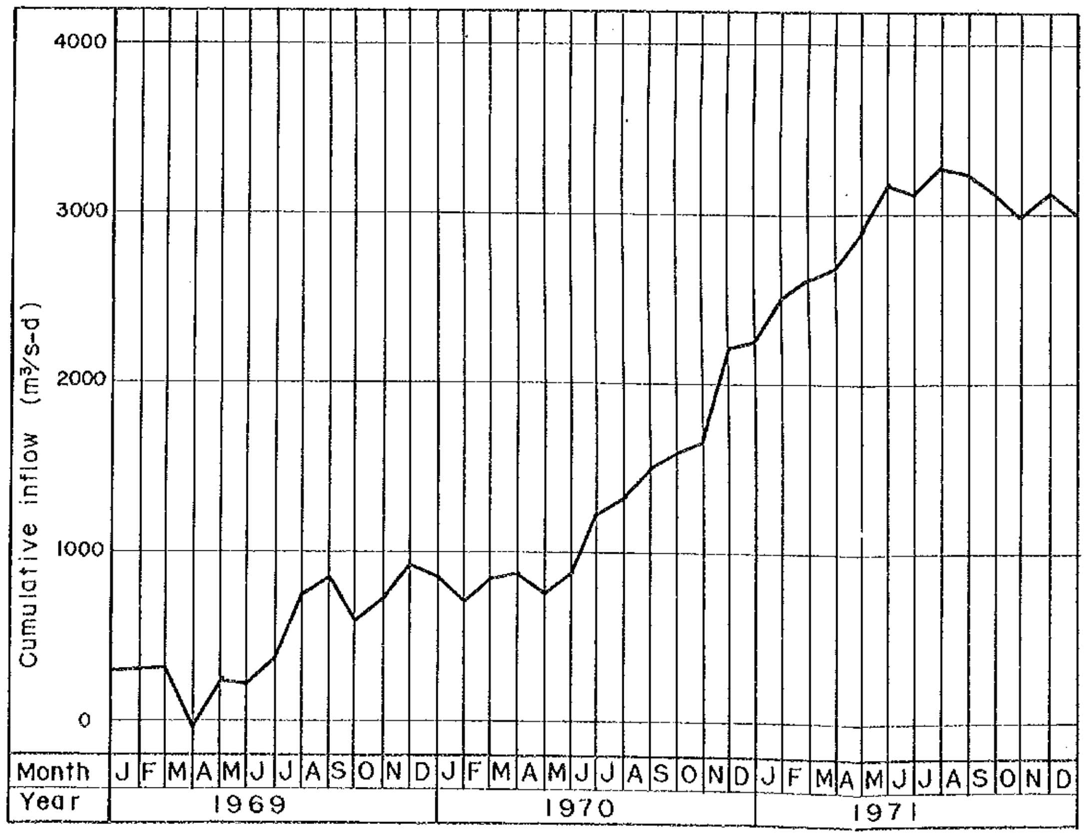
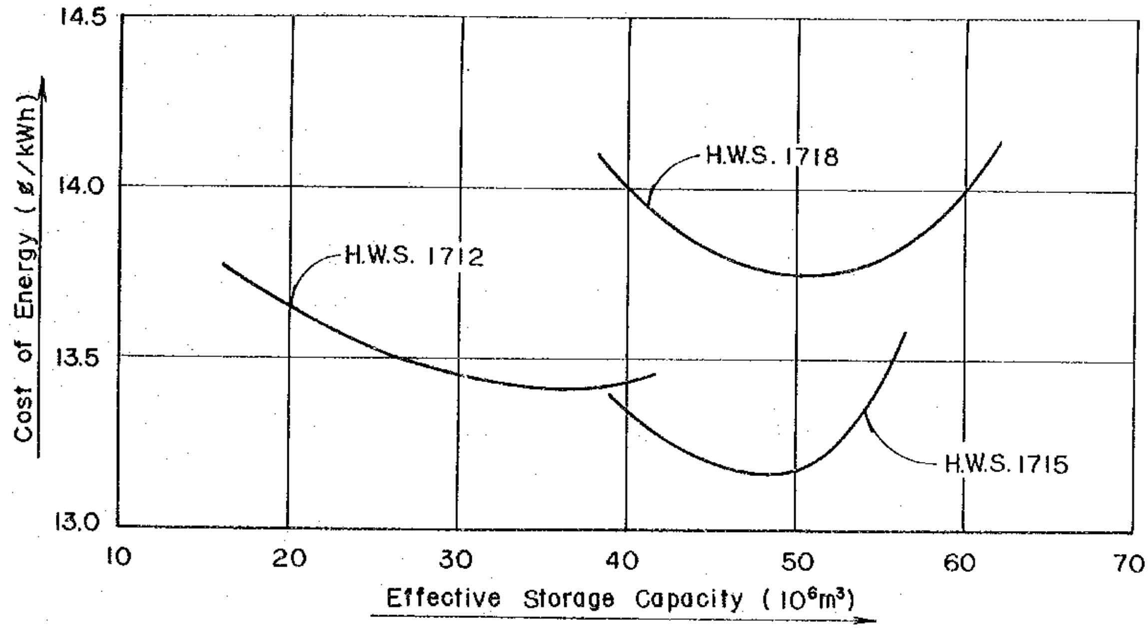
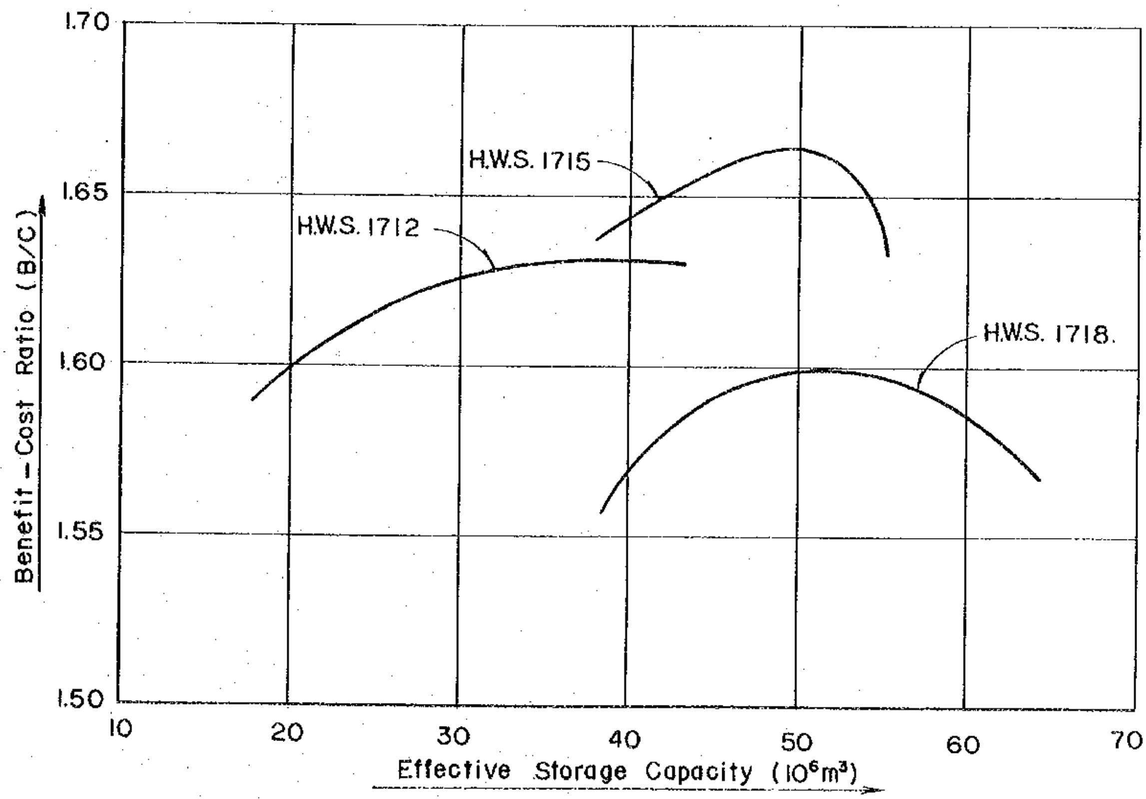


Table 7-2 Study on Optimum High Water Surface and Effective Storage Capacity

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 (10^6 \$)	Annual Cost (10^6 \$)	Cost of Energy (¢/kwh)	B/C
		Gross	Effective									
1			20	23.1	46.2	125.8	49.2	263.6	341.7	35.98	13.65	1.600
2	1,712	47.6	30	23.7	47.4	124.7	50.1	267.7	342.0	36.01	13.45	1.626
3			40	24.4	48.8	123.0	50.8	271.1	345.8	36.41	13.43	1.630
4			40	24.4	48.8	127.1	52.5	279.1	353.8	37.26	13.35	1.643
5	1,715	60.8	50	25.0	50.0	125.5	53.0	282.6	353.6	37.23	13.17	1.665
6			55	25.3	50.6	124.4	53.3	281.1	358.9	37.79	13.44	1.638
7			40	24.4	48.8	130.7	54.0	286.3	380.5	40.07	14.00	1.570
8	1,718	77.5	50	25.0	50.0	129.8	55.0	291.1	380.1	40.02	13.75	1.599
9			60	25.7	51.4	128.6	56.0	289.5	385.5	40.59	13.02	1.586

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





(2) The reservoir is to be operated in accordance with the operation rules described in 7.3, and the annual energy productions are to be calculated for the 10-year period from 1962 through 1971.

(3) As dependable discharge, a discharge which can be guaranteed for roughly 10 years for each case based on the reservoir operation described in 7.3 is taken.

(4) The installed capacity is to be determined based on the normal effective head and plant factor of 50%.

(5) The benefit is to be calculated with the annual costs per kW and kWh of the alternative thermal power station described in 10.3.2 as criteria (see 10.3.2).

(6) The annual costs are calculated multiplying the construction cost by the annual cost ratio. The construction costs of major structures are to be obtained through preliminary designing.

The results of studies made according to the above are indicated in Table 7-2 and Fig. 7-4. As seen from these study results, it may be said that the case of reservoir high water surface level of 1,715.0 m and effective storage capacity of 50 x 10^6 m^3 would have the highest economy and is the optimum scale.

Therefore, it is decided for the reservoir of this Project to have a high water surface level of 1,715.0 m, available drawdown of 15.0 m and effective storage capacity of 50.0 x 10^6 m^3.

7.3 Reservoir Operation

The operation rule of Julumito Reservoir is established considering the various points indicated below.

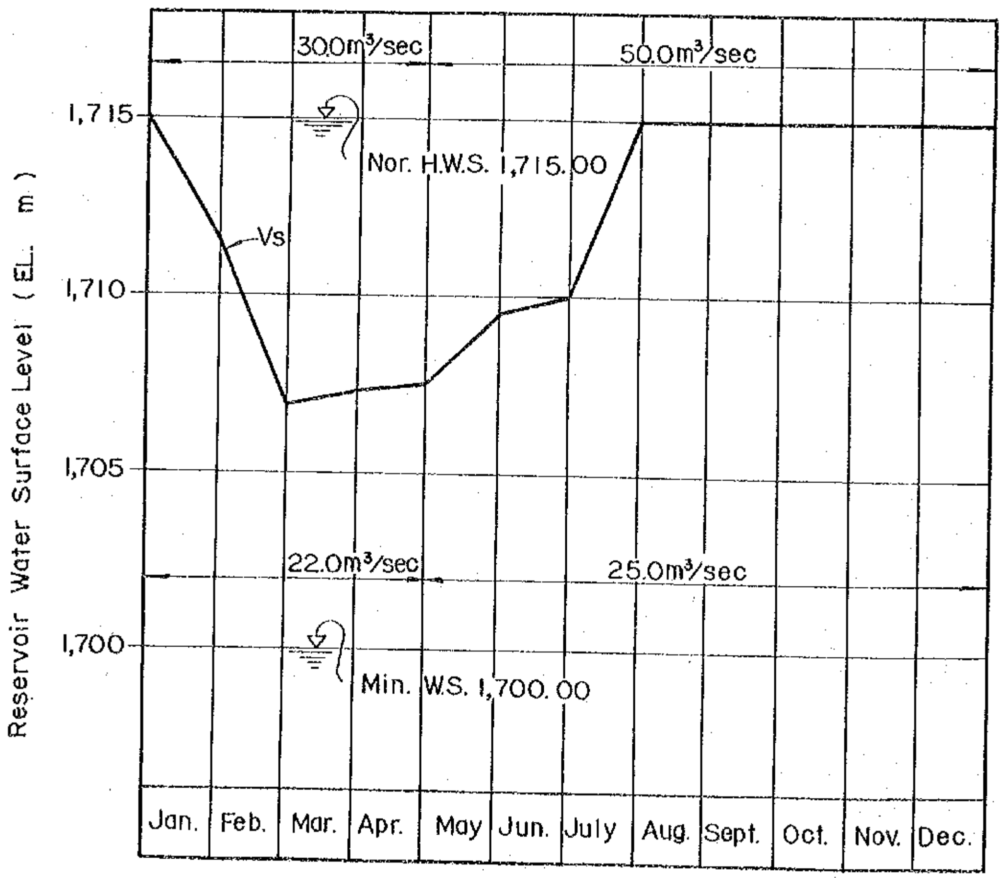
(1) So-called seasonal regulation during a year by storing the inflow of the high-water season and discharging this in the dry season will be carried out.

(2) Operation is to be carried out in a manner to secure as large dependable discharge as possible storing the run-offs of wet years for supplementing water in dry years.

(3) Operate in a manner to minimize ineffective overflow of the reservoir as much as possible.

Fig. 7-5 Operation Rule of Reservoir

Month	Vs		
	m	$10^6 m^3$	m^3/sec-day
Jan.	1, 711. 7	34.00	393.5
Feb.	1, 707. 0	17.48	202.3
Mar.	1, 707. 3	18.47	213.8
Apr.	1, 707. 5	19.15	221.6
May	1, 709. 5	26.30	304.4
Jun.	1, 710. 0	28.26	327. 1
Jul.	1, 715.0	50.00	578.7
Aug.	1, 715.0	50.00	578.7
Sept.	1, 715.0	50.00	578.7
Oct.	1, 715.0	50.00	578.7
Nov.	1, 715.0	50.00	578.7
Dec.	1, 715.0	50.00	578.7



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 Apr.
50.0 m^3/sec May to Dec.

Q_L = 22.0 m^3/sec Jan. to Apr.
25.0 m^3/sec May 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

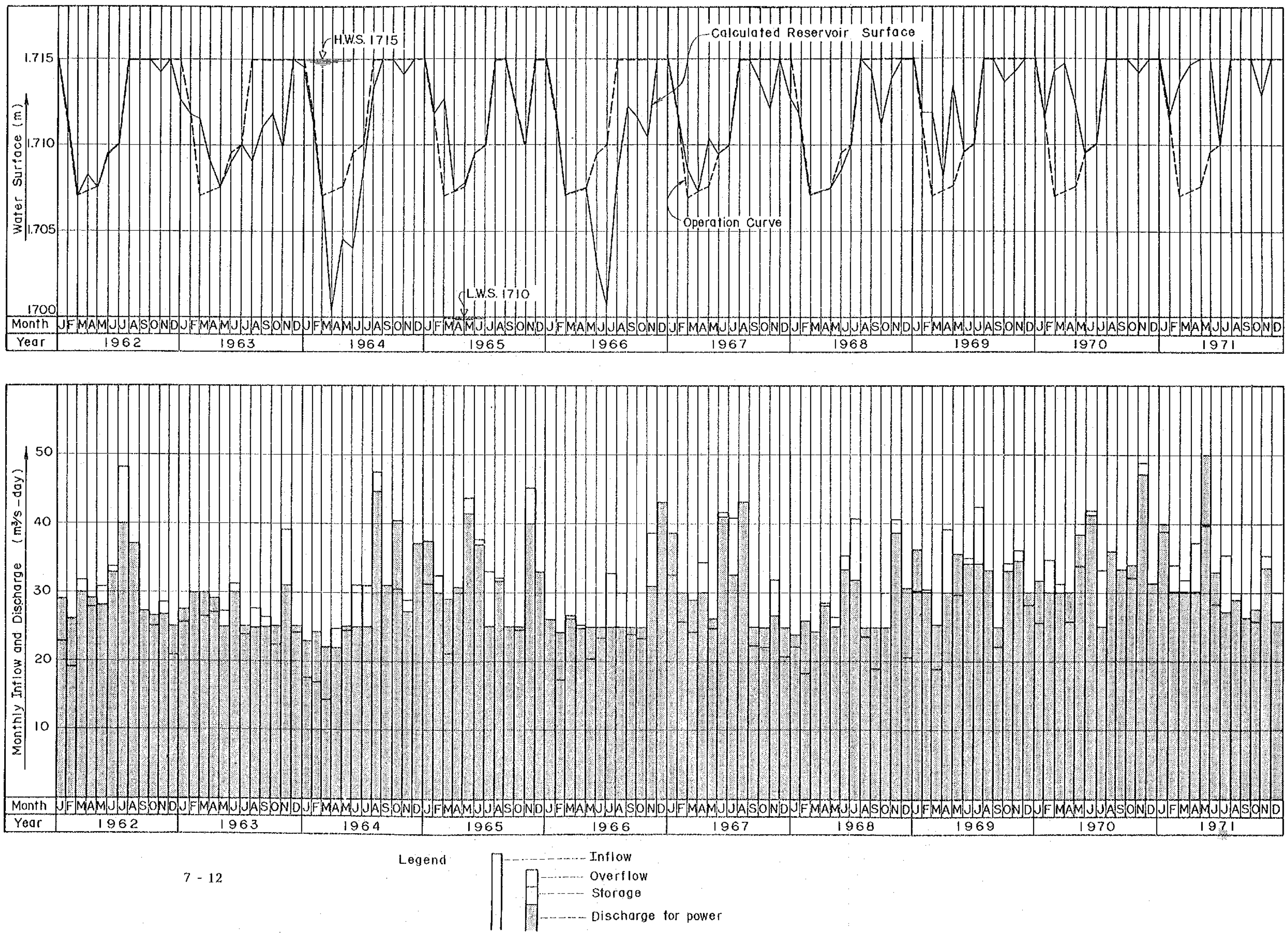
1. $V_{n-1} + q_n > V_s$

(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 1971)



(4) Operate in a manner that a stable output can be secured, while moreover, energy production is increased over long periods of time.

Fig. 7-5 gives the Julumito Reservoir operation rule established based on the above. This operation rule has been established only tentatively for the purpose of studying the Julumito Hydroelectric Project, and when Julumito Power Station is completed and the time for actual operation arrives, operation should be carried out with more precise operation rule.

The available discharge, storage and supplementation, and reservoir water level in case of operation of Julumito Reservoir for the 10-year period from 1962 through 1971 based on the reservoir operation rule given in Fig. 7-5 are as indicated in Fig. 7-6.

The energy production of Julumito Power Station based on the abovementioned operation rule are indicated in Table 7-5 and Fig. 7-8.

7.4 Dependable Discharge

The water level fluctuation after operation of Julumito Reservoir according to the operation rule described in the preceding paragraph is indicated in Fig. 7-6. According to this, during the 10-year period from 1962 through 1971, only the 4-month period from January through April in 1964, a dry year, shows available discharge to be slightly under 25.0 m^3/sec, but for the 9 years and 8 months other than the above period, discharge of 25.0 m^3/sec or more is available. As is clear also from the mass curve of Fig. 7-3, the first half of 1964 from January to May showed an abnormal drought which is considered to be an extremely rare phenomenon.

If 22.0 m^3/sec is taken as the dependable discharge, it would mean that the water resources could not be utilized to the maximum extent and consequently it is liable to lower the economy of the Project. Rather, it would be more rational to set the dependable discharge slightly higher than 22.0 m^3/sec and supplement the shortage of electric power in such a rare case with the reserve capacity of the power system or by the power improted through the interconnecting transmission line.

Upon these consideration, the dependable discharge of Julumito Power Station is selected to be 25.0 m^3/sec.

7.5 Maximum Discharge

The maximum discharge and installed capacity of Julumito Power Station must be decided so as to fill the power supply deficiencies of the CEDELCA and CEDENAR service areas in the most economical manner.

As has already been contemplated in Chapter 3, "LOAD FORECAST" it is thought that Julumito Power Station should carry the peak parts of the load together with other power stations possessing regulating reservoirs.

In other words, in regard to daily load curves from 1985 and after, the balance of the demand after deducting the supply capacities of power stations presently existing or under construction will be as follows:

Year	Peak Difference (MW)	Energy Difference (MWh)	Load Factor (%)
1985	66.7	1,139	71
1990	133.8	2,121	66
1995	237.1	3,633	64

This balance of demand would be supplied through new power source development (Julumito Power Station etc.) and other means, an interconnecting transmission line, etc.).

As is clear from the above balance of demand, the immediate resulting load factor for at least 20 years from 1982 is supposed to be roughly 70 to 60%.

After 1985 when the facilities of Julumito Power Station become actualized, Julumito power Station would carry the peak part of the resulting load with 50% of plant factor and it is considered that the base portion should be carried by the interconnecting transmission line.

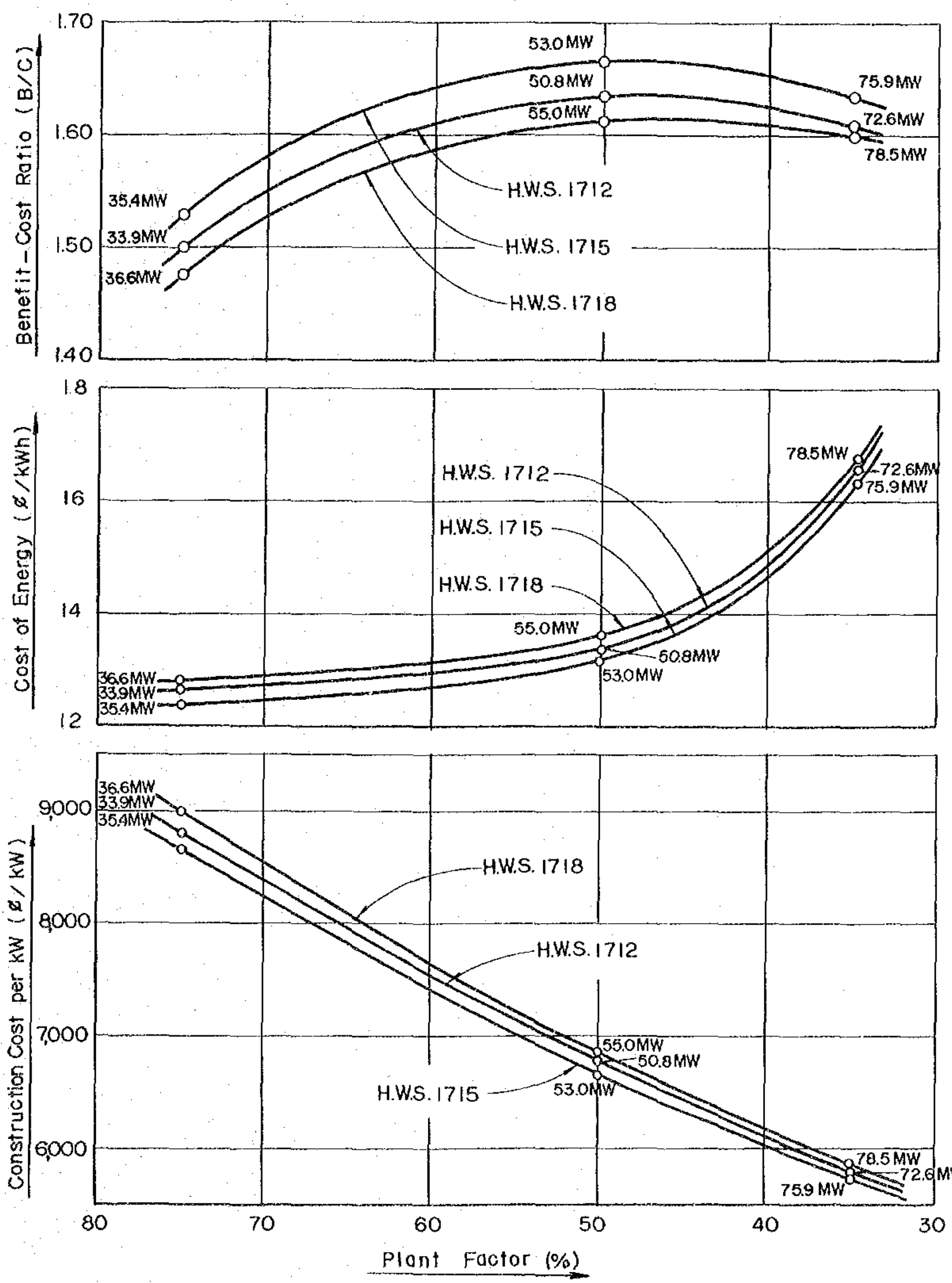
Julumito Power Station is a reservoir-type power generation project having an effective storage capacity of 50×10^6 m^3 and capable of seasonal and yearly regulation. However, the length of the headrace tunnel is about 1,800 m and relatively long, while moreover, there are diversion waterways amounting to approximately 11,000 m. Therefore, there is a natural limit to the economy as a peak power station. In effect, it is thought to have the characteristics of a site where the economy worsens sharply when the plant factor is taken at too low a rate.

There is a necessity for thorough study to be made of the above site characteristics also in deciding the plant factor.

Table 7-3 Study on Optimum Maximum Discharge and Installed Capacity

Case	H. W. S. (m)	E. Storage Capacity (10^6 m^3)	Drawdown (m)	Firm Discharge (m^3/sec)	Plant Factor (%)	Max. Discharge (m^3/sec)	Installed Capacity (MW)	Annual Energy Production (10^6 kWh)	Construction Cost Total (10^6 $)	Construction Cost ($/kW)	Annual Cost (10^6 $)	Cost of Energy (¢/kWh)	B/C
1					75	32.5	33.9	249.0	298.6	8,810	31.44	12.63	1.501
2	1,712	40.0	13.4	24.4	50	48.8	50.8	271.1	344.7	6,790	36.30	13.39	1.635
3					35	69.7	72.6	271.1	424.2	5,840	44.67	16.48	1.609
4					75	33.3	35.4	260.0	306.1	8,650	32.23	12.40	1.529
5	1,715	50.0	15.0	25.0	50	50.0	53.0	282.6	353.6	6,660	37.23	13.17	1.665
6					35	71.4	75.9	282.6	436.3	5,750	45.94	16.26	1.634
7					75	33.3	36.6	270.7	328.9	8,990	34.63	12.79	1.477
8	1,718	50.0	11.1	25.0	50	50.0	55.0	291.1	376.6	6,850	39.66	13.62	1.614
9					35	71.4	78.5	291.1	459.6	5,850	48.40	16.63	1.601

Fig. 7-7 Result of Study on Optimum Max. Discharge and Installed Capacity



The economies of Julumito Power Station corresponding to the various plant factors studied based on ① construction cost per kW of installed capacity, ② energy cost per kWh ③ evaluation (B/C) with cost of alternative thermal power station as a criterion are indicated in Table 7 - 3 and Fig. 7 - 7.

According to these, it is seen that the Julumito Hydroelectric Project has the following site characteristics:

1. When the plant factor is taken to be lower than 50%, the construction cost per kW becomes lower, but there is a tendency for the energy cost per kWh to be fairly high.

2. Even though the plant factor is taken to be higher than 50%, the energy cost per kWh is not lowered very much.

3. This tendency is clearly indicated with respect to B/C also.

Also, the Julumito Hydroelectric Power Project, as indicated in Chapter 10, "ECONOMIC EVALUATION", is a project with fairly good economic soundness compared with construction of an alternative thermal power station or purchase of power from CVC System via the interconnecting transmission line, but it is considered desirable for the energy cost per kWh at Popayan Substation to be about 13 CVS or lower when the related expenses to be incurred at and beyond the substation are taken into consideration.

Consequently, the plans are established with the plant factor of Julumito Power Station at 50% for a dry year, 56% for a normal year with maximum discharge being 50.0 m^3/sec. Also, in the future, it will be possible to make Julumito even more of a peaking station by adding some generating facilities as necessary.

7.6 Dependable Peak Discharge

The dependable peak discharge for the dependable peak output of Julumito Power Station, is the available discharge that the turbines of this power station can take in at the time of low water level of the reservoir and is set at 48.1 m^3/sec.

7.7 Installed Capacity and Dependable Peak Output

The installed capacity, and dependable peak output are calculated based on the normal effective head and low water level respectively with the results shown in Table 7-4, reservoir for dependable peak output. The tailrace water level is taken to be at the estimated water level at the outlet of the power station at the available discharge.

Table 7-4 Calculation of Output

	Unit	Installed Capacity	Firm Peak Output (Dependable Capability)
Available Discharge	(m^3/sec)	50.0	48.1
Intake Water Surface	(m)	1,710.0	1,700.0
Tailrace Water Surface	(m)	1,577.0	1,577.0
Effective Head	(m)	125.5	116.1
Output	(kW)	53,000	47,300

7.8 Quantities of Main Equipment

Regarding the numbers of main equipment of Julumito Power Station, a one-unit plan and a two-unit plan are conceivable.

In general, when the number of units is small but the capacities are large, there are the merits that

(1) the machine installation cost becomes cheaper,

(2) the line charging capacity is increased, and

(3) the installation period is shortened,

but on the other hand, there are the demerits that

(4) the effect on the system during stoppage due to fault is great,

(5) it is difficult from the standpoint of system operation for stoppages to be made for periodical inspections, etc.,

(6) the ineffective discharge during stoppage is omcreased,

(7) the efficiency at partial load is bad,

(8) transportation during construction is difficult,

(9) crane capacity is increased, and

(10) spare parts become costlier.

Further, for determination of the number of units, or unit capacity, the maximum unit capacity, the maximum unit capacity of existing facilities within the system, the reserve capacity of the system and the economy of the project must be considered. Florida II Power Station will have been in operation at the time Julumito Power Station becomes operative and the 12,000 kW of Florida would be the maximum unit capacity. Therefore, the unit capacity of Julumito Power Station, when the possibility of fault is considered, is not desirable to be very much large, but since the CEDELCA System will be interconnected with the Central System, and this interconnecting transmission line is considered as reserve capacity as previously mentioned, the limitations from this aspect would be alleviated. On the other hand, it would be more economical if unit capacity is increased and the number of units is decreased, but in view of the demerits (4) ~ (10) listed above and due to the fact that Julumito Power Station is of extreme importance in the system, it is thought that stoppage would have a great influence even though there would be the interconnecting transmission line, and it is considered that adoption of a single turbine-generator unit of large capacity should be avoided from the standpoint of system operation, and for this Project, two units will be adopted.

7.9 Producible Energy

Using the available discharge for power generation and the reservoir water level calculated based on the Julumito Reservoir operation rule established in Paragraph 7.3, the monthly available energies for the 10-year period from 1962 through 1971 are calculated. The results are as shown in Table 7-5 and Fig. 7-8 and the average energy production during the above 10 years would be as follows:

Primary energy	237.7×10^6 kWh
Secondary energy	47.7×10^6 kWh
Total	285.4×10^6 kWh

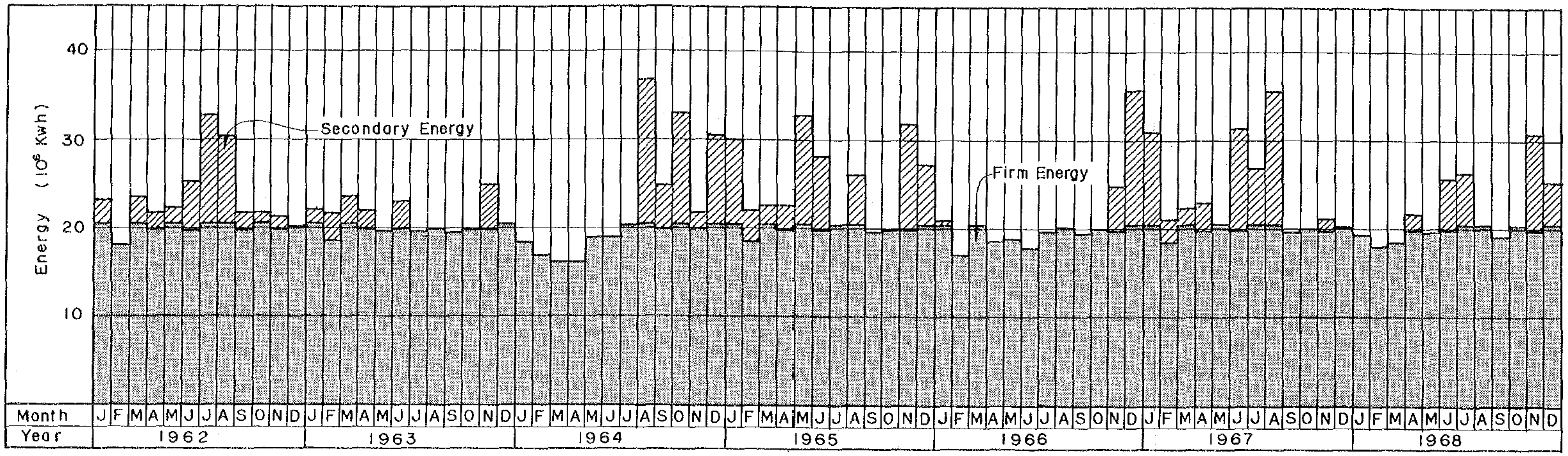
Further, the secondary energy should be an energy exceeding the dependable output (the output, 27,500 kW, in case of use of dependable discharge at normal intake level).

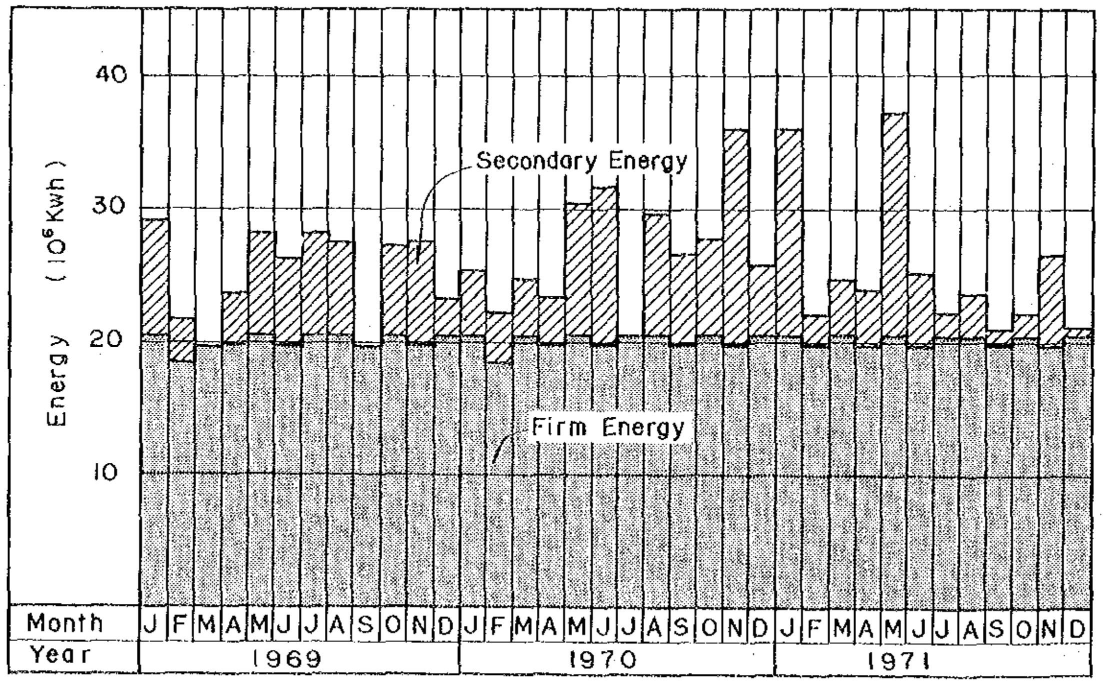
Table 7-5 Energy Production

(Unit : 10^6 kWh)

Month / Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	July.	Aug.	Sept	Oct.	Nov.	Dec.	Annual
1,962	23.1	18.1	23.4	21.7	22.2	25.1	32.7	30.4	21.7	21.7	21.2	20.2	281.5
1,963	22.0	21.7	23.5	21.9	19.6	23.0	19.6	19.9	9.4	19.8	24.8	20.5	255.7
1,964	18.3	17.0	16.2	16.1	18.8	18.9	20.3	36.7	24.8	33.0	21.7	30.5	272.3
1,965	29.9	21.9	22.5	22.5	32.7	28.2	20.5	26.0	19.5	19.8	31.8	27.2	302.5
1,966	20.9	16.9	20.4	18.6	18.7	17.8	19.6	20.1	19.4	19.9	24.7	35.6	252.6
1,967	31.0	21.2	22.4	23.0	20.6	31.4	26.8	35.6	19.7	20.1	21.2	20.2	293.2
1,968	19.3	18.1	18.5	21.2	19.6	25.6	26.3	20.5	19.2	20.4	30.7	25.2	264.6
1,969	29.1	21.7	19.6	23.6	28.1	26.3	28.2	27.5	19.7	27.2	27.5	23.2	301.7
1,970	25.4	22.2	24.6	23.4	30.4	31.6	20.5	29.6	26.6	27.7	36.0	25.8	323.8
1,971	36.0	22.0	24.6	23.9	37.2	25.1	22.3	23.7	21.0	22.2	26.6	21.1	305.7
Average	25.5	20.1	21.6	21.6	24.8	25.3	23.7	27.0	21.1	23.2	26.6	25.0	258.4

Fig. 7-8 Energy Production





(10^6 kWh)

Item / Year	Firm Energy	Secondary Energy	Total
1962	240. 5	41. 0	281.5
1963	237. 7	18. 0	255.7
1964	226. 7	45. 6	272.3
1965	240. 2	62. 3	302.5
1966	232. 2	20. 4	252.6
1967	240. 4	52. 8	293.2
1968	236. 0	28. 6	264.6
1969	240. 2	61. 5	301.7
1970	241. 2	82. 6	323.8
1971	241. 2	64. 5	305.7
Average	237. 7	47. 7	285.4

Note :

Based on Reservoir Operation Rule in Fig. 7-5

CHAPTER 8

PRELIMINARY DESIGN

CONTENTS

FIGURE LIST

DRAWING LIST

CHAPTER 8 PRELIMINARY DESIGN

8.1 Design

8.1.1 *Civil Structures*

(1) Main Dam

For the type of the dam it is judged most suitable for a rockfill dam to be adopted in consideration of the topography of the dam site and the geological conditions described in Chapter 6, "GEOLOGY AND CONSTRUCTION MATERIALS", the characteristics of the dam embankment materials available in the area surrounding the dam site, and further, the behavior of the dam after completion.

The construction of a concrete dam, as stated in Chapter 6, "GEOLOGY AND CONSTRUCTION MATERIALS" is considered to be unfeasible technically and economically for the reasons that the geology of the ground which would serve as the foundation of the dam is mainly andesite lava but partially includes volcanic ash and that good quality aggregate for concrete is not possible to obtain in large quantity.

Also, the impermeable soil materials which are possible to collect at the dam site are weathered residual soils from volcanic ash layers and andesitic lava. Both have fairly high impermeabilities as core materials, although they are generally very fine-grained with high natural water content.

In consideration of the above properties of impermeable soil materials (especially compressibility), the characteristics of the rock materials to be used, the topography of the dam site and outcropping of bedrock, the type of the fill dam is selected to be a curving rockfill dam of inclined core design with upstream slope of 1 : 2.3 and downstream slope of 1 : 1.8.

Diversion of the river flow during construction is to be by a diversion tunnel of inner diameter of 2.2 m to be provided on the left bank. The capacity of the diversion tunnel in consideration of safety during the period of construction of the dam will be 25.0 m^3/sec.. This diversion tunnel will be converted to a draw-off tunnel after completion of the dam construction, adding an intake structure and a valve in the tunnel.

In regard to the embankment materials of the fill-type dam and the geological condition of the dam foundation, further detailed investigations are necessary and the final design of the dam should be made based on the results.

(2) Spillway

As stated in Chapter 5, "HYDROLOGY", the design flood discharge of the main dam site (Rio Sate) is 50.0 m^3/sec. The spillway is to be a chute-type of a structure capable of discharging 50.0 m^3/sec under high water surface level of 1,715.00 m. The spillway will be provided at the left bank of Dike No. 1 where it will be possible to construct it economically from a topographical standpoint while the design will be for regulation of discharge to be made by one slide gate.

(3) Intake

The intake structures is to be an inclined type of 1 : 0.7 located on the left bank approximately 200 m upstream from the dam site. The bed elevation of the intake, in consideration of the geological state of the site, center line elevation of the headrace tunnel and construction methods, is established at an elevation of 1,665.5 m which is 34.5 m below the low water level of the reservoir. Trash rack and rake guide rails are to be provided in front of the intake. A roller gate is to be installed along the slope of the intake orifice for the purpose of maintenance and inspection of the headrace tunnel.

The intake structure is also designed so as to prevent the inflow of soil or sand to the headrace tunnel and powerhouse.

(4) Headrace Tunnel

The type is to be a circular pressure tunnel. Regarding the cross-sectional dimensions of the tunnel, the annual cost and the loss of benefit due to head losses are calculated for a number of inner diameters and an inner diameter by which the sum would be a minimum is selected. The result of examination is as shown in Fig. 8-1 and the inner diameter of the tunnel is taken to be 4.20 m.

The geology between the intake and the powerhouse where the headrace tunnel will be provided is comprised of a volcanic ash layer from the top to approximately 30 ~ 40 m underneath where there is thick andesite lava. The center line of the route of the headrace tunnel is selected so that it would be inside this layer of andesite lava, moreover with ample coverage provided by this layer.

It is planned for the tunnel to be lined with reinforced concrete over its entire length with the intake, surge tank vicinity or places of poor geological conditions to be further reinforced with inner-lining steel pipe, etc.

Also, as stated in Chapter 6, "GEOLOGY AND CONSTRUCTION MATERIALS", it is thought there is no risk of faults and large amount of water springs existing in the tunnel route, so that there should be no special problems in construction. Adits are to be provided at both the intake side and powerhouse side.

(5) Surge Tank

For the type of the surge tank, in consideration of the difficulty of construction due to the topography of this site, and the geological conditions because of which the upper half must be situated in a volvanic ash layer, complicated structures are avoided and an orifice type surge tank is selected, the cross-sectional shape being circular, with an inner diameter of 8.0 m.

In design of the surge tank, the conditions stipulated are that in case total load is broken at high water surface level of reservoir and that when 50% — 100% load is suddenly increased at low water level of reservoir, there would be no adverse effects on the headrace tunnel and the turbines.

(6) Penstock

In regard to the design of the penstock, as the result of the study for a surface type and a diagonal type upon consideration of the topography of the installation site, especially the inclination and geology of the ground surface, the surface type is adopted, which is possible to install on the ground embankment materials of the main dam have been collected, diagonal shaft type penstock would require additional adits and costly.

The penstock will be comprised of welded steel pipe in one line which is to be bifurcated into two lines immediately prior to entering the powerhouse. The pipe shell materials are to be chiefly SM 53 or SM 58 (JIS) or equivalent material.

Regarding the inner diameter of the pipeline, as in the case of examination of the inner diameter of the pressure tunnel, the annual cost and the loss of behefit fue to head losses are calculated and a result in which the sum is a minimum is adopted. The result is as indicated in Fig. 8-2 with the average inner diameter of 3.60 m. Based on this, the inner diameter will be made to vary from 4.0 m to 3.2 meters between No. 1~No. 3 anchor block.

The design of the penstock is to be considered with hrad due to surging and water hammer pressure as the internal pressures.

Fig. 8-1 Study on Headrace Tunnel Diameter

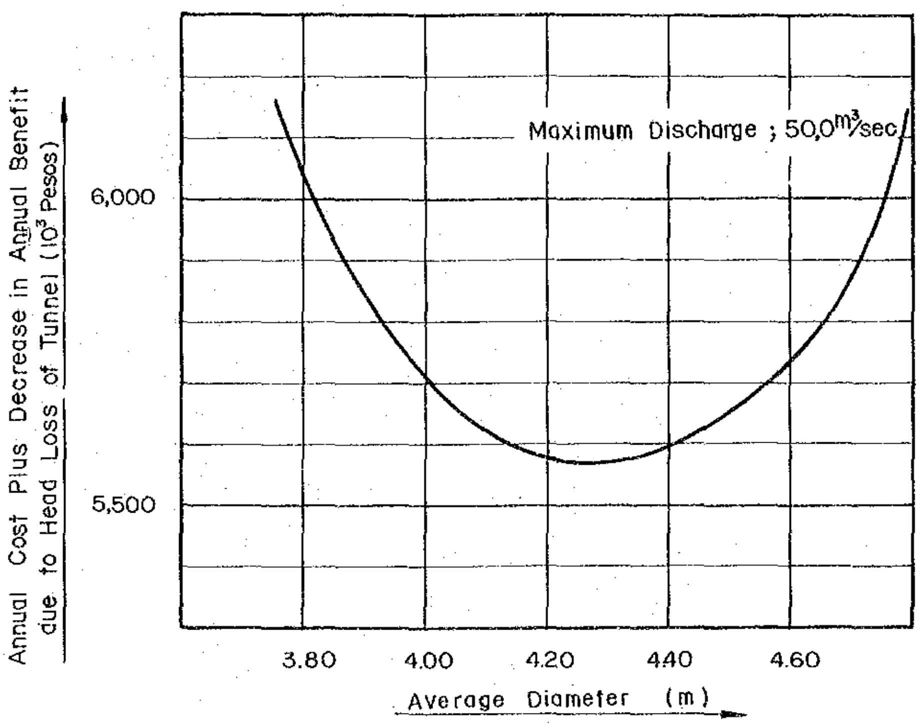
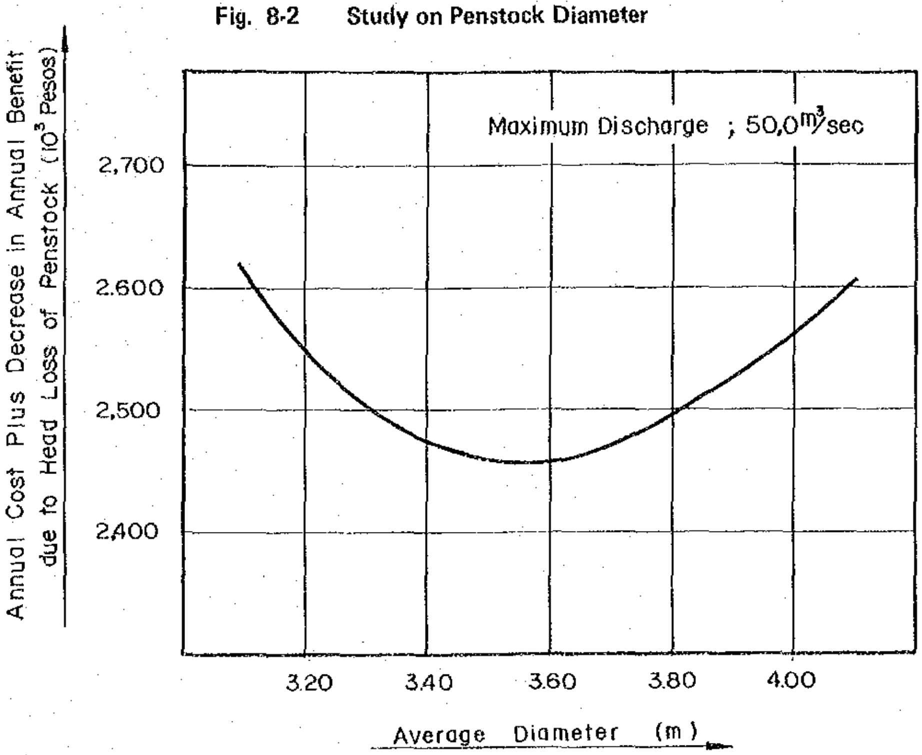


Fig. 8-2 Study on Penstock Diameter



Further, the excavated rock from the penstock route will be diverted to serve as the rock material of the main dam as stated previously.

(7) Powerhouse, Outdoor Switchyard

The powerhouse and switchyard are to be surface-type in consideration of the geology. The draft tubes are to be L-type with gates provided at outlets.

The outdoor switchyard is to be installed adjacent to the powerhouse on the upstream side.

Rock excavated at the powerhouse and switchyard sites are to be diverted insofar as practicable to embankment material for the main dam.

(8) Rio Cauca Diversion Dam

The type is to be a concrete gravity dam with the crest length of 71.0 m, and the hight of 11.0 m.

A sand flush gate, 6.0 m x 3.0 m, is to be installed near the intake at the right bank.

As for the sand-gravel layer which is the foundation of the dam, grouting will be performed along the dam axis at intervals of 1 m~2 m to depths of 20 m~25 m.

The intake bay is to be provided with a sand flush sluice of 1.0 m x 1.5 m for a structure to permit flush-out of sediment flowing into the intake.

(9) Rio Palacé Diversion Dam

The structure will be of the same type as Rio Cauca Diversion, with a free overflow section. The crest length of the dam is 30.4 m and the height is 7.7 m.

A sand flush gate of 4.0 x 3.0 m is to be installed near the intake at the left bank. A sand flush sluice 1.0 m x 1.0 m is to be installed at the intake bay.

(10) Rio Blanco Diversion Dam

The dam is to be a free overflow concrete gravity type as in the cases of the beforementioned 2 dams. Palacé Diversion Waterway will be of a design to pass through the inside of the dam in the form of an inverted siphon.

Intake of water from the Rio Blanco is to be provided on the left bank at the dam site and water of Rio Blanco is conducted to the Palacé Diversion waterway through a short channel.

(11) Cauca Diversion Waterway

Cauca Diversion Waterway is to consist of a 2,400-m open concrete canal and a 220-m tunnel. The concrete canal is to be selected to have a cross section for safe and economical downflow of a maximum water passage of 40.0 m^3/sec at a gradient of 1 : 600.

A side overflow section is to be provided near the intake of the open concrete canal for a design to cause overflow of any inflow in excess of the maximum capacity of the waterway.

The tunnel section is to be designed as a concrete-lined non-pressure tunnel of standard horseshoe type.

(12) Palacé No. 1 and No. 2 Diversion Waterways

The No. 1 Diversion Waterway is to be an open concrete canal of a length of 4,000 m. The design is to be for a cross section for safe and economical downflow of a maximum water passage of 12.0 m^3/sec at a gradient of 1 : 600. Immediately downstream from the intake, a side overflow section is to be provided in a design for overflow of any inflow in excess of the maximum water passage capacity of the waterway.

The No. 2 Diversion Waterway is to be comprised of a 3,660-m open concrete canal section and tunnels at 3 places totalling 770 m in length. The open canal section is to have a cross section for safe and economical downflow of a maximum water passage of 13.8 m^3/sec at a gradient of 1 : 600.

The tunnel sections are to be designed as concrete-lined non-pressure tunnels of standard horseshoe type.

8.1.2 Turbine and Generator

The normal effective head of this power station is 125.5 m, available drawdown 15.0 m and the maximum discharge per turbine 25.0 m^3/sec. To match these conditions, it would be most suitable to select vertical shaft Francis turbines.

The outputs of the turbines are to be 27,500 kW per unit with the revolution speed of 400 rpm.

Butterfly valves will be provided as inlet valves.

The generators are to be 29,500 kVA per unit at rated power factor of 0.9 (lag) with voltage of 11 kV, enclosed hood type.

For auxiliary equipment of the turbine and generator, a unit system is to be adopted.

Two 29,000 kVA, 3-phase, oil-immersed self-cooled and forced air cooled transformers will be installed at the outdoor switchyard to be provided adjacent to the powerhouse. The secondary side voltage of the transformer is to be 115 kV as will be described later (See 8. 1. 3 (1)).

The 115 kV out-going facility to Popayan Substation is to be single-circuit. A circuit breaker is to be provided at the outgoing point, but since the distance to Popayan Substation is only 10 km, in case a transfer trip system is employed as the protective system, it would be possible for the circuit breaker to be omitted.

For the operation of turbine, generator and various outdoor switchyard equipment, a one-man control system is adopted and it will be designed for operation of all equipment to be possible from the control room. Switchyard equipment of the power station are shown in Fig. 8-3.

The utmost precautions are to be taken in the protective system for the transmission line adopting a carrier relay system. The details are given in DWG. NO. 22, "SINGLE-LINE DIAGRAM."

8.1.3 Transmission Line and Telecommunication Facilities

(1) Transmission Line

In order to transmit the electric power produced at Julumito Power Station to load areas through Popayan Substation, a transmission line is to be installed between Julumito Power Station and Popayan Substation.

The transmission capacity of the transmission line, in consideration of the projected maximum output of Julumito Power Station of 53 MW and of the voltage of the existing power system, is to be 115 kV, 1 circuit.

The scheduled transmission line route with the exception of the vicinity of power station is a gentle hill area, and as there are no special difficulties with respect to transmission line construction and maintenance, a route crossing the Rio Cauca at the power station site and pointed more or less in a straight line to Popayan Substation is adopted. The route map is indicated in Fig. 4-2 and the transmission diagram in Dwg. No. 23.

The standard figure for steel towers to be used for the transmission line is indicated in Fig. 8-4.

Based on the fact that the route of this transmission line is around 1,700 m above the sea level and considering corona phenomena, the most economical 160 mm^2 (outer diameter 18.2 mm, strand composition Al 30/2.6 mm + Steel 7/2.6 mm) is adopted.

For insulation, strings of 6 insulators of 250-mm suspension type are adopted with arcing horns attached.

In order to avoid direct lightning, a overhead ground wire is to be installed.

The outline of the transmission line is indicated in 8.1.5, "Major Specifications."

(2) Transforming Facilities

The transmission line from Julumito Power Station will be connected by 115 kV bus bars with the interconnecting transmission line to the CVC System and Rio Mayo Power Station at Popayan Substation existing and in operation at the south end of Popayan City.

Also, one main transformer bank of 30 MVA is to be added for supply of electric power to the load in the city through this substation. The outline of the transforming facilities to be added to Popayan Substation is indicated in 8.1.5., "Major Specification."

The arrangement of the phase modifying facilities for the 115 kV transmission line is to be similar to that of the existing facilities taking into consideration adjacent bus connection facilities to be added in the future.

As a protection device for the transmission line, it would be possible to adopt a transfer trip system for a line of such a short length in which case it would be possible to omit the outgoing circuit breaker at the power station, but to match the existing facilities, a power line carrier relaying system will be adopted here.

However, thorough consideration should be given to the transfer trip system at the stage of a Definite Study.

The main transformer, in consideration of the results of load forecast and the capacity of the existing bank, is to be 30 MVA with on-load tap chager and the tap range is to be $\pm$ 13.2 kV to match that of the existing bank. The transformer, in consideration of economy, is to be phase and the circuit breaker on the primary side is to be an oil circuit breaker equipped with bushing current transformer in order to achieve the same equipment arrangement as the adjacent bank. It should be noted that the #925 circuit breaker is not budgeted for this Project.

The system of protection for the main transformer is to be the same as the existing equipment.

In consideration of the fact that Popayan Substation will become an extremely important substation in the electric power system in the present design, a bus protective relay system will be provided.

The single line diagram is shown in Fig. 8-5, and the layout is indicated in Fig. 8-6.

(3) Telecommunication Facilities

As telecommunications facilities, it is planned for the following to be installed for communication for security purposes between Julumito Power Station and Popayan Substation:

(a) load dispatching telecommunications facilities,

(b) telecommunications facilities for carrier relaying system.

Regarding telemetering, as a result of discussions, it was decided this would be unnecessary. It is planned for all load dispatching instruction to be given to Julumito Power Station from Popayan Substation.

As for telecommunications facilities for transmission line maintenance, it was decided the existing mobile VHF station would suffice and these are not included in the present plan.

(a) Load Dispatching Telecommunications Facilities

A single channel, power line carrier equipment is to be provided between Julumito Power Station and Popayan Substation to form a load-dispatching telephone channel, for exclusive use.

(b) Carrier Facilities for Transmission Line Protective System

For protection of the 115 kV transmission line, a power line carrier relaying system is to be adopted and carrier apparatus for single channel carrier relaying will be installed at Julumito Power Station and Popayan Substation.

Fig. 8-3 Switchyard of Julumito Power Station

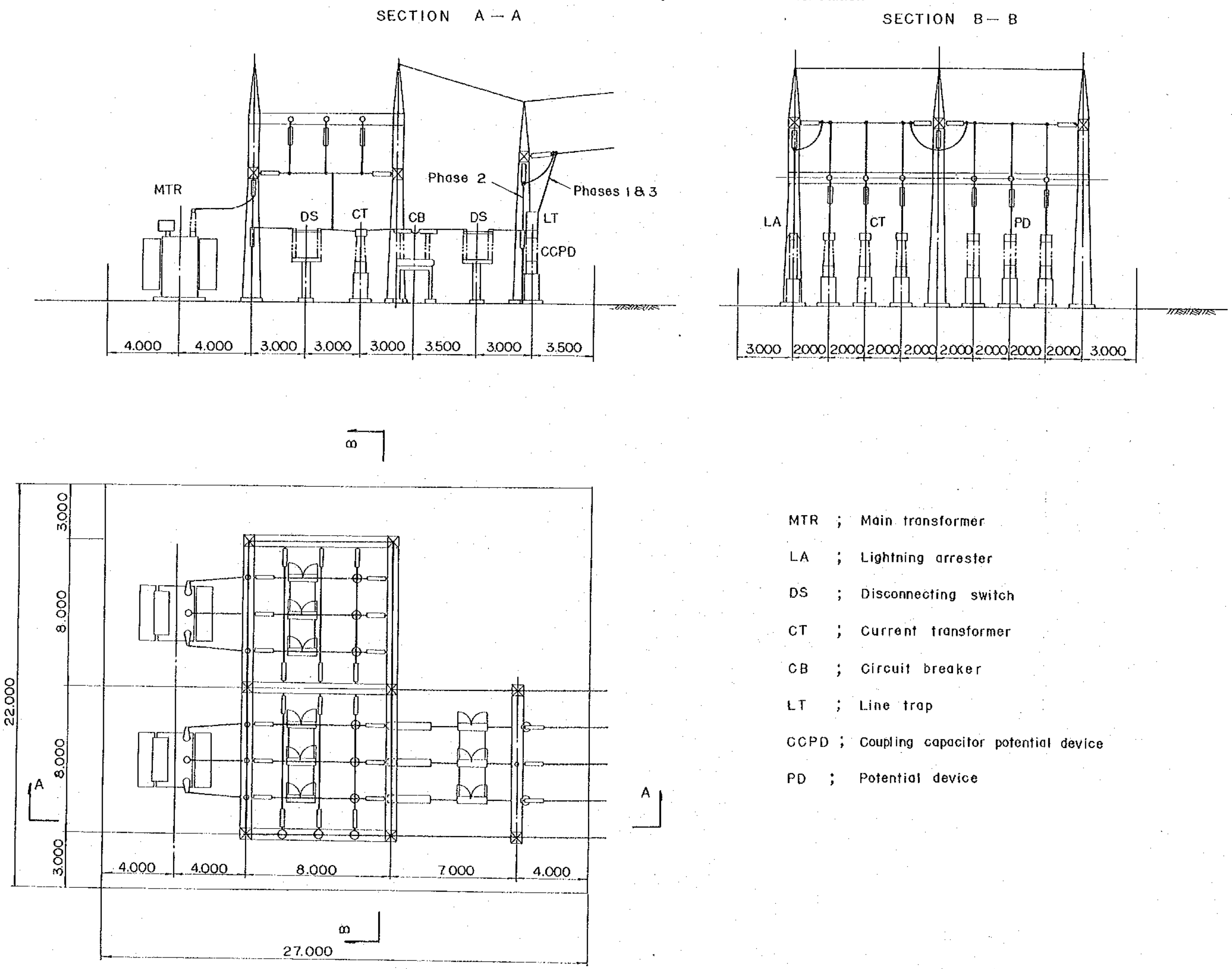


Fig. 8-4 Transmission Line Tower Configration

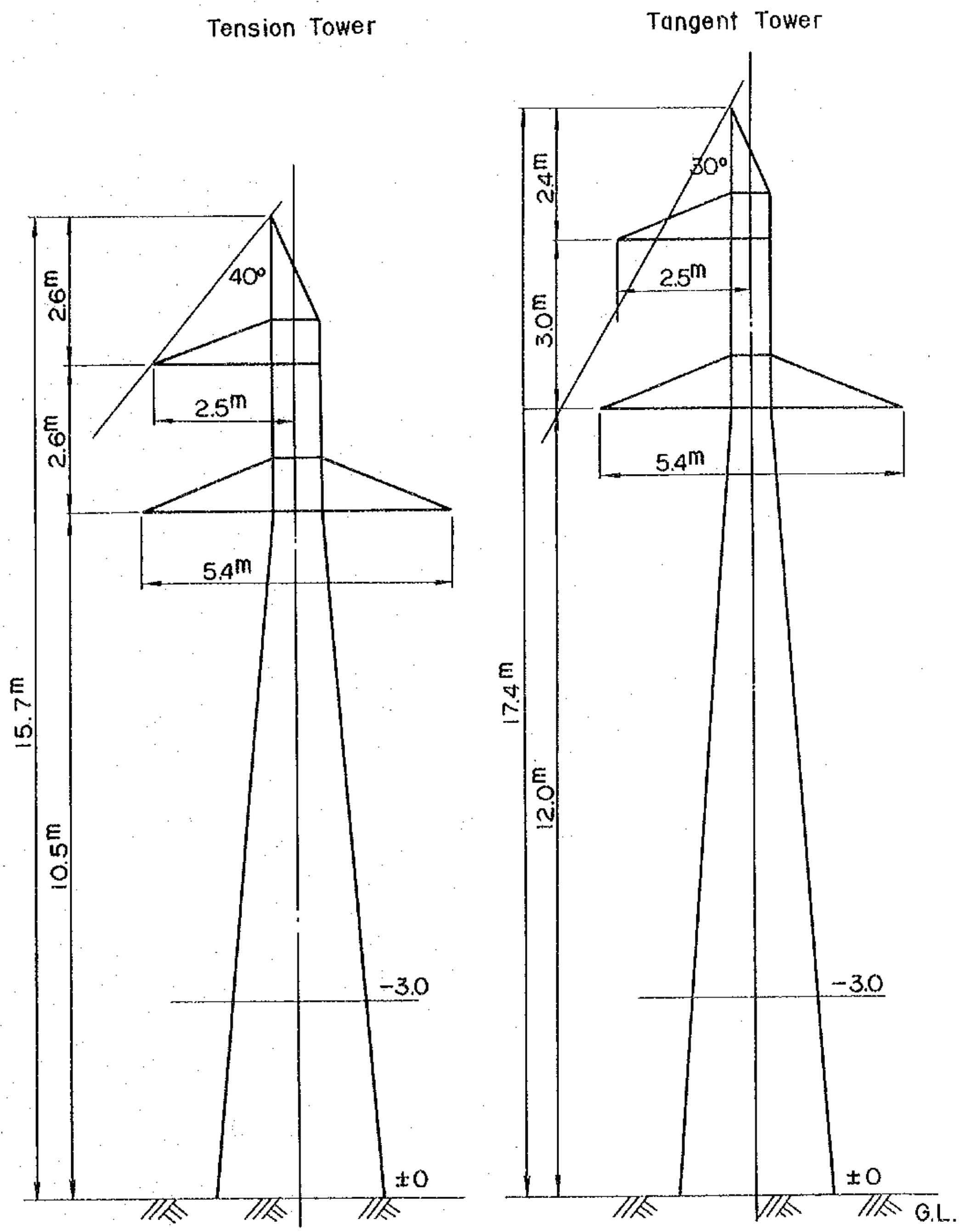


Fig. 8-5 Single Line Diagram of Popayan Substation

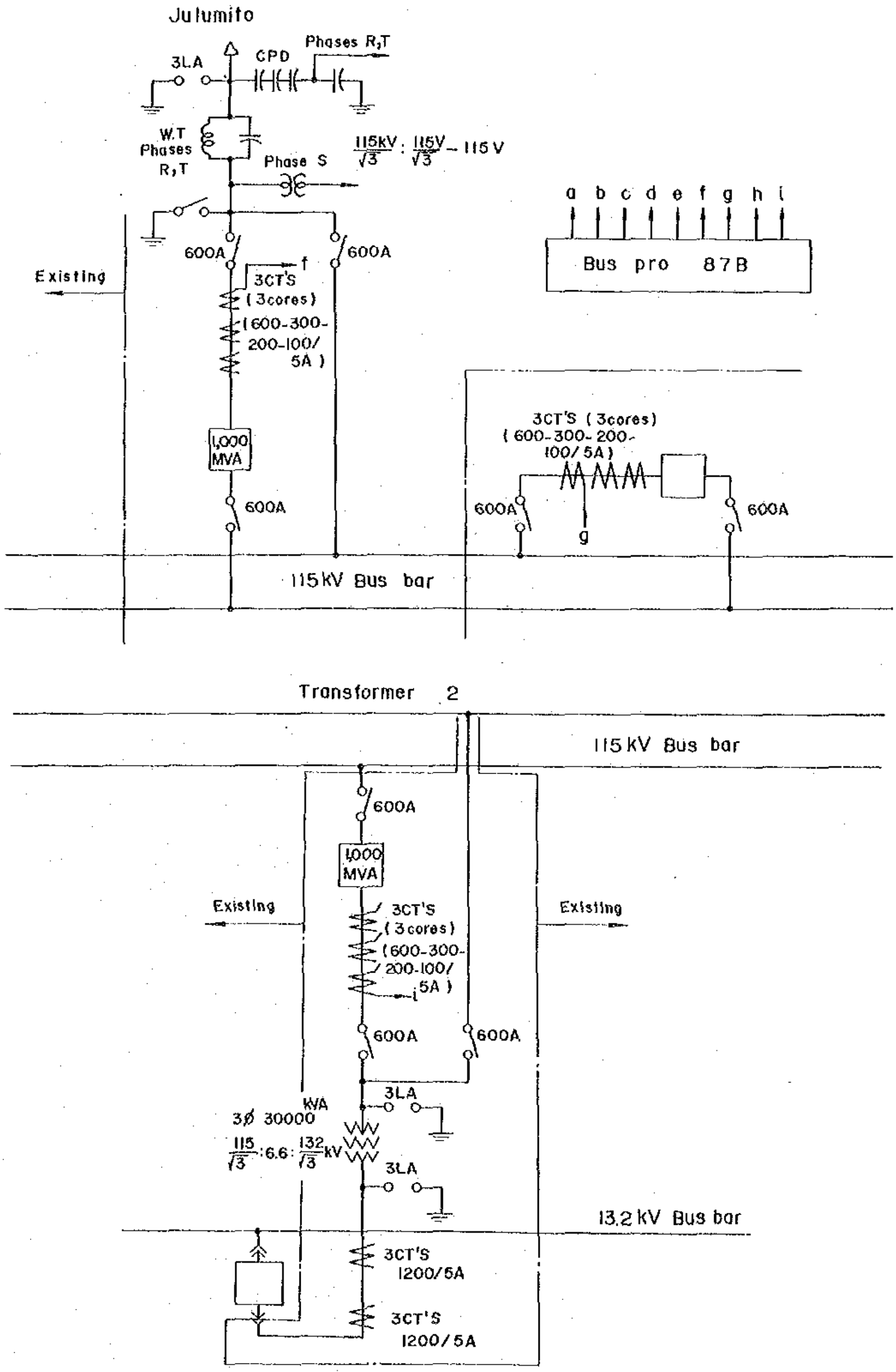
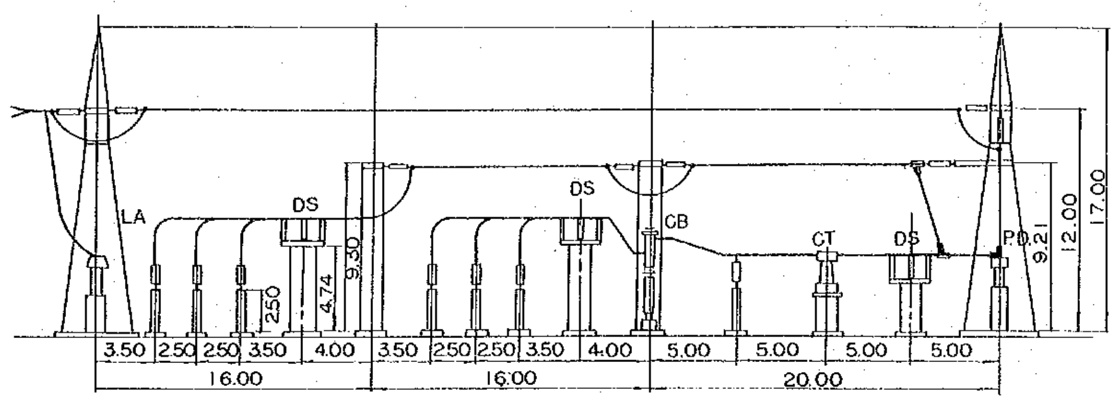
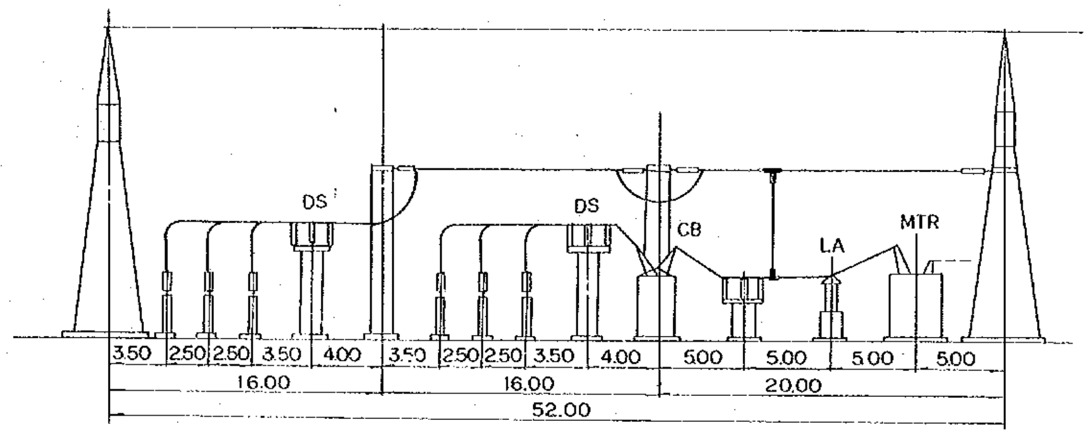


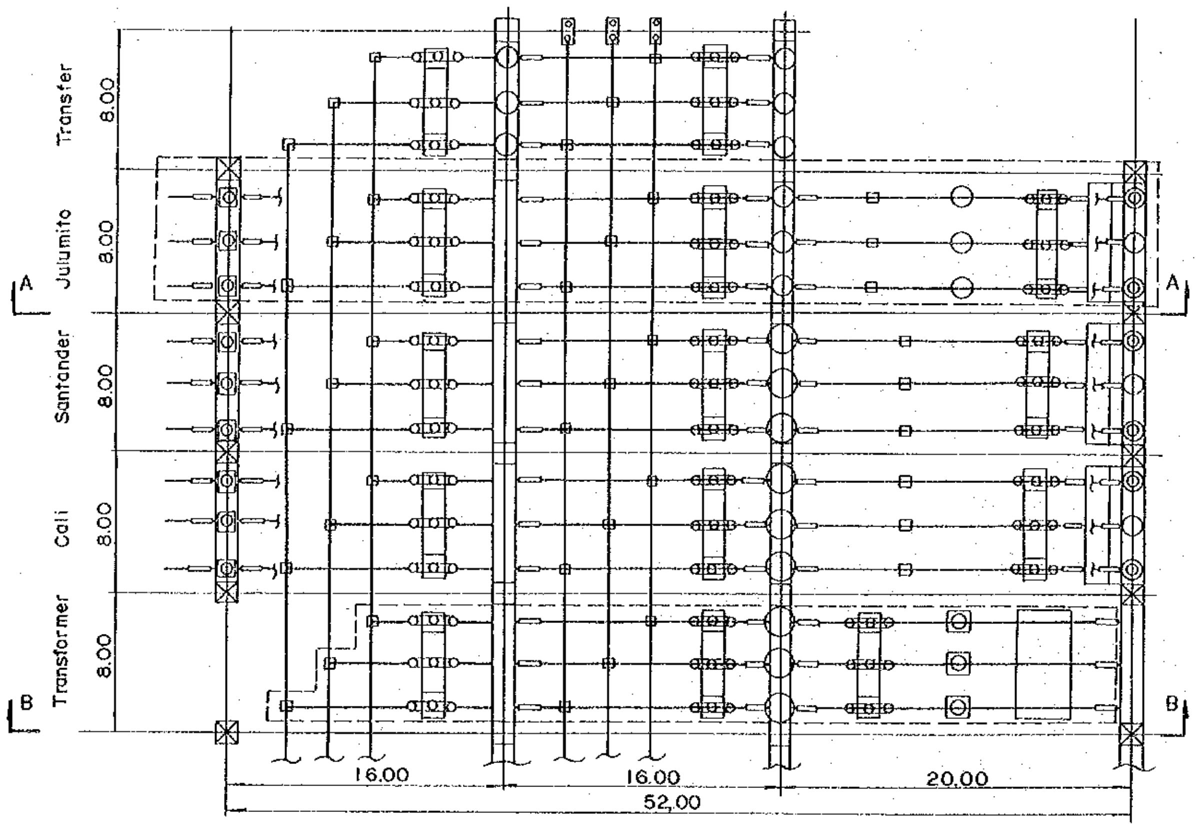
Fig. 8-6 Popayan Substation

SECTION A – A



SECTION B — B





Remerk

[dashed box] Addition

Legend

MTR ; Main transformer

LA ; Lightning arrester

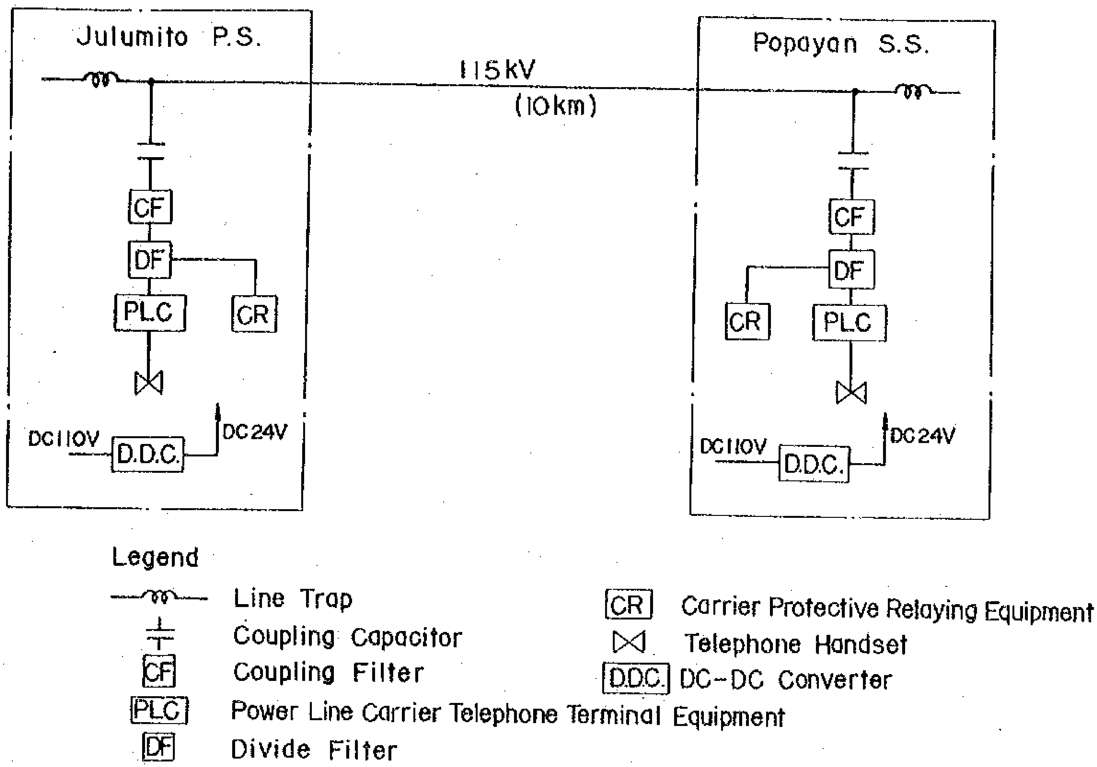
DS ; Disconnecting switch

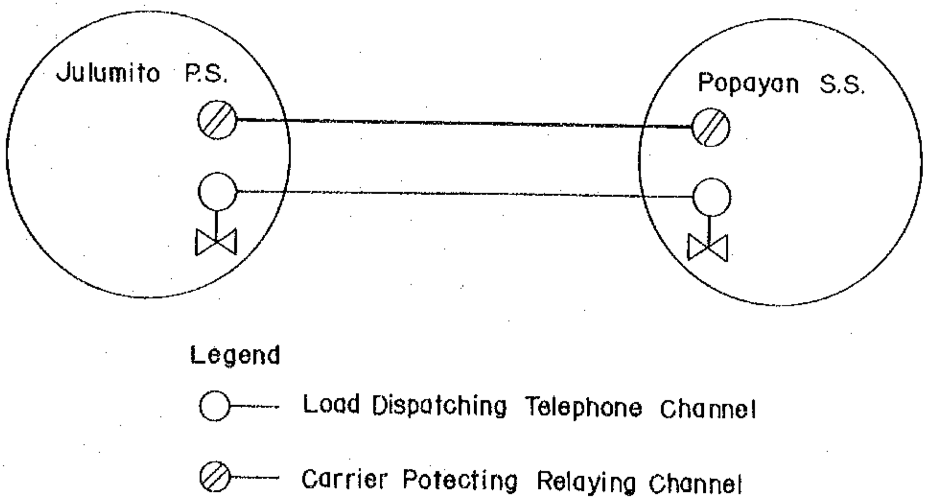
CT ; Current transformer

CB ; Circuit breaker

PD ; Potential device

Fig. 8-7 Telecommunication System Diagram





The telecommunications system diagram for the Julumito Project is indicated in Fig. 8-7.

8.1.4 System Analysis

The results of analyses by digital computer of power flows, voltages, stabilities, short-circuit capacities and short-circuit currents of the CEDELCA and CENENAR power systems before and after start of operation of Julumito Power Station are attached as Appendix II, the outline of which is given below.

(1) For the purpose of supplying electric power of good quality, the transformers of the various substations are to be equipped with ± 10% LRT.

(2) In order to maintain necessary and stable voltages it is necessary for static condensers to be installed at Pasto Substation and Ipiales Substation from 1976.

(3) For alleviating losses, saving phase modifying equipment and improving supply dependability, it is recommended that the line between Popayan Substation and Pasto Substation be made double circuit from 1980.

(4) In case a three wire grounding fault should occur at a place very close to Popayan Substation, the existing small hydroelectric power stations will tripped therefore, it will be necessary to provide a stepout relay at each of these power stations.

(5) The system short-circuiting capacity will be a maximum of 540 MVA at Popayan Substation and the short-circuit current 2,680 A.

8.1.5 Major Specifications

The major specifications of the various structures in the Julumito Project are as indicated below.

Item	Description	
CIVIL STRUCTURE		
Dam	Inclined core type, Rockfill dam	
	Elevation of crest	1,717.0 m
	Height	80.0 m
	Length of crest	350.0 m
	Width of crest	8.0 m

Item	Description	
	Slope of upstream face	1 : 2.3
	Slope of downstream face	1 : 1.8
	Volume of dam	1,050,000 m^3
Draw-off	Tunnel type	
	Length	352.0 m
	Type of valve	Hollow Jet Valve
Dike No. 1	Earthfill	
	Height	22.5 m
	Length of crest	190.0 m
	Slope of upstream face	1 : 2.5
	Slope of downstream face	1 : 2.0
	Volume of dike No. 1	65.000 m^3
Spillway	Chute spillway with control gate	
	Design flood discharge	50.0 m^3/sec
	Type of gate	Slide gate
	Number of gate	1
	Dimension of gate	8.0 x 3.0 m
Dike No. 2	Earthfill	
	Height	3.0 m
	Length of crest	560.0 m
	Volume of dike No. 2	15,000 m^3
Intake	Inclined type, reinforced concrete structure	
	Maximum discharge	50.0 m^3/sec
	Type of gate	Roller gate
	Dimension of gate	5.5 m x 6.5 m
	Screen	6.0 m x 31.0 m
Headrace	Pressure tunnel	
	Length	1,793.0 m
	Shape	Circular
	Inside diameter	4.2 m

Item	Description	
Surge tank	Orifice type	
	Inside diameter of tank	8.0 m
	Inside diameter of orifice	2.2 m
	Height	65.6 m
Penstock	Welded steel, ring girder type	
	Materials	SM 58 or SM 53 (JIS)
	Length	239.0 m
	Number of Line	1
	(bifuracate into 2 lines at No. 3 anchor blocks)	
	Inside diameter	4.0 x 3.2 m
	(2.0 ~ 1.6 m after a bifuracation)	
Powerhouse	Reinforced concrete structure	
	Length	27.0 m
	Width	20.1 m
	Height	29.5 m
Rio Cauca diversion dam	Free overflow type concrete gravity dam	
	Height of dam	11.0 m
	Length	71.0 m
	Sand flash gate	Slide gate 60 x 3.0 m
	Volume of concrete	7.200 m^3
Rio Palacé diversion dam	Free overflow type concrete gravity dam	
	Height of dam	7.7 m
	Length	30.4 m
	Sand flash gate	Slide gate 4.0 x 3.0 m
	Volume of concrete	1,400 m^3
Rio Blanco diversion dam	Free overflow type concrete gravity dam	
	Height of dam	7.5 m
	Length	19.4 m
	Volume of concrete	900 m^3

Item	Description	
Cauca Diversion Waterway	Open channel and tunnel	
	Capacity	40.0 m^3/sec
	Length of open channel	2,400.0 m
	Length of tunnel	220.0 m
Palacé Diversion Waterway	Open channel and tunnel	
	No. 1 Waterway	
	Capacity	12.0 m^3/sec
	Length of open channel	4,000.0 m
	Length of tunnel	—
	No. 2 Waterway	
	Capacity	13.8 m^3/sec
	Length of open channel	3,600.0 m
	Length of tunnel	770.0 m
ELECTRIC EQUIPMENT		
Turbine	Vertical shaft Francis type	
	Output	27,500 kW
	Maximum discharge	25.0 m^3/sec
	Revolution	400 rpm
	Number of units	2
Generator	Three-phase synchronous generator, vertical shaft, rotating field closed type	
	Capacity	29,500 kVA
	Voltage	11.0 kV
	Frequency	60 cycles
	Number of units	2
Transformer	Three-phase outdoor, oil-immersed self-cooled type	
	Capacity	29,500 kVA
	Voltage	10.5/115 kV
	Number of units	2

Item	Description	
Outdoor Switchyard	Transmission voltage	115 kV
	Area	27.0 x 22.0 m
Transmission Line		
	Distance	10 km
	Voltage	115 kV
	Number of circuits	1 cc
	Conductor	160 mm^2 ACSR
	Insulator	250 mm Suspension Insulator Ball and Socket
	Overhead ground wire	45 mm^2 GSC
	Support	Steel tower
ADDITIONAL TRANS-FORMER IN POPAYAN SUBSTATION		
	Main transformer	1 unit
	Capacity	30 MVA
	Voltage	115 kV/13.2 kV with on load tap changer
	Circuit breaker	115 kV, 1,000 MVA
TELECOMMUNICATION SYSTEM		
	For communication system	1 ch.
	For Carrier relaying system	1 ch.

8.2 Construction Schedule and Construction Scheme

8.2.1 Construction Schedule

The construction period for the Julumito Project, as a result of study on the scale of the Project, arrangement of structures, the construction capabilities expected of contractors and regional conditions, requires about 32 months.

From the standpoint of power demand and supply it is necessary for Julumito Power Station to go into operation in December 1981. Therefore, tracing back the period of water impoundment prior to start of operation, time required for trial operation and the abovementioned period required for construction, work must be started in January 1979 at the latest.

Various preparatory works required before start of construction must all be completed by the end of 1978.

The above construction schedule is indicated in Fig. 8-8.

The preparatory works to be completed in 1978 consists of newly constructing access roads or the improvement of existing roads to the dam site, powerhouse vicinity and borrow areas and installation of various facilities such as electric power equipment for construction.

In the first year of start of construction (1979), after completion of the diversion tunnel for the main dam, excavation of the dam foundation and embankment of the main dam are to be started, while excavation would also be carried out for the Rio Cauca diversion dam, Palacé Diversion Waterway, headrace tunnel, surge tank, penstock and powerhouse foundation.

Orders would be issued for the main equipment of the power station and for penstock.

In the second year (1980), embankment of the main dam, construction of the foundation and the building of the powerhouse, concrete lining of the headrace tunnel, concreting of the surge tank and penstock, and construction of the Cauca and Palacé diversion waterways will be mainly carried out, while the Rio Palacé, Blanco Diversion Dams, Cauca Diversion Waterway and embankment of Dikes No. 1 and No. 2 will be started.

Inside the powerhouse, installation of turbines will be completed.

In the third year (1981), embankment of the main dam, concrete placement work for the various structures, installation of the penstock, generators, transformers, etc., and further, all other works necessary for start of operation of the power station such as the transmission line, telecommunications facilities, etc. will be completed. During October November, water passage tests are to be conducted after completing water storage into the reservoir and in the middle of December commercial operation of the power station is to be started.

In order for the above works to be carried out smoothly, the various preparatory works required for construction and other advance preparations must have progressed satisfactorily in accordance with the schedule indicated in Fig. 8-8.

8.2.2 Construction Scheme

(1) Regional Conditions and Related Matters

(a) Transport Route

The Julumito Project site is situated at a point approximately 10 km northwest of Popayan City and is favored with good conditions of site from the standpoint of construction.

The distance from Cali City which would be the main source of construction materials is approximately 150 km over which there is a good concrete-paved road. Also, the port of Buenaventura on the Pacific Ocean coast west of Cali would serve as the landing port for imported materials and equipment and possesses equipment capable of landing heavy articles while the 140-km road to Cali is amply maintained in good condition.

Over the approximately 10 km from the center of Popayan City to the dam site, there is an existing road which would be adequate so long as it is partially improved at a section inside Julumito Village. Between the main dam site and the powerhouse, a distance of approximately 3 km, there exists a survey road and as this would be used for hauling rock materials for embankment of the dam, and for hauling heavy equipment such as turbines, generators, and transformers, improvements such as widening of the road will be necessary.

Besides the above, construction of access roads will be necessary, such as the approximately 500-m distance between the borrow area for dam embankment and the dam site, to the vicinities of the Rio Cauca and Rio Palacé diversion dams and to the Cauca and Palacé diversion waterway sites, but these can all be constructed quite easily and will pose no problems.

(b) Electric Power for Construction

The electric power required for construction of Julumito Power Station is estimated to be a maximum of approximately 3,000 kW. This electric power for construction is to be supplied by newly constructing a temporary transmission line (34.5 kV) of approximately 10 km from Popayan Substation of CEDELCA to a temporary project substation (34.5 kV/6.6 kV) to be provided in the vicinity of the dam site. It is planned for power to be supplied from this substation to the various construction sites by means of 6.6 kV distribution lines.

(c) Water Supply Facilities

The source of water for construction and drinking is to be water stored upstream of the coffer dam of the main dam site on the Rio Sate. This water will be pumped up for supply to the various adits of the headrace tunnel, the powerhouse, the aggregate plant and other facilities.

As for the diversion dam and diversion waterway sites, the river water upstream of the various diversion dam sites will be taken in for water supply.

(2) Procurement of Construction Materials

The chief materials to be used in construction are estimated to be 24,400 tons of cement, 1,500 tons of reinforcing steel, 1,000 tons of other steel, 300 tons of explosives, and approximately 3,000 kl of petroleum products such as kerosene, gasoline, heavy oil, etc. As most of these materials are produced in Colombia, domestic products will be used, but steel pipe, gates, outdoor steel structures, steel forms, supports, rods, bits, etc. will be imported.

Concrete aggregates are planned to be artificially manufactured by crashing rocks from quarries near the powerhouse and excavated rock from the penstock route and the powerhouse, but there are also plans to supplementarily use river deposits of the Rio Timbio and Rio Ondo.

(3) Construction of Major Structures

(a) Construction of Main Dam

A diversion tunnel is first to be provided at the left bank of the Rio Sate to divert the river flow.

Excavation of the dam foundation is to be carried out by first removing overburden and then excavating downward from parts of higher elevation with the river bed the last to be excavated. Curtain grouting is to be carried out on the bedrock serving as the foundation for the impervious soil core in order to prevent permeation of water.

Consolidation grouting will be performed where required to improve bearing power of the foundation or to improve the ground.

The volume of embankment for the dam will be 120,000 m^3 of impervious soil materials for the center, 80, 000 m^3 of filter materials on each side of the core, and further outside on both sides, 850, 000 m^3 of rock materials or a total of 1, 050, 000 m^3.

As stated before, rock materials and filter materials are planned to be collected from both banks of the Rio Cauca near the powerhouse site and hauled to the dam site. Impervious soil materials are to be borrowed from a site approximately 500 m downstream from the dam site on the Rio Sate, hauled to the dam site and used for embankment. The period of embankment of the dam is planned to be approximately 23 months taking into consideration the volume of embankment, the heavy equipment to be employed, hauling distances, weather, etc.

The heavy equipment to be used for embankment would be shovels with dipper capacities of 1.2 to 2.0 m^3 class, dump trucks of 10 to 15 ton class, buldozers of about 20 tons and motor scrapers of 10 m^3 class.

The transport road from the rock quarry areas to the dam site is to have adequate width for two ways and gentle gradient to enable trucks to pass at high speeds.

Lift heights and compaction methods of embankment should be determined from trial embankments to produce good results.

For 3 months after plugging of the diversion tunnel, in May 1981 the natural flow of only the Rio Sate will be stored in the reservoir and the work will be carried out to convert the diversion channel into a draw-off tunnel during this period.

From August 1981, both diversion waterways will be employed for storage of water.

(b) Construction of Headrace Tunnel

Construction of the approximately 1, 800 m long headrace tunnel will be carried out from both the intake end and the surge tank end. Work adits will be provided at the sides of the two portals and approximately one-half of the length or 900 m each will be constructed from each portal.

Excavation of the tunnel will be entirely mechanized. It is anticipated that full-face excavation can be adopted over approximately the entire length.

After the tunnel excavation is completed, the concrete lining will be placed. Concrete will be placed for the entire cross section at once using travelling steel forms.

Following the concrete lining, mortar grouting and high-pressure grouting will be performed in sequence.

(c) Construction of Other Structures

Excavation of the vertical shaft portion of the surge tank will be carried out by first driving a pilot vertical shaft of cross section of 6 8 m^2 after which enlargement to the specified cross section is to be made in sequence from the top, and upon completion of excavation, concrete will be lined.

Regarding the penstock, after collection of rock materials for embankment of the dam, excavation for the foundation of the penstock will be carried out and concrete for the penstock line and anchor blocks will be placed.

For the powerhouse, after diversion of the river by providing a diversion tunnel at the opposite bank (left bank of the Rio Cauca), excavation of the foundation and placement of concrete will be carried out and the outer finish of the building will be completed and the crane for setting heavy equipment will be provided by the time for installation of the turbines.

The Rio Cauca Diversion Dam will be constructed employing the river bed release method. First, a diversion channel will be provided at the left half of the dam site and the upstream and downstream sides of the right half will be coffered to divert the river flow to the diversion channel at the left half. Following this, excavation of the dam foundation and excavation of the intake will be performed, base concrete will be placed after which curtain grouting is to be carefully carried out to prevent water permeation.

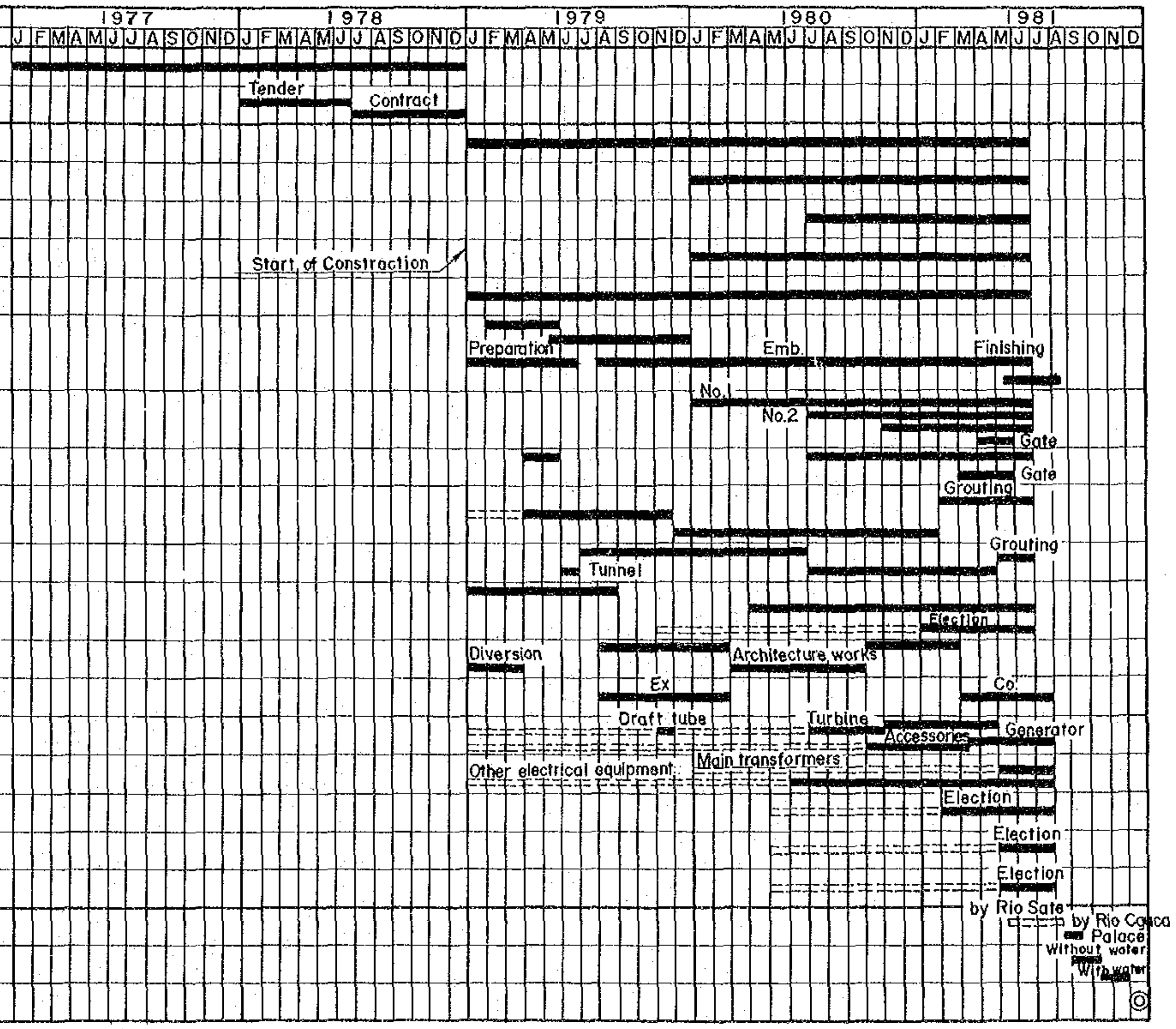
After completion of the right half of the dam, the upstream and downstream sides of the left half will be coffered and the river diverted to the right half, followed similarly by excavation, curtain grouting and concrete placement.

The Rio Palacé and Rio Blanco diversion dams will be constructed in the same manner.

There are no special comments to be made on construction of the Cauca and Palacé diversion waterways and the No. 1 and No. 2 dikes.

Fig. 8-8 Construction Schedule

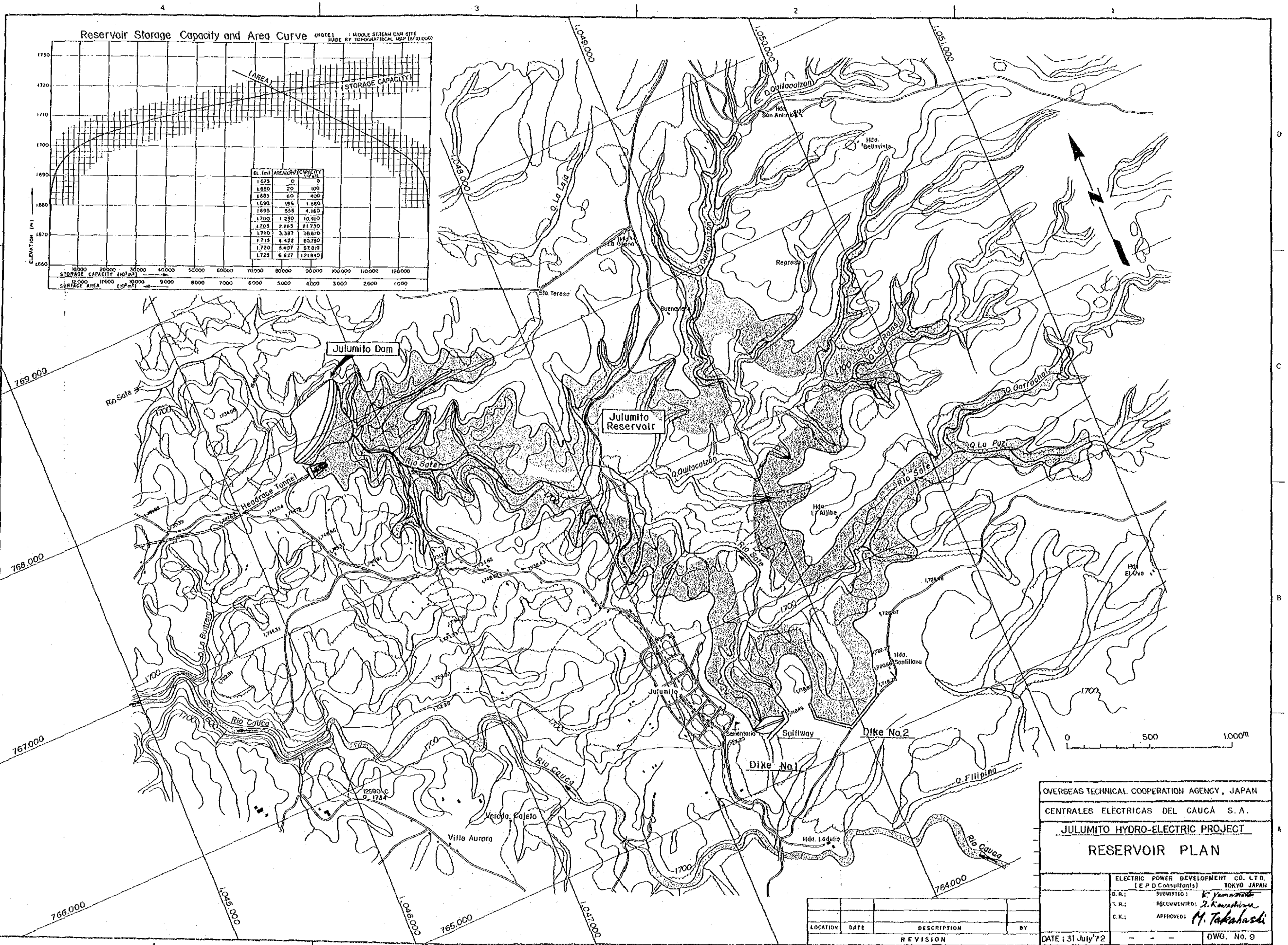
Item			Year / Month
Preparation Works			
Tender and Contract			
Civil Works	Diversion Dam	Rio Cauca	Ex.=20.800 m³ Co.= 7.160 m³
		Rio Palacé	Ex.= 6.100 m³ Co.= 1.410 m³
		Rio Blanco	Ex.= 5.300 m³ Co.= 870 m³
	Diversion Waterway	Cauca	L.= 2.62 km
		Palacé No.1, No.2	L.= 8.43 km
	Dam		Diversion Tunnel Ex.=125,110 m³ Emb.=1,050,000 m³ Draw. off Tunnel
	Dike No.1, No. 2		Emb.=65,000 m³ Emb.=15,000 m³ Spillway
	Intake		Ex.= 21,170 m³ Co.= 4,790 m³
	Headrace Tunnel		L.= 1,793.0 m Ex.=35,060 m³ Co.=10,590 m³
	Surge Tank		Ex.= 6,640 m³ Co.= 2,860 m³
	Penstock		Ex.=8,470 m³ Co.=3,800 m³ Penstock
	Powerhouse		Ex.=21,380 m³ Co.= 5,600 m³
	Foundation of Switchyard		
Electrical Equipment	Turbines and Generators		
	Main Transformers and Other Equipment		
Transmission Line and Others	Transmission Line		L.= 10 km
	Substation		
	Telecommunication System		
Water Storage			
Test			
Start of Operation			



LEGEND

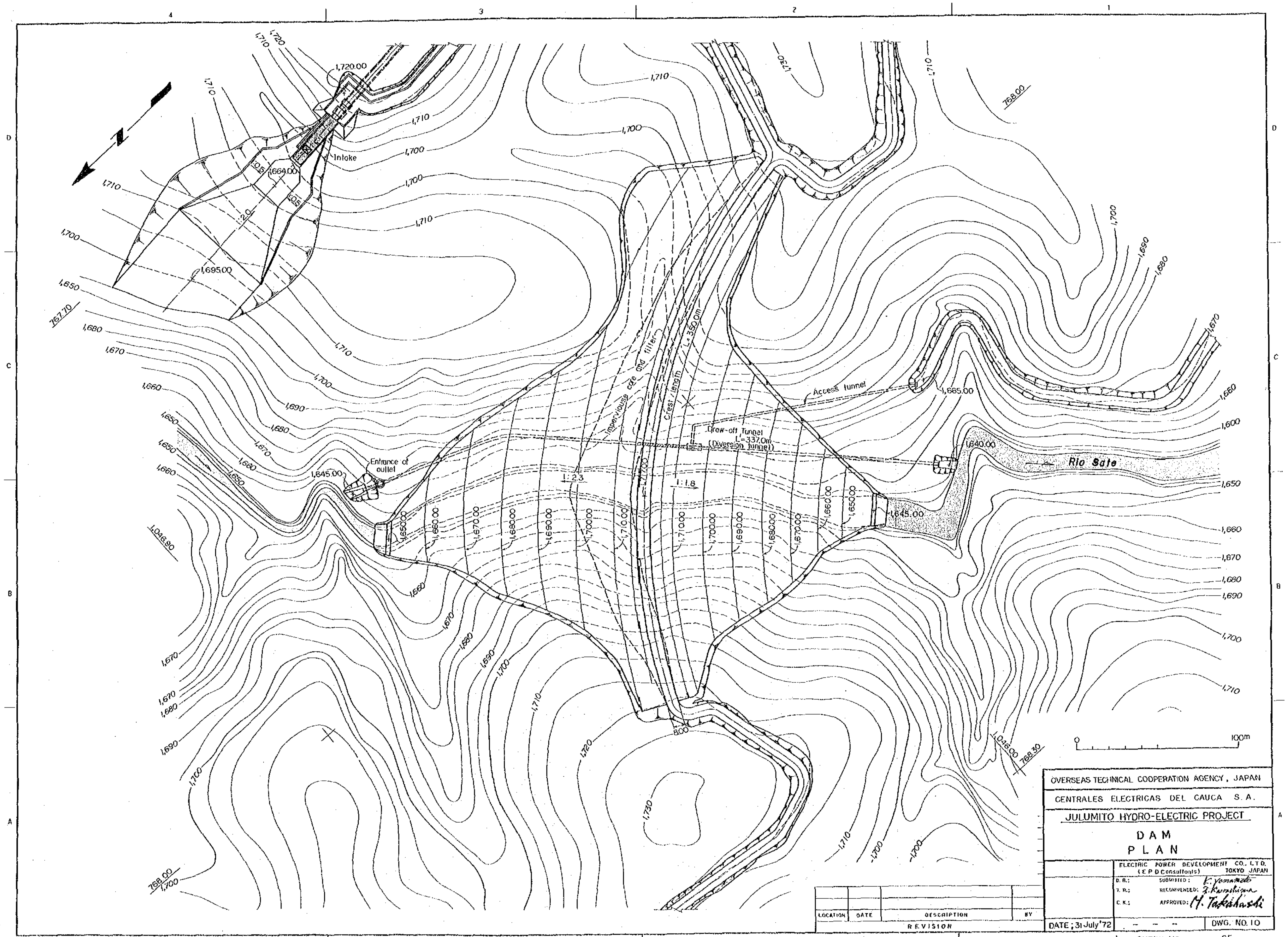
Manufacturing and Transportation

Field Works



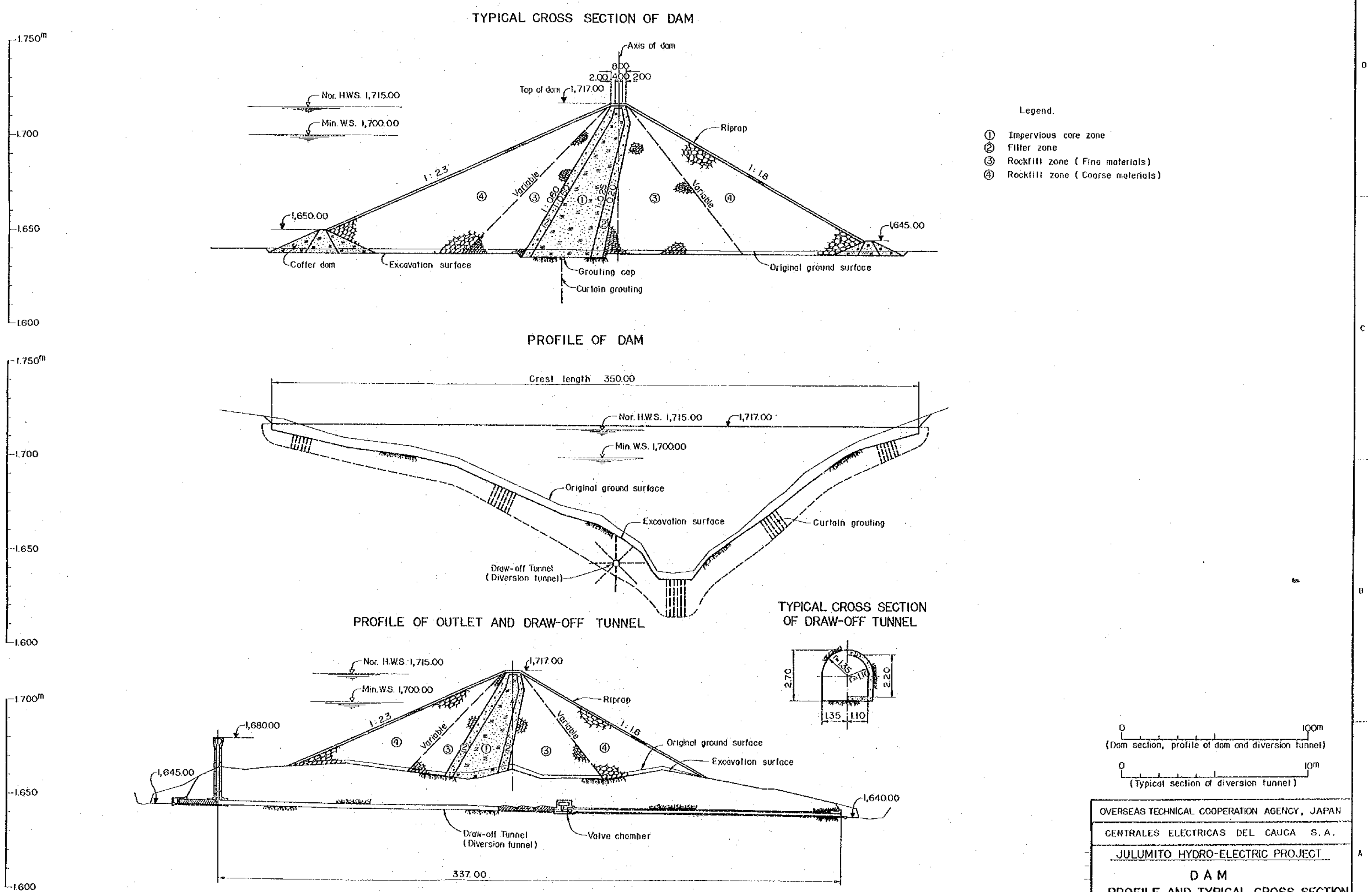
EL. (m)	AREA (10^3m^2)	CAPACITY (10^3m^3)
1.675	0	0
1.680	20	100
1.685	60	400
1.690	196	1.380
1.695	556	4.160
1.700	1.250	10.410
1.705	2.265	21.730
1.710	3.387	38.670
1.715	4.422	60.780
1.720	5.407	87.810
1.725	6.827	121.940

SHEET NO. OF



Intake
Access tunnel
Draw-off Tunnel L=337.0m (Diversion tunnel)
Crest length L=350.0m
Impervious core and filter
Entrance of outlet
Rio Sate
1:2.3
1:1.8
100m
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
DAM
PLAN
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE ; 31 July '72
DWG. NO. 10

SHEET NO. OF



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S. A.

JULUMITO HYDRO-ELECTRIC PROJECT

DAM

PROFILE AND TYPICAL CROSS SECTION

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

D. R.: SUBMITTED: K. Yamamoto

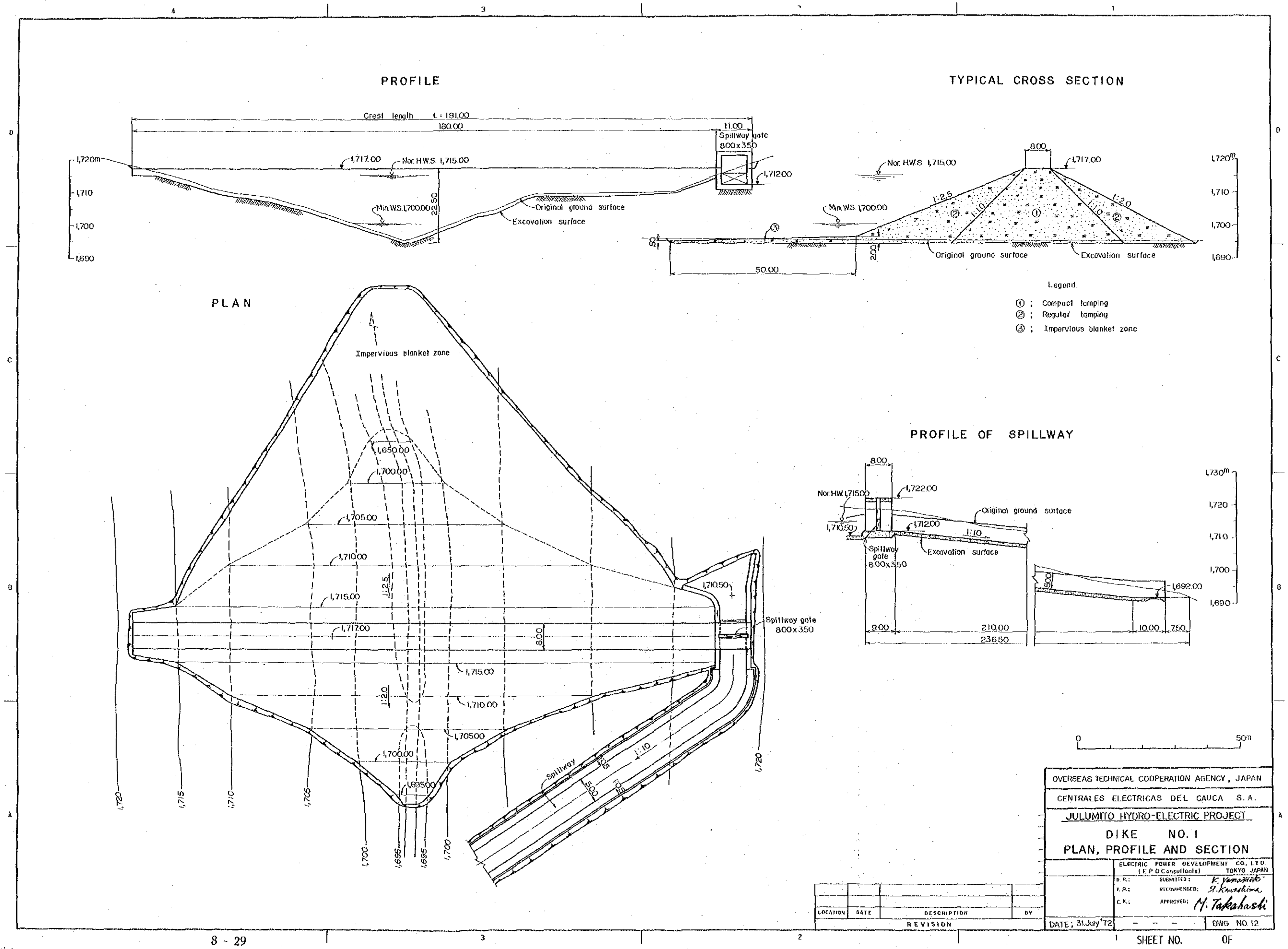
T. R.: RECOMMENDED: T. Kawashima

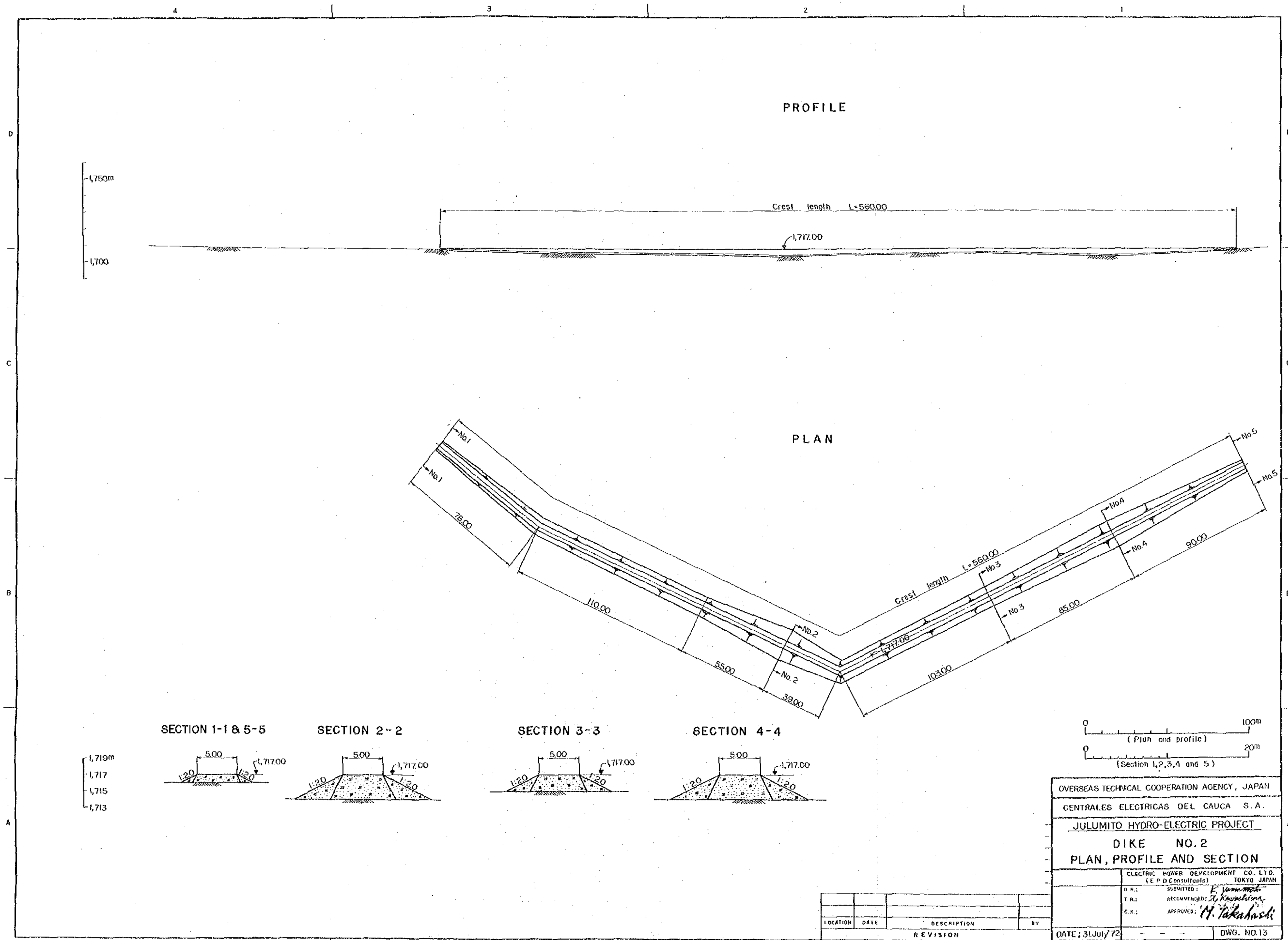
C. K.: APPROVED: M. Takahashi

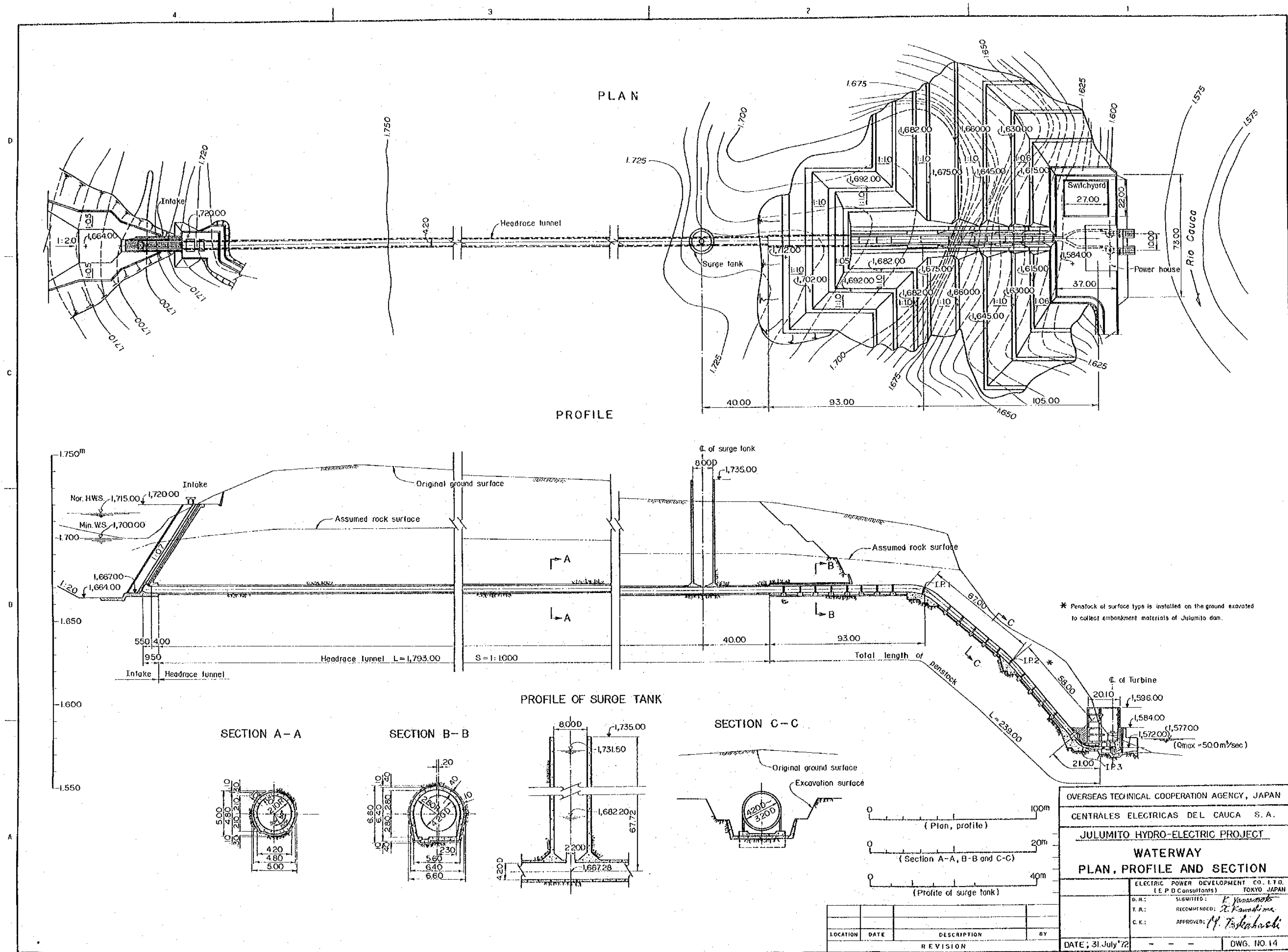
LOCATION	DATE	DESCRIPTION	BY
REVISION			

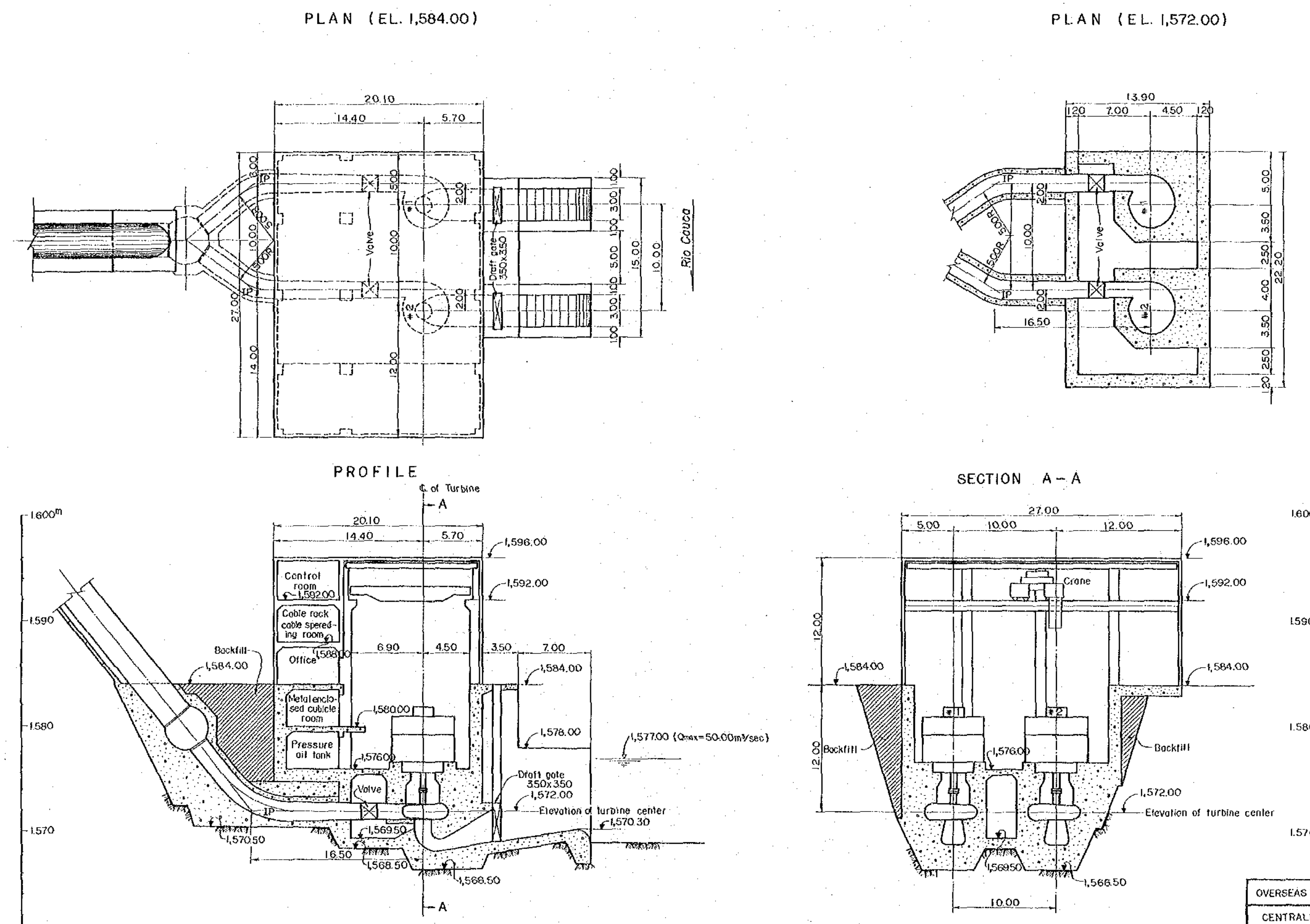
DATE: 31 July '72 — — — DWG NO. 11

SHEET NO. OF


PROFILE
Crest length L = 191.00
180.00
11.00
Spillway gate 800x350
1,720m
1,710
1,700
1,690
1,717.00
Nor. H.W.S. 1,715.00
1,712.00
Min. W.S. 1,700.00
22.50
Original ground surface
Excavation surface
TYPICAL CROSS SECTION
8.00
1,717.00
Nor. H.W.S. 1,715.00
Min. W.S. 1,700.00
1:2.5
1:1.0
1:2.0
1:1.0
50.00
2.00
Original ground surface
Excavation surface
Legend.
① ; Compact tamping
② ; Reguler tamping
③ ; Impervious blanket zone
PLAN
Impervious blanket zone
1,650.00
1,700.00
1,705.00
1,710.00
1,715.00
1,717.00
8.00
1:2.5
1:2.0
1,710.50
Spillway gate 800x350
1,715.00
1,710.00
1,705.00
1,700.00
1,695.00
Spillway
1:10
5.00
1,720
1,715
1,710
1,705
1,700
1,695
PROFILE OF SPILLWAY
8.00
1,722.00
Nor. H.W. 1,715.00
1,710.50
1,712.00
1:10
Original ground surface
Excavation surface
Spillway gate 8.00x3.50
1,692.00
1,730m
1,720
1,710
1,700
1,690
9.00
210.00
236.50
10.00
7.50
0
50m
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S. A.
JULUMITO HYDRO-ELECTRIC PROJECT
DIKE NO. 1
PLAN, PROFILE AND SECTION
ELECTRIC POWER DEVELOPMENT CO., LTD. (E.P.D.Consultants) TOKYO JAPAN
SUBMITTED: K. Yamamoto
RECOMMENDED: S. Kuroshima
APPROVED: M. Takahashi
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE; 31.July '72
DWG NO. 12
SHEET NO.
OF


PROFILE
1,750m
1,700
Crest length L=560.00
1,717.00
PLAN
No.1
No.2
No.3
No.4
No.5
76.00
110.00
55.00
39.00
103.00
85.00
90.00
Crest length L=560.00
1,717.00
0
100m
(Plan and profile)
0
20m
(Section 1,2,3,4 and 5)
SECTION 1-1 & 5-5
SECTION 2-2
SECTION 3-3
SECTION 4-4
1,719m
1,717
1,715
1,713
5.00
1,717.00
1:2.0
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
DIKE NO.2
PLAN, PROFILE AND SECTION
ELECTRIC POWER DEVELOPMENT CO., LTD.
(E P D Consultants) TOKYO JAPAN
D.R.:
T.R.:
C.K.:
SUBMITTED:
RECOMMENDED:
APPROVED:
LOCATION
DATE
DESCRIPTION
BY
REVISION
DATE; 31 July '72
DWG. NO.13
SHEET NO.
OF


PLAN
Intake
Headrace tunnel
Surge tank
Switchyard
Power house
Rio Cauca
PROFILE
Original ground surface
Assumed rock surface
Nor. H.W.S. 1,715.00
Min. W.S. 1,700.00
℄ of surge tank
℄ of Turbine
Headrace tunnel L=1,793.00 S=1:1000
Total length of penstock L=239.00
(Qmax =50.0m³/sec)
* Penstock of surface type is installed on the ground excavated to collect embankment materials of Julumito dam.
PROFILE OF SURGE TANK
SECTION A－A
SECTION B－B
SECTION C－C
Original ground surface
Excavation surface
(Plan, profile)
(Section A-A, B-B and C-C)
(Profile of surge tank)
LOCATION
DATE
DESCRIPTION
BY
REVISION
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S. A.
JULUMITO HYDRO-ELECTRIC PROJECT
WATERWAY
PLAN, PROFILE AND SECTION
ELECTRIC POWER DEVELOPMENT CO. LTD. (E.P.D.Consultants) TOKYO JAPAN
DATE; 31.July '72
DWG. NO.14
SHEET NO.
OF



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S.A.

JULUMITO HYDRO-ELECTRIC PROJECT

POWER HOUSE
PLAN AND PROFILE

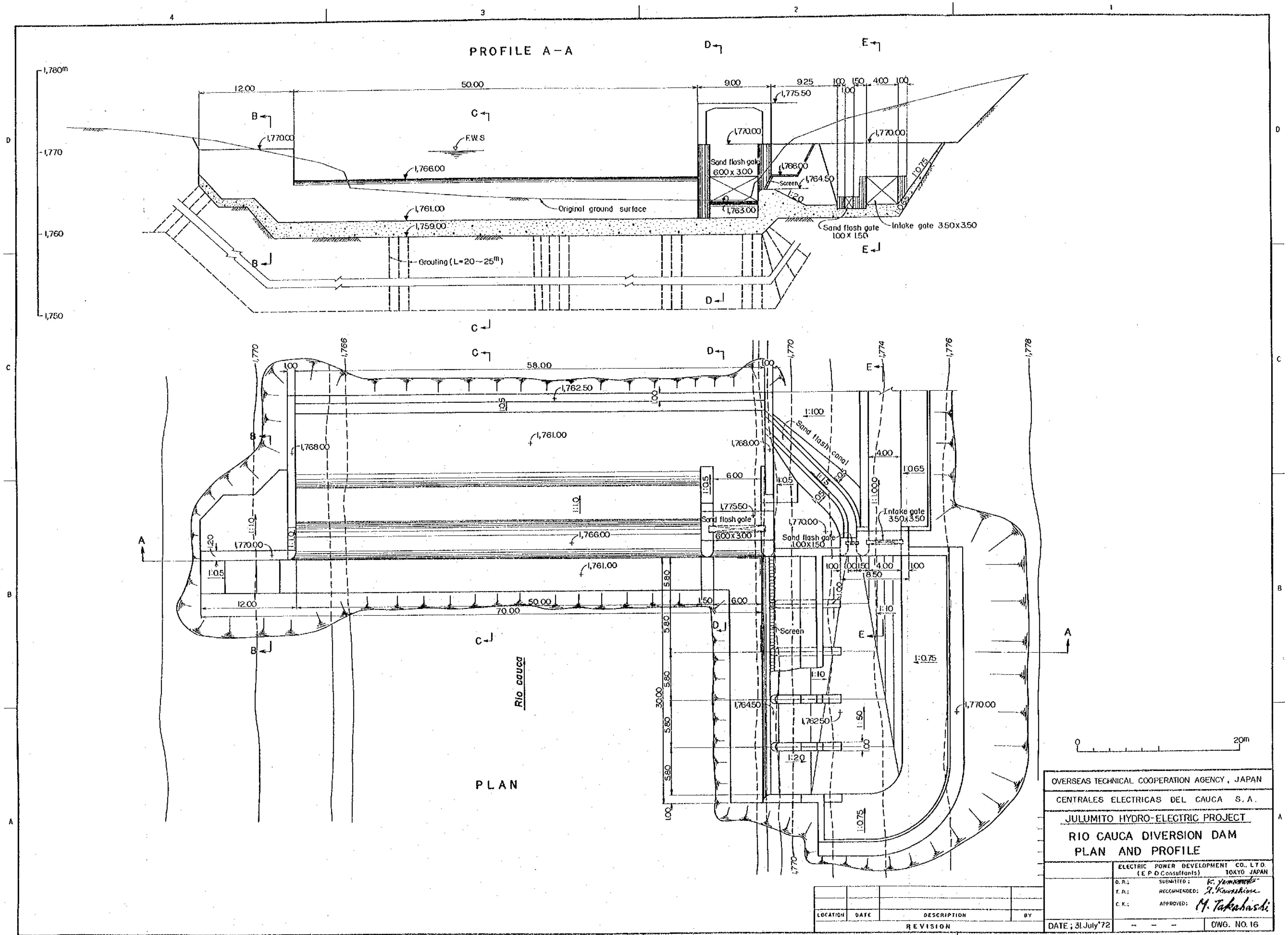
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

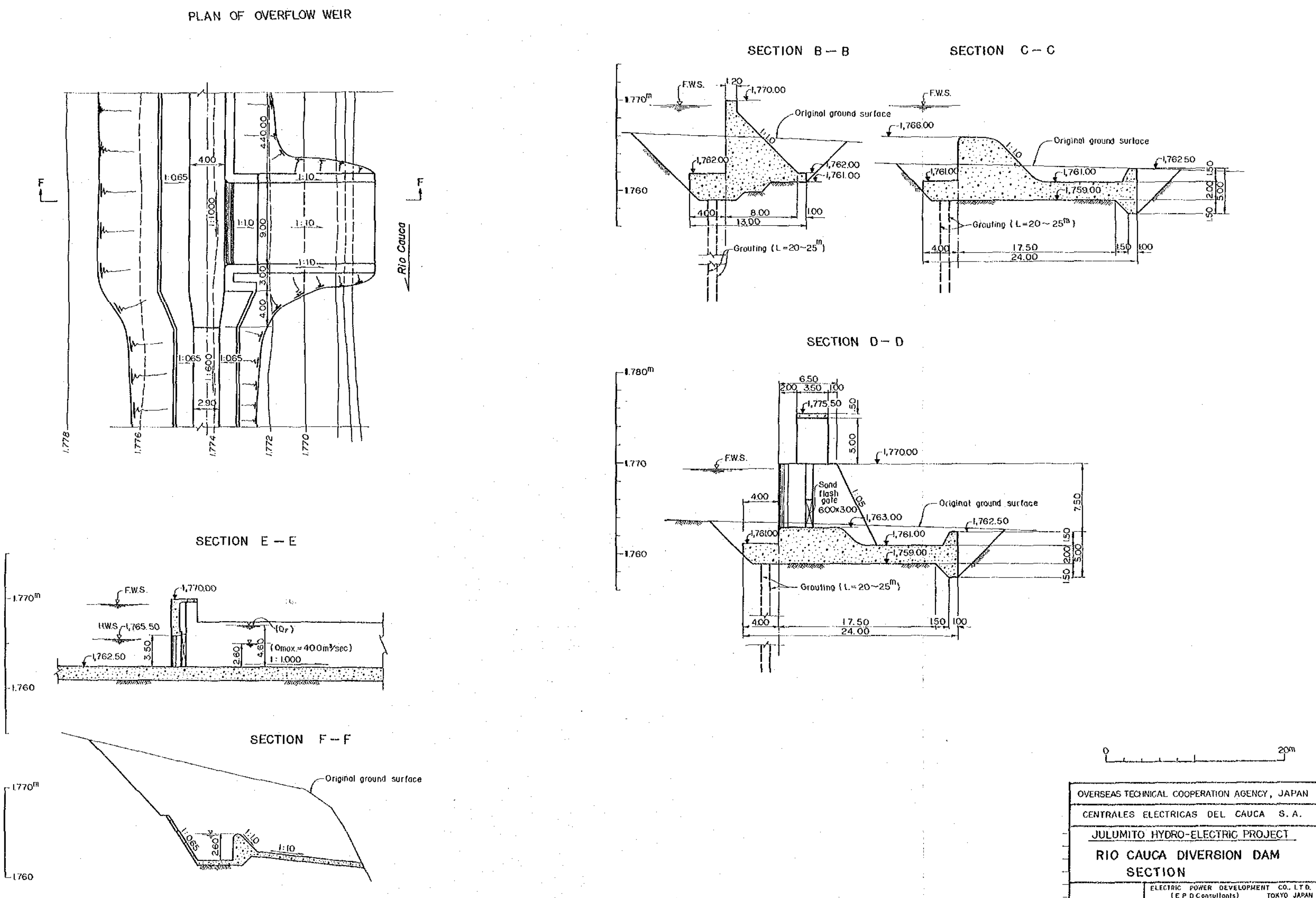
SUBMITTED: K. Yamamoto
RECOMMENDED: T. Kawashima
APPROVED: M. Takahashi

LOCATION	DATE	DESCRIPTION	BY
REVISION			

DATE: 31 July '72 — DWG. NO. 15

SHEET NO. OF

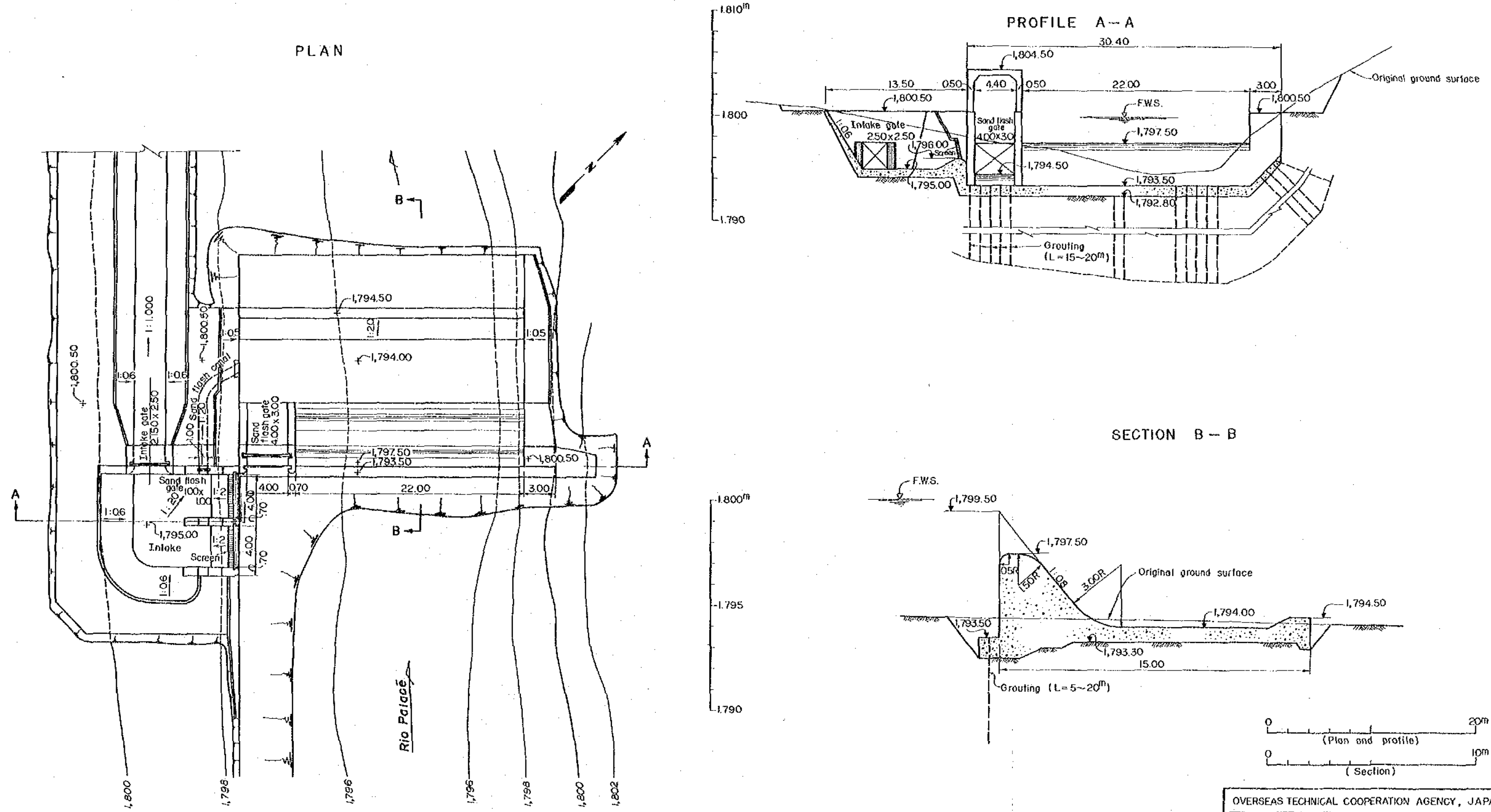

PROFILE A-A
PLAN
Rio cauca
Sand flash gate 6.00 x 3.00
Sand flash gate 1.00 x 1.50
Intake gate 3.50x3.50
Original ground surface
Grouting (L=20~25m)
F.W.S
Screen
Sand flash canal
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
RIO CAUCA DIVERSION DAM
PLAN AND PROFILE
ELECTRIC POWER DEVELOPMENT CO., LTD. (E.P.D.Consultants) TOKYO JAPAN
DATE ; 31 July '72
DWG. NO. 16
REVISION
SHEET NO. OF



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICAS DEL CAUCA S. A.
JULUMITO HYDRO-ELECTRIC PROJECT
RIO CAUCA DIVERSION DAM SECTION
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN
SUBMITTED: F. Yamamoto
RECOMMENDED: Y. Kawashima
APPROVED: M. Takahashi
DATE; 31 July '72
DWG NO. 17

LOCATION	DATE	DESCRIPTION	BY
REVISION			

SHEET NO. OF



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S.A.

JULUMITO HYDRO-ELECTRIC PROJECT

RIO PALACÉ DIVERSION DAM
PLAN, PROFILE AND SECTION

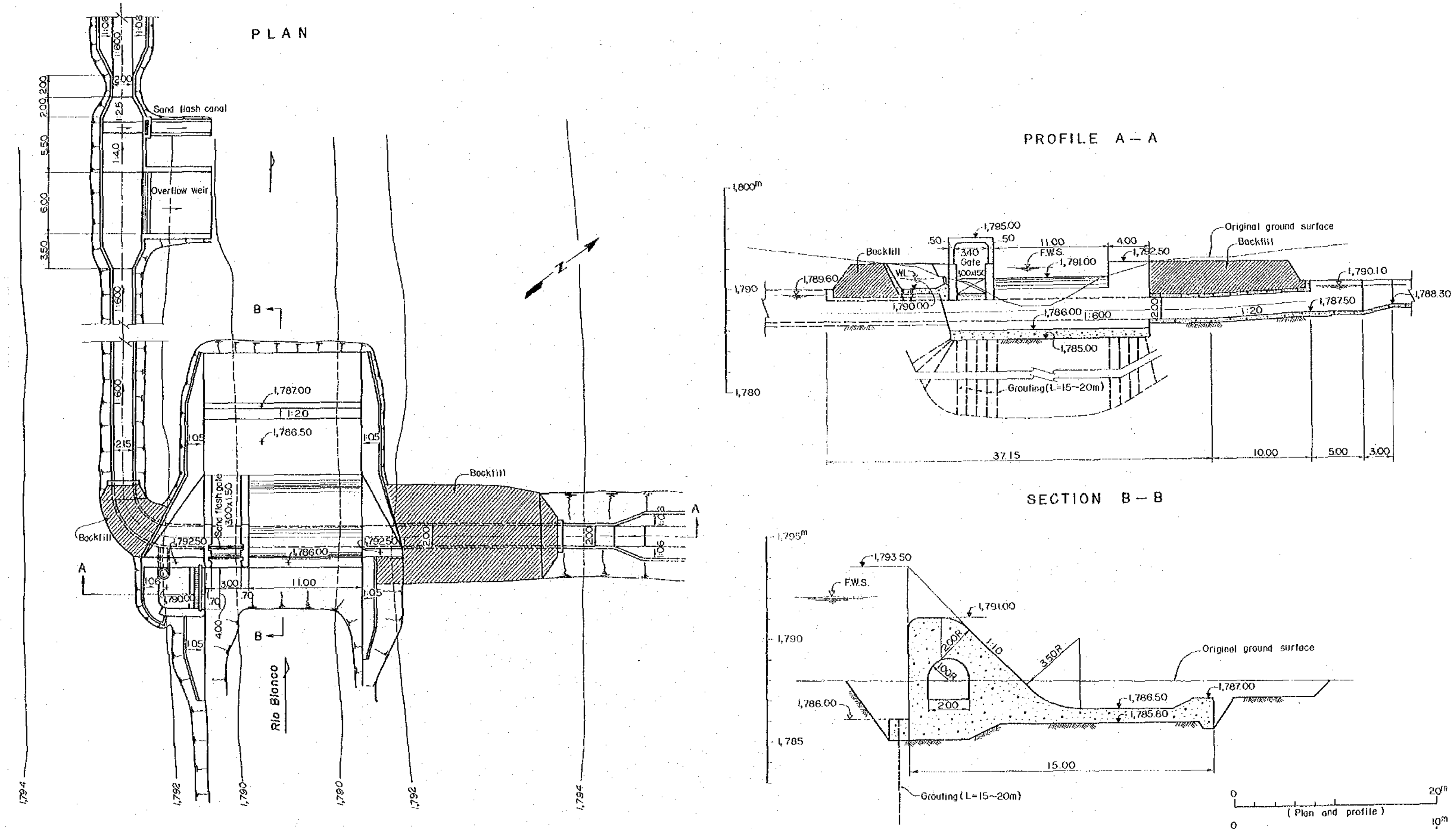
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D C consultants) TOKYO JAPAN

D. R.:	SUBMITTED:	K. Yamamoto
T. R.:	RECOMMENDED:	T. Kawashima
C. K.:	APPROVED:	M. Takahashi

LOCATION	DATE	DESCRIPTION	BY
REVISION			

DATE: 31 July '72 — — — DWG. NO. 18

SHEET NO. OF



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S. A.

JULUMITO HYDRO-ELECTRIC PROJECT

RIO BLANCO DIVERSION DAM
PLAN, PROFILE AND SECTION

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

SUBMITTED: K. Yamamoto
RECOMMENDED: T. Kawashima
APPROVED: M. Takahashi

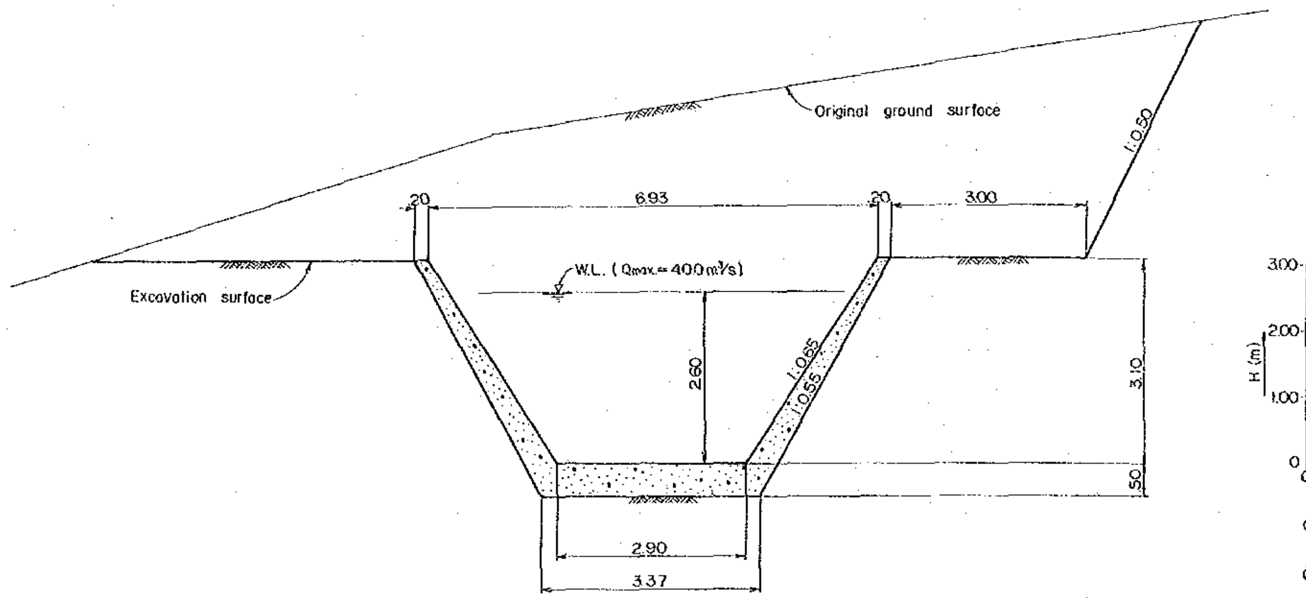
LOCATION	DATE	DESCRIPTION	BY
REVISION			

DATE; 31 July '72

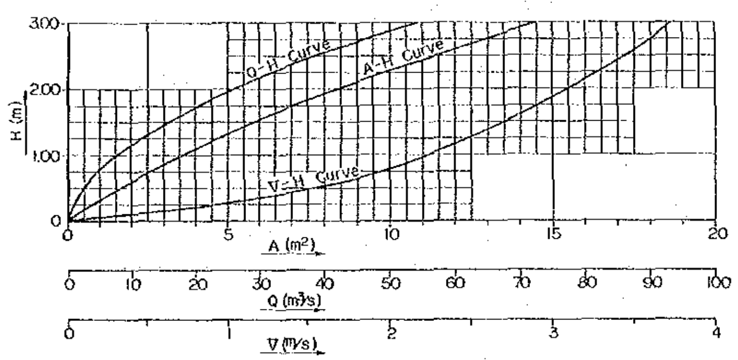
DWG. NO. 19

SHEET NO. OF

TYPICAL CROSS SECTION OF OPEN CHANNEL



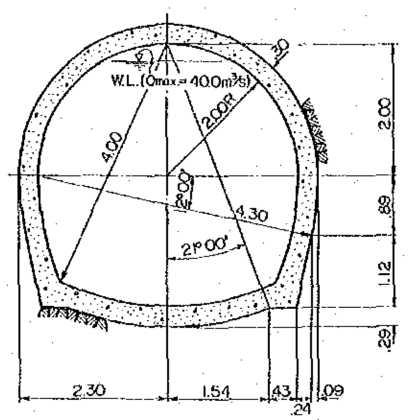
HYDRAULIC CHARACTERISTIC CURVE



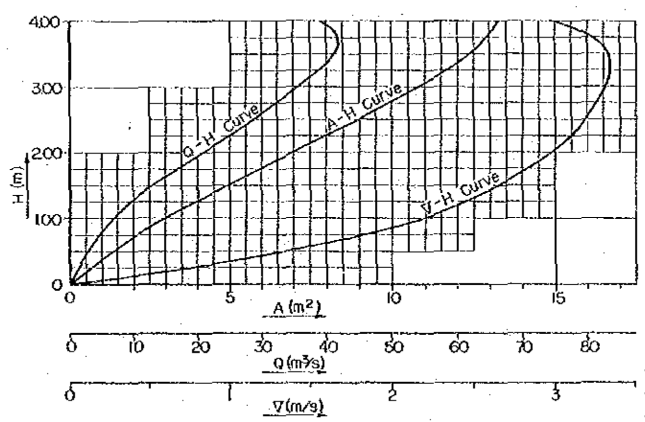
Legend

H : Water depth (m)
A : Area (m²)
Q : Discharge (m³/s)
V : Velocity (m/s)

TYPICAL CROSS SECTION OF TUNNEL



HYDRAULIC CHARACTERISTIC CURVE



LOCATION	DATE	DESCRIPTION	BY
REVISION			

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S.A.

JULUMITO HYDRO-ELECTRIC PROJECT

CAUCA DIVERSION WATERWAY

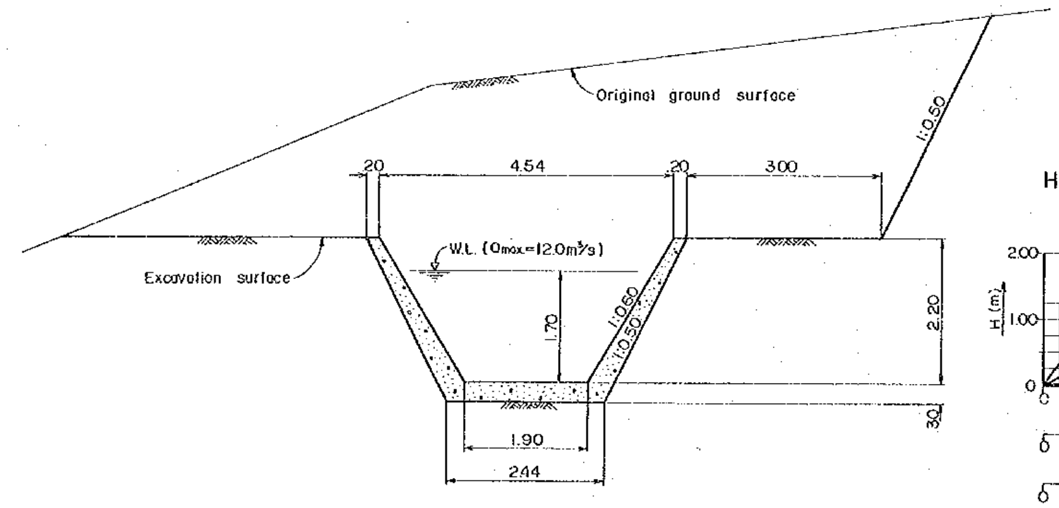
ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

D. R.: SUBMITTED: K. Yoko...
T. R.: RECOMMENDED: R. Kawashima
C. K.: APPROVED: M. Takahashi

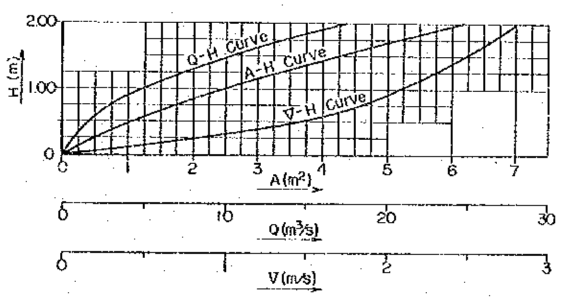
DATE ; 31 July '72 — — — DWG NO. 20

SHEET NO. OF

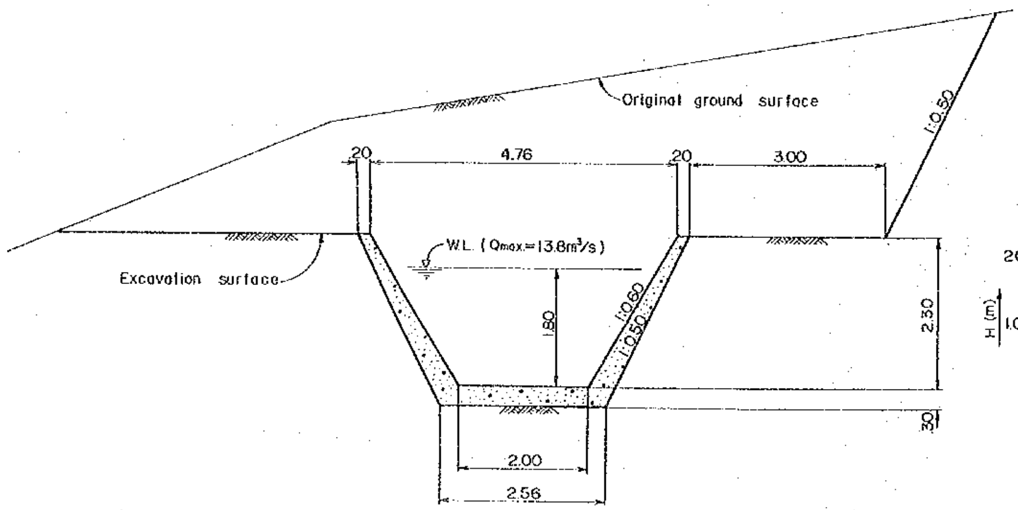
TYPICAL CROSS SECTION
OF NO.1 OPEN CHANNEL



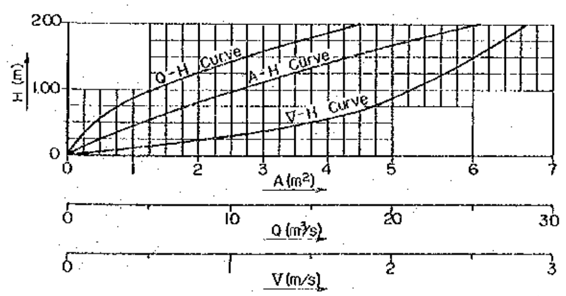
HYDRAULIC CHARACTERISTIC CURVE



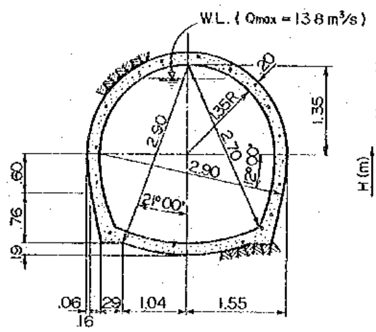
TYPICAL CROSS SECTION
OF NO.2 OPEN CHANNEL



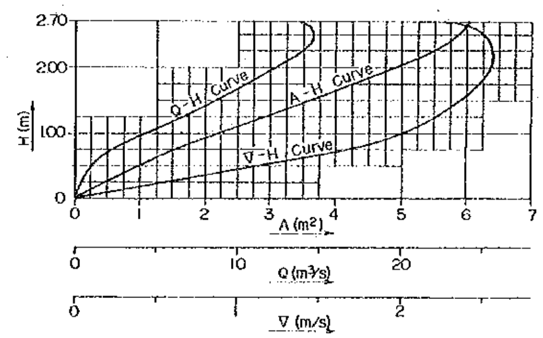
HYDRAULIC CHARACTERISTIC CURVE



TYPICAL CROSS SECTION
OF TUNNEL



HYDRAULIC CHARACTERISTIC CURVE



Legend

H :	Water depth	(m)
A :	Area	(m²)
Q :	Discharge	(m³/s)
V :	Velocity	(m/s)

LOCATION	DATE	DESCRIPTION	BY
REVISION			

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S.A.

JULUMITO HYDRO-ELECTRIC PROJECT

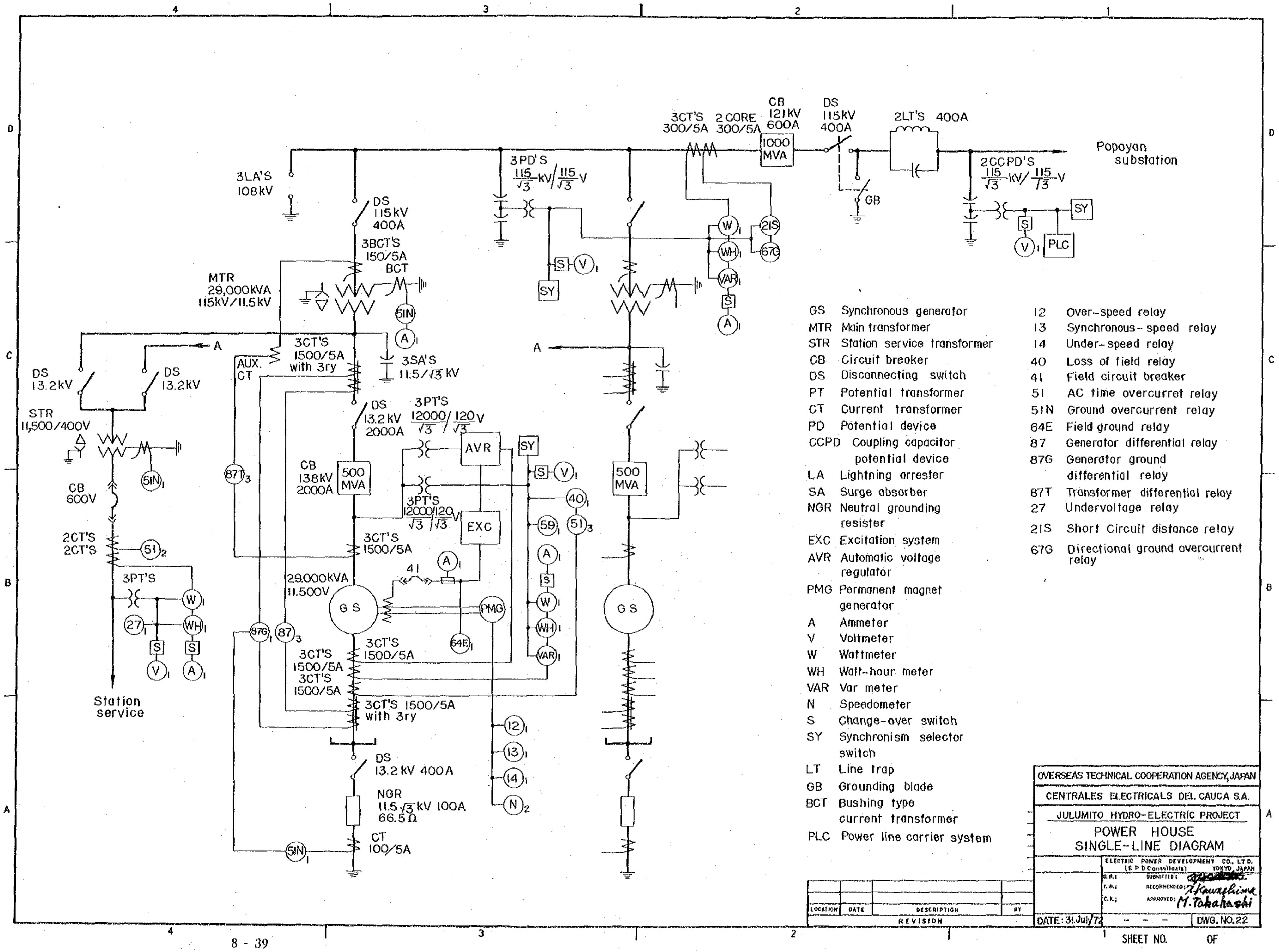
PALACÉ DIVERSION
WATERWAY

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

D.R.: SUBMITTED: Y. Yamamoto
T.R.: RECOMMENDED: Y. Kawashima
C.K.: APPROVED: M. Takahashi

DATE: 31 July '72 — — — DWG NO. 21

SHEET NO. OF


3LA'S 108kV
DS 115kV 400A
3BCT'S 150/5A
BCT
MTR 29,000KVA 115kV/11.5kV
3CT'S 1500/5A with 3ry
AUX. CT
3SA'S 11.5/√3 kV
DS 13.2kV
DS 13.2kV
STR 11,500/400V
CB 600V
2CT'S 2CT'S
3PT'S
Station service
DS 13.2kV 2000A
3PT'S 12000/√3 / 120/√3 V
CB 13.8kV 2000A
500 MVA
AVR
SY
EXC
3CT'S 1500/5A
41
29,000KVA 11,500V
GS
PMG
3CT'S 1500/5A
3CT'S 1500/5A
3CT'S 1500/5A
3CT'S 1500/5A with 3ry
DS 13.2 kV 400A
NGR 11.5/√3 kV 100A 66.5Ω
CT 100/5A
3PD'S 115/√3 kV/115/√3 V
3CT'S 300/5A
2 CORE 300/5A
CB 121kV 600A
1000 MVA
DS 115kV 400A
2LT'S 400A
GB
2CCPD'S 115/√3 kV/115/√3 V
PLC
Popoyan substation
GS Synchronous generator
MTR Main transformer
STR Station service transformer
CB Circuit breaker
DS Disconnecting switch
PT Potential transformer
CT Current transformer
PD Potential device
CCPD Coupling capacitor potential device
LA Lightning arrester
SA Surge absorber
NGR Neutral grounding resister
EXC Excitation system
AVR Automatic voltage regulator
PMG Permanent magnet generator
A Ammeter
V Voltmeter
W Wattmeter
WH Watt-hour meter
VAR Var meter
N Speedometer
S Change-over switch
SY Synchronism selector switch
LT Line trap
GB Grounding blade
BCT Bushing type current transformer
PLC Power line carrier system
12 Over-speed relay
13 Synchronous-speed relay
14 Under-speed relay
40 Loss of field relay
41 Field circuit breaker
51 AC time overcurret relay
51N Ground overcurrent relay
64E Field ground relay
87 Generator differential relay
87G Generator ground differential relay
87T Transformer differential relay
27 Undervoltage relay
21S Short Circuit distance relay
67G Directional ground overcurrent relay
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
CENTRALES ELECTRICALS DEL CAUCA S.A.
JULUMITO HYDRO-ELECTRIC PROJECT
POWER HOUSE
SINGLE-LINE DIAGRAM
ELECTRIC POWER DEVELOPMENT CO., LTD. (E.P.D.Consultants) TOKYO, JAPAN
LOCATION DATE DESCRIPTION BY
REVISION
DATE: 31.July.72
DWG. NO. 22
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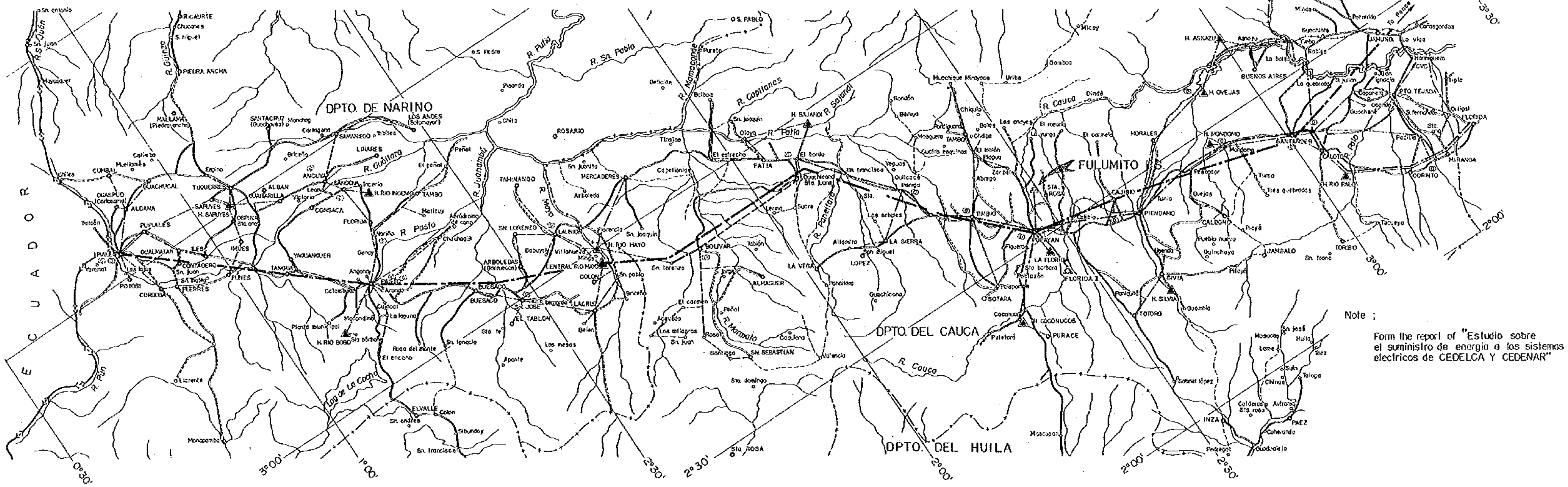
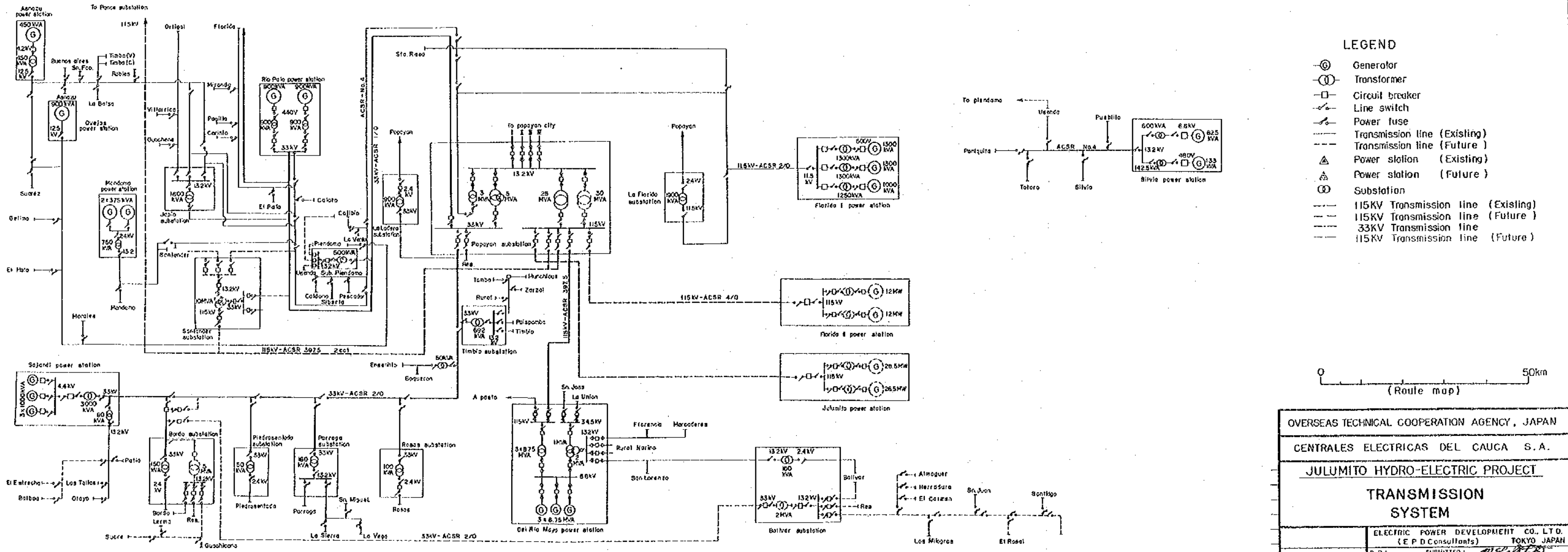
ROUTE MAP OF TRANSMISSION LINE





OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

CENTRALES ELECTRICAS DEL CAUCA S.A.

JULUMITO HYDRO-ELECTRIC PROJECT

TRANSMISSION SYSTEM

ELECTRIC POWER DEVELOPMENT CO., LTD. (E P D Consultants) TOKYO JAPAN

LOCATION	DATE	DESCRIPTION	BY
		REVISION	

DATE ; 31 July '72 — DWG. NO. 23

SHEET NO. OF