MINISTERIO DE MINAS Y ENERGIA

PROYECTO DEL RIO GUAVIO

ISA

1974

ISA Interconexión Eléctrica S. A.

PROYECTO DEL RIO GUAVIO

APENDICE A

INFORMES DE CONSULTORES

INGETEC

BOGOTA, COLOMBIA

AGOSTO, 1974

INTERCONEXION ELECTRICA S.A.

PROYECTO DEL RIO GUAVIO

ANALISIS DE

ALTERNATIVAS DE APROVECHAMIENTO HIDROELECTRICO

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INTERCONEXION ELECTRICA S.A.

PROYECTO DEL RIO GUAVIO

ALTERNATIVAS DE APROVECHAMIENTO HIDROELECTRICO

APENDICE A

INFORMES DE CONSULTORES

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II. HIDROLOGIA

Informe del Consultor Franklin F. Snyder (Mayo, 1972): Hidrología de crecientes e hidrógrafas de diseño para los rebosaderos y las obras de desviación de las presas del Proyecto del Río Guavio.

Informe del Consultor Dwight E. Nunn (Septiembre, 1972): Estimativos de precipitación máxima probable para determinación de las crecientes de diseño en los sitios de presa del Proyecto del Río Guavio.

III. INFORME DEL CONSULTOR J. BARRY COOKE

Informe del Consultor J. Barry Cooke (Agosto, 1974): Rebosaderos en túnel para el Proyecto del Río Guavio.

INFORME 1

GEOLOGIA

GEOLOGICAL RECONNAISSANCE OF DAM SITES FOR POWER DEVELOPMENT OF THE GUAVIO RIVER AND TRIBUTARIES

by Richard E. Goodman

July 1971

INTRODUCTION

This reports a first geologic reconnaissance of several sites key to hydroelectric power development on the Guavio River and its tributaries. The thalweg of Rio Guavio is shown in figure 1. Flowing from above the town of Gacheta, Rio Guavio joints Rio Farallones near Gachala, at elevation 1450 meters. About 1 Km upstream from its junction with Rio Batatas, at about elevation 1400 m, it begins descending through an increasingly deep canyon, with average side slopes of 40 - 50° reaching upwards 500-800 meters. In this region, the river is flowing through hard rocks -- phyllite, slate, hornfels, and quartzite -- formed from low grade metamorphism of sandstones, siltstones and shales. Upstream, the Guavio Valley is incised in sedimentary rocks -- sandstones and shales -- except near Rio Farallones where metamorphic rocks similar to those at the Chivor dam site are encountered. Figure 2, reproduced from the Appendix to Chingaza Project bid documents, presents an estimate of the regional geology.

Based on topographic considerations alone, the initial scheme presented in figure 3 was conceived, the principal features being: 1) a 130 m high storage dam (Gacheta dam) with reservoir elevation 1725 m; and 2) a 115 m diversion dam (Cobre dam) with reservoir elevation 1450 m feeding a power tunnel to Rio Bata with surface or underground power house near Mambita at elevation 600. Additional storage reservoirs for later addition, shown on figure 3, are Farallones, Tigre, and Rucio reservoirs. A tunnel would carry water from Rucio and Tigre to Cobre reservoirs, while Farallones reservoir water would be released to the river.

During this reconnaissance, made in the company of Dr. Ricardo Cajiao, Guillermo Jimenez, Camilo Torres, and Eduardo Chavez, visits were made to Gacheta dam site on Tuesday, July 6, and Cobre low dam site on Wednesday, July 7. A heavy rain (perhaps 10 cm), which began in the early morning of July 7 and continued all day, created flood conditions allowing only a casual study of Cobre dam sites. Present plans are to continue the reconnaissance in early September.

GACHETA DAM SITE

The location of Gacheta dam site, accessible by road, is shown on figure 3 at 1:100,000 scale. Figure 4 is a 1:10,000 topographic map with reconnaissance geology superimposed. The Guavio River (also called Gacheta River on figure 3) flows S 50° E in passing through a narrow cayon 600 meters long. Abrupt bends of the river, and widening of the valley at both ends of the canyon restrict the possible dam axes to a limited choice. Figure 4 shows 3 possibilities, labelled AA', BB', and CC'. The canyon is maintained by a ledge of hard silica-cemented quartz sandstone striking obliquely across the valley and dipping upstream (strike S 20 W dipping 37° to the west).

The sandstone ledge (member II on figure 4) is generally a hard, competent rock, seemingly acceptable as the foundation of a high dam of any design, and probably suitable as a source of rock fill. On closer inspection, however, it is seen to grade downwards (geologically) into shales. The sandstone member has a maximum thickness of perhaps 100 meters but contains frequent interbeds of black shale in its lower half. Below the sandstone member is inter stratified shale and sandstone (member III) grading downwards into pure shale (member IV) about 130 m below the base of the sandstone. (These figures and the map on which they are based are only rough estimates. However the geology is tractable and well exposed so that accurate maps and projection can quickly be developed). The cross sections of axes A, B, and C, show that the dam can not be laid on a homogeneous foundation. Since the sandstone is likely to be more permeable than the shale, a somewhat unfavorable situation is presented in which relatively pervious rock lies under the upstream and relatively impervious rock under the downstream parts of the foundation. However, the site is basically a good one. Axis BB' is preferable.

This site could possibly accommodate an arch dam. However this is not recommended because the particular ledge upon which it must rest is the only suitable arch dam foundation in the region. Should excavation during construction reveal any serious defects, there would be no alternative but to initiate major design changes. Since, in fact, there is shale in abundance within the sandstone member, serious defects could be uncovered.

For a rock fill dam the site presents adequate strength. The problem is to find enough rock. Spillway excavation will not produce a sufficient quantity of suitable rock and a quarry will have to be carried perhaps up to elevation 2200 meters downstream of the left abutment.

A large debris slide downstream on the right side, which covers the sandstone member, has created terraces just below the site, probably of reworked shale. One solution worth exploring would be to compact a sloping core of these terrace materials to reduce the quantity of sandstone required.

As noted later, it may not be necessary to build Gacheta dam. But if it is, a good site is available.

COBRE DAM SITE

The site suggested for diverting flow to the power tunnel, and for collecting subsidiary flows, is shown on figure 5, just below Rio Chivor. Reconnaissance of this section was made from the right bank from, the trail to Minas de Cobre.

The most striking topographic feature of this section of Rio Guavio is an incised horseshoe bend defended by a steep, narrow spur (axis AA' and BB'). A saddle in this spur invites an off channel spillway which could be developed by a 35 meter excavation. The spur is a strike ridge of competent rock (probably argillite) dipping upstream, while its saddle is possibly the topographic expression of a minor cross fault or a flexure in the stratum.

The geologic structure of this section is dominated by a sharp syncline striking across the river at Rio Chivor, which may be the continuation of the chevron syncline clearly visible in the top of Cuchilla El Diamante. An anticline is visible in the left bank above axis BB' (figure 5) but is continuation toward the river is unclear. A well marked cross fault can be seen in the left side near axis DD' and probably continues across the river through the end of the previously noted spur.

A deep cover of debris is visible in a slide scar below axis DD' on the right bank. Also a relatively deep mantle of debris is suggested on the left bank at axis CC'.

There is certainly at least one good site for a dam in this stretch of the river. The rocks are generally hard and the valley walls steep. However all the axes marked seem to have some problems. Axis AA' seems to be the best. The natural spillway can be developed for overflow, or if the ridge proves too narrow for safety a tunnel spillway could be cut through the spur. An earth or rock fill dam here would suffer the consequences of placement in a tight curve. Because of the narrowness of the right abutment, however, an arch dam could not even be discussed without closer geological inspection. Both axes AA' and BB' invite leakage possibilities across the narrow spur, which may possess a cross fault as previously noted. Axis BB' complicates spillway development as the toe of the embankment would block the natural discharge routes.

Axes CC' and DD' have no spillways. Further a considerable thickness of debris may need to be stripped from the left bank. The left abutment is weakened in this section by the fault(s) previously noted.

Below DD', no site seems feasible till near axis EE' because of slides and debris mantling the right bank. The river is dropping quickly here so that EE' is already a 15 meter higher dam if one attempts to maintain the reservoir at elevation 1450 m.

GACHALA DAM - AN ALTERNATIVE SCHEME

The Cobre dam produces a small reservoir (of the order of 55 million cubic meters) while Gacheta dam creates about 500 million cubic meters reservoir. A single high dam upstream of the junction with Rio Batatas could provide this combined volume performing both storage and diversion functions with a single structure. Upon cursory inspection it seems that a highly desirable site is available at axis FF' (figure 5) where the Guavio River flows in a straight narrow canyon just before receiving Rio Batatas. Side slopes average 50 to 55° for more than 300 meters of elevation. A 200 meter high dam at this site (to elevation 1600 meters) would provide a reservoir of about 600 million cubic meters, mainly in Rio Murca Valley, with an embankment of only 7 million cubic meters (figure 6). It may also be possible to develop an arch dam.

Though upstream of both Rio Batatas and Rio Chivor these significant flows could be diverted. Also a spillway and diversion of the river during construction can take advantage of the short path through the right bank to Rio Batatas. A significant advantege lies in the increase of 150 meters of head to the power plant. An unknown is the source of suitable materials.

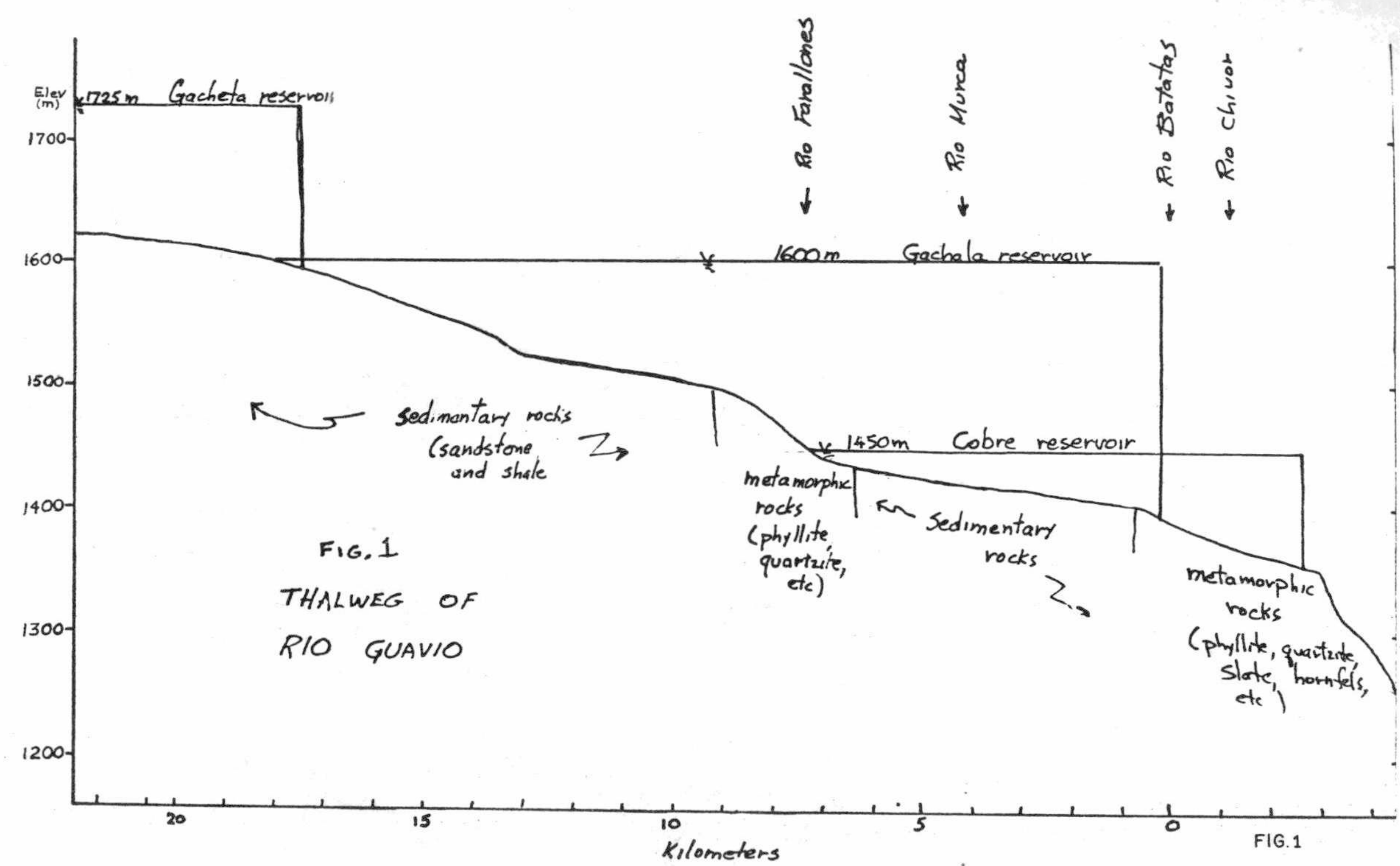
As seen from the trail above the right side of Rio Batatas, the major valley seems to be Batatas, so narrow is the Rio Guavio canyon. The site could not be inspected during this reconnaissance; geologically it remains almost unknown. Aerial photo inspection shows bold formations high above the left abutment dipping toward the Rio Chivor syncline.

This high dam scheme seems attractive. Therefore a detailed geological inspection is recommended, preferably in the dry season.

SUMMARY OF RECOMMENDATIONS

This reconnaissance is in progress. Major decisions should await further study, in drier season. The power tunnel and power house remain unknown. While Gacheta storage dam site is acceptable and a satisfactory site for diversion at El Cobre seems attainable, a more attractive scheme seems to be one high dam (200 m) above Rio Batatas (Gachala dam).

A second geological reconnaissance, mainly to inspect Gachala dam site and the power house sites is planned for early September.



Elev (m)
1725m Gacheta reservoir
1700
1600
1500
1400
1300
1200
Rio Farallones
Rio Murca
Rio Batatas
Rio Chivor
1600m Gachala reservoir
1450m Cobre reservoir
Sedimentary rocks (sandstone and shale
metamorphic rocks (phyllite, quartzite, etc)
Sedimentary rocks
metamorphic rocks (phyllite, quartzite, slate, hornfels, etc)
FIG. 1
THALWEG OF
RIO GUAVIO
20
15
10
5
0
Kilometers

FIG.1

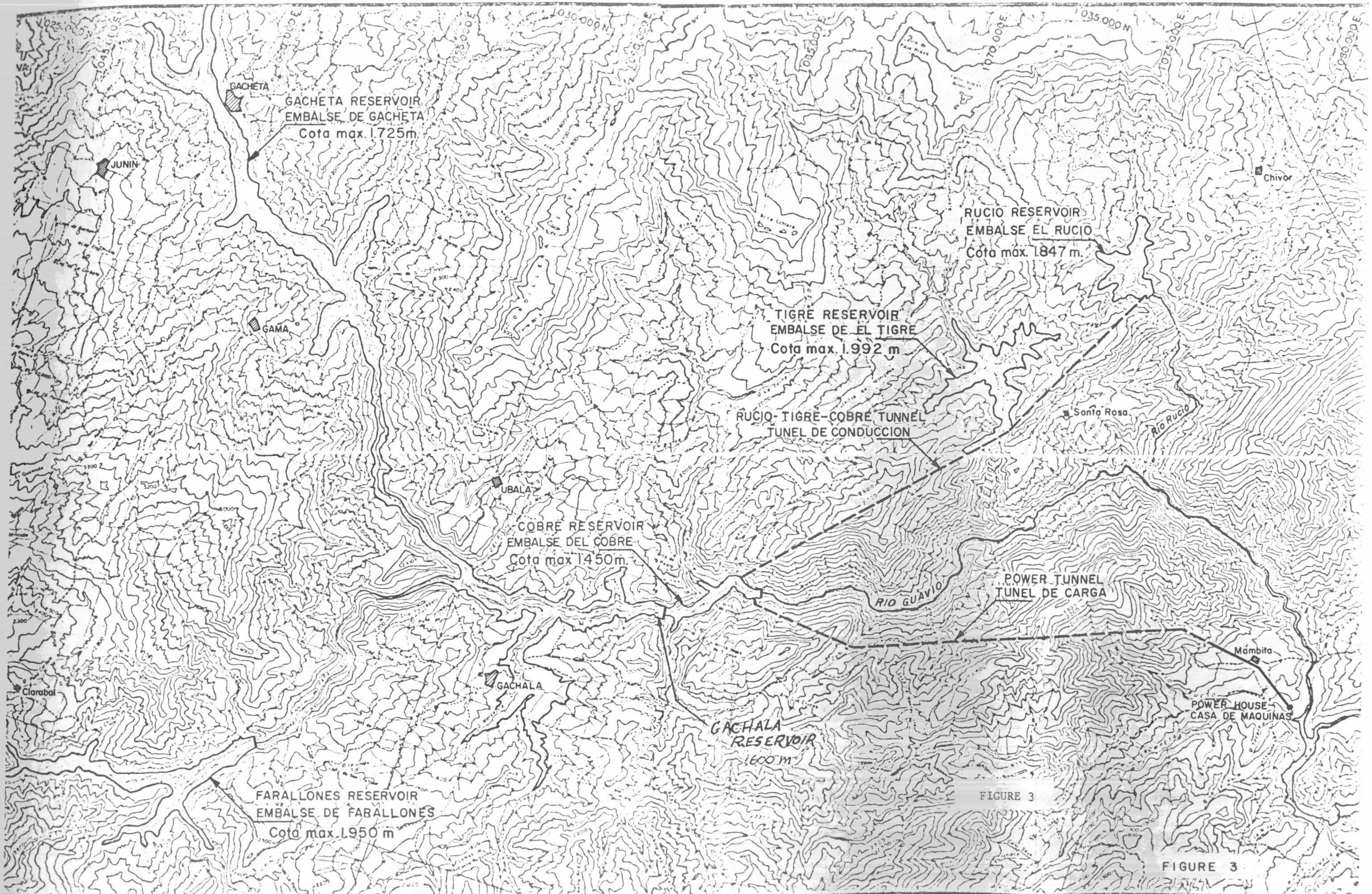


FIGURE 3

FIGURE 3

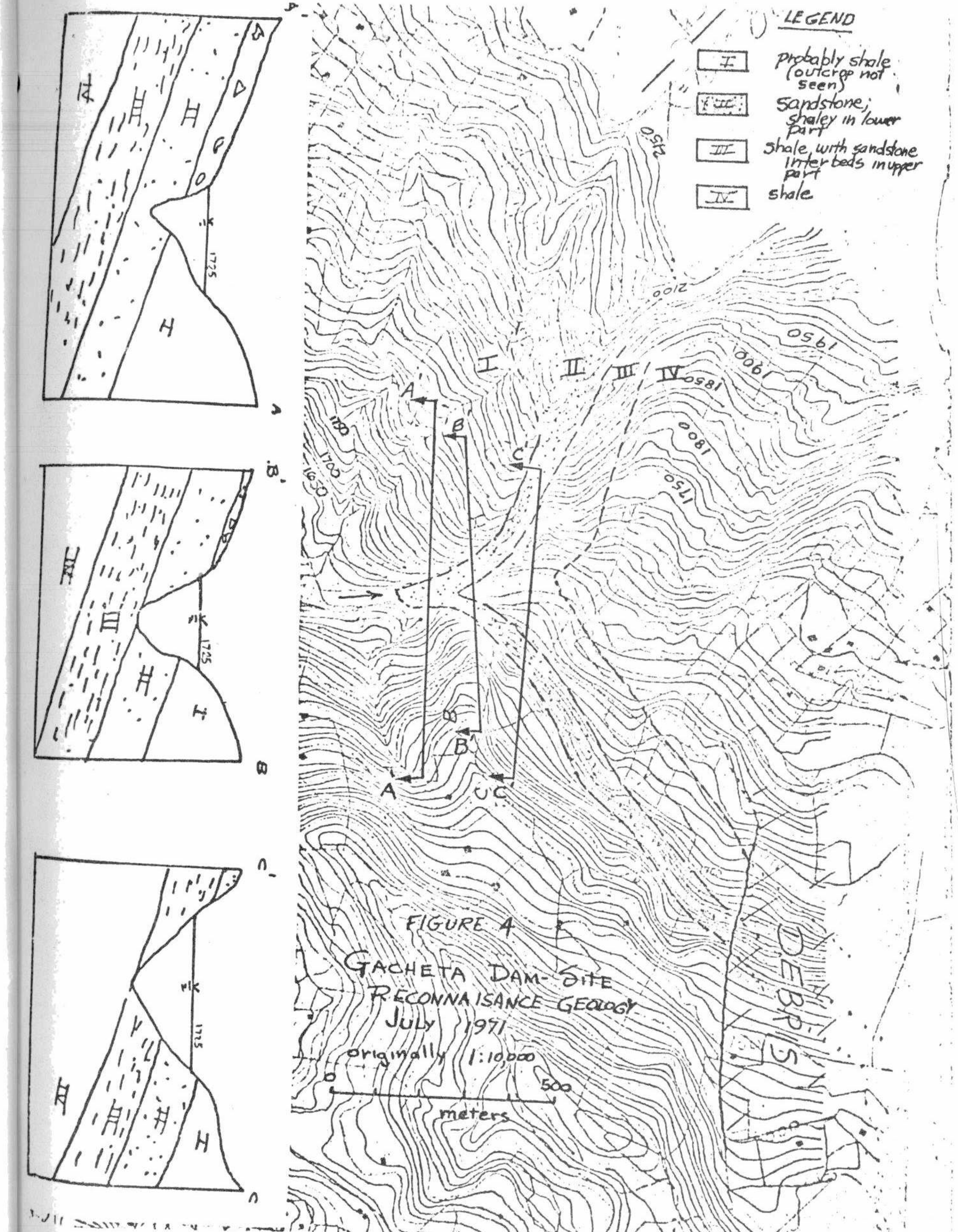


FIGURE 4
GACHETA DAM-SITE
RECONNAISANCE GEOLOGY
JULY 1971

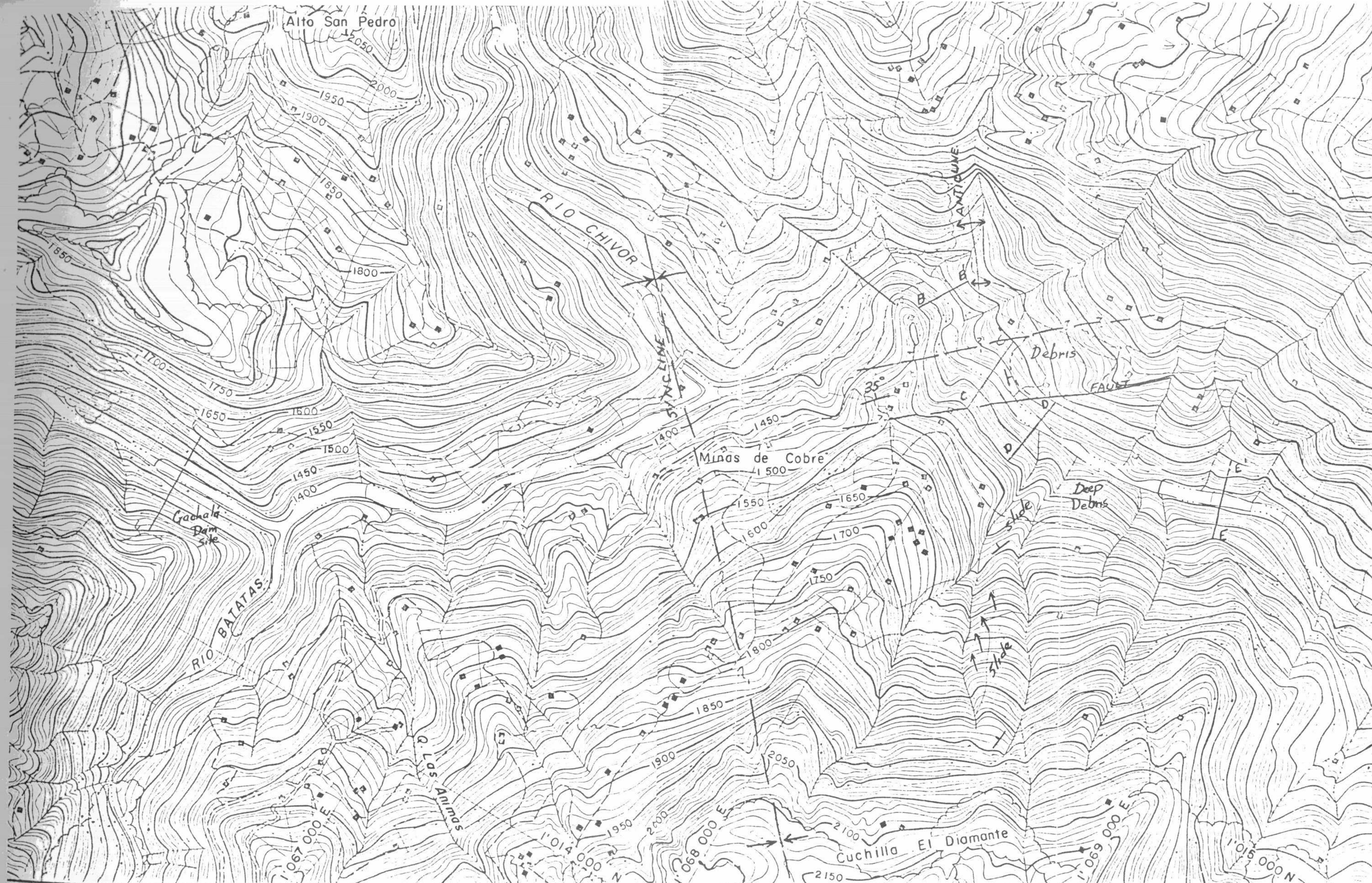


FIG. 5

FIGURA 5 - COBRE AND GACHALA DAM SITES

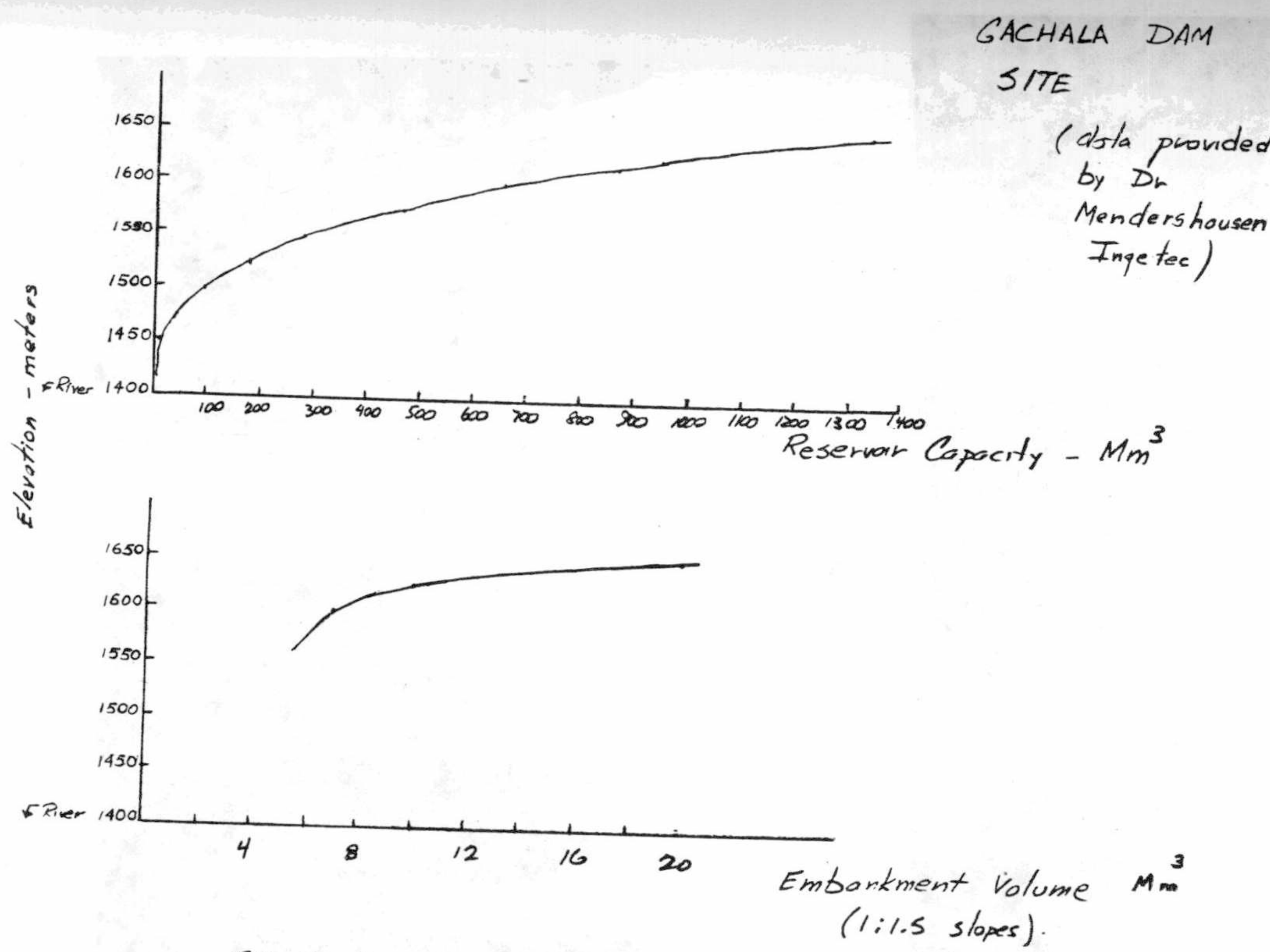
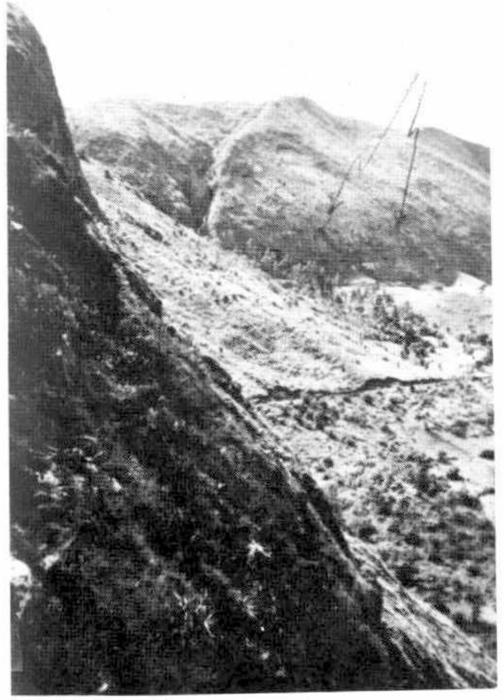
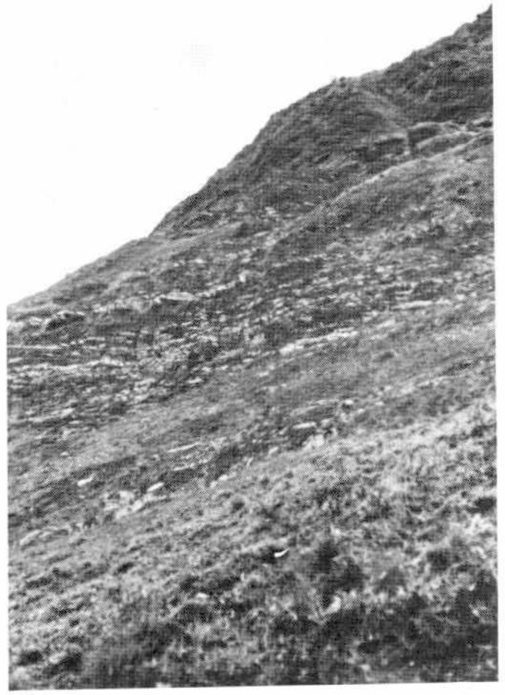


FIGURE 6 - EMBANKMENT, AND RESERVOIR CAPACITY, VOLUME VS ELEVATION

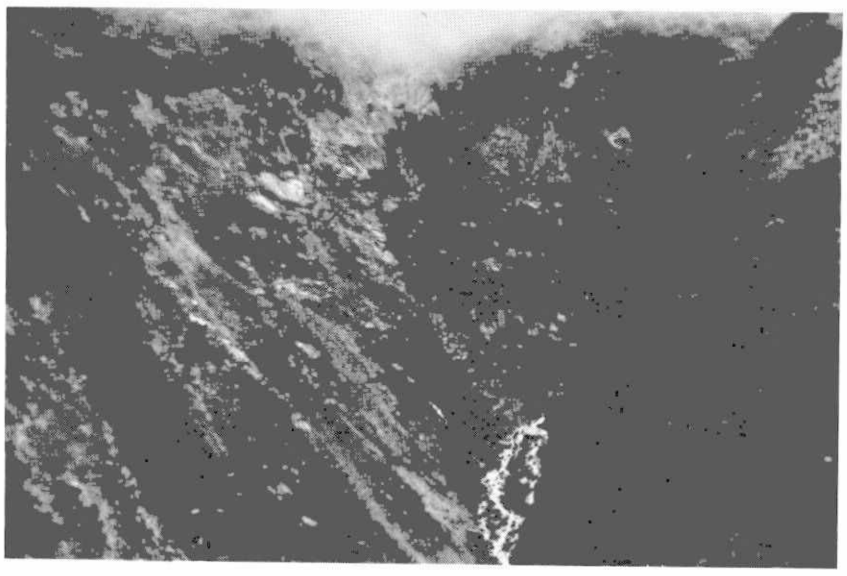
VIEW OF GACHETA RESERVOIR SITE FROM DAM SITE



SANDSTONE IN LEFT BANK OF GACHETA SITE ABOVE ROAD



SHALE AND SANDSTONE (MEMBER III) BELOW SANDSTONE MEMBER AT GACHETA DAM SITE

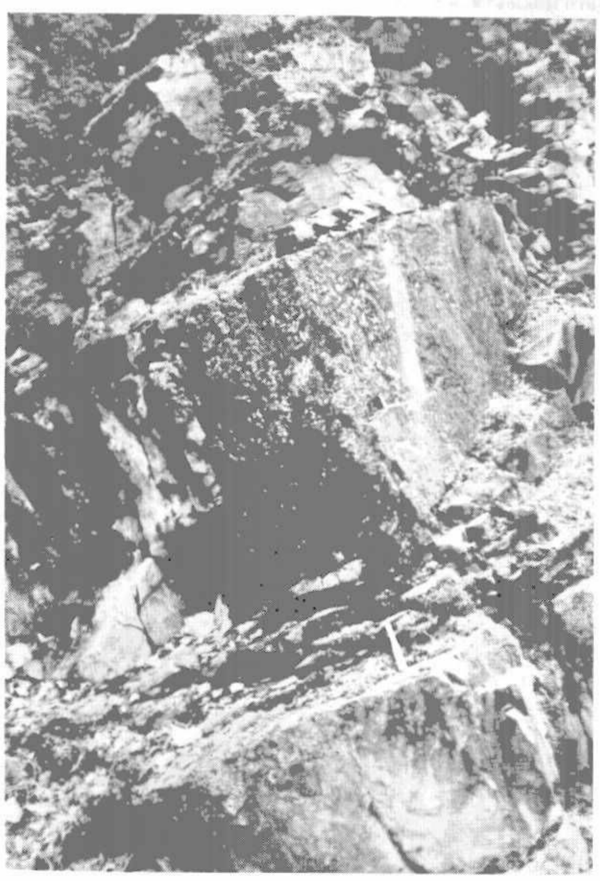


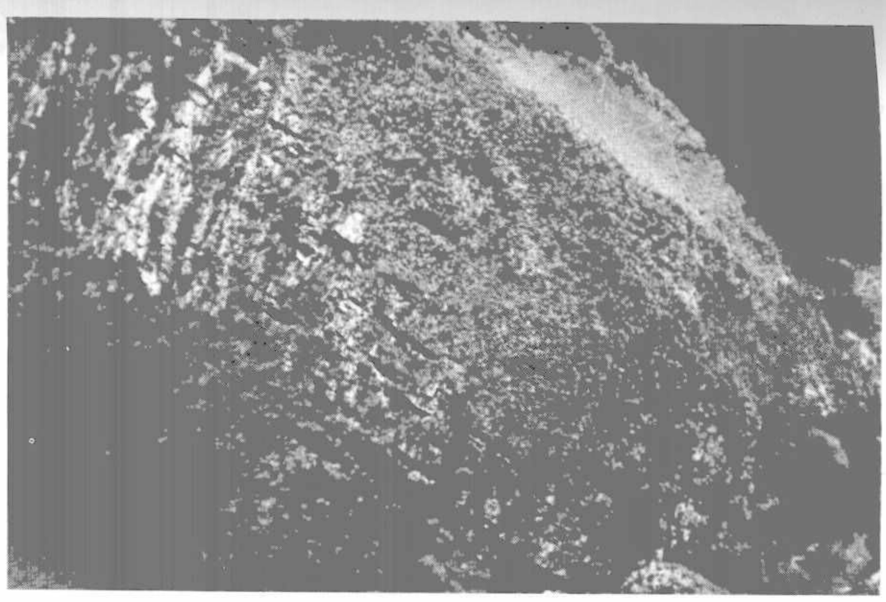
ABOVE: DEBRIS SLIDE ON A BEDDING PLANE
NEAR MINAS DE COBRE

TOP RIGHT: SYNCLINE CROSSING EL DIAMANTE
(PROBABLY CONTINUATION OF RIO CHIVOR
SYNCLINE)

LOWER RIGHT: THICK DEBRIS EXPOSED
IN SLIDE ZONE DOWNSTREAM OF EL COBRE
DAM SITE ON THE RIGHT BANK.

MOSAIC OF PHOTOS SEEN LOOKING UPSTREAM THROUGH GACHETA DAM SITE:
DIP OF STRATA SEEMS TO STEEPEN IN LEFT BANK (RIGHT SIDE OF PHOTO)

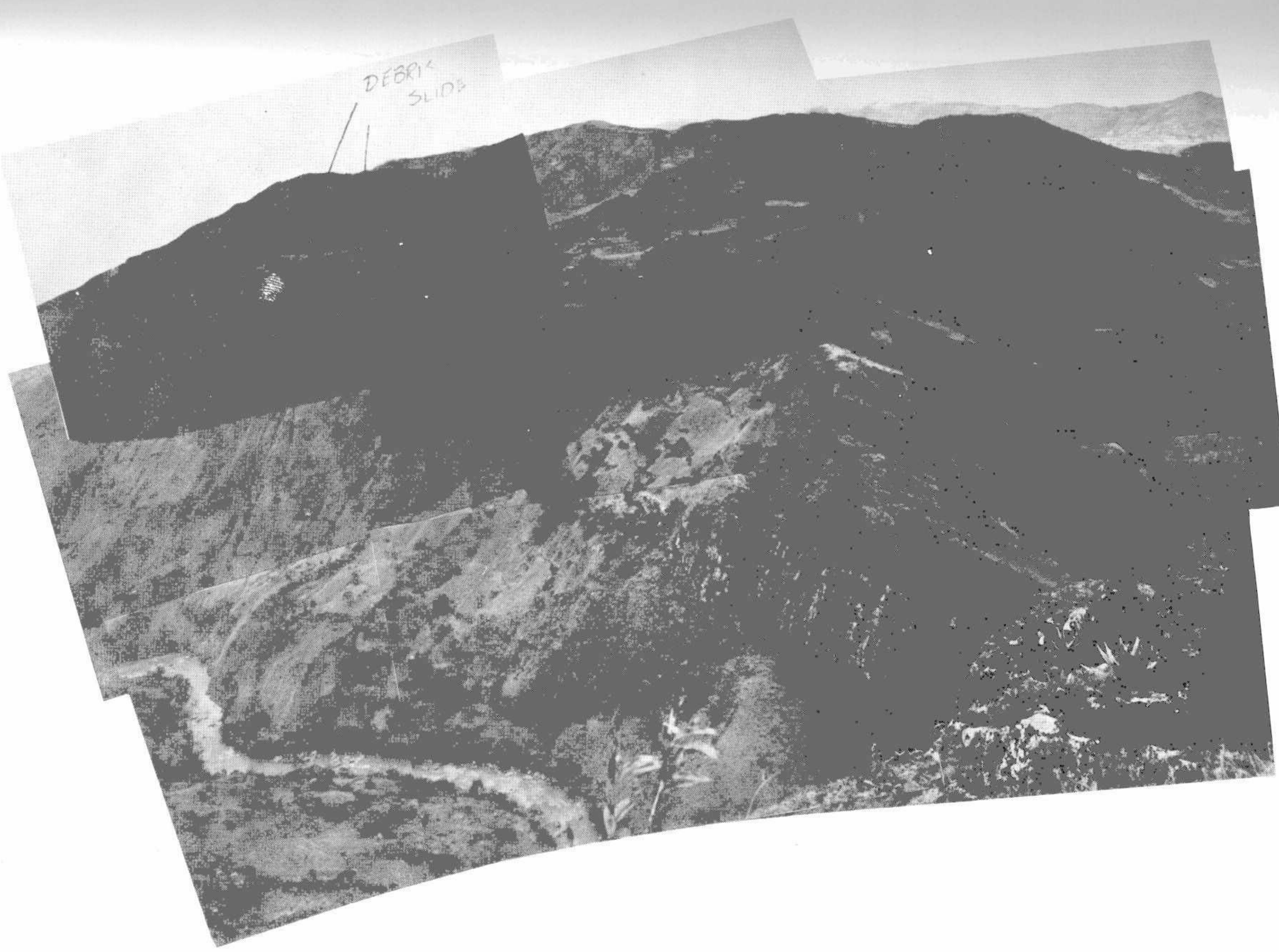




ABOVE: SHALE INTERBEDS IN THICK BEDS OF HARD QUARTZ SANDSTONE AT GACHETA DAM SITE

TOP RIGHT: FULL THICKNESS OF SAND- STONE MEMBER (MEMBER II) AT GACHETA DAM SITE (RIGHT BANK)

RIGHT, BELOW: CROSS FAULT EXPOSED IN ROAD AT GACHETA DAM SITE -- ONE OF A SYSTEM OF SUBPARALLEL FAULTS; SEEMS TO ACT AS A WATER CONDUIT



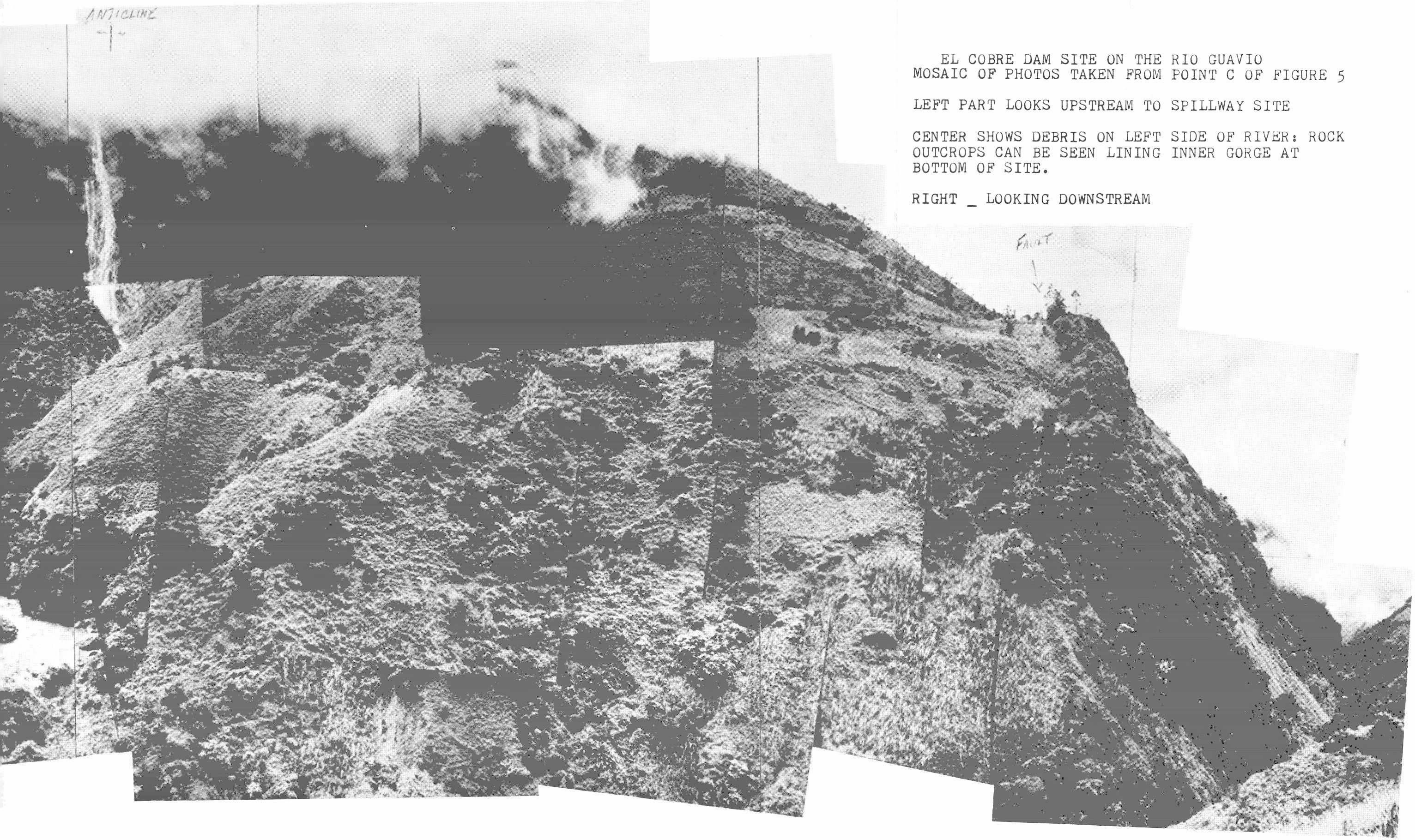
RIGHT BANK OF GACHETA DAM SITE; PHOTO MOSAIC TAKEN FROM
A POINT ON THE LEFT BANK JUST DOWNSTREAM FROM AXIS BB' (FIG $)

ABOVE: LOOKING DOWN RIO GUAVIO TOWARDS
EL COBRE SITE: LINE MARKS AXIS AA' OF
FIGURE 5

TOP RIGHT: RIO GUAVIO BETWEEN RIO CHIVOR
AND RIO BATATAS

LOWER RIGHT: RIO BATATAS


NATURAL
SPILLWAY
SITE



EL COBRE DAM SITE ON THE RIO GUAVIO
MOSAIC OF PHOTOS TAKEN FROM POINT C OF FIGURE 5

LEFT PART LOOKS UPSTREAM TO SPILLWAY SITE

CENTER SHOWS DEBRIS ON LEFT SIDE OF RIVER: ROCK OUTCROPS CAN BE SEEN LINING INNER GORGE AT BOTTOM OF SITE.

RIGHT _ LOOKING DOWNSTREAM

INFORME 2

GEOLOGIA

September 20, 1971

Doctor Carlos Ospina
INGETEC
Apartado Aéreo 5099
Bogotá, Colombia

Reconnaissance of Aguila (Gachala) dam site and Guavio tunnel line.

Dear Carlos:

On September 8, 9, and 10 I completed a reconnaissance of the Guavio River project, with Dr. R. Cajiao, E. Chavez, and C. Torres of INGETEC. We proceeded by horse from Santa Maria to Mambita, a possible powerhouse site (6 hours) on Wednesday, September 8. On Thursday, September 9, we travelled generally along the tunnel route from Mambita to Cuchilla El Diamante (8 hours) where a waiting jeep took us to Gachala. On Friday, we made a brief inspection of the left bank at Aguila dam site. The weather was favorable and reasonably good exposures of rock along the way made it a useful reconnaissance. Geologic control points and the general composition of the rocks at the dam sites and tunnel lines are now known in rough outline. Accordingly, some tentative planning decisions can now be made so that more detailed geological studies can begin.

<u>General Geology of the Tunnel Line</u>

Mambita, a town of 80 persons, lies atop a large fluvial terrace at elevation 800 meters. Fissile shales, resembling the Caqueza formation, underlie the valleys of Rio Rucio and Rio Trompeta south of the Mambita terrace. Rising above the terrace to the Northwest is a steep escarpment defended by cemented sandstone and conglomerate striking NE and dipping steeply to the south east. The first line of outcrops resembles the ripple marked sandstone forming the bold cliffs visible from Santa Maria. The rocks of this complex, probably Paleozoic, including hard phyllite as well, form a belt 2 to 3 kilometers wide. To the west, this formation gives way to argillites and shales, continuing all the way to El Diamante. The dam site itself is composed of hard marble, phyllite, and argillite, as will be discussed later.

Figure 1 shows 2 reconnaissance geologic profiles of possible tunnel lines. Point A is on the right abutment of Aguila dam site. Point B is near Mambita. Point C is on the Guavio downstream from Mambita, about 2 kilometers. Profiles AB and AC will be compared and contrasted. Both profiles are about the same length, about 20 km. C is perhaps 20 meters lower than B. Both profiles would require long penstocks for surface powerhouse development. There is no easy place to make room for a surface powerhouse at C, and the foundation will be in shale. There is a good place for a surface powerhouse at B and the foundation will be in fluvial deposits. The penstock for AC is about 4-1/2 kilometers long. For AB, the penstock is longer, perhaps 7 kilometers. Natural slopes for a penstock on AB do not appear to be unstable. The downstream portion of AC was not studied and its degree of stability is unknown.

The belt of hard rock is closer to the downstream end of the tunnel in profile AB than in profile AC. This rock seems to be excellent from the point of view of an underground powerhouse. On profile AB, a powerhouse can be located with good access at optimum depth (300 meters above the roof) whereas on profile AC, the depth will be greater, ie 600 meters above the roof. AC has a longer tailrace tunnel and no easy system surge shaft location. However, the tailrace tunnel of AB may have to pass through terrace gravels.

An underground powerhouse in the conglomerate sandstone, phyllite complex can probably utilize partially rock supported pressure shafts. Most of the tunnel length, with either alternative, will be in argillite. In my opinion this looks like optimum rock for machine boring.

Recommendations for Investigations

I recommend that an accurate geological map, at 1:50,000 scale, now be prepared for the possible tunnel lines. Since the depth of terrace deposits at Mambita is a key point on line AB, which otherwise appears very favorable, an effort should be made to determine this depth. A drill hole at B is required. Seismic studies are appropriate now and have a good chance of success.

Aguila Dam Site

The geology of the Guavio canyon, upstream of Rio Batatas is as follows:

(1) Massive beds of crystalline marble form bold outcrops dipping gently (ie 20°) into the left abutment and slightly downstream. (N 30-40E strike). Hard phyllite, and perhaps some quartzite, are interbedded with the marble. The massive beds cross the canyon floor about 1 kilometer downstream from Rio Batatas and then rise gradually in the left bank. The geology of the right bank is not understood. The beds strike parallel to the marble of the left bank but much more steeply. Several interpretations are possible, including an unconformity, a thrust fault, and a monocline.

(2) Minor faults parallel to a slaty cleavage strike N65W dipping steeply to the south west.

(3) Vugs and surface solution can be seen in the marble outcrop. No caves or sinkholes were discovered.

Recommendations for Investigations

Detailed site selection at Aguila dam site can not be made until detailed geologic mapping has been completed. The geology is such that minor adjustments of the axes could produce significant changes in the quality of the site.

The site appears to possess axes topographically suited to an arch dam and rock of good quality exists in the canyon. If a rockfill is considered, it will almost certainly have to be a quarried rock dam.

Spillway possibilities exist on both abutments; but are better on the right of the reservoir at 1600 meters; the right could be cut to spill into Rio Batatas. The left bank possesses a natural saddle at elevation 1840 meters.

All decisions about the dam require a good geological map now. An outcrop geological map at 1:10,000 should be compiled. The limits of outcrops should be shown. Test pits and trenches should be used to expose the rock between outcrops.

To facilitate this work, a bridge should be constructed and trails should be developed.

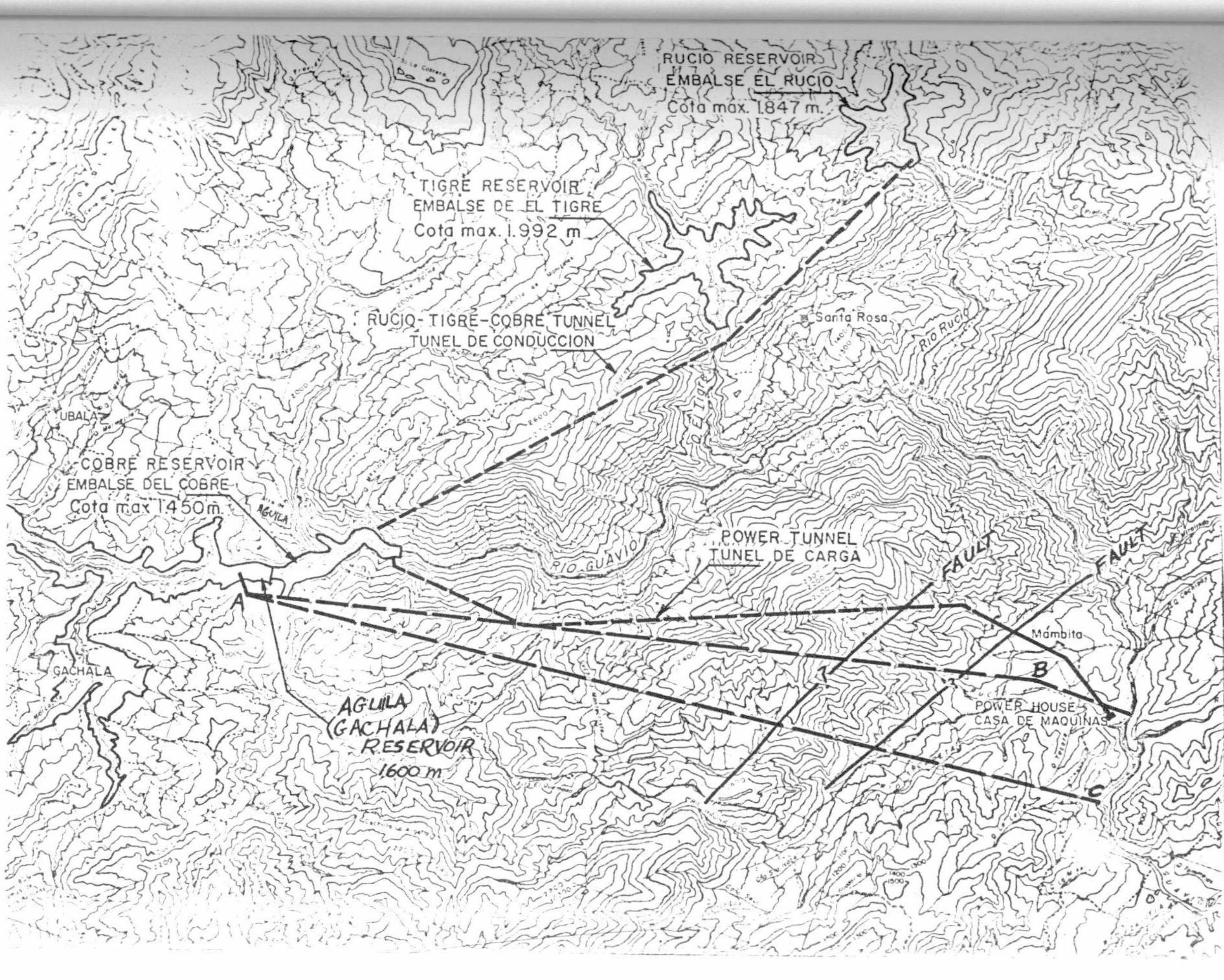
Drill holes are not necessary yet.

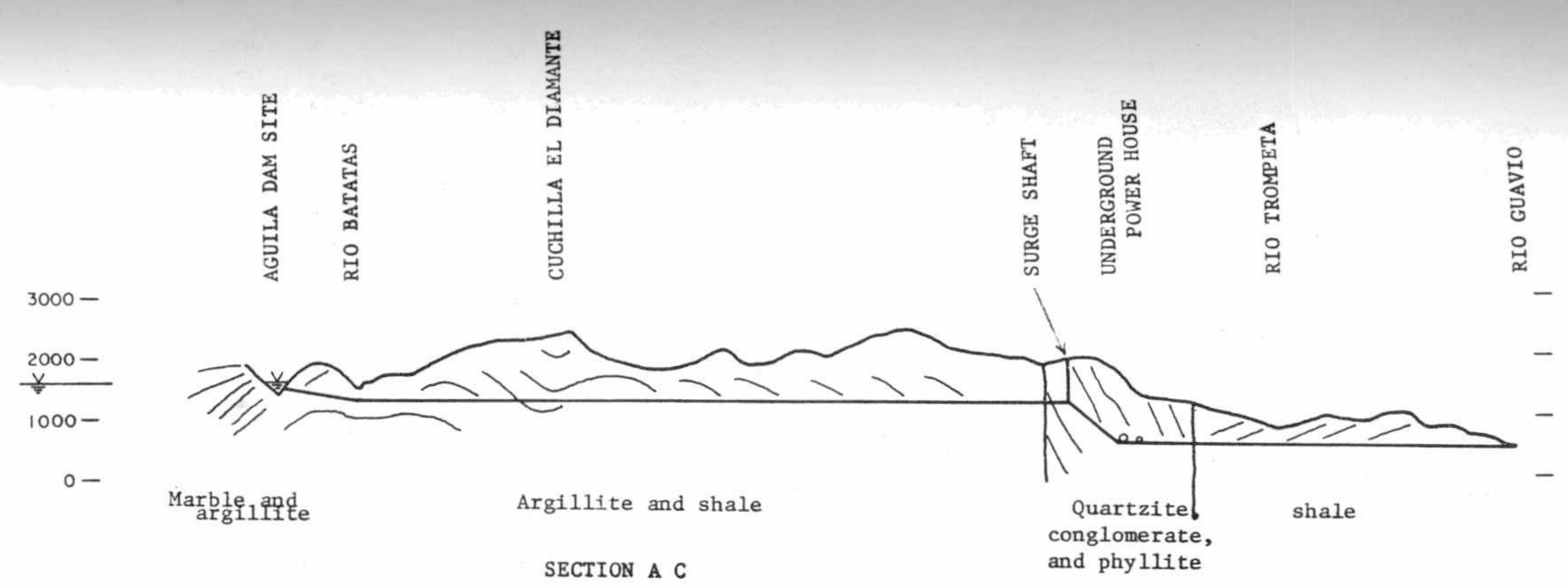
I hope the geological mapping of outcrops at Aguila dam site can be realized by early December. If so, I will plan to visit the site and review the work in December.

Very sincerely,

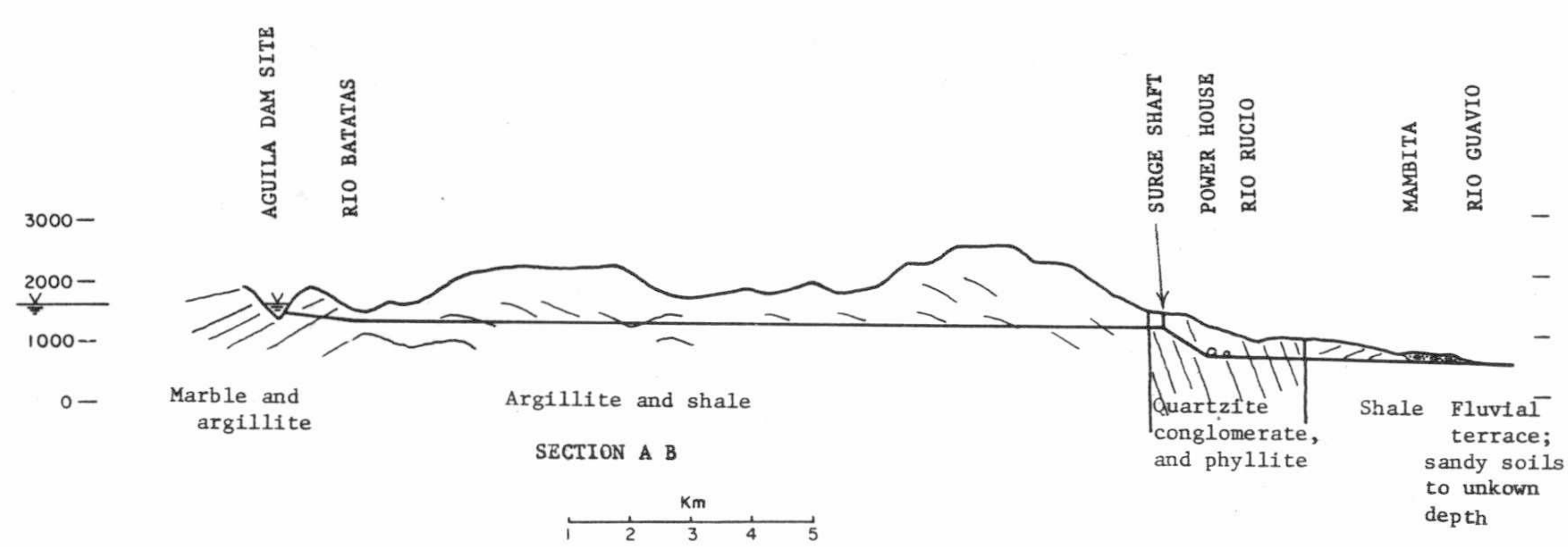
Richard E. Goodman

REG/nh

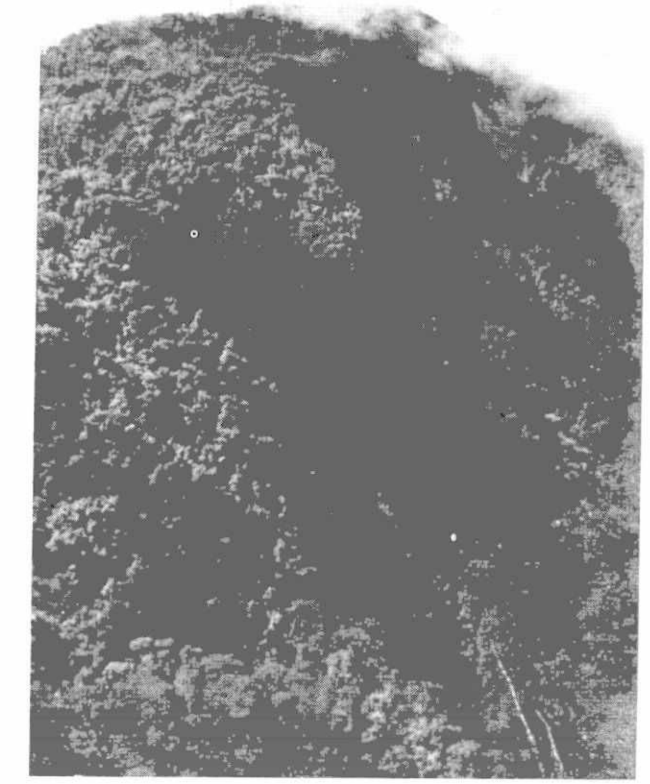

RUCIO RESERVOIR
EMBALSE EL RUCIO
Cota max. 1.847 m.
TIGRE RESERVOIR
EMBALSE DE EL TIGRE
Cota max. 1.992 m
Santa Rosa
Rio Rucio
RUCIO-TIGRE-COBRE TUNNEL
TUNEL DE CONDUCCION
UBALA
COBRE RESERVOIR
EMBALSE DEL COBRE
Cota max 1450 m.
AGUILA
RIO GUAVIO
POWER TUNNEL
TUNEL DE CARGA
FAULT
FAULT
A
Mambita
B
GACHALA
AGUILA
(GACHALA)
RESERVOIR
1600 m
POWER HOUSE
CASA DE MAQUINAS
C



SECTION A C



SECTION A B



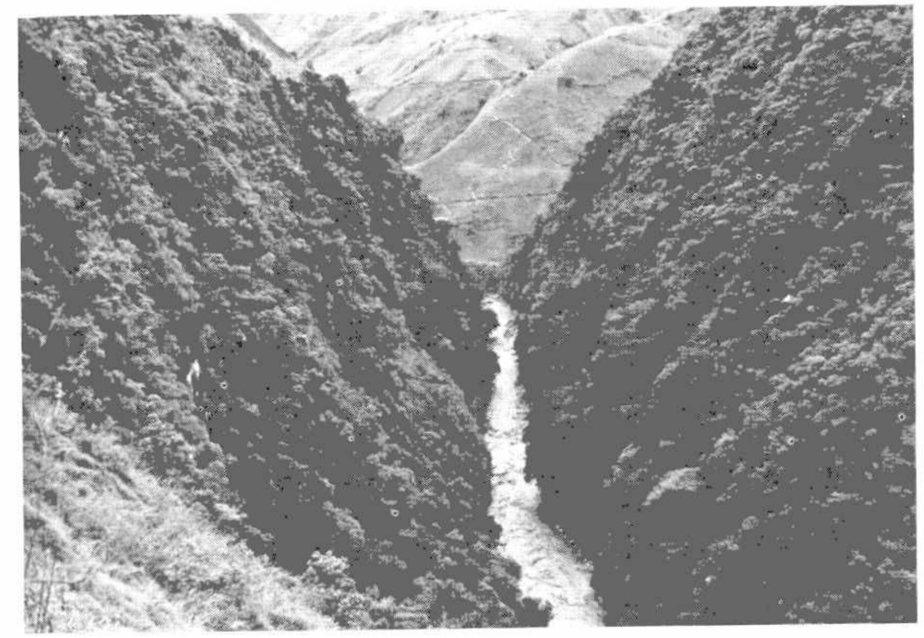
Outcropping quartzite on tunnel line
Above left bank of rio Trompeta

Boulders of conglomerate derived from vicinity of upper photo outcrops

Fluvial terrace of Mámbita

AGUILA DAM SITE

Looking towards downstream

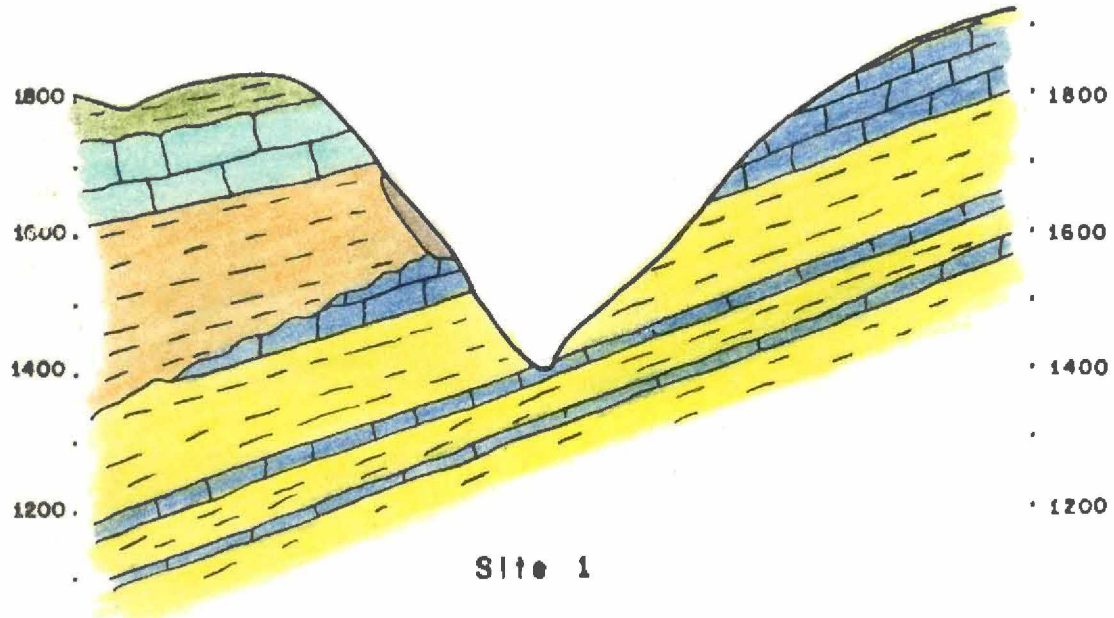


AGUILA DAM SITE

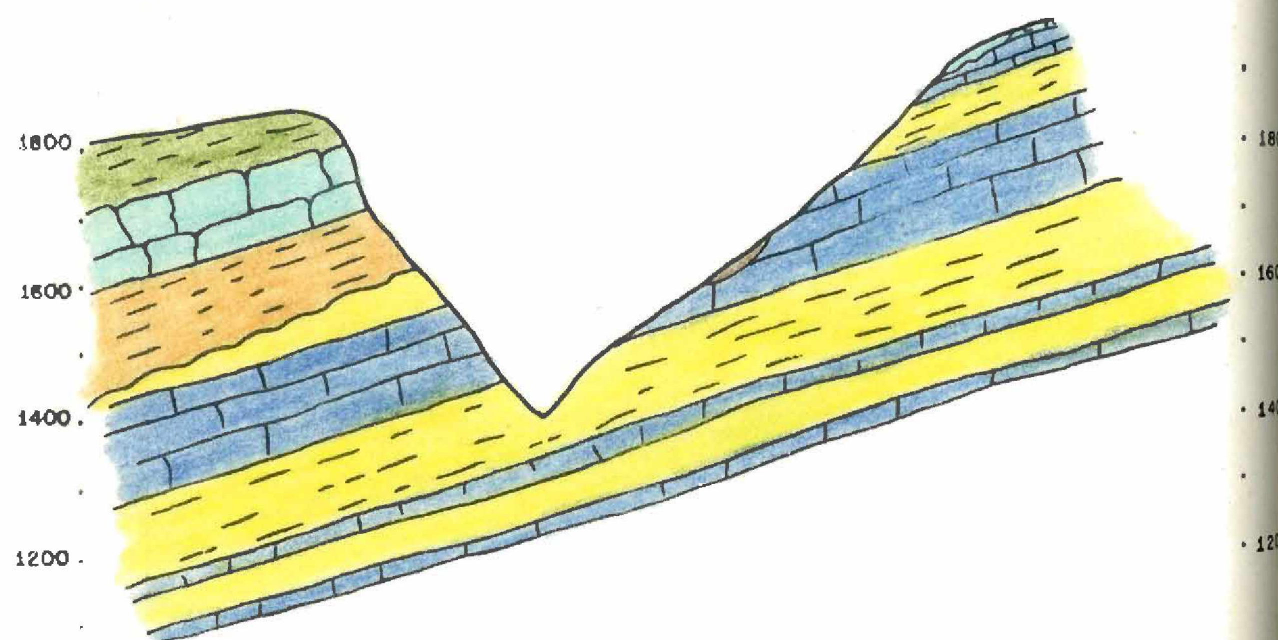
Looking downstream

SOLUTION IN MARBLE OUTCROP

Left bank of Aguila damsite



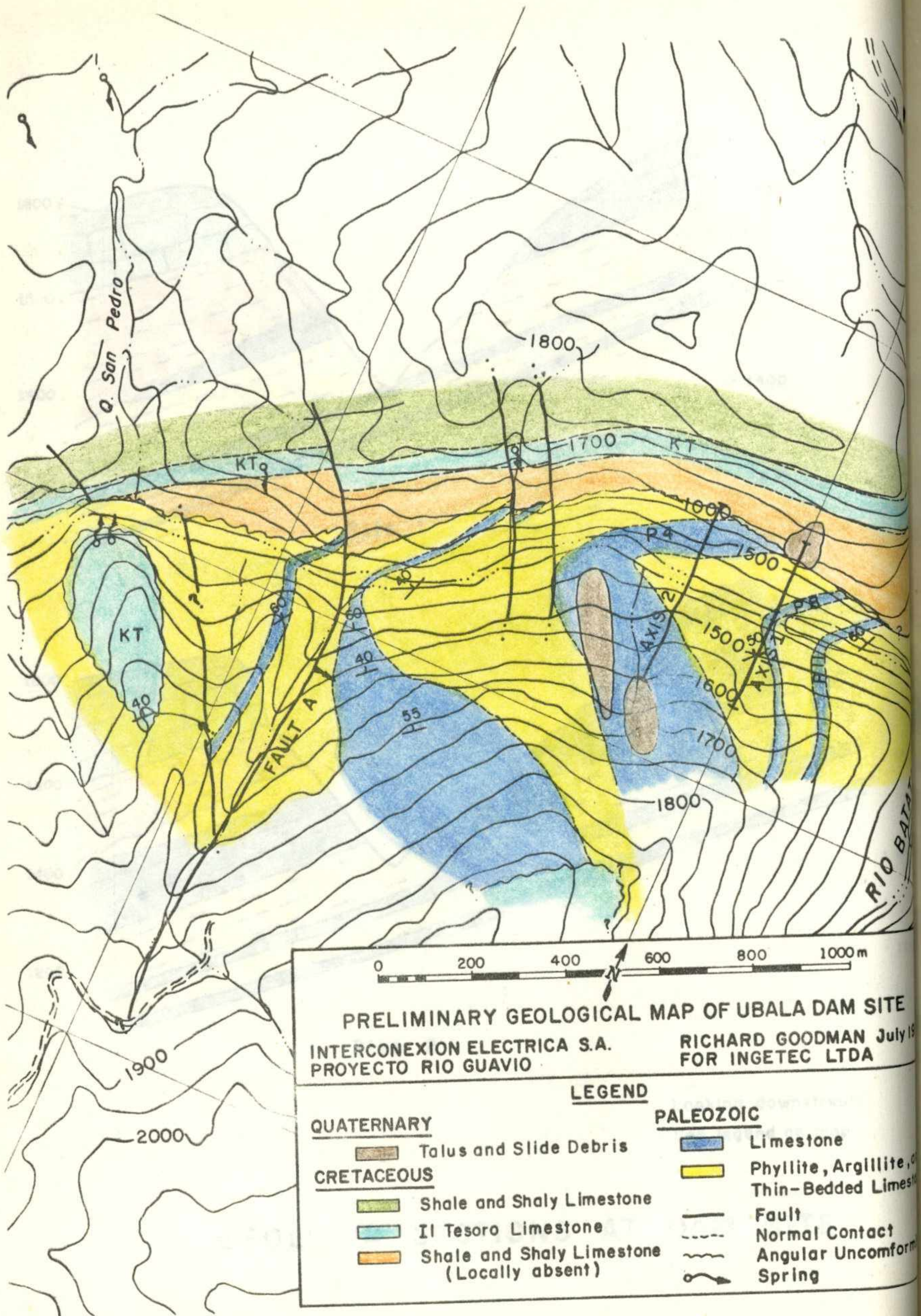
Site 1



Site 2

Looking downstream
See legend on map

GEOLOGIC SECTIONS AT DAM SITE


Q. San Pedro
1800
1700
KT
1000
1500
1600
1700
1800
1900
2000
FAULT A
AXIS 2
AXIS 1
RIO BATA
0 200 400 600 800 1000 m
N
PRELIMINARY GEOLOGICAL MAP OF UBALA DAM SITE
INTERCONEXION ELECTRICA S.A.
PROYECTO RIO GUAVIO
RICHARD GOODMAN July 19
FOR INGETEC LTDA
LEGEND
QUATERNARY
Talus and Slide Debris
CRETACEOUS
Shale and Shaly Limestone
Il Tesoro Limestone
Shale and Shaly Limestone (Locally absent)
PALEOZOIC
Limestone
Phyllite, Argillite, a
Thin-Bedded Limest
Fault
Normal Contact
Angular Uncomform
Spring

APPENDIX 1

GEOLOGIC MAP AND CROSS SECTIONS

INFORME 3

GEOLOGIA

January 21, 1972

Dr. Carlos Ospina, President
INGETEC
Apartado Aereo 5099
Bogotá, Colombia

Report of Visit to
Guavio River Project - Agüila Dam Site

Dear Carlos :

I visited the Aguila dam site on Monday and Tuesday, January 17 and 18 (1972). On Monday I studied the downstream portion of the left abutment with Ing. Chavez and Ing. Vela of Ingetec. On Tuesday, joined by geologists Monroy and Torres we traversed the right bank. The object of this visit was to review geologic mapping and to study rock exposures using new access created since my reconnaissance visit to the left bank in September (1971). The purpose was to recommend exploration procedures so that the quality of this site can be evaluated.

General geology of the site.

The Aguila dam site is just above the junction of Rio Batatas which enters the easterly flowing Guavio River from the right. The right (East) wall of the Batatas canyon is defended by a 70 - 80 meter thick ledge of crystalline limestone dipping gently to the north (Fig 1, layer A). A similar ledge, almost definitely the same member forms a cliff along the left bank of the Guavio canyon about 300 meters above the river. This ledge dips gently to the northwest at the dam site and turns to the west upstream, descending gradually to the river level at San Pedro creek. At first glance it might appear that ledges (Batatas right bank and Guavio left bank) are simply related by a uniform, continuous rock structure. But the rock block between the left side of the

Batatas River and the right bank of the Guavio River dips uniformly to the southwest (Fig. 1). The relations between these two blocks is not yet clear. They both seem to be formed of the same rock series crystalline limestone with calcareous argillite, and some chert and siliceous shale. The boundary along the Guavio canyon occurs in the left bank at about elevation 1550 meters. It is expressed by a striking angular discordance (line F. Figure 1). This line probably represents a fault. Along the Batatas creek a sharp anticline, and possibly a fault divide the two blocks. The regional structure will be more clear after access is completed to all parts of the site, data from trenches and test pits are obtained, and better aerial photographs are studied.

Exploration Objectives

The first phase of exploration consists of regional geological mapping at 1:10.000 scale. This is nearing completion by geologist Mario Monroy but should continue as access is improved by new trails and river crossings, and as new aerial photographs become available.

The second phase of exploration new beginning, consists of 1:1000 scale geologic mapping of the dam site axis. Data from outcrops will be supplemented by data from trenches and test pits, tentative locations for which were selected in consultation with Dr. Cajiao and Dr. Torres. The trenches and pits were located with the following objectives:

(1) to expose the rock and overburden on the left bank between the ledge A and the line F of figure 1. This strip is completely unknown geologically.

(2) to determine the orientation and nature of the surface whose trace is line F (see figure 1 and figure 2). If F is the expression of a horizontal plane, the material above it may be weak without seriously diminishing the quality of the site. If this surface is vertical, however, the majority of the volume of the left abutment remains, at this time, wholly unknown.

(3) to establish the nature of rock and depth of overburden between outcropping ledges on both banks.

In addition to test pits and trenches, it is necessary to have continuous access by trails to the river level, so that a complete stratigraphic section can be described and measured. A bridge or cable crossing near the dam site axis and a good trail on either of the banks are required.

The third phase of exploration will consist of diamond drill holes and exploratory adits on both abutments and below the river. The purposes of these measures are :

(1) to study the rock stratigraphy and rock character in depth for the full range of influence of a 220 meter high dam, (2) to furnish samples for laboratory testing; (3) to furnish holes for pump-in water pressure tests; and (4) to allow plate bearing and seismic velocity measurements within the rock mass.

It is hoped that any tendency for cavernous conditions to exist in the rock at Aguila dam site will be detected during the phase 2 and especially the phase 3 exploration. Small voids were noted in localized horizons and several springs occur. No caves were seen or are known by the local residents, and in my opinion there is no great probability that they exist at this site as karst topography is absent.

Geological Factors Influencing Type of Dam

It would be premature at this time to discuss the balance of advantages and disadvantages favoring one type of dam versus another. Let us note simply the impressions gained from two visits to the site.

The surface topography appears suitable for an arch dam and it is unlikely that the sound rock surface will deviate greatly from this shape. Large supplies of limestone probably suitable for concrete aggregate lie at a favorable position for quarrying, processing, and batching concrete on the left abutment of the dam. The rock that outcrops is generally massive and sound. Local problems of shale members, seams and voids could present themselves after detailed exploration. However the stratigraphy is such that it ought to be possible to improve the site details through relatively minor shifts in the position of the axis.

The topography does not make an earth or rock fill dam a natural choice. There is no easy spillway, and construction would be congested in the narrow canyon. Shell material could be obtained from member A but mining of large volumes might require extensive stripping of overlying shale. Core material is not easily available.

It would be appreciated if geological data reported from the field be forwarded to me routinely so that I can keep currently informed of the developing geological picture.

Very sincerely,

R. E. Goodman

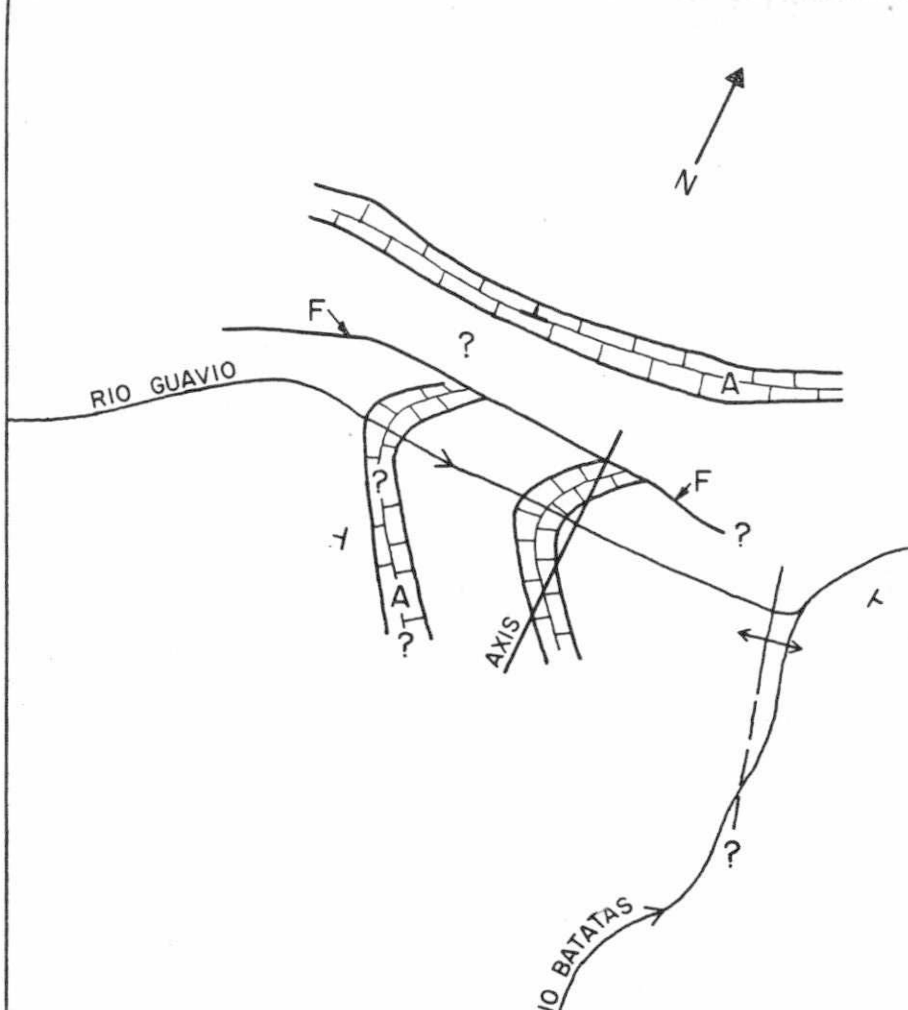


FIG. 1

DIAGRAMMATIC SKETCH MAP
LOOKING DOWNSTREAM

APPROX.
1:10,000

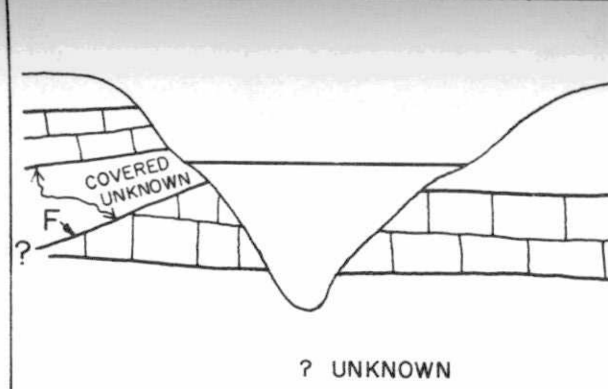


FIG. 2

DIAGRAMMATIC SECTION
LOOKING DOWNSTREAM

INFORME 4

GEOLOGIA

April 5, 1972

Dr. Hernando Quijano
Acting President
INGETEC
Apartado Aereo 5099
Bogotá, Colombia

RE: Guavio River Project

Dear Dr. Quijano:

I visited the Ubala dam site on Thursday March 23 and Friday March 25 in the company of geologists Camilo Torres, and Eduardo Moreno, and engineer Alberto Marulanda. On Saturday the 25 of March I chartered a light plane and over flew two alternative sites for the power station as well as the Upia and Humea dam sites. This is to report preliminary impression and ideas.

Ubala Dam Site (formerly called Aguila Dam Site) In January the geological mapping at the site had progressed to the point that it was possible to define principal exploration goals. The most important of these was unmasking of a zone extending from approximately elevations 1500 meters to 1650 on the left abutment; this belt, parallelling the contours, is grass covered and of gentle topography with very few outcrops. It is sandwiched between the bold north-dipping escarpment of limestone above and the west-dipping paleozoic rock mainly of limestones below. As noted in my previous report (January) either a fault of an unconformity separates these two geological plates. The belt in question on the left abutment can be the topographic expression of an unconformity or can represent a softer layer in the north dipping plate. I recommended that trenches and test pits be completed in this region. At the present time three trenches and one test pit have been completed in the left abutment with some additional exploration on the right. Logs and detailed geological maps in the region of the test pits are now being prepared by geologist Eduardo Moreno under the supervision of Dr. Torres and will be forwarded to me within several weeks time. Here I will only convey my general impressions of the materials exposed in these pits.

These exposures demonstrate that the region of interest is undesirable in many respects for the foundation of a high dam. There is evidence of considerable permeability for water pervious breccia, and karstified limestone. There is also badly weathered shale. The true nature of this zone is still not known nor the dip of its lower surface in contact with the right abutment plate. Based on the topography, it would seem that it is dipping gently to the west conformably with the overlying limestone; this favors the unconformity interpretation. It would be a more serious problem if the boundary between the west dipping plate in the valley bottom and the north-dipping plate above the left abutment were a steep fault as this would mean that the left abutment consists of a pillar of sound rock with unknown material behind it.

Whithout further data it is not possible to reach conclusions on the overall quality of this dam site. But my general impression at the present time is much less favorable than it was upon first inspection of the site last year. A pervious abutment is difficult to treat unless it is a small zone well definied by the program of exploration and bounded by impervious features. A continuous pervious zone can be treated by blanketing and drainage but this is not a pleasant prospect. Chief Joseph Dam on the Columbia River, a Corps of Engineers project, is a case history of successful analysis and treatment of a pervious abutment.

My impressions of the structural adequacy of the site are also not as favorable as I had hoped they would be. One would not want to put a heavy load on the types of materials exposed in these test pits without some kind of load distribution structure.

Reservoir Conditions. More serious potentially than the problems mentioned above at the dam site is the discovery of karstic conditions within the reservoir. The general absence of sink holes in the area gave me the first impression that the limestones were essentially free from caves. On my last trip we interviewed a farmer who claimed that there were no caves to his knowledge within the vicinity of the Guavio River. However, geological mapping has proved the contrary to be true; there are many caves within the bluffs of limestone outcropping within the reservoir some of large dimension. In my brief visit it was not possible to see most of these but I did inspect one very impressive feature. This is a subterranean portion of the Murca River. The Murca flows underground for a length of perhaps 150 meters in a tunnel of minimum dimension on the order of 16 meters. Inside the tunnel, which we completely traversed on a pebbly beach, it is possible to see evidence of cavernous joints. In an outcrop near the Murca tunnel we entered one cavernous joint to find a shaft of about 7 meters in height.

Implications of these Geological Findings. One should not prejudge the quality of the site without further investigations. There have been no drill holes yet and geological mapping around the reservoir is as yet incomplete. However it is difficult to emerge from the limited investigations with an optimistic view of the Ubala project.

The originally conceived Guavio project consisted of a storage reservoir at Gacheta dam site and a diversion dam at El Cobre. The recognition of a topographically superior site at Aguila Mountain above the confluence of the Batatas and Guavio led to a new conception for the Guavio power project embracing a single high dam combining regulation and diversion functions. It now appears that this critical section of the river contains cavernous formations complicating location of a regulatory reservoir upstream of El Cobre until above the confluence with the Farallones and Guavio Rivers. If this proves to be the case or if the Ubala dam site proves to be inappropriate for a high dam, it may be preferable to return to the Gacheta dam site scheme with a low diversion structure either at the Ubala site or downstream of El Cobre; in the latter case, one might consider an intermediate powerhouse.

Recommendations for Continued Exploration of the Guavio Project. Further trenching should be completed at the Ubala site to gain a better picture of the distribution of rocks in the left abutment. A 1:1000 scale map showing the geology of both abutments should be prepared. It is desirable to have some drill holes at this time high in the left abutment penetrating the previously discusses breccia and weathered zone. It is very important to develop a regional geologic map extending from Batatas creek confluence downstream well beyond El Cobre dam site. Geological mapping should therefore be accelerated. The present work of Eduardo Moreno forms a good basis for understanding the regional structure in the vicinity of the Ubala site. To accelerate mapping of the same scale downstream I recommend a second geologist be called to the area to complete the 1 : 1000 scale mapping and to log the trenches and test pit. A preliminary geological report showing the 1 : 1000 map of the site and the logs of test pits and trenches and the regional geology thus far should be produced for office use only in the near future (ca. one month).

It may seen somewhat outside of the scope of the geological mapping to elaborate the hydrology of the Guavio River. However as a fundamental question concerns water tightness, two programs of measurement seem appropriate : 1. observation of water levels along the reservoir rim in drill holes; and 2. stream flow data at a number of points along the Guavio, above and below the confluence of tributaries.

Aerial Reconnaisance of Powerhouse Sites. On March 25 I made an air trip with Camilo Torres and Eduardo Moreno. Clear weather made it possible to get a good view of the geology of Mambita. Three powerhouse alternatives have been considered. One is based on a tunnel line heading towards Medina; the other two are related to a tunnel line heading towards Mambita. One powerhouse is behind Mambita with a second along the Rio Trompeta (Algodones site). Both sites are in the same rock formation (the Quetame); the difference is in the relative lengths of tunnelling in various formations. The tunnel for Algodones powerhouse is about two kilometers shorter with the pressure tunnel six kilometers shorter and the tailrace 4 kilometers longer. The tailrace tunnel would be driven at a significant depth in what appears to be the Guadalupe formation, mainly sandstones and interbedded sandstone and siliceous shale. The Algodones arrangement seems better than the Mambita powerhouse which will require a tailrace underneath the Mambita terrace. From the air it appears that this terrace could be quite deep and in fact the tailrace tunnel could actually require tunneling in terrace materials. Drill holes and geophysical investigations would be helpful; however further exploration work at Mambita should await a decision about the location of storage and diversion reservoirs as discussed earlier. The third powerhouse arrangement is considerably to the west, in the vicinity of Medina. An underground powerhouse at Medina would not be satisfactory because it would be in the Guadalupe formation unless the powerhouse were placed at considerable depth as an intermediate station. In this event a very long access tunnel would be required. In flying from Mambita to Santa Maria it was possible to see the continuous belt of Quetame formation rocks extending from the Mambita and Algodones powerhouse locations right along strike to the mountain above Santa Maria. This is comforting.

Aerial Reconaissance of Upia and Humea Dam Sites. Unusually clear weather at these sites made the aerial reconaissance extremely rewarding. Both of these sites are on essentially the same hogback ridge of Teriary rock, apparently sandstone, dipping upstream. At the Upia site the rocks are dipping 30 to 40 degrees towards the upstream while at Humea the dip is somewhat less, perhaps 15 - 25 degrees. Humea has a more complicated topography being formed by more than a single thick bed. Upia, on the other hand, seems to be a simple hogback formed of essentially one hard ledge. Both sites appear to be free from landslides or other geologic problems and look remarkably good from this cursory inspection.

In order not to delay the submission of these preliminary impressions, I have not included photographs taken during the trip. A photographic appendix will be sent at a later date.

Very sincerely,

Richard E. Goodman

INFORME 5

GEOLOGIA

PRELIMINARY GEOLOGIC REPORT

UBALA DAM SITE, GUAVIO PROJECT

FOR INGETEC LTDA.

Richard E. Goodman, August 1972

PRELIMINARY GEOLOGICAL REPORT OF UBALA DAM SITE*

GUAVIO PROJECT

1. INTRODUCTION

The Ubalá dam site is located on the Guavio River 500 meters upstream from its junction with Rio Batatas. This site will be developed for storage of Guavio River waters nourished by flows from the Murca River, the Farallones River and other tributaries. As first conceived the Guavio project consisted of a storage reservoir at the Pingaro site downstream from Gachetá, and a low diversion dam at El Cobre to conduct water into the power tunnel. The recognition of a topographically superior site above the confluence of the Batatas and Guavio Rivers offered an alternative power scheme consisting of a single dam for both regulation and diversion functions. The remarkable topographic aspects of the Ubalá dam site are not matched by equally excellent geological attributes. However it now appears that a high dam at Ubalá is feasible and it is recommended that continued exploration be pursued as described later. The purpose of this report is to summarize findings about the quality of the site at this moment, one year after its original selection for study. Reference is made to my letter reports dated July 24, 1971, September 20, 1971, January 21, 1972, and April 5, 1972 as well as a preliminary geological report dated June 1972 prepared by the soils department of Ingetec.

The basis for this report consists of a review of previous work cited above and a visit to the site between July 7 and 10, 1972. During this visit a geological map, attached to this report, was prepared. This map presents my interpretation of the regional geological picture in the vicinity of Ubalá dam site based upon surface outcrops and three test pits completed along dam site axis # 1 (1,066.00E). Five of these trenches and one test pit are on the left abutment while the remaining three trenches and test pits are on the right abutment directly opposite.

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* Previously called Gachalá and Aguila Dam

The organization of the report includes: 1) a discussion of the general geology of the region; 2) detailed attributes of the specific dam sites; 3) recommendations for further exploration; 4) a discussion of reservoir leakage possibilities; 5) a listing of factors influencing the selection of the type of dam; and 6) a summary of recommendations for further exploration. A geological map and two cross-sections are appended followed by 13 photographic plates. Appendix 3 is a discussion of simple permeability tests that can be conducted in exploration drill holes. This appendix was prepared by a colleague, Dr. Tidu Maini. Appendix 4 is a reprint of an article on bore hole deformability measurements.

2. GENERAL GEOLOGY OF THE REGION

The geology of this portion of the Guavio River canyon may be termed "layer cake geology". Two systems of rocks are separated by an angular unconformity. High above the walls of the canyon and occasionally plunging into the canyon itself is a system of rocks of Cretaceous age, characterized by shales and an approximately 100 meter thick limestone formation. This Cretaceous limestone has been named the Tesoro formation, the name deriving from a legend about a treasure hidden in a cave near the arrival of the Río Batatas. Underlying the Cretaceous rocks and separated from it by an angular unconformity is a sequence of Paleozoic rocks easily distinguished from those above by its fossiliferous nature and frequent slaty cleavage (see Plate XIII, photograph 4). (The following fossils are present: Chonetes, Archimedes, Spirifer and unidentified Crinoid plates). The Paleozoic suite of rocks consists of recrystallized, occasionally marblized, limestone, argillite, phyllite, and calc-arenite and calc-conglomerite. Reference is made to the geological map to be found at the end of the report in Appendix 1. It will be seen that there are four, possibly five prominent limestone ledge layers. The structurally lowest of these is a calc-conglomerate exposed along the spur on the right bank at the confluence with Rio Batatas. The next, labelled (P-11) on the map, and the third, labelled P-8, are thin bedded limestone layers. P-4 is a thicker unit with massive beds of marble. The rock in between these limestone units, colored blue on the map, is also competent, hard rock although it is composed of low grade metamorphic rocks derived from clays and calcareous clays. These members, colored blue on the geological map, are characterized as ridge making units, that is they are relatively more competent than the yellow colored units.

The Paleozoic rocks strike generally to the northeast and dip the northwest at 40 to 60 degrees. There are numerous small structures within the Paleozoic rocks and the detailed structural relationships have not been worked out. The Cretaceous rocks above dip to the north at a much lower angle, about 15^{o}.

The unconformity between the Paleozoic and Cretaceous rocks does not follow the exact structural orientation of the Cretaceous layers but instead rises and falls since it represents the former surface of the land. The topography of the erosion surface developed on the Paleozoic rocks was such that there was a high point, or a high ridge overlying what now constitutes the right bank of the river. The topographic surface on the Paleozoic sloped to the north and east. Opposite Q. San Pedro on the right bank, it will be seen that the unconformity directly underlies the Tesoro limestone, whereas farther to the east the Tesoro limestone rests on some 100 meters thickness of Cretaceous shales with the unconformity below. The disappearance of the Cretaceous shales is interpreted as a stratigraphic "wedge-out" against a rough surface.

Plate V presents two excellent exposures of the angular unconformity and a third which is inferred. In figure 1, of Plate V the Cretaceous limestones rest with angular discordance upon the Paleozoic rocks in the left bank of the Guavio just upstream from the confluence with the Farallones River. A very similar appearing outcrop in Figure 2 is just upstream from the dam site at Quebrada San Pedro. In contrast to these photos in which the Tesoro limestone is resting directly on the Paleozoic rocks, Plate I shows the Cretaceous overlying the Paleozoic at the dam site itself. The unconformity is shown by the inked line on the photos. It can be seen that there is a thick sequence of grassy slope between the bluffs marked in the outcrop line of the limestone and the Paleozoic rock below. The unconformity surface is itself folded, the folding evident from the structural attitudes of the Tesoro limestone throughout the region. Generally the Cretaceous rocks and the unconformity are topographically high. For example, at the dam site the uncorformity outcrops around 1600 meters elevation on the left bank and at 1900 meters elevation on the right bank. Downwarping of the unconformity into a broad synclinal structure has brought the Tesoro limestone to river level at several points, most notably opposite Quebrada San Pedro as shown on the western edge of the geological map and in the reservoir area along the valley of Rio Murca. A very preliminary geological map of the reservoir area is contained in the Ingetec Preliminary Geological Report of June 1972 previously noted. This structural interpretation has ramifications for the storage capacity of the reservoir as will be discussed later.

The rock is not extensively faulted, however at least three minor faults have developed since the Cretaceous as shown on the geological map. These are all north-south trending, eastward dipping reverse faults. Plate IV presents ground photographs of the three post-Cretaceous faults shown on the geological map. A fourth fault shown to the west of fault A is inferred to be older than Cretaceous. However the extensive soil cover in this area makes it difficult to confirm its existence or its structural relationships.

The geological map presented reflects an interpretation based on rather limited opportunity to observe good outcrops in the region. It should be noted that other structural interpretations are possible so that this geological map may be refined as the project develops.

3. GEOLOGICAL CONDITIONS AT THE DAM SITE

The first site considered is labelled axis 1 on the geological map. It is located along the north-south line corresponding to coordinate 1,066.000 east. The geologic cross-section along this line (Appendix 1) shows that a dam to crest elevation 1610 meters would be at least 60 meters above the unconformity. This potentially troublesome region of the left abutment was investigated with several trenches. Plate II shows the left abutment at axis 1. It was not appreciated at the time explorations were first laid out that a slide involving one large loose block of limestone and considerable debris was masking the crest elevation of the dam. Plate XII presents photographs of material exposed in trench 6 in this region. The upper part is occupied by a mixture of karstified limestone blocks and soil. The lower portion is a deep soil material. Test pit 3 in the same vicinity was 5.8 meters deep and not clearly in bedrock at this depth. The karstified knob of limestone shown in figures 1 and 2 at the base of the test pit may well belong to a loose block. This material would have to be removed for the dam; what underlies it has not yet been determined. Its exploration awaits the program of drilling and underground excavations.

Two hundred meters west of axis 1, a dam site can be developed which seems in many respects better than site 1. At this site, denoted axis 2, the unconformity reaches its highest elevation of any point on the left bank -- above 1600 meters; therefore it is possible to locate almost the entire dam on Paleozoic rock. An arch dam here would rest almost entirely on a thick bedded limestone member within the Paleozoic as shown in the geological cross-section, (Appendix 1). Plate II, figure 1, shows the location of this site at A. The left abutment is heavily covered with brush but the massive rock can be seen cropping out here and there between the bushes in the very steep valley side below the unconformity. Plate III is a photographic mosaic of the right abutment at this site. As can be seen, the right bank presents restricted siting possibilities. If an arch dam were to be constructed, the site could not move very far downstream from A before it would run off of the outcrop of thick bedded limestone onto the heavy soil-covered argillite with less satisfactory topography and rock conditions.

Just upstream from letter A (Plate III) the right bank is occupied by a reentrant formed presumably on phyllite and argillite layers and covered by extensive debris. At location B the topographic expression and photographic appearance suggest a mound of deep debris which presumably slid from the scallops in the ridge line above. The spur underlying the letter D is composed of loosened blocks of karstic limestone. This is probable the remains of a talus pile at the base of an ancestral cliff.

Plate III contains evidence of an additional potential problem in the right abutment upstream of the axis at point E. The bending of the layers is interpreted as a product of buckling of the layers composing this dip slope. Because the topographic surface closely follows individual bedding planes, the weight of each layer is directed essentially along its plane; this converts the slope into a succession of inclined "columns". Two possibilities emerge from this observation. Due to outward flexing there may be gaps between layers and therefore large grout takes are possible (in addition to those related to cavernous conditions within the rock). Also, the slabs of rock in the valley side may become delodged due to the action of water so that landsliding could prove a nuisance.

For these reasons on the right bank it would not be desirable to move the dam site upstream as far as E or downstream much below A. Furthermore the left abutment becomes less desirable at other locations. It therefore seems warranted to consider site 2 only at the axis shown. It is further recommended that site 1 be discontinued and that exploration of site 2 begin now. It should be noted that there has been virtually no exploration at this location other than the making of trails. The previous trenches add to the general understanding of the site in that they expose the stratigraphy and the range of conditions. But now it will be necessary to add another series of trenches and test pits on both abutments to determine the character of the bedrock and the depth of soil cover.

4. STAGE II EXPLORATION AT THE DAM SITE

Further exploration, predominantly at site 2, will have the following goals: 1. To determine the stratigraphic succesion as far as possible, on both abutments; 2. To determine the nature and attitude of the unconformity and its water carrying properties; 3. To search for possible water conduits in the Cretaceous rocks above the unconformity; 4. To evaluate the water permeability of fractures, vugs, and caverns in the limestone below the unconformity; 5. Ultimately it will be necessary to determine the depth to rock below the river itself.

At this stage, further trenches and pits along the axis of site 2 will be very helpful. But in addition it is necessary to execute a series of drill holes, and ultimately to drive several adits on each abutment. The drill holes should be at least NX (three inches diameter) at the collar. Pump-in pressure tests will be very helpful but it is extremely important that the results indicate the true water losses from the packed-off test section into the formation rather than leakage around the packers. Appendix 3 is a discussion of water permeability testing in drill holes prepared by Dr. Tidu Maini. Dr. Maini suggests that the packers should be at least 2 meters long ensure that there not be any leakage past the packers. However it is possible to check for this by a simple graphical device described by Dr. Maini in his Section D and figures 3 and 4. It is also of interest that for pump-in pressure tests above the water table, Dr. Maini suggests that the water flow should be continued for three hours before taking any reading of water loss or pressure. A novel alternative to the usual pump-in pressure tests (Lugeon tests) is to conduct falling head tests in the drill holes. These types of tests do not require extensive equipment; the test consists simply of filling up the hole to an artificially high level and making a record of the fall of this level with time. This type of test can also be done in test pits.

Since it is contemplated to consider an arch dam resting on the Paleozoic rocks, it would be helpful now to have information on the rock deformability and strength variations in the foundation. Samples of rock obtained from the drill holes should be tested thoroughly in a rock mechanics laboratory. If possible, the field moisture content of the samples should be preserved. The samples may be shipped to Berkeley for testing or arrangements can be made at a laboratory in Mexico or in France if preferable. The walls of the bore hole also offer an opportunity for an introductory sequence of tests of the mechanical characterists of the rock. It would be possible to conduct plate bearing tests within the NX borehole using a borehole jack rented from the Slope Indicator Company, Seattle Washington. This is a very simple test which can be conducted in each drill hole to a depth of several hundred feet. Another possibility is to use a dilatometer type device. An article summarizing various types of borehole deformability measuring instruments is attached to this report as Appendix 4.

Finally in connection with explorations within the drill holes themselves, it would be very useful to know if water is entering or leaving the drill holes from particular horizons. There are several methods of determining this. One is to measure vertical flow velocity within the borehole by lowering down the hole a propeller, mounted on a horizontal shaft. Depths at which the velocity abruptly changes indicate depths at which water flows into or exits from the borehole. By calibrating the device in a pipe of the same size as the drill hole it is possible to determine how much water is entering or leaving. Such a device was used on the investigation of the Grand Rapids project by H. R. Grice. This was a complex of dikes and dams on cavernous dolomites. Another possibility is to observe the rate of decay of a sudden charge of salt introduced between packed-off sections of the borehole. If there is interest in this, I can invite Dr. Tidu Maini to prepare a memorandum on procedures for this type of test.

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5. RESERVOIR LEAKAGE POSSIBILITIES

Problems at the dam site have been discussed; the question now is whether or not a reservoir to elevation 1600 meters can be maintained behind a dam at the Ubalá site. There is ample evidence of serious karstic phenomena in the rocks at low elevations within the reservoir, as discussed in my report of April 5, 1972. Observations of caves in the Río Murca led me to a pessimistic evaluation of the reservoir site at that time. However geological work since the April 5 report has returned a more optimistic evaluation. It seems that most of the cavernous condition is confined to the Cretaceous rocks. While the Paleozoic rocks present vugs and small caves, the large caves and springs, to the best current knowledge, are confined entirely to the rocks of Cretaceous age. The importance of this statement is great since the Cretaceous rocks are generally high above the reservoir except in specific downwarpings, as previously noted; thus caverns in the Tesoro limestone member at river level are always in low structural positions and have to climb to conduct water away from the reservoir. Numerous springs, indicated on the geological map (Appendix 1) indicate that water is currently flowing towards the Guavio River through subterranean routes. See plates IX and XI (figure 1). If the reservoir is to fill up the low-lying portions of the Tesoro limestone, it will create a bank storage effect. But, will it induce leakage ? It is questionable that there could be significant leakage as there is no place for the water to go. The water could not flow over ground water divides above the reservoir elevation. The only possible leakage paths would be through some pathway of subterranean channels into caverns emptying on the west banks of Río Chivor and Río Batatas. It is recommended that very careful geological examination of these areas be made, if necessary cleaning large areas of rock to explore for caverns.

The contrast in perviousness and cavernessness of the Cretaceous and Paleozoic rocks is documented in Plates VI through X. In Plate X is shown the manner is which the Paleozoic limestones dissolve. Plate X, figure 1, shows a small cave of unknown depth in one stratum of Paleozoic limestone and small caverns along cross-joints along another stratigraphic member. Figure 3 in Plate X shows smaller vugs in yet another Paleozoic limestone member. Vugs and open joints are likely to be distributed widely in the Paleozoic limestones. Along faults which cut these rocks, larger caverns are possible. The karstic characteristic of the Cretaceous Tesoro limestone is much worse, as exemplified by Plates VI - IX. Figure 1 of Plate VI shows the cliff of the Tesoro limestone in the erosional outlier opposite Quebrada San Pedro on the right bank. On this cliff can be noticed a number of black dots; these seem to be portals of caverns. Furthermore there are numerous stalactites hanging from the cliff. It seems that this region of the Tesoro limestone is honeycombed with caverns. Plate VII shows the natural tunnel through which the Murca River flows underground for more than 100 meters. Plate VIII shows the entrance to a small gallery in the left bank near fault A. As shown in Plate IX a significant flow, perhaps 20 liters per second, comes from the head of the cavern discharging into the Guavio canyon. There are many other caverns no doubt which have not been shown in these photographs. However, unless the Paleozoic rocks can be proved to be cavernous, the outlook is optimistic for reservoir storage without serious complications; it is recommended that the high dam scheme at Ubalá site be retained for the present.

6. FACTORS INFLUENCING THE SELECTION OF THE TYPE OF DAM

The steep topography and narrowness of the Guavio canyon suggest that an arch dam may well prove to be the most economical solution for this site. A factor contributing to the economics of an arch dam is the lack of a natural saddle spillway, which really is preferable to a tunnel spillway for an embankment dam. If an earth dam or rock fill dam were to be constructed, it is not clear which material will be best for its construction. A rock fill dam would surely have to be constructed with quarried rock; the most plentiful rock in the region is Tesoro limestone; it is not clear whether its characteristics would lend themselves to a dam without continuing settlement owing to solution at points of contact between rock particles. This can be tested in a triaxial or plane-strain testing vessel under constant load with Guavio River water circulating the pores. On the other hand, for an arch dam the limestone could probably offer suitable material for concrete aggregate. The question of abutment load carrying capacity is particularly acute on the left side in the vicinity of the unconformity. At axis 1 it might be necessary to construct a gravity abutment or load distributing structure in the upper portion of the abutment. However, at the new site it is possible that an arch could be carried all the way up. It is too early to discuss the suitability of the rock foundation. Superficially the rocks appear appropriate for consideration in the foundation of a structural dam. It will be necessary to complete the program of drill holes and, more importantly, adits and cleaning of the rock walls under the arch before any decision can be made on this point. For any type of dam grouting will surely be necessary at this site and grout takes will be high. It is recommended that both arch and rock fill structures be estimated for the Ubalá site.

7. SUMMARY OF RECOMMENDATIONS

1. Move site to "axis 2" along line 1,065.800 E. (200 meters west of site 1).

2. Explore axis 2 with trenches and test pits on both abutments.

3. Drill NX boreholes at site 2. The holes should be 400 feet deep if possible. Representative samples should be shipped to me for laboratory testing. Also falling head tests and Lugeon water loss tests should be conducted during drilling. Some dilatometer or borehole jack tests should also be conducted to probe the deformability of the rock on the borehole walls.

4. A geologist should be assigned to log the trenches, pits and drill holes semi-continuously during exploration. He should reside at the field camp in Gachalá throughout the exploration period.

5. The west banks of Río Chivor and Río Batatas below elevation 1600 meters should be examined carefully to search for possible outlet conduits from the reservoir.

6. Access to the left bank of the dam site should be improved.

7. Both arch and rock fill structures should be cost estimated at this time.

APPENDIX II

PHOTOGRAPHIC PLATES

PHOTOGRAPHIC PLATES

PLATE I

The canyon of the Guavio River

1. Looking downstream towards dam site

2. Looking upstream towards dam site.

The unconformity between the Paleozoic rocks below and the Cretaceous rocks above is indicated in both views by the wavy line. The new axis for the Ubala dam site is at the highest point of the unconformity in the left abutment as shown by the letter B. Point A is the same location in both photos.

PLATE II

Views of the left abutment at the old and new Ubala Dam site axes.

1. (Upper) General view of both old and new axes.

   A. Crest of dam site at new axis

   B. Crest of dam at old axis

   C. Loose block of limestone

   D. Trench # 4

   E. Trench # 7

2. (Lower) Detailed view of lower portion of left abutment. Identifing letters have the same meaning as given above.

PLATE III

View of right bank of Guavio River at the new Ubala Dam site.

A. Crest at new axis

B., C, D, and E indicate preliminary geological interpretations as follows:

B. Deep slide debris which will have to be removed for construction.

C. Moderately deep debris and blocks.

D. Blocks of limestone, very deep

E. Buckled strata of limestone on the dip slope; gaps between the beds may exist as discussed in the text.

PLATE IV

Faults which cut across the Cretaceous strata and therefore must offset the unconformity.

1. (Left) A, B, and C designate faults which are shown on the geological map.

2. Closer view of fault A on the left bank of the Guavio River.

3. (Right) Inferred trace of fault A on the right bank of the Guavio River as shown on the geological map.

PLATE V

Exposures of the unconformity between Cretaceous and Paleozoic rocks in the Guavio canyon.

1. (Upper left) and photo 2 (upper right) offer convincing evidence of the unconformity interpretation for the regional structure of the Guavio canyon. In order to avoid marking the geological details the line of the unconformity has not been marked. However two points along the surface are shown in each of these photos by arrows indicated by the letter U. The Cretaceous is

the Tesoro limestone; the Paleozoic rock below in angular unconformity is a slightly metamorphosed suite of limestones and argillites. The rock in the foreground of photo 2 is in the Paleozoic series.

3. (Lower left) the top of the hill is believed to belong to the Cretaceous suite while the rocks below are Paleozoic, with the unconformity along the line shown. This point has not been visited in the field and the line is only interpreted.

The locations of the photos are as follows:

1. Above the left wall of the Guavio canyon just above the junction with the Farallones River.

2. Taken from the right side of the Guavio River opposite Quebrada San Pedro.

3. On the right abutment of the new dam site (compare with Plate III).

PLATE VI

Solution effects in the Cretaceous limestone (Tesoro Formation)

1. Cretaceous limestone of the Tesoro formation exposed on the right side of the Guavio River opposite Q. San Pedro. The black dots on the cliff are portals of caves. Notice the stalactites.

2. A view of the Murca Valley above a natural tunnel shown in Plate VII. The prominent valley may be a collapse feature formed by subsidence over a previous tunnel of the Murca River. The Murca River now flows in a natural tunnel under the dashed line. The tunnel is shown in Plate VII.

PLATE VII

A natural tunnel of the Murca River in the Cretaceous limestone.

1. Entrance to the tunnel from upstream.

2. and 3. Inside the tunnel looking upstream.

4. Inside the tunnel looking downstream. The tunnel emerges into a natural shaft and crosses under a natural bridge returning to its open air valley.

PLATE VIII

Solution effects in the Cretaceous limestone - Tinajo cavern. This cavern is located below fault A (see geological map) in the left side of the Guavio canyon at about 1570 meters elevation. See also Plates IX.

PLATE IX

Inside the Tinajo cavern

1. Water flows from the rear of the accessible portion of the tunnel, emanating from a continuation to the left.

2. Looking from the rear of the tunnel towards the portal.

3. The portal of the tunnel, from within.

PLATE X

Solution effects from the Paleozoic limestone along the bed of the Guavio River upstream from the dam site.

1. Two cavernous strata are shown in the left bank.

   A. Is a cave, probably shallow, confined to one stratum

   B. Is a stratum dissolved along cross joints.

2. A closeup of the cave A of photo 1.
3. Large vugs.
4. The appearance of stratum B on the right bank.

PLATE XI

Views into the reservoir area.

1. San Pedro Valley on the left bank of the Guavio. Springs in the Cretaceous are marked by the arrows and the letter S. The springs probably issue from caverns in the limestone layers above the Tesoro limestone in the Cretaceous rocks.

2. View upstream towards the junction with Rio Farallones in the distance; Murca Valley to the left.

PLATE XII

Test pits and trenches.

1. Test pit # 3 from the left bank at the old axis 5.8 meters deep; a karstified block of limestone is visible at the bottom of the pit.

2. A closer view of the knob of limestone.

3. Trench # 6, lower portion; deep soil with blocks of limestone.

4. The upper part of trench # 6 showing blocks of limestone.

PLATE XIII

1. The canyon of the Guavio River upstream from the confluence with the Farallones River, formed from Paleozoic and Cretaceous limestones. Unfortunately there is only small storage behind a high dam at this site.

2. Quebrada Moncobita, at its confluence with the Murca River in the reservoir area of Ubala dam site. The shales in this region contribute large sediment load.

3. A trail under construction along the right bank downstream from the Ubala dam site. The Paleozoic rock possesses jointing and foliation which tend to be loosened by the trail making operations and normal weathering on the valley sides.

4. Slaty cleavage in the phyllitic rocks near photo 3.

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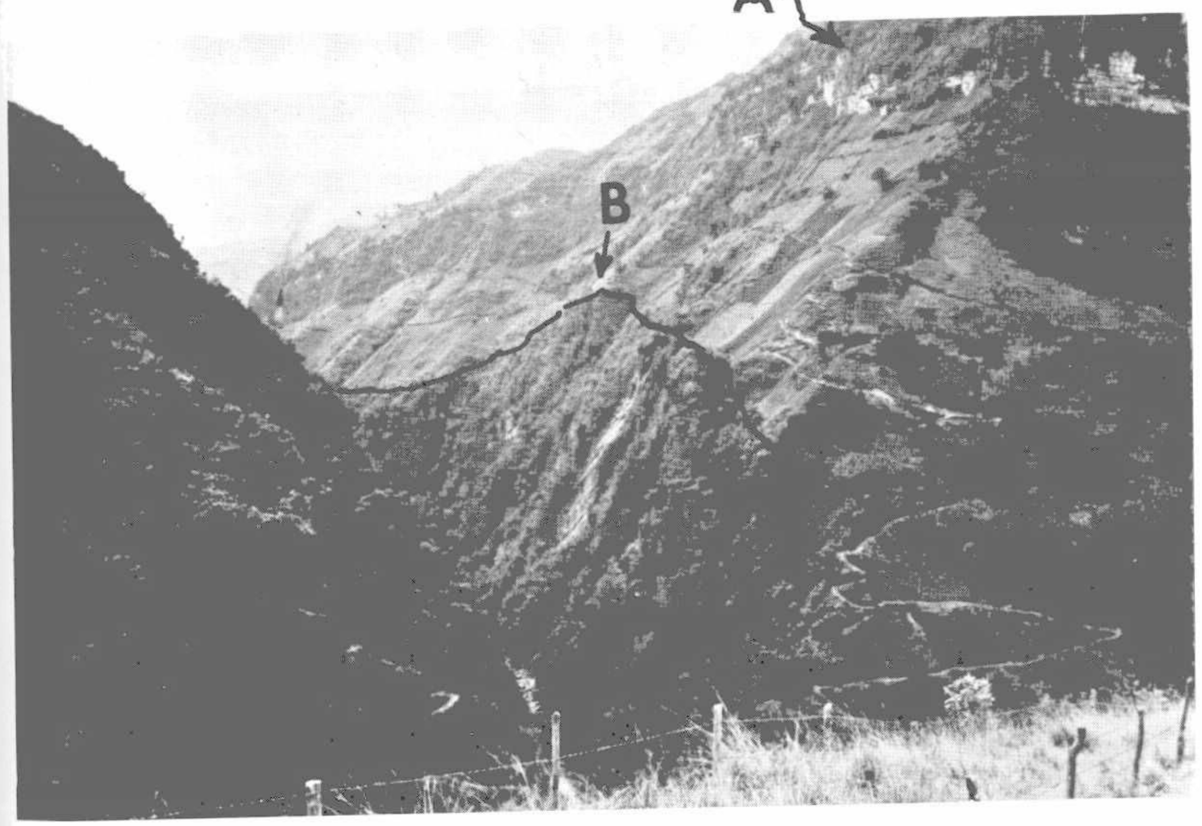

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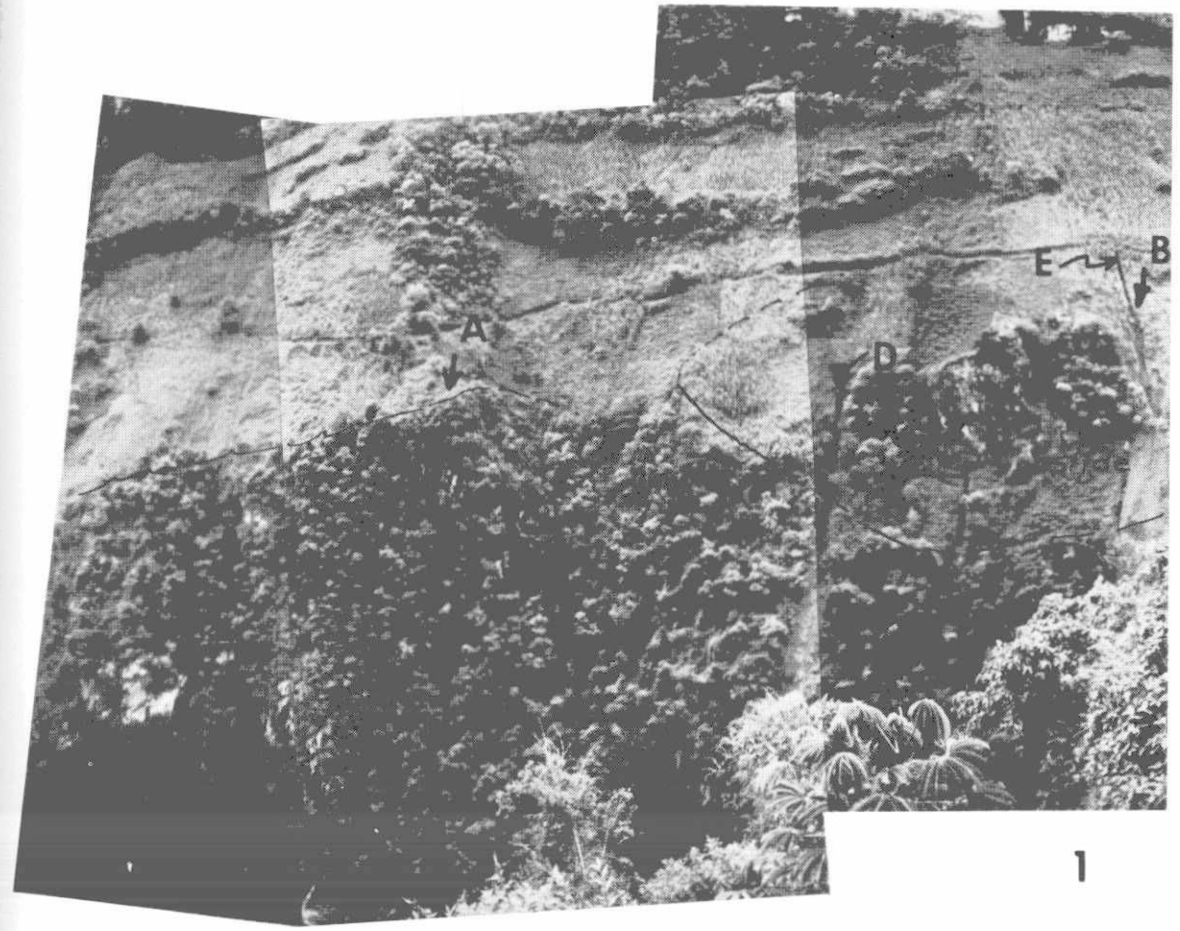

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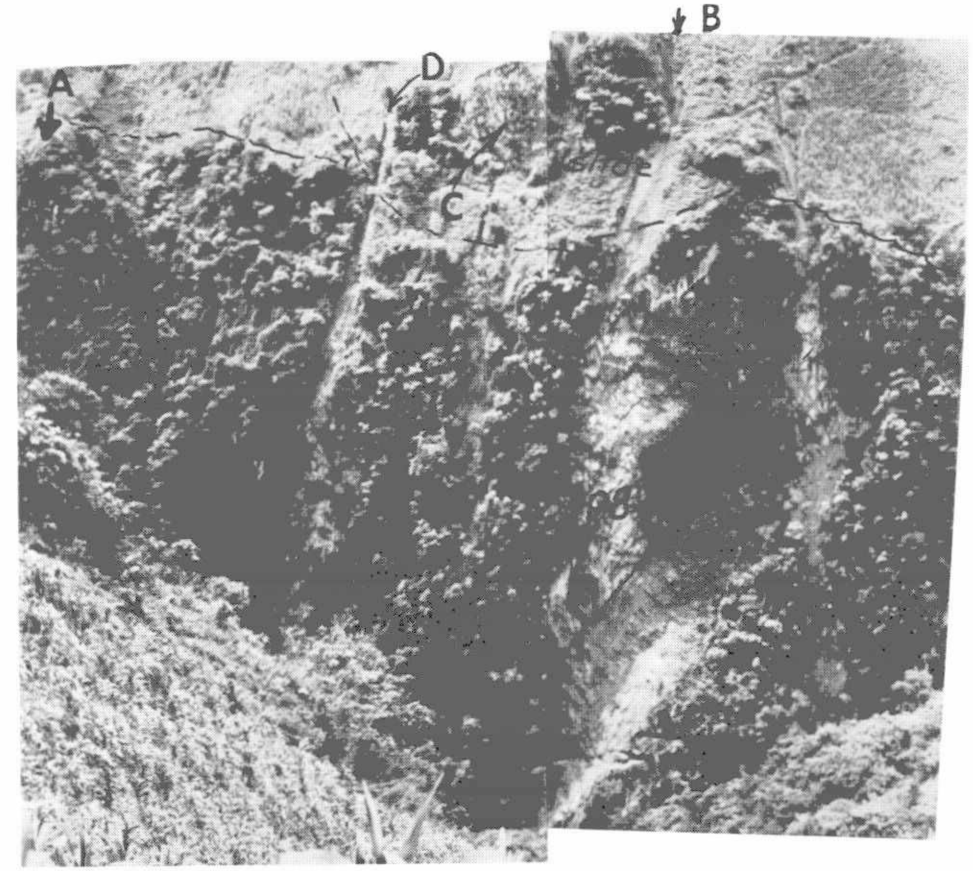

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PLATE II

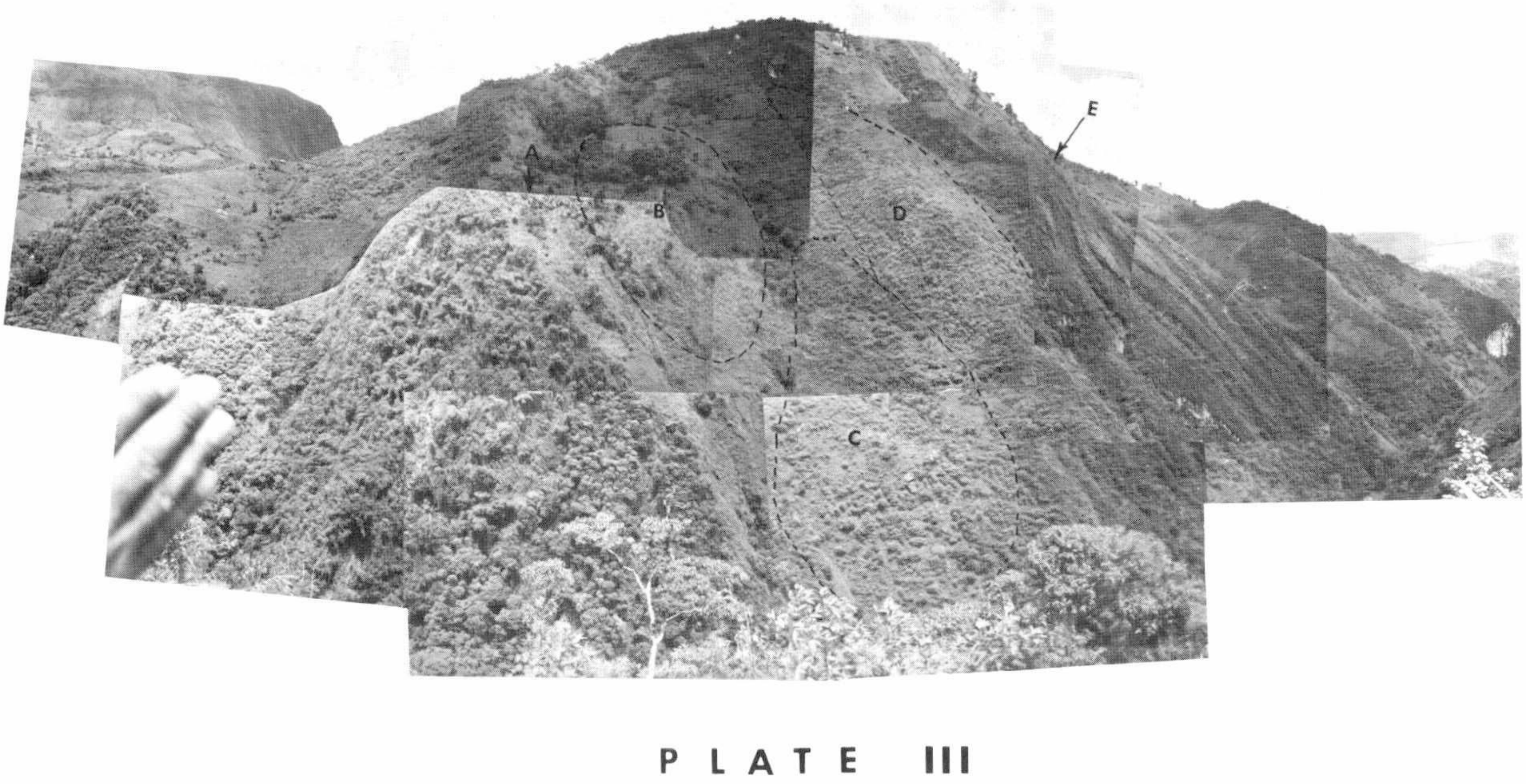

PLATE III

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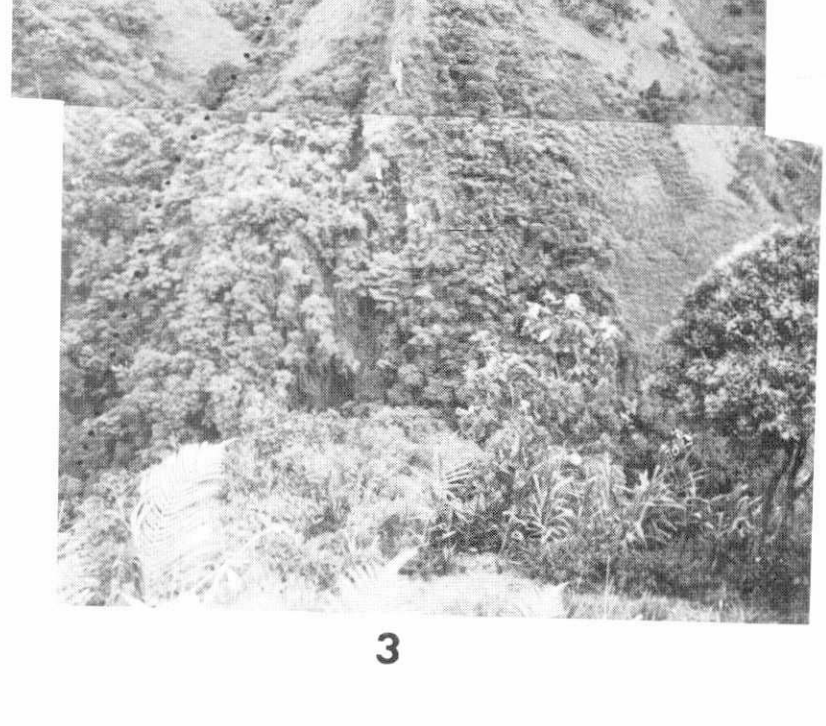

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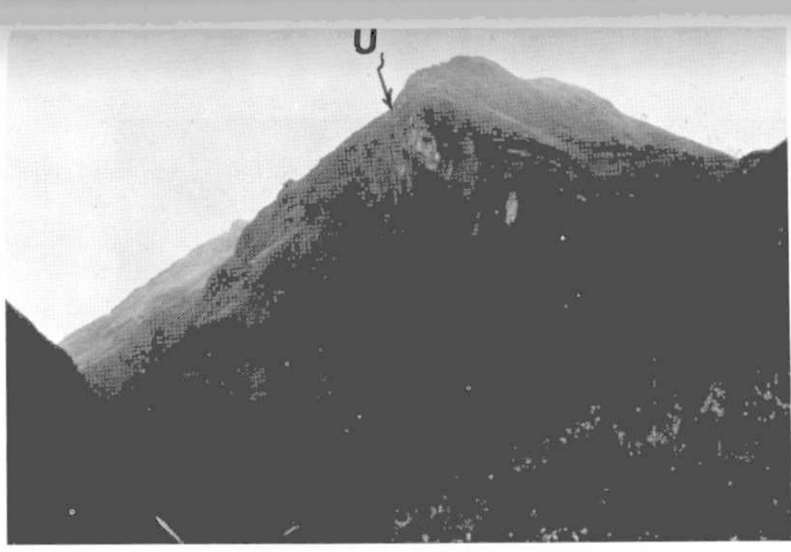

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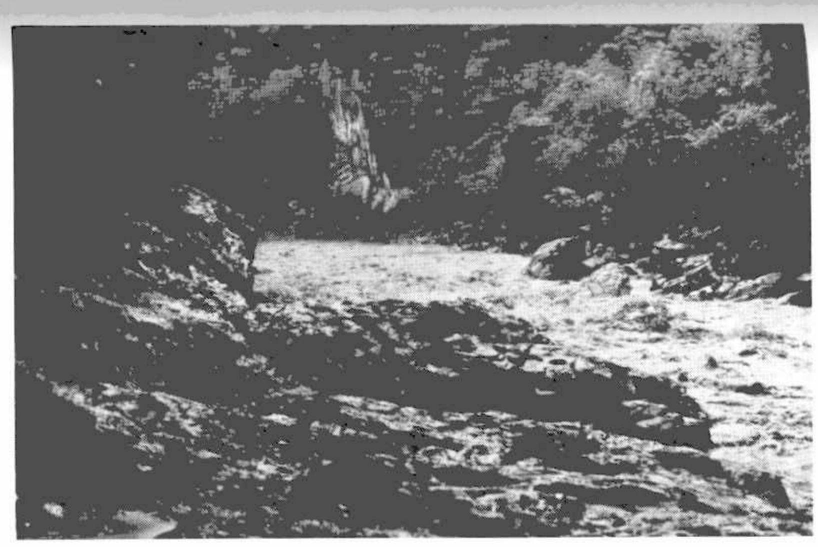

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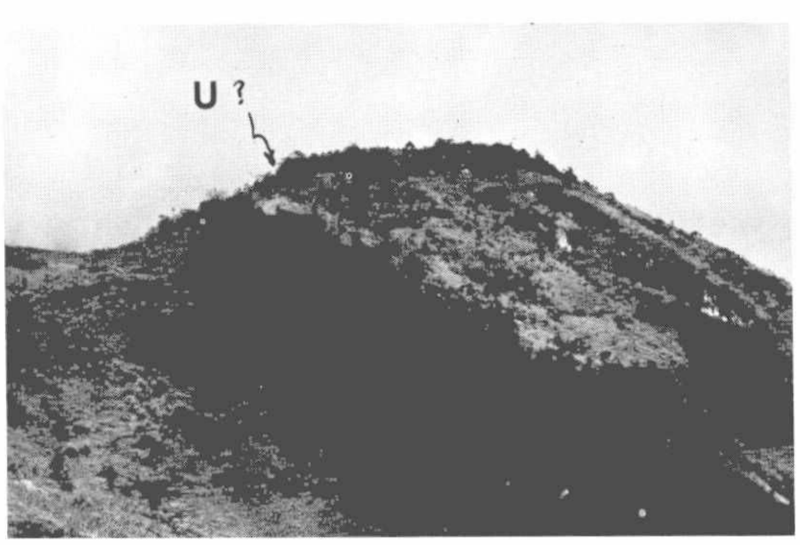

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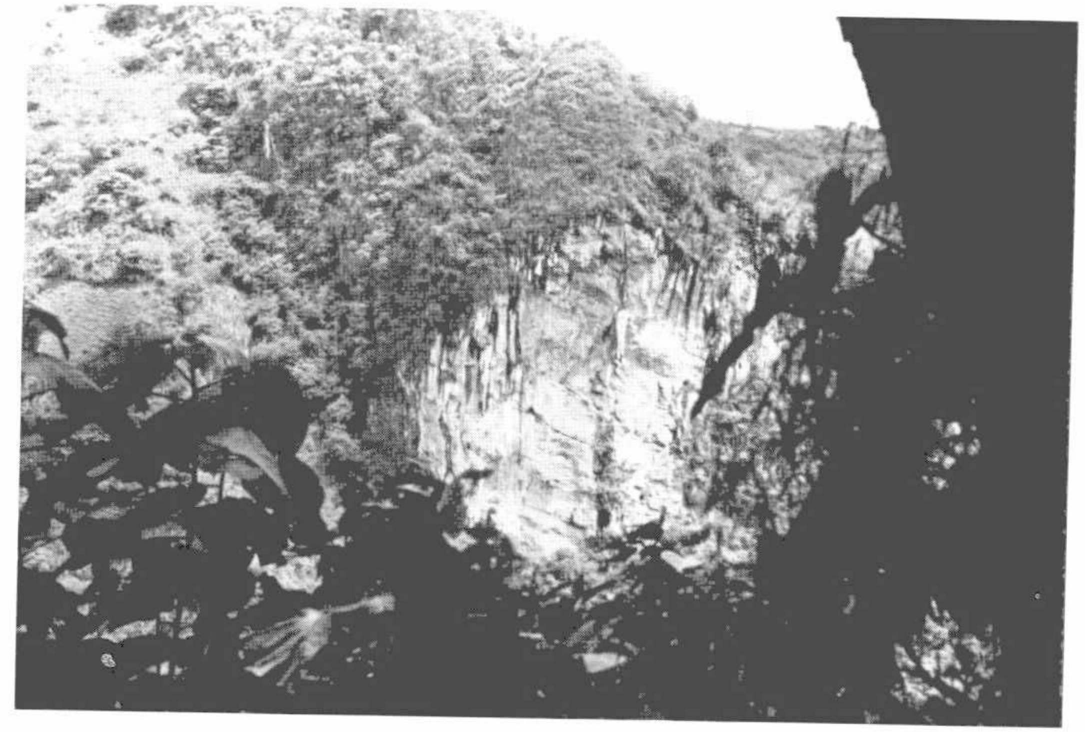
PLATE V



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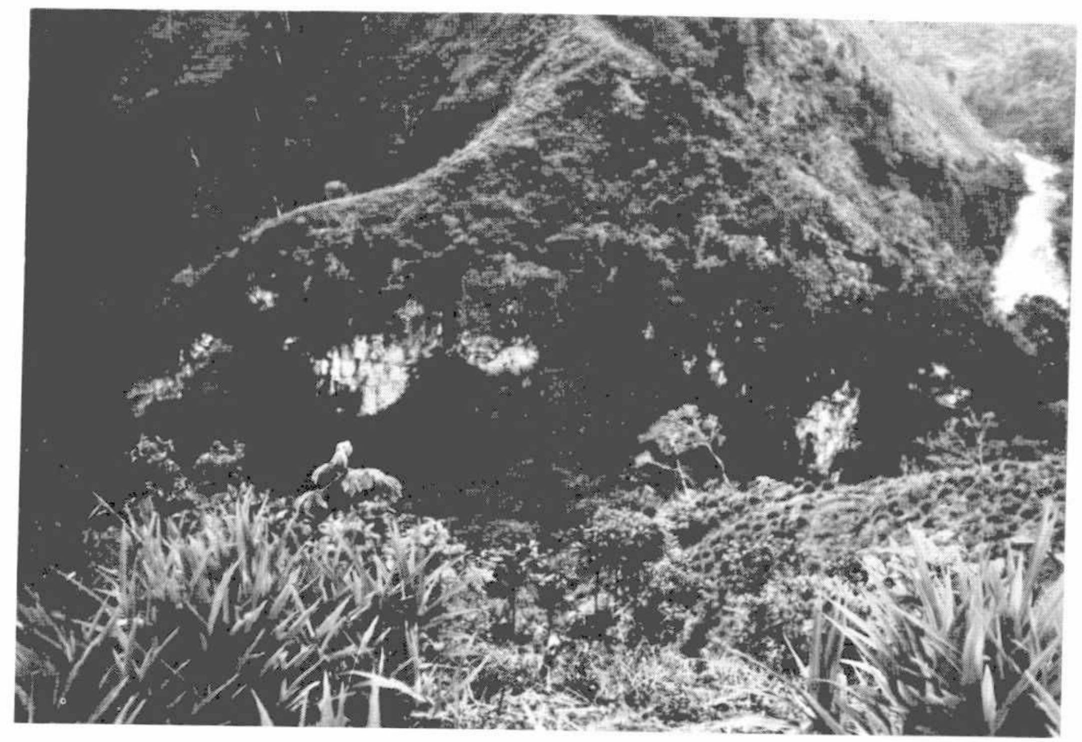

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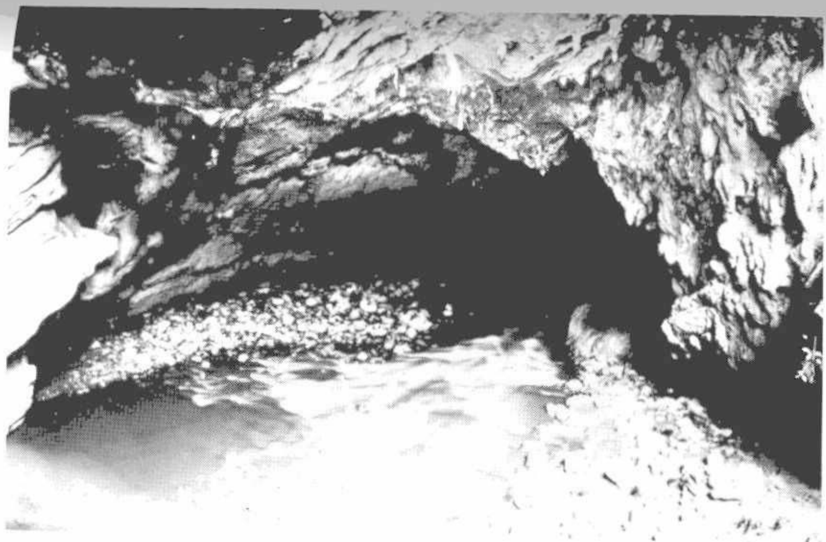
PLATE VI



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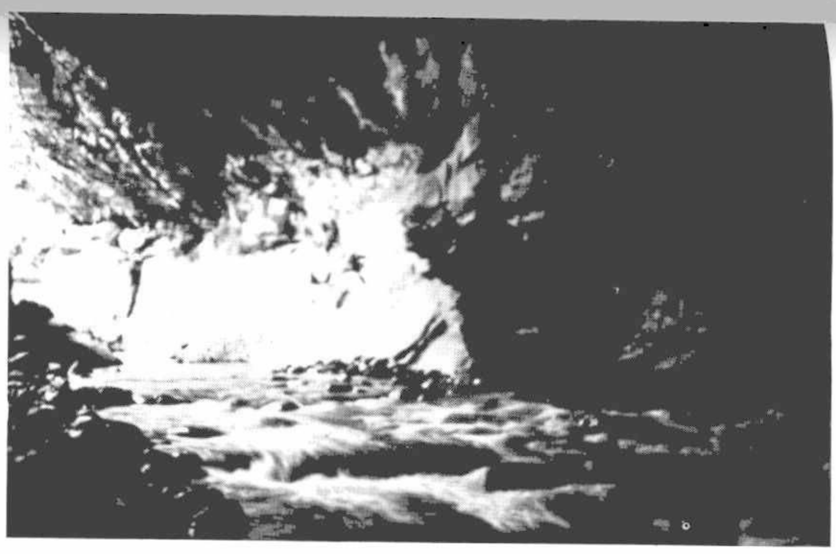

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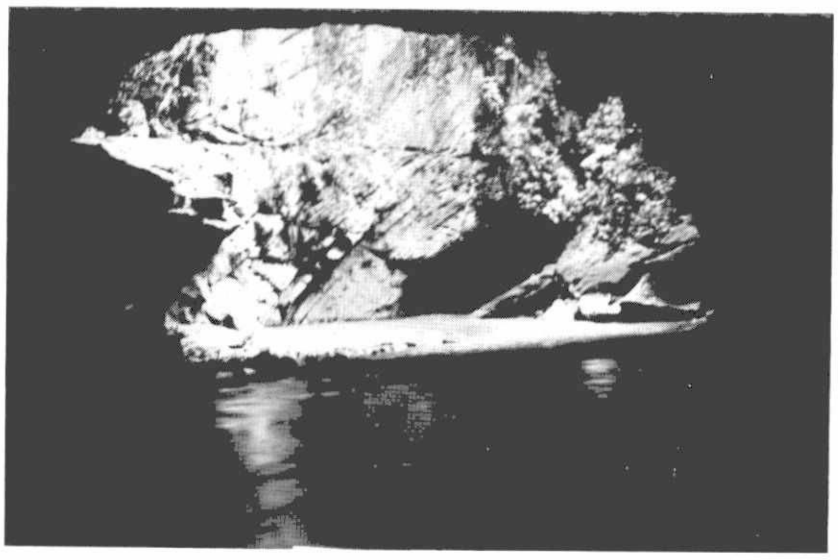

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PLATE VII

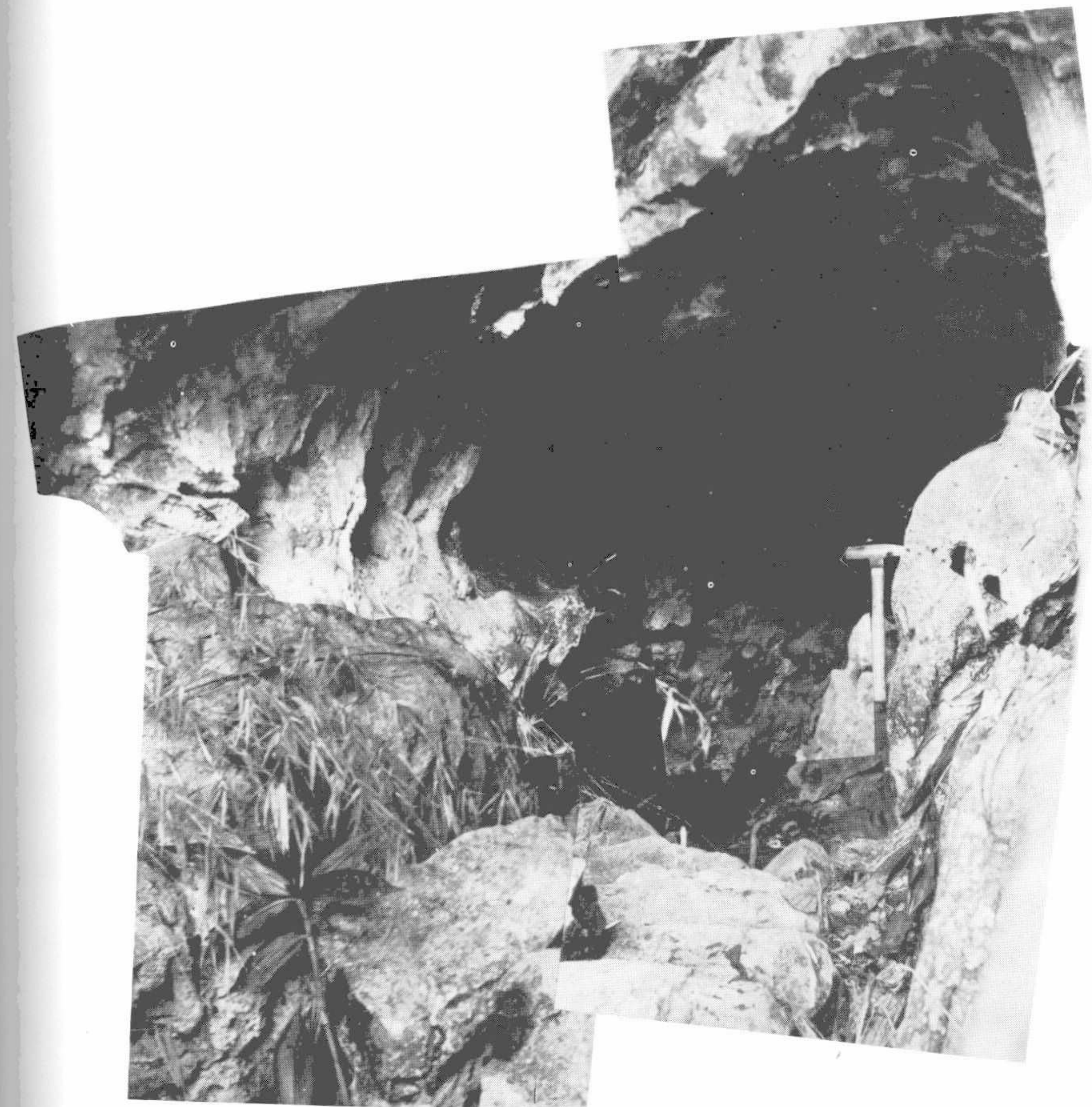
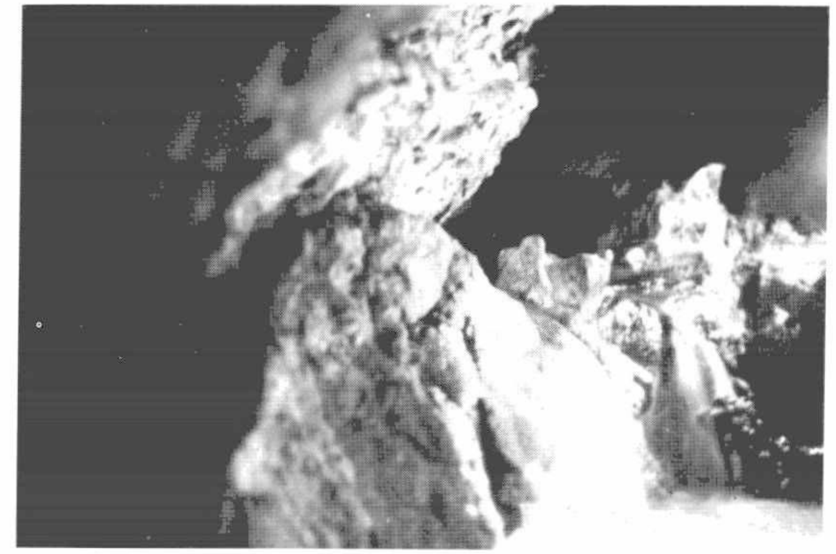


PLATE VIII



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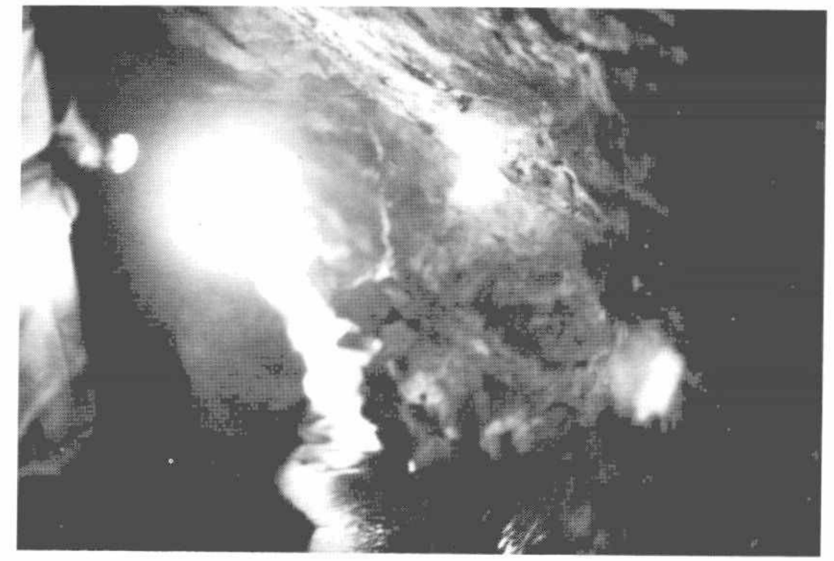

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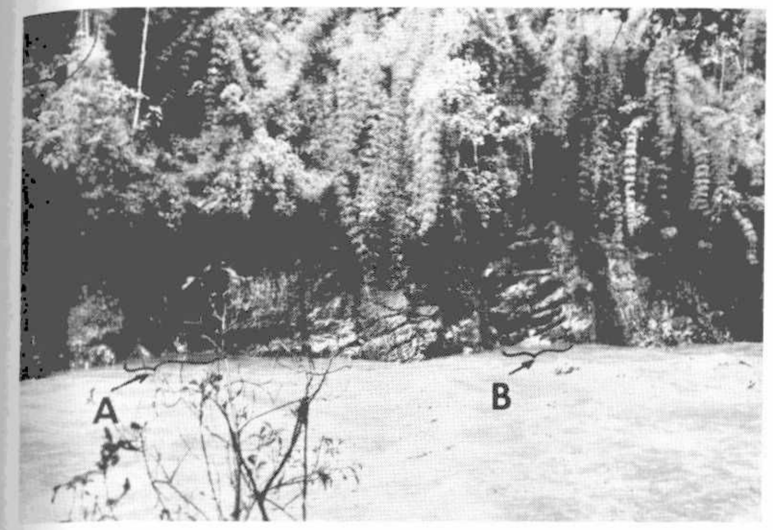
PLATE IX



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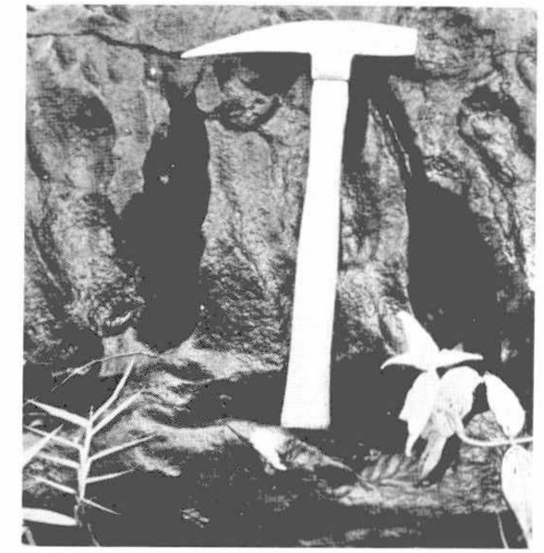

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PLATE X

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PLATE XI

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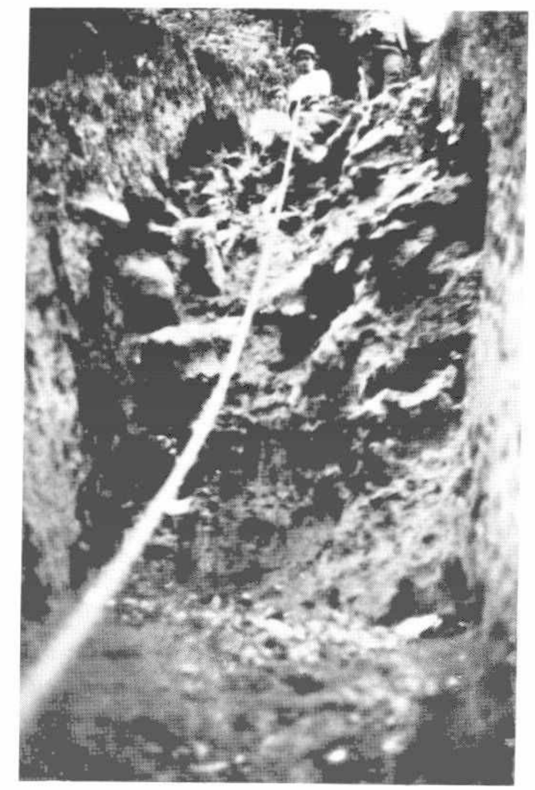

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PLATE XII

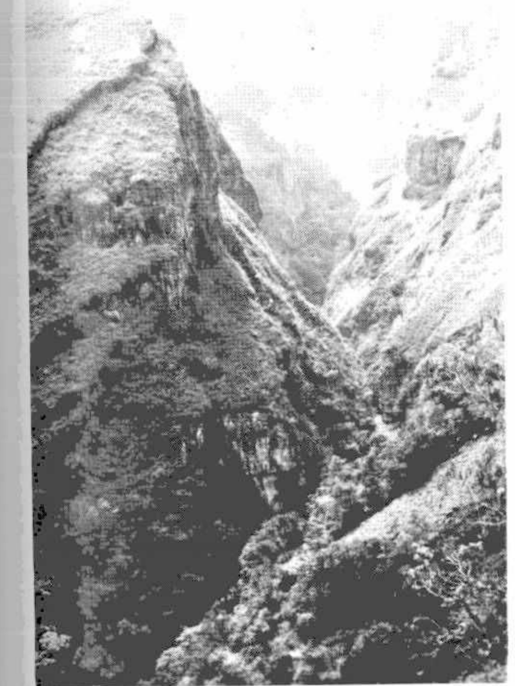

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PLATE XIII

APPENDIX III

DISCUSSION OF PERMEABILITY TESTING IN NX BOREHOLES

by

Dr. Tidu Maini

A. OBJECTIVES

The object of this report is to outline methods which can be used to determine insitu hydraulic conditions. If this determination is carried out correctly it should be possible to calculate (a) the uplift pressure beneath the dam and the pore pressures within the abutments of the dam (b) the leakage in an area of rock mass underneath and surrounding the dam. The determination of insitu hydraulic conditions includes an idea of the present water regime, the permeability of the rock mass and presence of large scale heterogenieties such as faults. Unfortunately, the task of collecting such information is a formidable one and involves the iteraction of technology, economics and logistics. Owing to the inaccessible nature of the proposed site, a set of priorities was established before going into the detailed ramifications of test procedures, etc. These are principally:

(a) Location is inaccessible.
(b) Limited availability of technology, i.e. no electronics or mechanical workshop facilities on site.
(c) Limited availability of electrical power, i.e. no 240/120 volts A.C. Mains.
(d) Difficulty in operating and maintaining pumps for water and compressed air.
(e) Limited number of boreholes.

B. BACKGROUND

In view of the diverse nature of literature on the subject, it is proposed to briefly outline the theory of groundwater flow as applied to the design of dams so as to put the proposed tests in true perspective.

Prior to planning any tests which give insight into the conditions of the rock mass, it is suggested that the existing groundwater table is established within the area surrounding the damsite. The total area should include the foundation of the dam along with the immediate upstream and downstream areas, and also the sides of the valley which will undergo considerable changes in their pore pressure distribution due to reservoir impounding. The simplest approach of establishing the water table is to locate the depth of water in an array of boreholes. The surface joining the top of the water levels in the various holes will establish the approximate phreatic surface. The porepressure can then be easily deduced at any point in the rock mass.

The presence of large scale heterogenieties can cause serious problems in terms of leakage and uplift pressures in a dam. It is therefore important to recognize such features during the geological investigation stage so that the implications of such heterogenities can be fully assessed. Large scale heterogenities include impermeable/permeable faults highly systematic fracture patterns and karstic formations. Hopefully the tests to be described will be sensitive to these anomalies, but it must be emphasized that the geologist ought to look out for such structures.

C. DETERMINATION OF PERMEABILITY OF A SOIL/ROCK SYSTEM IN THE FIELD

At a typical damsite one of the most important factors that has to be established is whether the proposed permeability measurements are to be carried out in the unsaturated or saturated zones. The analysis and tests for the two conditions are quite different and if incorrectly applied yield results which can be different by orders of magnitude. In general there are three kinds of tests which can be used to determine permeability and they all give different answers, the reason being that the scale of test is an important factor. It is therefore essential prior to testing to decide how the results are to be used in any subsequent analysis. The three principal tests that can be carried out to measure permeability are:

(i) Large scale pump tests, such as those used in groundwater studies. The Theis method is a typical way of analyzing the results for permeability. This method will not be further pursued because it is a test more relevant to calculating bulk flow rates in large systems and also it does not provide a distribution of permeabilities with depth. Finally it is an expensive test needing highly qualified personnel to conduct and interpret the results.

(ii) Lugeon or modified Lugeon tests, these tests consist of pumping water in specific zones, isolates within a borehole by inflatable packers. This is a steady state test requiring a fair degree of technology to make sure that the tests are carried out correctly. The Lugeon test because of its relative simplicity has been one of the standard tests in major dam projects throughout the world. Unfortunately no specific standards exist on this test and they are carried out quite arbitrarily in practice.

(iii) Falling head tests with and without packers. This type of test is fairly popular because of the bare minimum of equipment that is required. Once again no standards exist and the execution and interpretation [more difficult than (b)] depend on the whims of the engineers.

D. LUGEON TEST: TEST SECTION BELOW THE GROUNDWATER TABLE

(i) Procedure and Sources of Error. The test consists of inflating or mechanically securing the two packers (Fig. 1) in position so that they are hydraulically sealing the test section from the rest of the hole. The input of water is into the main cavity and should be such that it is possible to vary either the pressure or flowrate of the fluid that is being injected. If it is not possible to have two packers, tests can be carried out using only one packer as shown in Fig. 2, but unfortunately this makes the analysis of the data a little more difficult.

The first important thing that has to be established is the scale of the test, i.e. the length L of the test cavity. In theory L should be such that any increase in the dimension should

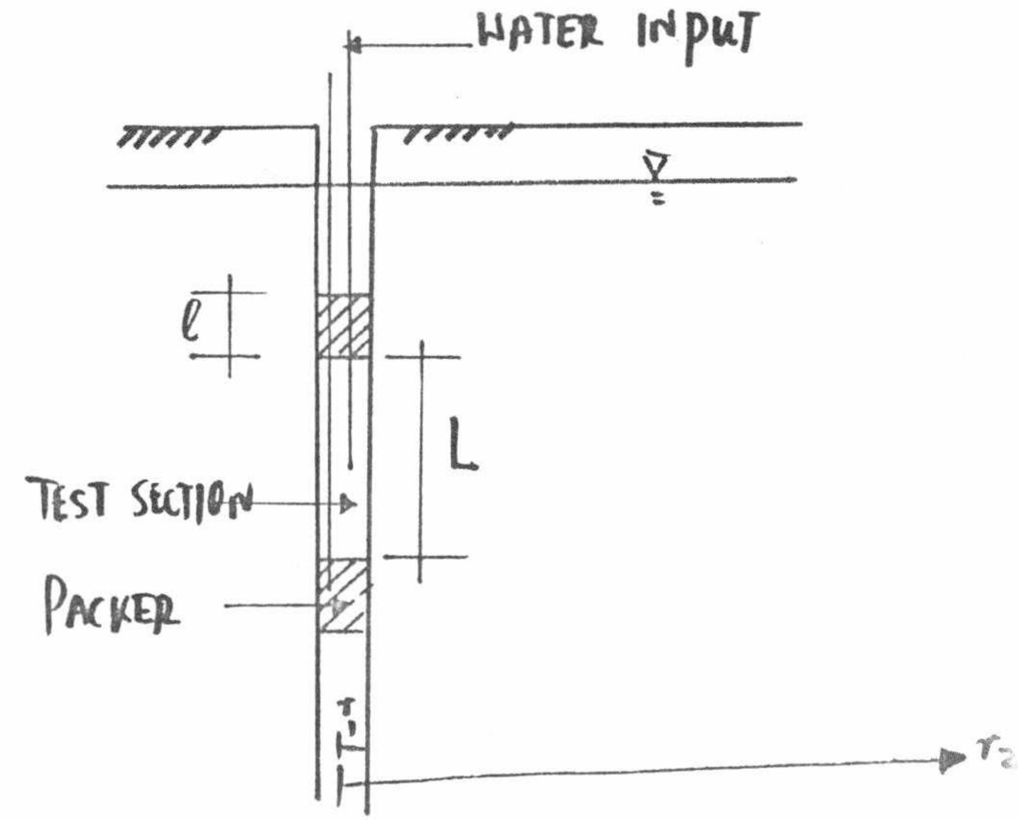


FIG 1

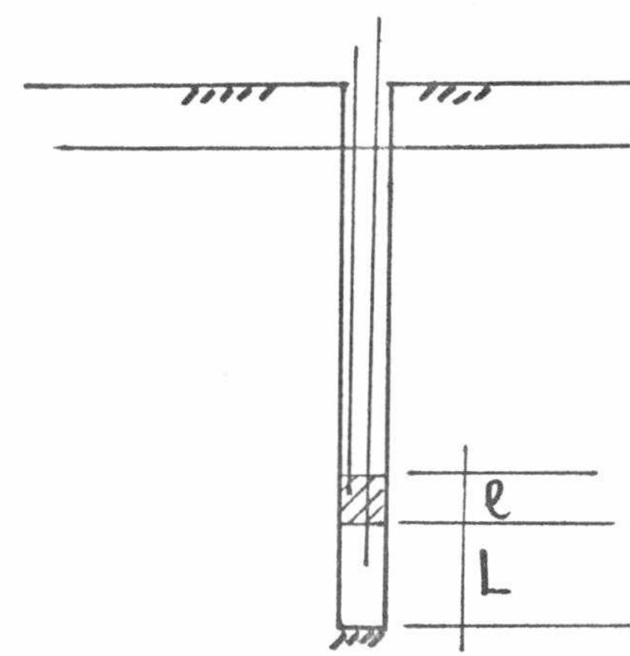


FIG 2

not alter the permeability significantly. Therefore if time permits the length L should begin at 1 meter and it should be increased to see how much difference there is in the value of permeability. Fixing the final length is very much a question of judgement, but it ought to be somewhere where there is little change in permeability in change in L. It is worth noting that the test could be carried out in the entire length of the hole. This gives a value of permeability which may represent an area too big to be of any use in our analysis. The second important factor is,to observe the pressure during pumping in an adjacent or nearby hole,since permeability is defined by (see Appendix for derivation).

$$K = \frac{Q \log [r_2/r_1]}{2\pi L [P_1 - P_2]}$$

K	permeability
Q	is the flowrate into the test section cm^3
r_2	distance of observation hole cms
r_1	radius of test hole cms
L	length of test section L
P_1	pressure at injection hole cms of H_2O
P_2	pressure at observation hole cms of H_2O

Thus by observing Q for different values of P it is possible to calculate K. The maximum value of P_1 should be the maximum expected pore pressure at a given depth - this simply may be thought of the height of the waterlevel in the reservoir above a given datum.

If it is impossible to have an observation hole the value of r_2 and P_2 will have to be guessed and unfortunately it is difficult to give a prescription, short of saying that the error introduced due to the log (r_2/r_1) relationship may not be serious if the rock mass is randomly or heavily fractured. An important part of the procedure,and a common source of error, in estimating K is the leakage past packers. This occurs because the packers are not hydraulically sealed or alternatively they are not long enough. A minimum recommended length is 2 meters and if possible it should be extended. Without sophisticated electronic equipment it is difficult to check this leakage, but a useful hint is that if at low hydraulic pressures the relationship between Q and P is nonlinear the packers ought to be tested carefully. A nonlinear relation as in Fig. 3 may be described by,

$$P = AQ^2 + BQ \quad (1)$$

$$\frac{P}{Q} = AQ + B \quad (2)$$

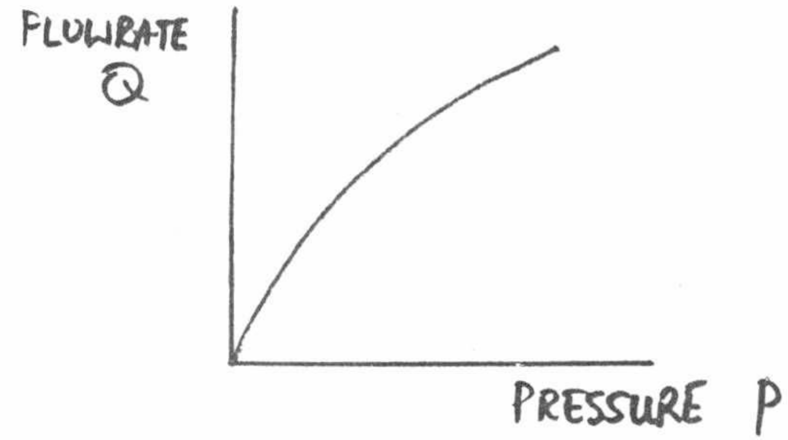


FIG 3

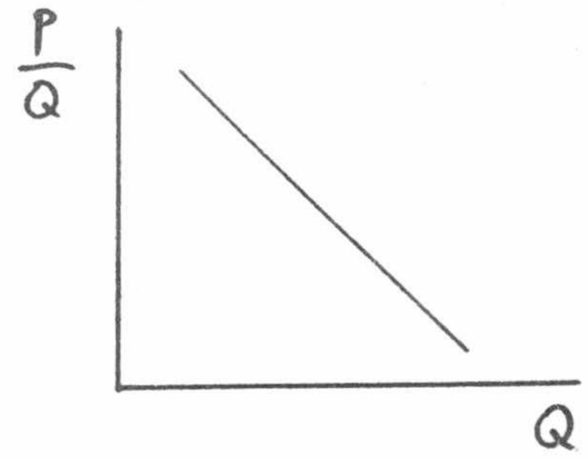


FIG 4

The term AQ^2 reflects the kinetic energy in the system [proportional to $V^2/2g$ and BQ is the frictional component. Therefore the slope of (2) should tell us whether the kinetic energy is increasing with increased flow or decreasing with, a leaky packer the kinetic energy will decrease suggesting that there is a hole in the system which is responding to pressure changes. A relatively non-deformable soil or rock fracture would tend to increase the kinetic energy losses due to greater inefficiency of the system. Care should be taken to apply this argument to systems with high pressures or fractures which are very small and systematic.

(ii) Test Layout. As can be seen a fair amount of equipment such as pumps, tanks, etc. is needed for the test and this is not always possible if the site is remote or the funds are limited.

E. LUGEON TEST ABOVE THE WATER TABLE

The determination of permeability in an unsaturated zone is a highly complex and unreliable process. It is suggested to carry out the constant pressure test for up to 3 hours before taking any reading of Q or P. The test should then be repeated with a different initial period to see the difference in K. Since the flow through an unsaturated media is really an nonsteady state phenomenon, an approximate method will be described in a subsequent section.

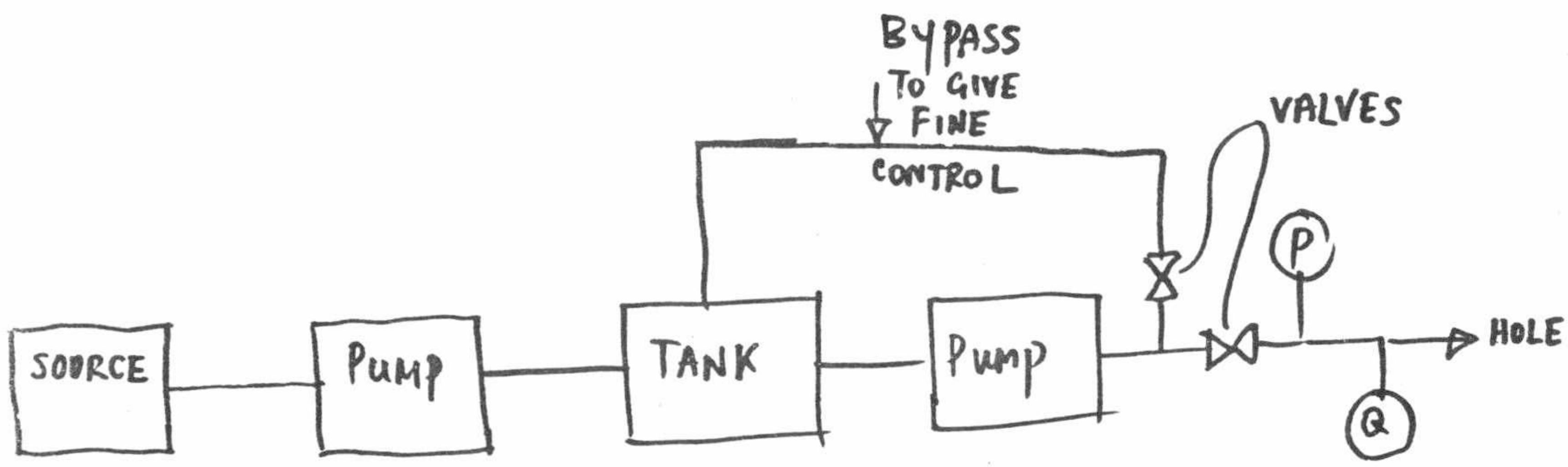
F. FALLING HEAD TEST BELOW THE WATER TABLE

This test has the great attraction that it does not require a large amount of cumbersome equipment such as water tanks and pumps, not to mention water pressure gauges [Bourdon type] and flow meter. A simple device shown in Appendix will be completely adequate to measure the pressure of water in the test section. As can be seen from theory in Appendix the permeability is given by

$$K = \frac{\log_e [\frac{h_1}{h_2}]}{(t_2 - t_1)} \cdot \frac{r_0^2 \log_e (\frac{r_e}{r_0})}{2L}$$

An initial record of the water level is made in the hole and then the hole is very quickly filled up with water to the a given level. The decrease in water level is then recorded with time. A typical curve looks as in Fig. 6 and if it is replotted as a semilog scale Fig. 7 it is possible to obtain the permeability of the system from the above equation.

The test can be carried out with or without the packers depending on the scale at which the answer is required. The great advantage of this type of test is that it requires little equipment, and there is a smaller chance for water to leak past packers since the pressures used are small.



P= PRESSURE
Q= FLOWRATE

FIG 5

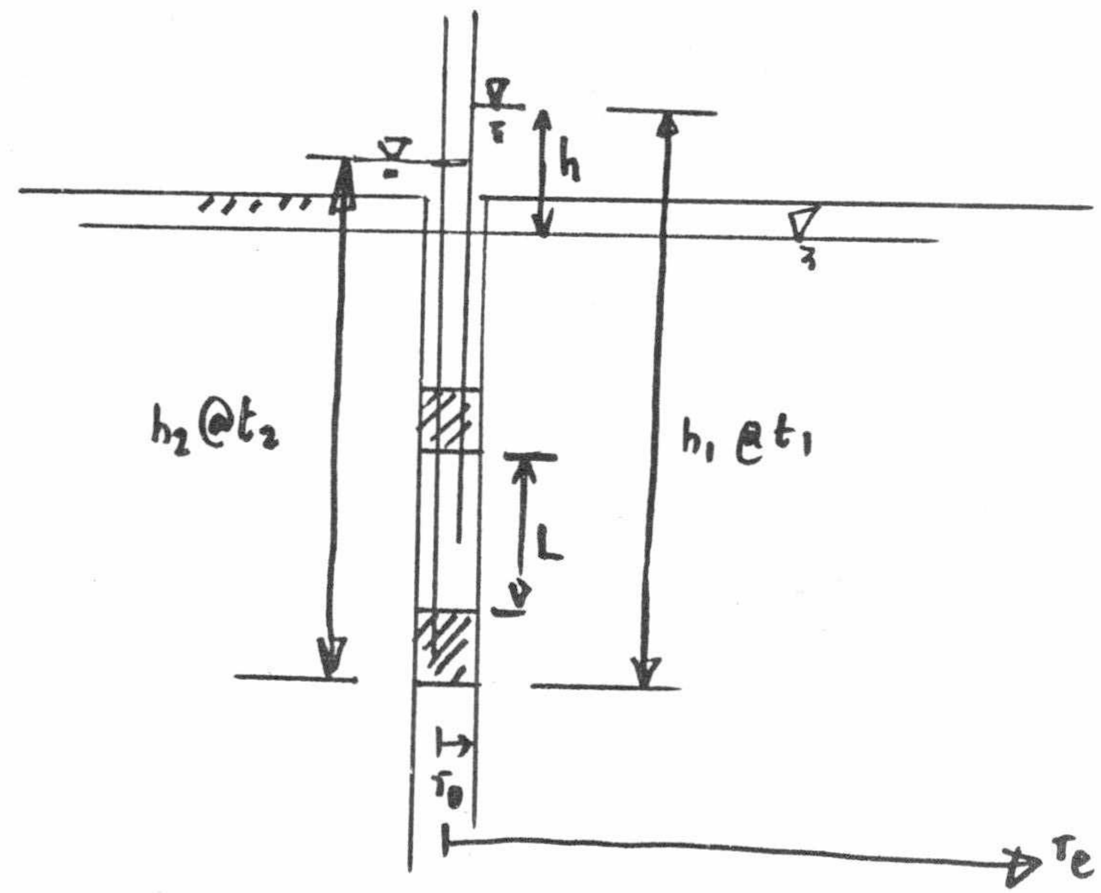


FIG 5

G. FALLING HEAD TEST ABOVE THE WATER TABLE.

As was mentioned previously this test is very difficult and ought to be carried out with caution. The analysis of such a problem requires a very careful evaluation of all the parameters. A graphical and approximate method is suggested. As in the previous case a plot of H vs. t is made an from this if the plot in Fig. 8 is made the slope of the straight line

$$\text{the mean effective opening of the fracture} = \sqrt[3]{\frac{3\mu\, r_0^2}{2\gamma}}$$

This being an indirect measure of permeability and for purposes of design could be used more as a basis of comparison than exact computation of leakage or uplift pressure.

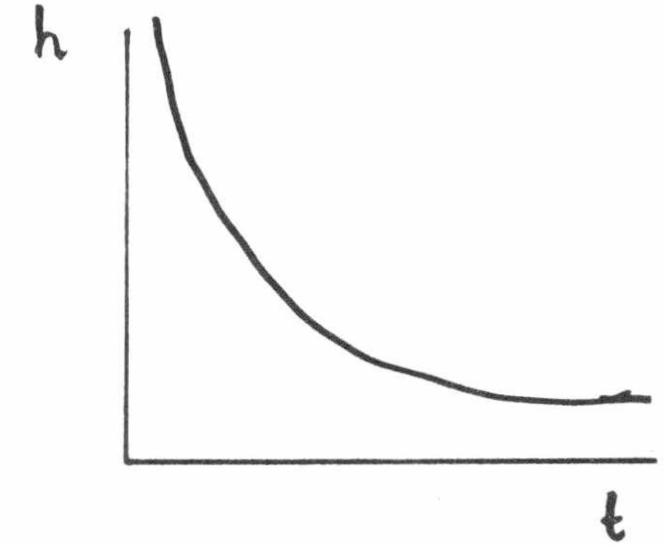


FIG 6

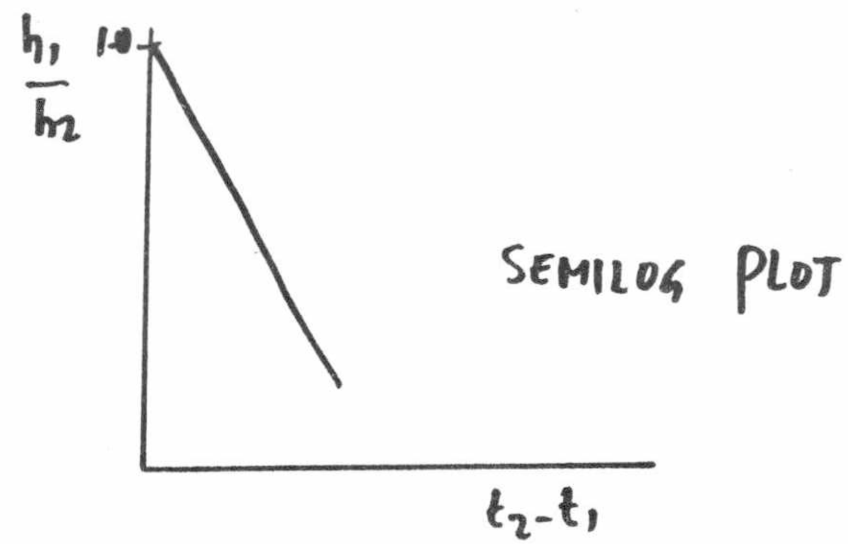


FIG 7

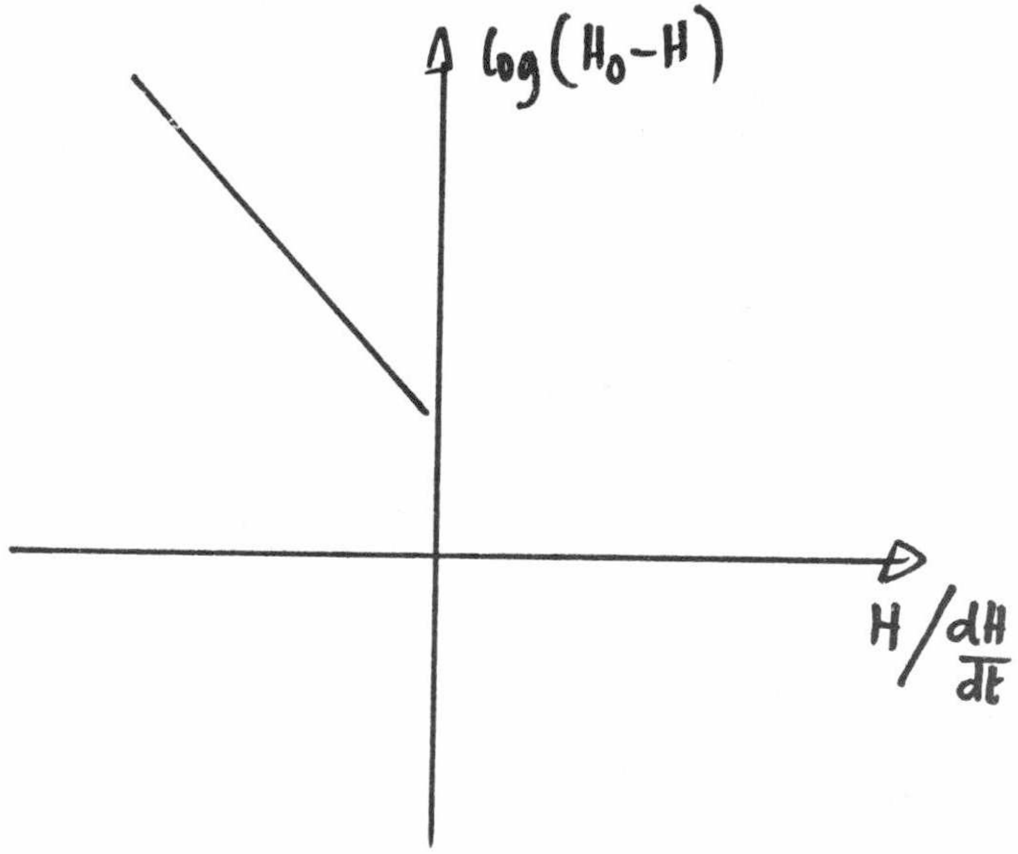


Fig 8

APPENDIX

2.1.0 AXISYMMETRIC SEEPAGE

It is proposed that equations of motion of a fluid are developed for the case where pumping-in of a fluid takes place. The following assumptions are made:

(a) Fissure of infinite radial extent (SINGLE FISSURE).

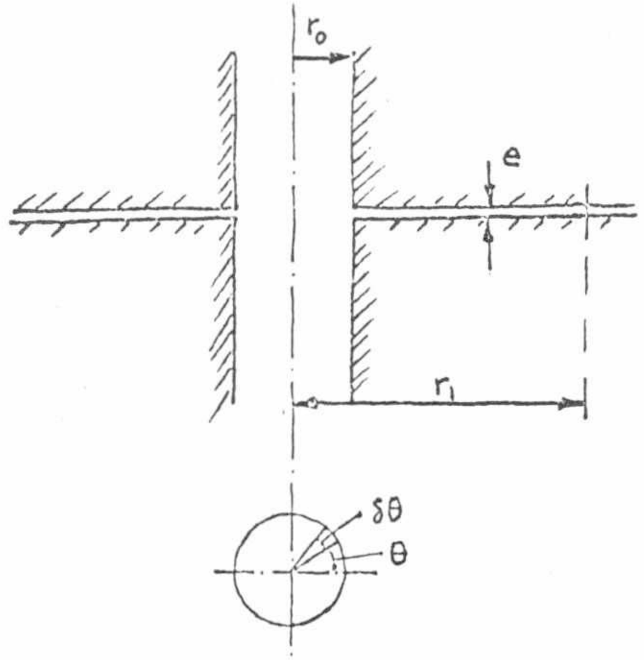
(b) Linear Flow Law.

(c) Radial Flow, ie. equipotential lines are circular - isotropic in fissure plane.

(d) No inertia effect as a result of acceleration of fluid.

(e) No influence of secondary fissures, parallel to the hole, eg. no influence of gravity.

Note: This special case of a fissure at right angles to the borehole is considered to make the mathematics simple and clear. Influence of gravity, as would be the case in an inclined joint, is simply allowed for by linearly superimposing the gravitational component g.sind to the gradient (see Ref.10).



Consider Fig.2.1 using polar coordinate system:
Darcy's Law applied to a parallel plate model

$$V = K_J \frac{\partial P}{\partial r} \qquad (2.1)$$

where K_J = joint permeability

V = mean velocity

e = mean opening

The flow from a cavity into a fissure is

$$Q = 2\pi re.\bar{V} \qquad (2.2)$$

Substituting 2.1 into 2.2 and integrating gives

$$\frac{Q}{2\pi re} = K_J \frac{\partial P}{\partial r}$$

$$\int_{r_0}^{r_1} \frac{dr}{r} = \int_{P_0}^{P_1} \frac{2\pi e}{Q} K_J \, dP$$

$$\log_e \left(\frac{r_1}{r_0}\right) = \left(\frac{2\pi e}{Q}\right) K_J \left(P_0 - P_1\right)$$

$$\therefore \quad Q = \left\{ \frac{2\pi e K_J}{\log_e \left(\frac{r_1}{r_0}\right)} \right\} (P_0 - P_1) \qquad (2.3)$$

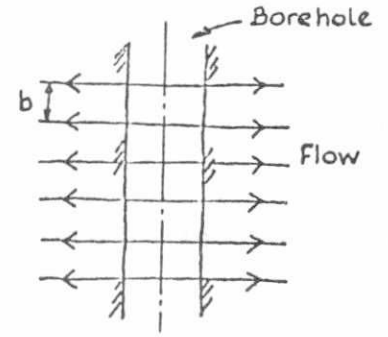
2.1.1 INFLUENCE OF FISSURE FREQUENCY IN DIRECTION ORTHOGONAL TO BOREHOLE.

If within a given test section the fracture frequency orthogonal to the hole is considered as regular, ie. the spacing between joints is constant, equation 2 . 3 is very simply modified to the following, by the principle of superposition:

$$Q = \left\{ \frac{2\pi e n K_J}{\log_e \left(\frac{r_2}{r_1}\right)} \right\} (P_o - P_1) \qquad (2.4)$$

where n is the number of fissures.

In such a case, it now becomes possible to use the term equivalent permeability if the fissures are identical in aperture.



If we consider flow from a borehole into a set of fissures, the discrete element approach gives us the flowrate as:

$$Q = \left\{ \frac{2\pi e n K_J}{\log_e \left(\frac{r_1}{r_o}\right)} \right\} (P_o - P_1) \qquad (2.5)$$

where K_J is the fissure permeability.

Using the continuum concept, for the same problem

$$Q = \left\{ \frac{2\pi b n K_S}{\log_e \left(\frac{r_1}{r_o}\right)} \right\} (P_o - P_1) \qquad (2.6)$$

where K_S is the system permeability.

Since flow is the same in both cases : $e K_J = b K_S$

$$\therefore \quad \frac{e}{b} K_J = K_S \qquad (2.7)$$

5.1.2 Falling head test carried out when test section is below the groundwater table.

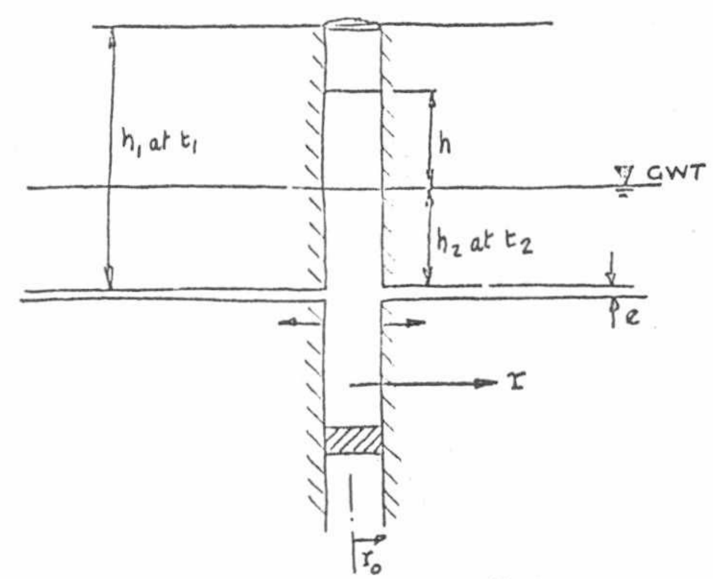


FIG 5.2

If we assume radial flow in the fracture and we use Darcy's law at a distance r

$$Q = -2\pi r e K_J \frac{dh}{dr}$$

$$\therefore \int_0^h dh = -\frac{Q}{2\pi e K_J r} \int_{r_o}^{r_e} dr$$

$$h = \frac{Q}{2\pi e K_J} \log_e\left(r_o/r_e\right)$$

$$h = \frac{Q}{2\pi e K_J} \log_e\left(r_e/r_o\right)$$

Kg = joint conductivity $= \frac{e^2 g}{12U}$

e = mean aperture of the fissure

rate of change of water level in the hole = $Q = \pi r_o^2 \frac{dh}{dt}$

$$\therefore \quad h = \frac{\pi r_o^2}{2\pi e K_J} \frac{dh}{dt} \log_e\left(r_e/r_o\right)$$

$$\int_{h_1}^{h_2} \frac{dh}{h} = \frac{2\pi e K_J}{\pi r_o^2} \log_e\left(\frac{r_o}{r_e}\right) \int_{t_1}^{t_2} dt$$

$$\log\left(\frac{h_1}{h_2}\right) = \frac{2eK_J}{r_o^2} \log_e\left(\frac{r_o}{r_e}\right)(t_2 - t_1)$$

$$\frac{\log\left(h_1/h_2\right)}{t_2 - t_1} = \frac{2eK_J}{r_o^2 \log_e\left(r_e/r_o\right)} = \frac{2e^3 g}{12U r_o^2 \log_e\left(\frac{r_e}{r_o}\right)} = \frac{e^3 g}{6U r_o^2 \log_e\left(\frac{r_e}{r_o}\right)}$$

5.2.0 EQUATIONS FOR A FALLING HEAD TEST IN AN UNSATURATED FISSURED MEDIA:

Assumptions:

(a) Fissure is unsaturated, ie. above the groundwater table.
(b) Linear flow-pressure relationship.
(c) Axsymmetric flow.
(d) Homogenious and isotropic fissures.
(e) Kinetic energy is small.
(f) Infinite aquifer.
(g) It is assumed that there is no significant flow in the fissure while the borehole is filled with water.
(h) Viscous flow.

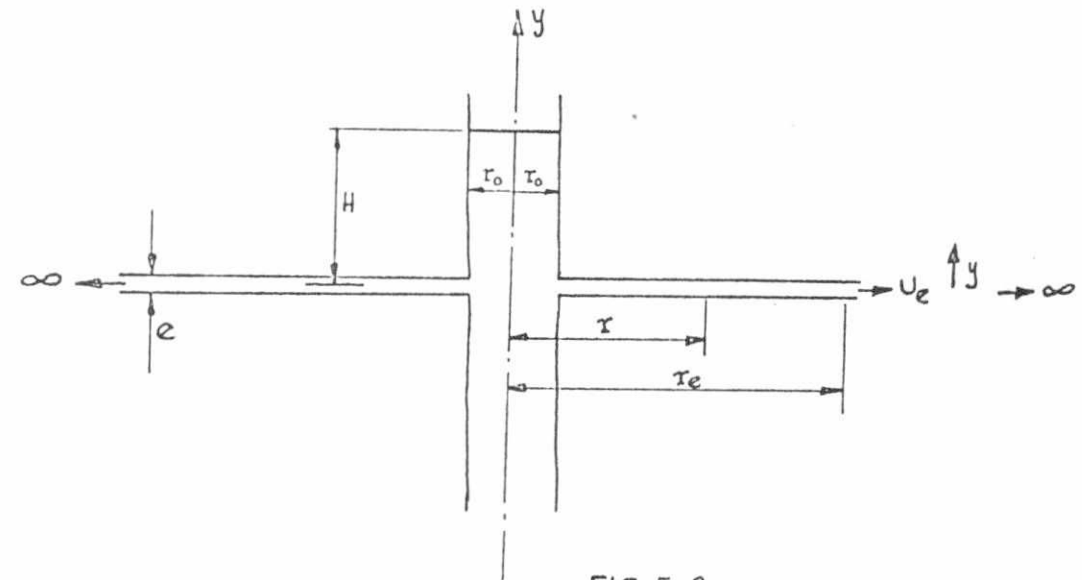


FIG 5.3

5.2.1 DERIVATION OF PERMEABILITY FROM RELATIONSHIP BETWEEN PRESSURE AND TIME

From the general equilibrium equation

$$\frac{\partial \sigma_r}{\partial r} + \frac{1}{r}\frac{\partial \tau_{r\theta}}{\partial \theta} + \frac{\partial \tau_{yr}}{\partial y} + \frac{\sigma_r - \sigma_\theta}{r} = 0$$

for radial flow : $\frac{\partial}{\partial \theta} = 0$; $\sigma_r = \sigma_\theta = p$, say

$$\therefore \quad \frac{\partial \sigma_r}{\partial r} + \frac{\partial \tau_{yr}}{\partial y} = 0 \qquad (1)$$

for viscous flow $\tau_{yr} = -\mu \dfrac{\partial u}{\partial y}$ where μ viscosity, τ drag, u velocity

$$\therefore \quad \frac{\partial \tau_y}{\partial y} = \mu \frac{\partial^2 u}{\partial y^2}$$

$$\therefore \quad \frac{\partial \sigma_r}{\partial r} - \mu \frac{\partial^2 u}{\partial y^2} = 0$$

$$\text{or} \quad \frac{\partial p}{\partial r} = \mu \frac{\partial^2 u}{\partial y^2}$$

Integrate $u = \dfrac{y^2}{2\mu}\dfrac{\partial p}{\partial r} + By + C$ where B & C are constants of integration

$B = 0$ since $\dfrac{du}{dy} = 0$

Boundary Conditions

$$y = \pm e/2 \quad ; \quad u = 0$$

$$\therefore \quad C = -\frac{1}{2\mu}\frac{\partial p}{\partial r}\left(\frac{e^2}{4}\right)$$

$$\therefore \quad u = -\frac{1}{2\mu}\frac{\partial p}{\partial r}\left[\frac{e^2}{4} - y^2\right] \qquad (3)$$

$\therefore$ Velocity distribution parabolic

$$\text{Rate of flow, } Q = 2\pi r \int_{e/2}^{e/2} u\,dy = -\frac{\pi r e^3}{6\mu}\cdot\frac{\partial p}{\partial r}$$

$$\therefore \quad \frac{\partial p}{\partial r} = -\frac{6\mu Q}{\pi e^3}\cdot\frac{1}{r}$$

$$\text{Integrating } p = -\frac{6\mu Q}{\pi e^3} \times \left[\log r + \text{constant}\right]$$

when $r = r_e$ outer radius of fissure $p = 0$

when $r = r_o$ at borehole, $p = \gamma H$, γ specific wt. of water

$$\therefore \quad 0 = -\frac{6\mu Q}{\pi e^3}\left[\log r_e + \text{constant}\right]$$

$$\gamma H = -\frac{6\mu Q}{\pi e^3}\left[\log r_o + \text{constant}\right]$$

$$\therefore \quad \gamma H = \frac{6\mu Q}{\pi e^3}\log\left(\frac{r_e}{r_o}\right) \qquad (4)$$

Initial conditions

$$H = H_o \ , \ r_e = r_o$$

When head drops to any other value, H

Volume of water displaced = $(H_o - H)\pi r_o^2$

also $\pi (r_e^2 - r_o^2) e$

$$\therefore \left(\frac{r_e}{r_o}\right) = \left[1 + \frac{H_o - H}{e}\right]^{\frac{1}{2}} \qquad (5)$$

$$\text{Also} \quad Q = -\pi r_o^2 \frac{dH}{dt} \qquad (6)$$

Substitute (5) and (6) into (4)

$$\gamma H = -\frac{3\mu r_o^2}{e^3} \log \left(1 + \frac{H_o - H}{e}\right)^{\frac{1}{2}} \frac{dH}{dt} \qquad (7)$$

Integrating in time $t = \int dt$

$$= \int \left(-\frac{3\mu r_o^2}{e^3}\right) \cdot \frac{\log \left(1 + \frac{H_o - H}{e}\right)^{\frac{1}{2}} dH}{H} \qquad (8)$$

If this can be evaluated we have related $H \to t$ for the case of flow from a borehole into a fissure of infinite radius.

Noting that the aperture e is small compared with $(H_o - H)$ except when the hole is full we can rewrite (7)

$$\frac{1}{2} \log \left[H_o - H\right] = \frac{1}{2} \log e - \frac{\gamma e^3}{3\mu r_o^2} \cdot \frac{H}{\left(\frac{dH}{dt}\right)} \qquad (9)$$

$\therefore$ plot $\log\left[H_o - H\right]$ vs $\dfrac{H}{\left(\frac{dH}{dt}\right)}$ should be straight line

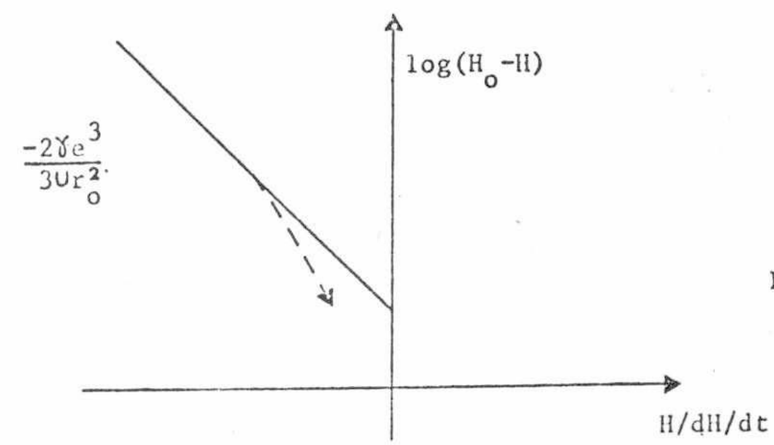


FIG 5.4

At the beginning of the test, $H = H_o$ $\log(H_o - H) \rightarrow -\infty$

This is not given by straight line plot becouse

(a) Eqn (9) we assumed e negligible compared with $(H_o - H)$

(b) Ignored the kinetic energy.

Hence there will be a departure from the straight line as indicated in Fig. 5.4.

APPENDICE III (Continuación)

R. E. Goodman (Agosto, 1972)

La segunda parte de este apéndice corresponde al siguiente artículo:

EVALUATION OF HYDRAULIC PROPERTIES OF ROCK MASSES INFLUENCED BY MAJOR HETEROGENEITIES

By: J. C. Sharp
Y. N. T. Maini
T. L. Brekke

Este artículo se publicó en las memorias del 14º Simposio de Mecánica de Rocas, celebrado en Pennsilvania, U.S.A., en junio de 1972.

APENDICE IV

Corresponde al siguiente artículo:

THE MEASUREMENT OF ROCK DEFORMABILITY IN BORE HOLES

By: R. E. Goodman
T. K. Van
F. E. Heuze

Este artículo se publicó en las memorias del 10º Simposio de Mecánica de Rocas celebrado en Texas, U.S.A., en Mayo de 1968.

INFORME 6

GEOLOGIA

23rd January, 1973

Dr. Hernando Quijano,
INGETEC,
Apartado Aereo 5099,
BOGOTA,
Colombia

Dear Dr. Quijano,

This is to report the proceedings of my trip to the Guavio Project. I arrived in Bogotá on the morning of January 2nd and, after reviewing project developments with you in the office, travelled to Gachala. I examined possible sources of core material for dams at El Cobre or Ubala. On January 5th I reviewed the geology of Gacheta dam in the field and studied sources of borrow for this site. Then, in the morning of January 6th, I studied various tunnel alternatives, via helicopter, accompanied by Camilo Torres, yourself and Mr. Ochoa of ISA. I returned to the United States on January 6th and 7th.

I understand that economic evaluations have led to a reconsideration of the two dam alternative, and accordingly I studied the El Cobre site. The economy of this site resides in the natural spillway which the topography allows. Unfortunately, this natural saddle owes its existence to a major fault, with a zone of silty mylonite about 60 meters wide. The mylonite gives the impression of affording cheap excavation. However, is probably erodible, and further will soften and disintegrate if subjected to accelerated wetting and drying. Construction of a spillway along this way will require a water tight lining. Because the required spillway width is greater than sixty meters, part of the gate structure will lie on rock while part lies on the fault zone. The foundation conditions will have to be improved to even the load distribution. Alternatively, the spillway can be moved. However, a move to the right will enlarge the excavation, reducing the possible site economies connected with the short spillway. A move to the left will place the chute on a very steep, obsequent slope formed by thin bedded limestone dipping upstream. I am concerned about the stability of this slope if a major structure is built upon it in such a way as to subject the rock to accelerated wetting and drying, vibrations, or large dead or live loads. I should also like to point out that the bedrock in the vicinity of the El Cobre dam site is crossed by a number of faults. In fact I believe that the existence of the transverse ridge forming the downstream end of the dam is due to parallel transverse faults, as shown on the accompanying map. A detailed appraisal of the rock quality at El Cobre is not possible without a preliminary program of explorations.

In examining the rock along the river at the El Cobre dam site, I again was impressed with the soundness of the Paleozoic bedrock when it is free of faults (see enclosed photographs). Again I think in terms of a concrete dam at Ubala as a viable alternative. If preliminary estimates of an arch dam for Ubala show favourable cost, it is important that we proceed immediately with the program of drill holes agreed upon previously. We will not be able to know the true qualities of this site without some drilling. I was surprised to see the excellent drilling machines lying idle at Gachala.

I examined two possible sources of borrow in the vicinity of Gachala. The Guzman Creek deposits are located about 5 kilometers from the mouth of Batatas Creek, within easy reach of either El Cobre or Ubala site. The Tunjita borrow area is across the Murca River from Gachala and upstream from Ubala dam site. It would not be useful for El Cobre.

The Guzman Creek deposits consist of a shaley colluvium with blocks of limestone up to several meters in diameter. The material is quite dense in the two test pits I observed. Since it lies on a moderate slope and is elevated above the creek, it should be possible to exploit the material without moisture content difficulties. There is a great volume available, perhaps of the order of 6 million cubic meters. Large blocks will be a nuisance but the volume is greater than the need so that selectivity is possible to minimise problems associated with blocks. It may be possible to use limestone blocks for rip rap; however, a quarry on the abutment of the dam would probably be more economical as a source of rip rap since haulage is such a high cost element with this material.

The Tunjita borrow area consists of an extensive field of shallow shaley colluvium perhaps 3 meters deep on the average, overlying decomposed shale. It should be possible to excavate the shale as a source of material for compaction in the core. I estimate at least 30 meters depth of shale could be easily excavated for this purpose, but test pits are necessary to confirm this. The shale decomposes to a silty clay.

Considering the availability of good sources of quarry stone for shell material or for concrete aggregate, and the large volumes of potential core material within short haul distance of the site, we can conclude that the unit prices for material should be low. As I pointed out during our discussions, it would also be feasible, from a material point of view, to establish a cement plant near Ubala, as the raw materials, namely limestone and shale, are both plentiful in the region. The quality of cement produced from limestone depends to a great extent on the chemical composition of the limestone. There are restrictions on the quantities of magnesium and iron, I believe. It would be wise to obtain a chemical analysis of several limestone sources within the Ubala region if there is any serious consideration of establishing a cement plant.

Gacheta dam would require more rock fill than is available in the sandstone ledge below the road on the left bank. A road relocation will be required; a 500 m. tunnel would allow a sufficient length of quarry to supply all shell material needed. Core material is more difficult. The only proximate possibilities are to take colluvium and decomposed shale. A possibly sufficient source lies downstream of Rio Murchindoto, opposite Q. de los Robles. This would represent a haul distance of several kilometers but has the virtue of an elevation below the present road. At a higher elevation, the spoon - shaped re - entrant in the right bank ridge three hundred meters below the same site may contain enough decomposed shale and slide material to supply the core. It remains to be determined if the plasticity and water content of the shale as well as the pore pressure attributes, are conducive to its use in a core of reasonable dimensions.

The helicopter flight on January 6th allowed a summary overview of all project features, except the Gacheta dam site, and was particularly helpful in evaluating power house alternatives. The possibility for an underground powerhouse near Algodones (Mambita alternative III) could not be assessed properly as the cost of an underground powerhouse is heavily dependent upon the detailed qualities of the rock, in particular the spacing and orientation of seams. A program of deep exploration, including an underground adit, will be essential to design an underground plant, and a program of exploratory drilling is required even before the feasibility can be established. For first order cost comparisons, which might be the most telling aspects of the feasibility determination, I believe optimistic assumptions of rock quality can be made.

The two surface powerhouse alternatives at Mambita area both on rockhard shale, conglomerate and limestone. The ridge at the north end of the Mambita terrace that has been considered for the penstock leading to both powerhouse alternatives, is sound in appearance. Perhaps the most advantageous alternative will be to lower the grade of the tunnel so that its portal is approximately at elevation 950; in this event, the best appearing location for the tunnel portal will be at the north end of the spur between Rio Rucio and Ouebrada Seca. The conduction from the tunnel portal to the powerhouse alternative near the present bridge would then pass just south of the village of Mambita.

The tunnel line from Mambita to the Batatas will pass through competent, outcropping formations for approximately one half of its length, and through rocks of medium competency - hard shale, phyllite and argillite, for the other half. I would guess that at least one third, and possibly one half of the total length of the tunnel, can be left unlined, without any support of treatment except for dental work in erodible seams, placed after a final inspection of the tunnel. The tunnel to Chivor will pass through considerable limestone, perhaps for one third of the total length; at least one half of the tunnel will be in hard shale, which is to be compared with the hard shale of the current Chivor tunnels. Shotcrete lining may be necessary in most of the phyllite and hard shale sections to prevent deterioration of the tunnel due to the action of the water. The limestone sections should not require lining.

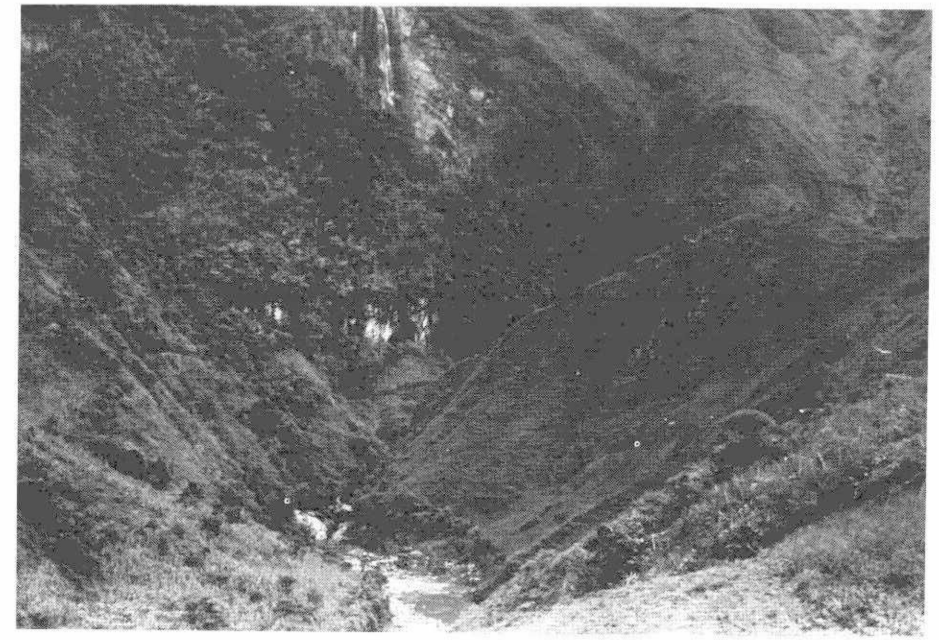
From the geological point of view, the various tunnel and powerhouse alternatives all appear quite similar (with the exception of the Medina alternative, not discussed here). Therefore, the selection of the most feasible alternative should be fairly straightforward, once the principal control points are determined. These are the dam, and the powerhouse. From a geological point of view, the Ubala site is preferable to the El Cobre site. The Mambita powerhouse alternative also seems to lie on better rock and to provide better penstock foundation conditions than the Chivor III alternative. Drill holes should be pursued, now, to check the acceptability of Ubala dam site and any other major sites, before final lines and grades are set for the project.

A series of ground and helicopter photographs are enclosed, together with negatives. I lack facilities here to integrate them in the report and hope that Mr. Torres will be able to prepare suitable examples for distribution from the collection.

Very sincerely,

Richard E. Goodman

2:1
18:1
45
32
48
1.400
DEFLECTOR
1.450
1.058.000 E
REBOSADERO PARA PRESA
CRESTA EL. 1530
NELES DE DESVIACION
FAULT
FAULT
1.500
1.550
1.600
0
50
150
200m



COBRE DAM SITE

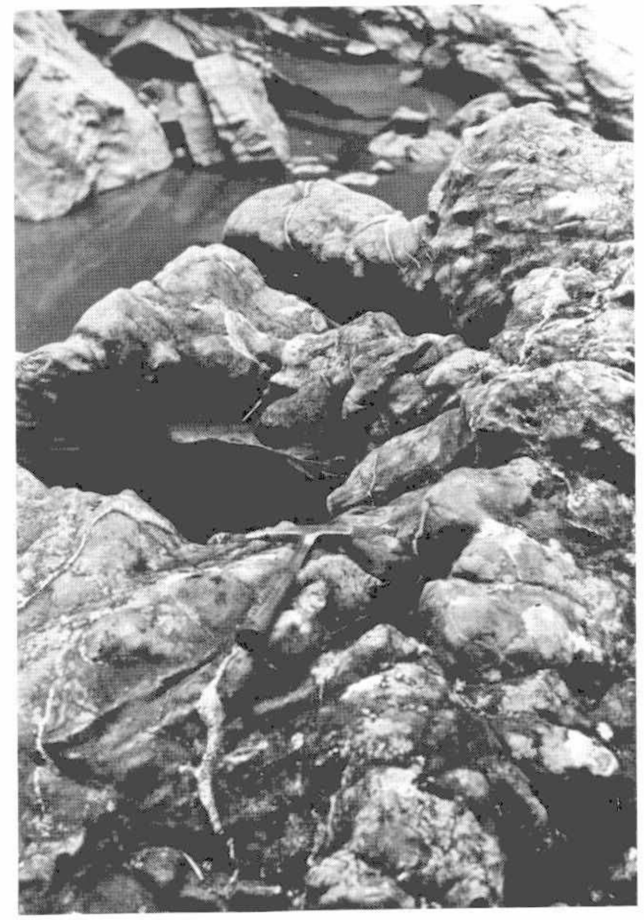
Looking downstream

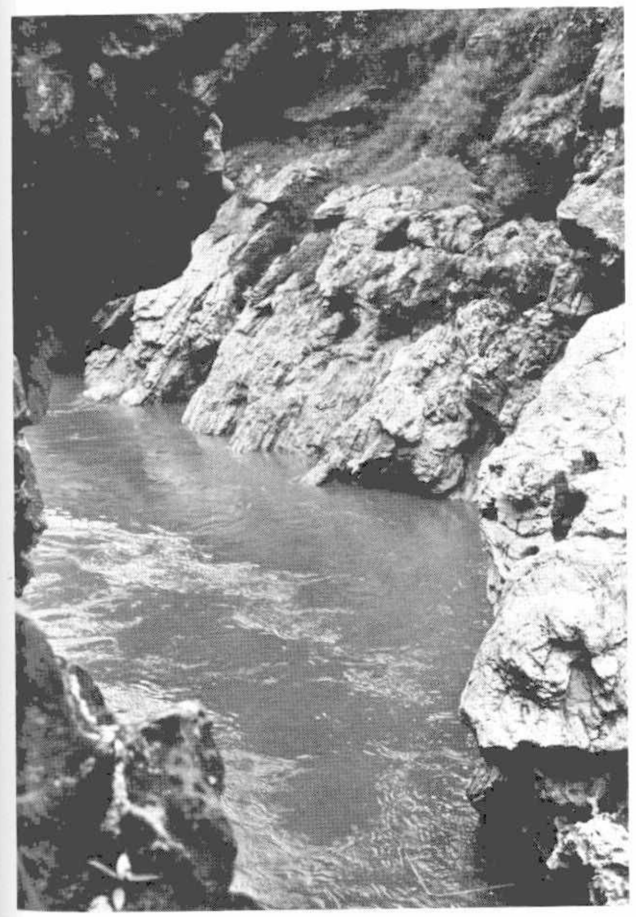
COBRE DAM SITE

Ridge to be cut for construction of the spillway

COBRE DAM SITE

Sound rock outcrops



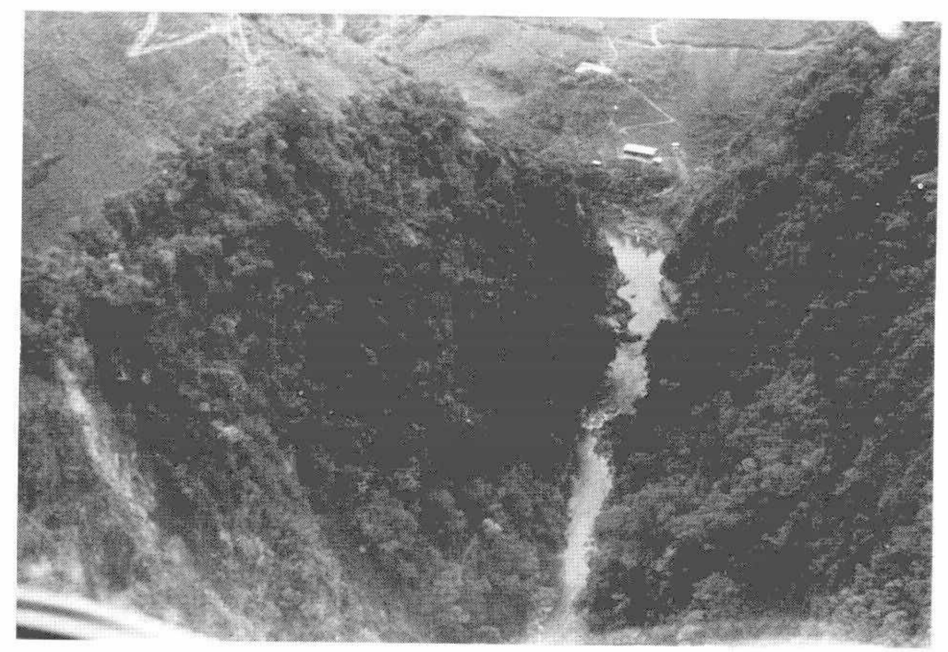


COBRE DAM SITE

Sound rock outcrops

UBALA DAM SITE

Left abutment looking downstream



UBALA DAM SITE

Downstream end. Temporary field camp is ahown

GACHETA DAM

Borrow area - General view

TUNJITA BORROW AREA

General view

GUZMAN CREEK BORROW AREA

Panoramic view

GUZMAN CREEK

BORROW AREA

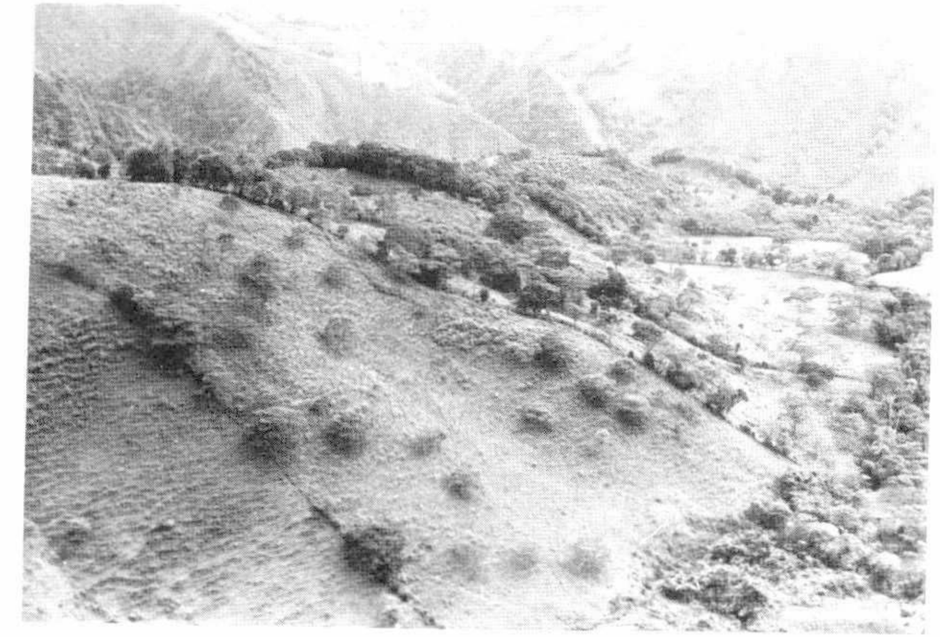
Large blocks of limestone present in the area

GUZMAN CREEK BORROW AREA

Material existing in the zone

GUZMAN CREEK
BORROW AREA

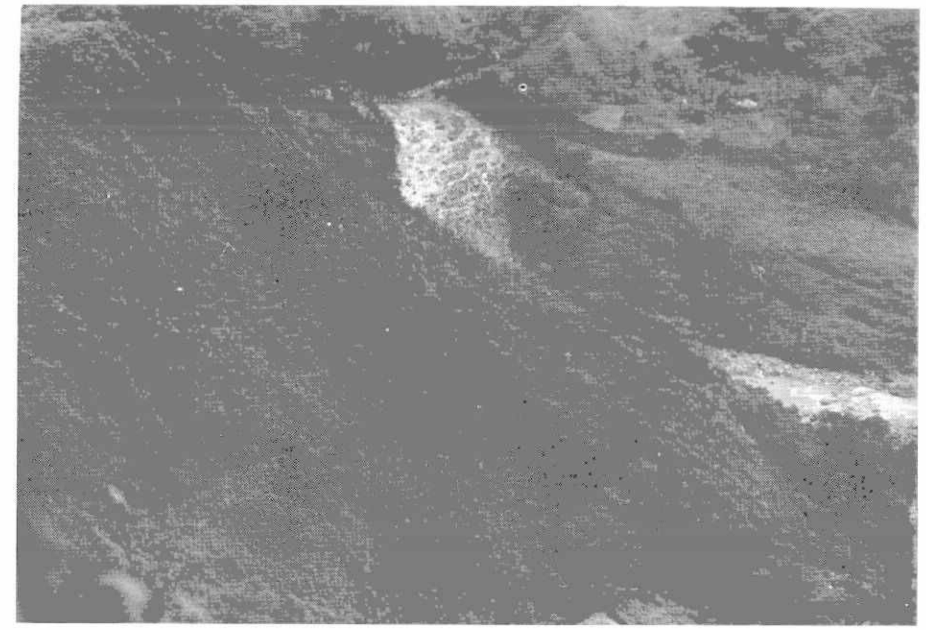
Material existing in the zone



Partial view of ridge and terrace along the alignment of surface penstock-Mámbita I alternative

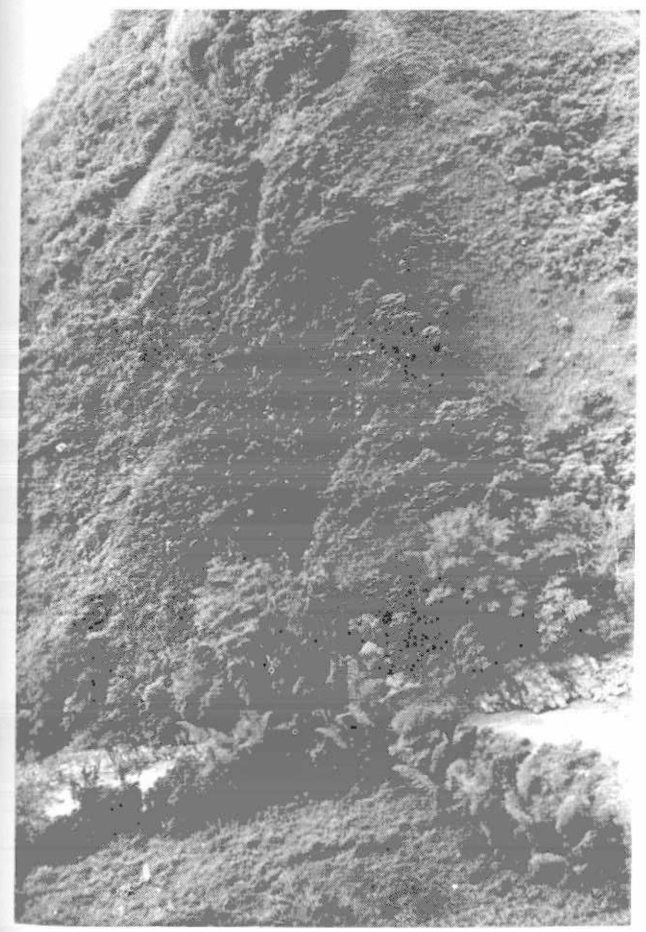
Power tunnel outlet portal area

Mámbita alternative



Surface powerhouse site

Mámbita alternative looking from
Mámbita terrace



Partial view of ridge along the last portion of surface penstock alignment. Looking upwards from the powerhouse site

Powerhouse site - Mámbita I alternative

Rock outcrops

Powerhouse site - Mámbita I alternative

Rock outcrops

INFORME 7

GEOLOGIA

16, Sept. 1973

Dr. Hernando Quijano
INGETEC
Apartado Aereo 5099
Bogotá, Colombia

Dear Dr. Quijano:

This is a report of my review of the Guavio Project exploration, September 3 and 4, 1973. I arrived in Bogotá, from Medellin, on Monday, September 3, reviewed Ubala dam site exploration results with you and Messers Rojas, Castellano and Vela, and then travelled to Gachala, where Mr. Molina of ISA joined us. On Tuesday, I reviewed the drilling program in the field, inspected the new trails I had not previously seen and then new rock exposures, and observed the equipment and procedures for drilling, and water pressure testing. I was accompanied by Rojas, Castellano, Vela, and Molina. We then inspected most of the drill cores and reviewed the logs. On September 4, before travelling to Santa María, I discussed the accompanying(*) integrated exploration log and exploration objectives. The following is a record of the discussions in your office on September 6.

Eight drill holes have been completed to date, with a total length of 564 meters. These holes have generally confirmed preliminary ideas formed during surface mapping about the nature of the rock and its structure. Core recovery is good and the rock is mainly hard and sound. Fractures are widely to moderately spaced over most of the holes, although more closely fractured zones are numerous. A number of partly dissolved horizons, with interconnected voids occur (marked "d" on the section). No caverns were intersected with the possible exception of an occurrence in hole FP8 where the drill reportedly dropped under its own weight.

Water was lost from the drill holes over a significant footage but water levels have returned to a phreatic surface paralleling the topography on the right bank. This was disturbingly not the case in borehold FP8 on the left abutment, which remained dry (356 feet deep).

(*) Previously presented in your office.

Unfortunately, no piezometer has been set in this hole and further information will not be forthcoming. It is therefore urgent that FP6, being drilled now, be continued deeply, in fact as deeply as possible, to ascertain the cause and seriousness of the low phreatic levels. Another hole between FP8 and FP10 will be desirable as well.

Water pressure test results are also disturbing, with water losses of more than 10 Lugeons in most sections. I am suspicious about these results, however, as packers only 10 cm. long have been used. I recommend a packer length of 1 meter so that we can be certain water isn't flowing past the packers back into the hole. If a more suitable packer arrangement can not be obtained, a flow balance can be established to reveal true water loss data. This requires monitoring input and output water flows during steady state seepage. All water flows should be plotted against pressure for each test, past and future. The hydrologic data are important for the assessment of the dam site. Therefore every effort should be given to see that the water pressure test results are valid. If there is trouble obtaining a long packer arrangement in Colombia, I can contact sources in the U.S.A. on your behalf.

The results of the exploration program are incomplete. No preliminary conclusions can be drawn about structures and control elevations until the first phase of exploration has been completed.

Your geological and engineering staff should be commended for the orderly and efficient way in which the cores are being boxed, logged, and stored. I have made a few verbal suggestions about logging details and enclose a blank log form, following these discussions, which should be of interest to Mr. Rojas.

Very sincerely,

Richard E. Goodman

SNOWY MOUNTAINS HYDRO-ELECTRIC AUTHORITY

GEOLOGICAL LOG OF DRILL HOLE

PROJECT .... FEATURE ....
HOLE No .... CO-ORDINATES: E .... M .... , N .... M .... R.L. GROUND ....
LOCATION .... ANGLE FROM HORIZONTAL .... DIRECTION ....

ROCK TYPE & DEGREE OF WEATHERING SHOWN IN CORE	DESCRIPTION OF CORE OR CUTTINGS WHERE CORE LOST	R.L. / CASING	DEPTH / SIZE OF CORE	LOG	LIFT & CORE RE-COVERY %	STRUCTURES JOINTS, VEINS, SEAMS, FAULTS, CRUSHED ZONES	WATER LEVEL	WATER PRESSURE TEST GALLONS PER MINUTE LOSS 5 10

TEST PRESSURES SHOWN IN POUNDS PER SQ. INCH MEASURED AT GROUND SURFACE

DRILL No .... TYPE .... DRILLER .... COMMENCED .... COMPLETED	EXPLANATION CASING IN HOLE DURING DRILLING ▌ WATER LEVEL IN DRILL HOLE ◁ (DATE)	ENGINEERING GEOLOGY BRANCH LOGGED BY .... DRAWN BY .... CHECKED BY .... SHEET .... OF .... DRAWING No ....

INFORME 8

GEOLOGIA

9 January, 1974

Dr. Hernando Quijano
INGETEC
Apartado Aéreo 5099
Bogota, Colombia

Dear Dr. Quijano:

This concerns the Guavio project. On January 2 I discussed the results of most recent exploration activities at Ubala dam site with Messrs. Sierra, Marulanda, and Rojas at your office; then, on January 3, I took a helicopter trip to Mambita to inspect the powerhouse sites, as a side trip from Santa Maria.

I. The Ubala dam site continues to look feasible and the recent exploration results are encouraging. We should continue and complete exploration of a high dam scheme and hold as a second alternative a low Ubala dam together with a storage reservoir at Píngaro. The drilling program has reinforced the impressions from mapping of site geology that the rock, particularly the units we have been mapping as Paleozoic rock, are competent and relatively free of discontinuities. Except for some slick planes widely spaced in the core, and widely spaced solution zones, the core has been sound and the core recovery high. The origin and nature of the slick planes may be better understood after driving exploratory adits in the next stage of exploration. The principal questions of exploration have concerned the possibilities of leakage from the reservoir through karst formations, either near the dam site or at some point remote from the dam site. Several general observations are relevant to this question.

1) The dam site is on the main stream, which has a considerable flow. Serious leakage possibilities are more worrisome on tributary rivers, for two reasons. First the karstic formations on tributaries have the possibility of containing important transfer conduits conveying water underground to the main river valley. Secondly, the tributary river flow being less, leakage is more critical. The Guavio is the main river and the karstic formations that have been found, all in the Cretaceous rock along the Guavio canyon discharge water, i.e.,

they represent the downstreammost components of the underground water conveyance systems.

2) The Cretaceous rocks, which demonstrate karstic features much more prominently than the Paleozoic rocks, seem to lack potential leakage routes as they do not crop out below reservoir elevation in any other canyon. The only uncertainty is the left bank at the dam site and this will be checked in the exploration program yet to come.

3) The drill holes show water levels which generally follow the topographic profile at the dam site, suggesting that the valley is not leaking water to another place but rather that the ground water flow is towards the river.

4) The unconformity on the left side of the river near the top of the dam was considered as a possibly pervious horizon. It has been shown, in the one boring through it, apparently to be a tight contact between shale and limestone. Again, this will be checked in further exploration.

II. In the future exploration program we hope to establish definitely whether or not there can be any leakage problem. With regard to the eventualities of leakage from the reservoir, the following measures are proposed:

1) If gauge stations could develop river flow figures with a precision of say 0.2 m3/sec, it would be instructive to establish a number of stations above, in and below the reservoir to allow a flow balance to be constructed. This would show whether or not significant outflows or inflows from the ground water are occurring at the present time.

2) We have presumed that the Paleozoic rocks are reasonably water-tight and this is suggested by the results of exploration. However it would be helpful to establish this result more confidently. A large scale permeability test in situ is warranted. I suggest that the exploration adits be driven with a downgrade so that they can be flooded. Then the amount of water which has to be added to maintain them continuously full can be monitored. Also, tracers can be used to indicate where lost water reappears. In one adit, a bulkhead can be constructed

and a water pressure test conducted, to test the permeability at pressures comparable to those associated with the high dam, i.e. 300 psi.

3) Tracer studies should be undertaken in the reservoir area, and well above the reservoir maximum water surface, to establish the paths of subterranean flow. Where are the sources for the springs that have been observed ? New discharges may be identified as well through such a program, which can be conducted with radioactive tracers or with dyes.

4) In order to compare the geological situation of Ubala reservoir with other reservoirs in limestone terrains, prepare diagrams of minimum flow path for water to escape the reservoir, as suggested by R. Therond in his monograph on karstic reservoirs (Research on Watertightness of Dammed-up Lakes in Karstic Country - in French - 1973, Eyrolles).

III. With regard to the question of leakage in the rock at the dam site, the following program is proposed:

1) Map the rock walls of the Guavio Canyon, and the Batatas and Rio Chivor canyons to establish the elevations of the bottom of the Crestaceous limestone rather precisely. If this requires trenching, it should be done.

2) Make a careful and complete inventory of springs downstream from the dam site below the maximum water elevation.

3) Make a tracer study to find the sources of water for any springs identified.

4) Study the permeability of the bedrock with continued packer tests using the new hydraulic packers to see if these modify the previous conclusions from earlier tests. (I delivered two 1 meter long hydraulic packers manufactured by Drilling Accessory and Manufacturing Company, P.O. Box 5768, Dallas Texas (catalogue # 275) supplied to me by Sprague and Henwood Co., Scranton, Pa.) Continue analysis of all data from water pressure tests, including plotting of all test results and classification of flow regimes.

5) Drill additional holes to complete the profile of water levels across the valley at the dam site. This will require at least one deeper hole near PTFP 1A and one higher on the right bank.

6) Excavate and carefully log exploratory adits on both banks. The scheme for location and driving of adits previously prepared is adequate and is recommended. This includes two adits on the right bank and one on the left, with drifting up and down stream after gaining entry and driving along the dam axis for some distance. As noted above, the adits should be driven on a downgrade so that they can be flooded.

IV. In addition, to complete the exploration program at this stage, the following is recommended:

1) Drill one hole near the stream bottom on the dam axis to establish geological conditions for at least 100 meters below the river level. There is presently insufficient knowledge of the subsurface directly below the river.

2) Conduct a grout test to gain an estimate of the grout take in the bedrock. Extrapolation of the water pressure tests for this purpose may be unrealiable.

V. In the next stage of exploration at the dam site, other drill holes should be pursued to establish rock conditions below structures and excavations. But these are not essential for determining the feasibility of the project now.

VI. With regard to the power house sites: the surface power house alternative looks feasible as currently aligned. The powerhouse foundation should find rock - shale - at the elevation of the river. Drill holes necessary to confirm this should be bored now. The penstock will cross the Santa Maria fault. I recommend a trench on Mambita terrace to observe the fault trace. The trench walls should be photographed as well as logged.

The underground powerhouse alternative should be moved about 200 meters upstream to gain more distance from the Santa Maria fault.

The tailrace tunnel will cross the fault. Since the surface powerhouse seems satisfactory, it should be the preferred alternative.

Very sincerely,

Richard E. Goodman

HIDROLOGY

INFORME DE

MR. FRANKLIN F. SNYDER

HYDROLOGY STUDY FOR GUAVIO PROJECTS

COLOMBIA, SOUTH AMERICA

REPORT TO

INGETEC

Bogotá, Colombia

by

Franklin F. Snyder, CE

May 1972

HYDROLOGY STUDY FOR GUAVIO PROJECTS

GENERAL

1. Introduction. The Guavio hydroelectric project is located in the headwaters of Rio Guavio east-north-east of Bogotá, Colombia. Present planning of the project for Interconexion Electrica S.A. includes three main dams, power house, and connecting tunnels. The reservoir projects are Gacheta and El Aguila on Rio Guavio and El tigre on Rio Negro, a tributary to Rio Guavio below El Aguila. The Tigre reservoir is to be connected by tunnel to El Aguila with a power penstock from there to the powerhouse on Rio Guavio near the junction of Rio Bata.

A dam site on the Rio Farallones was under consideration at the time this study was initiated, but has since been eliminated. Material already prepared by the time notice was received of no further interest in the Farallones dam site is included herein.

2. Purpose and Scope. The purpose of this study is to develop the flood hydrology for the project basins including design hydrographs for the spillway and diversion facilities. The spillway design floods (inflow) are computed using the unit hydrograph procedure and estimates of probable maximum precipitation developed by Mr. Dwight E. Nunn (1).

3. Description of Basins

a. General. The project basins lie in the headwaters of Rio Guavio northeast of Bogotá. The Rio Guavio is a tributary of Rio Upia, a tributary to Rio Meta which flows into the Rio Orinoco and the Atlantic Ocean. The Gacheta dam site is on the Rio Guavio about eight kilometers above its junction with Rio Farallones near the upper end of the proposed El Aguila Reservoir. The Rio Negro joins Rio Guavio about twelve kilometers below the El Aguila damsite.

The headwaters of Rio Guavio constitute a broad valley high on the east side of the main ridge of the Cordillera Oriental of the Andes, reaching to the ridge and facing to the southeast and east. The upper divides are common with the Rio Chuza, Rio Blanco, Rio Tomine and

Rio Somondoco, and range from 3100 to 3700 meters above sea level.

b. El Tigre. El Tigre damsite is on Rio Negro at an elevation of 1800 meters about five kilometers (km) above its junction with Rio Guavio. The drainage area is 81 square kilometers (km^2) and the principal channel length is 17.5 km with a weighted slope of 4.6 percent.

c. Gacheta. The Gacheta damsite is on Rio Guavio at an elevation of 1570 meters about 8 km above the mouth of Rio Farallones. The drainage area is 700 km^2 and the length of the principal channel is 42 km with a weighted slope of 2.6 percent.

d. Farallones. The Farallones damsite is on Rio Farallones at an elevation of 1815 meters about 9 km above its junction with Rio Guavio. The drainage area is 301 km^2 and the principal channel length is 27.5 km with a weighted slope of 3.5 percent.

e. El Aguila. The Aguila damsite is on Rio Guavio at an elevation of 1410 meters about 7 km below the mouth of Rio Farallones. The drainage area is 1230 km^2 and the principal channel length is 57 km with a weighted slope of 2.2 percent.

4. Records. There are eight non-recording rainfall stations in the Guavio basin with 6 to 11 years of record. The station locations are such that estimates of average basin rainfall from observations are impractical.

Of the 4 non-recording stream gages, 3 are near the proposed damsites of El Tigre, Gacheta and El Aguila. The average flow for the Farallones site was estimated by comparison of two of the station records. The period of record is 8 years.

5. Hydrologic Data. On a basis of the runoff observations, the available rainfall data, and consideration of annual losses, the following estimates of average annual rainfall and runoff have been made:

Project	Average Annual Rainfall, mm	Average Annual Runoff, mm	Difference mm
El Tigre	3260	2570	690
Gacheta	1880	1380	500
Farallones	2080	1550	530
El Aguila	1980	1465	515

A little less than half of the runoff is estimated to be from groundwater sources.

6. Past Floods. Peak discharges are not available for the gaging stations. The highest values observed at the non-recording stream gages are as follows :

Location	Drainage Area, km^2	Highest Observed Discharge m^3/sec
Rio Nego, LaGloria	76	88
Rio Chivor, Ubala	68	122
Rio Guavio, Chusneque	690	520
Rio Guavio, Ubala	1110	560

SPILLWAY DESIGN FLOODS

7. Probable Maximum Precipitation. The probable maximum rainfall for the three project basins is given in Table 1 through 3. These one-hour values of rainfall were developed by Mr. Nunn and have been arranged in a critical sequence. The arrangement was accomplished by placing the rainfall (direct runoff) increments in reverse order of magnitude to that of the unit hydrograph ordinates for the respective project basins.

8. Losses. The losses given in Tables 1-3 were subtracted as shown to obtain the hourly values of direct runoff. The probable maximum storm could be expected to occur during a rainy period when the soil moisture capacity of the top soils would be essentially filled. Under such conditions the infiltration of rainfall would be limited to the seepage of gravity water through the ground water zone. The values of total storm loss were based on recharge and groundwater storage characteristics of the respective basins. The groundwater storage capacities of the project basins are relatively large and somewhat similar. Losses of 100 millimeters (mm) for El Tigre, 80 mm for Gacheta, and 86 mm for El Aguila were assumed for the 24-hour duration of the probable maximum storm.

9. Unit Hydrographs. Synthetic unit hydrographs (2) were computed for the three dam sites. Separate treatment for the surface areas of the proposed reservoirs was not necessary because they constitute a relatively small percentage of the basin-areas. The effects of the deep reservoirs at full pool conditions on the discharge characteristics were incorporated in the synthetic unit hydrographs by appropriate reduction in the lengths of the drainage basins. As no flood hydrographs were available, the discharge characteristics were based on those for the adjacent Chingaza and Bata rivers keeping in mind the rugged character of the Guavio basin.

A unit hydrograph constitutes the discharge distribution of unit volume, 1 mm over the basin area, of direct runoff resulting from rainfall of unit duration and specified areal distribution. One-half hour unit duration was used for El Tigre and one hour for Gacheta and El Aguila. The values of lag, t_p, and peak discharge, Q_p, for the unit hydrographs were computed as follows :

$$t_p \text{ equals } C_t \ (L \times L_{ca})^{0.3} \qquad (1)$$

where C_t is a coefficient varying from about 0.4 to 1.5 hours/km, t_p is the lag in hours from the center of mass of rainfall to time discharge, L is the length of the principal channel in kilometers (km) and L_{ca} is the channel distance from the center of area of the basin to the point of interest in km. The computed value of t_p is for a duration of effective rainfall equal to $t_p/5.5$. When a value of unit duration significantly different from $t_p/5.5$ is used, the computed value of t_p is adjusted to be consistent with the selected unit of time.

$$Q_p \text{ equals } A \times \frac{0.172}{t_p} \qquad (2)$$

where Q_p is the peak discharge in cubic meters per second (cms) for the unit hydrograph of one mm volume. A is the drainage area in square kilometers (km^2) and t_p is the basin lag in hours.

The following tabulation gives the input data and results of computations for the synthetic unit hydrograph characteristics. The peak discharges, Q_p, were computed using the values of t_p without adjustment for the adopted unit of rainfall duration to be conservative.

in which L' is the length of an equivalent channel having the same time of concentration, T_c, in hours, but with a standard slope of 1% and a standard friction factor of 0.10, L is the length of the principal channel in km, s is the weighted slope of the principal channel, in percent, determined as the mean height above the outlet of the channel profile divided by one-half the length, and n is the friction factor (Manning) for the channel.

$$Q_p \text{ equals } K\,A\,I_r \qquad (5)$$

where Q_p is the peak discharge in cms for the designated frequency from rainfall of that frequency and duration T_c, A is the drainage area in km^2, and I_r is the average rate of direct runoff, mm/hour, over the time of concentration. The coefficient, K varies with the basin storage and shape characteristics. The values of K used herein for Guavio project basins were patterned after those for the Chingaza and Bata rivers and are given in the respective computations, Tables 4-7.

13. Rainfall-Duration-Frequency. Rainfall-frequency studies were made for daily, monthly and annual data available for La Floresta and other stations in the Guavio basin. On a basis of the analyses of these data and comparison with the estimated annual rainfall for the project basins, the following values were adopted as representative of short period rainfall for the project basins.

Project & Duration*	Mean of Annual maximum**, mm	Coefficient of Variation, %	Standard Deviation mm
El Tigre, 3.3 hrs.	48.0	25	12.0
Gacheta, 6.6 "	41.6	20	8.32
Farallones 4.2 hrs	38.7	20	7.74
El Aguila 8.4 hrs	44.4	20	8.88

* Time of Concentration
** Frequency of 2.3 years after Gumbel

The rainfall amounts for other frequencies were computed on a basis of the Gumbel theory of extreme values using the 2.3 - year values and the coefficients of variation as given in the tabulation.

14. Rainfall-Runoff Relation. An average runoff condition is assumed in the synthetic flood frequency procedure because the frequency of peak discharge from a particular storm is not necessarily the same as that of the storm rainfall. As verified in many applications in humid regions, this can be accomplished by plotting the average monthly rainfall value against the quotient of average monthly runoff divided by average monthly rainfall, in % and drawing a percentage of runoff curve through that point parallel to a reference line (3). Since the reference line represents a condition where about 50% of the total runoff comes from groundwater sources, some adjustment of the plotted point (working line) is necessary for basins with groundwater runoff significantly greater or smaller than 50% of total runoff. The percentage of runoff is increased for basins with a low proportion of groundwater runoff and decreased for basins with a high proportion, because the desired runoff percentage is that for direct runoff. This procedure tacitly assumes that the percentage of direct runoff in a storm is about twice that of the monthly average.

15. Results. Tables 4-7 give the basic data and the results of the flood frequency computations for the four project basins. Since peak discharges are not available for the stream gaging stations in the Rio Guavio basin, no direct verification of the results is possible. However, the highest observed discharges at the gaging stations as given in section 6 provide some information in this connection.

16. Design Floods for Diversion Studies. The design flood peaks from the flood frequency computations are as follows :

Project	100-Year Discharge cms	1000-Year Discharge cms
El Tigre	288	402
Gacheta	928	1230
Farallones	592	786
El Aguila	1350	1800

The ordinates for the 100-year and 1000-year floods are given in Tables 1-3. The flood hydrographs are for natural conditions before construction and filling of the reservoirs and were based on synthetically derived hydrograph characteristics and the direct runoff volumes as given in Tables 4, 5, and 6. Normal values of base flow were added.

If flood volume becomes a consideration in the analysis of diversion facilities, allowance must be made for the fact that two floods of about the same magnitude can occur within a 24-hour period on the smaller basins.

17. Acknowledgments. The inspection of the Rio Guavio basin in the company of a representative of Ingetec was most informative and provided an opportunity to assess some of the surface features of the area. The provision of available hydrologic data as collected and processed is gratefully acknowledged.

Appreciation is also expressed for the efforts of Mr. Nunn in deriving the estimates of probable maximum precipitation for the project basins.

REFERENCES

1. Nunn, Dwight E., "Probable Maximum Precipitation for Guavio Projects, Colombia," Fairfax, Virginia, 1972.

2. Snyder, Franklin F., "Synthetic Unit Graphs", Trans. Am. Geophysical Union, 1938, Part I, pp 447-454.

3. Snyder, Franklin F., "Synthetic Flood Frequency", Jour. Hydraulics Div., A.S.C.E., Vol. 84, Paper No. 1808, October 1958.

TABLE 1. EL TIGRE DESIGN FLOODS

Time hours	Prob. Max. Precip. mm	Loss mm	Direct Runoff mm	UHG 1 mm in 1 hour cms	Spillway Design Flood cms	100 - Year Flood cms	1000 - Year Flood cms
0.5				1.0			
1.0	7	7	0	4.0		33	46
1.5				6.9			
2.0	8	8	0	8.0		93	130
2.5				7.1			
3.0	8	8	0	5.5		214	300
3.5				3.8			
4.0	9	9	0	2.7		292	408
4.5				1.9			
5.0	9	9	0	1.4		204	285
5.5				1.0			
6.0	10	8	2	0.7	13	94	131
6.5				.5			
7.0	10	7	3	.3	33	52	73
7.5				.2			
8.0	12	6	6	.1	64	34	48
9	12	5	7	45.1	104	23	32
10	13	4	9		142	15	20
11	14	3	11		183	10	14
12	14	2	12		222	7	10
13	16	2	14		257	1071	1497
14	16	2	14		289		
15	17	2	15		315		
16	18	2	16		335		
17	18	2	16		353		
18	20	2	18		374		
19	30	2	28		437		
20	35	2	33		550		
21	39	2	37		668		
22	56	2	54		827		
23	110	2	108		1230		
24	44	2	42		1520		
25	545	100	445		1180		
26					651		
27					330		
28					173		
29					94		
30					54		
31					35		
32					24		
					10457		

TABLE 2. GACHETA DESIGN FLOODS

Time hours	Prob. Max. Precip. mm	Loss mm	Direct Runoff mm	UHG 1 mm in 1 hour cms	Spillway Design Flood cms	100 - Year Flood cms	1000 - Year Flood cms
1	6	6	0	14		90	119
2	7	7	0	38		260	345
3	7	7	0	45		511	677
4	7	7	0	40		761	1010
5	8	6	2	24	48	902	1200
6	8	5	3	13	138	940	1250
7	8	4	4	8	280	913	1210
8	9	4	5	5.5	458	834	1110
9	9	3	6	4.0	644	635	842
10	10	3	7	2.5	832	396	525
11	10	2	8	1.0	1020	237	314
12	11	2	9	195.0	1220	138	183
13	11	2	9		1400	80	106
14	12	2	10		1560	47	62
15	12	2	10		1700	16	21
16	13	2	11		1810	6760	8974
17	16	2	14		1960		
18	16	2	14		2150		
19	20	2	18		2430		
20	24	2	22		2810		
21	42	2	40		3510		
22	65	2	63		4900		
23	30	2	28		6390		
24	23	2	21		6900		
25	384	80	304		6210		
26					4550		
27					2970		
28					1820		
29					1160		
30					780		
31					510		
32					300		
33					180		
34					100		
					60740		

TABLE 3. EL AGUILA DESIGN FLOODS

Time hours	Prob. Max. Precip. mm	Loss mm	Direct Runoff mm	UHG 1 mm in 1 hour cms	Spillway Design Flood cms	100 - Year Flood cms	1000 - Year Flood cms
1	7	7	0	13		98	131
2	7	7	0	34		268	357
3	7	7	0	52		460	613
4	8	8	0	56		670	894
5	8	8	0	53		920	1230
6	8	7	1	43	43	1140	1520
7	8	5	3	27	103	1300	1730
8	8	4	4	19	236	1370	1830
9	8	3	5	14	444	1340	1790
10	9	2	7	11	723	1220	1630
11	10	2	8	8	1060	1030	1370
12	10	2	8	6	1410	832	1110
13	12	2	10	4	1760	634	845
14	12	2	10	2	2100	484	645
15	12	2	10	1	2430	363	484
16	14	2	12	343	2740	262	349
17	14	2	12		3050	171	228
18	16	2	14		3360	90	120
19	23	2	21		3770	12652	16876
20	40	2	38		4520		
21	60	2	58		5960		
22	24	2	22		7680		
23	22	2	20		8940		
24	13	2	11		9320		
25	360	86	274		8780		
26					7450		
27					5710		
28					4280		
29					3110		
30					2270		
31					1660		
32					1230		
33					860		
34					565		
35					360		
36					216		
37					150		
38					116		
					96406		

TABLE 3-A. EL AGUILA SPILLWAY DESIGN FLOOD (Without Gacheta Reservoir)

Time hours	Prob. Max. Precip. mm	Loss mm	Direct Runoff mm	UHG 1 mm in 1 hour cms	Spillway Design Flood cms
1	7	7	0	6	
2	7	7	0	16	
3	7	7	0	30	
4	8	8	0	38	
5	8	8	0	41	
6	8	7	1	42	36
7	8	5	3	39	64
8	8	4	4	33	132
9	8	3	5	25	253
10	9	2	7	19	430
11	10	2	8	15	651
12	12	2	10	12	947
13	12	2	10	9	1270
14	12	2	10	7	1620
15	14	2	12	5	1970
16	14	2	12	3	2330
17	22	2	20	2	2710
18	24	2	22	1	3170
19	40	2	38	343	3810
20	60	2	58		4780
21	23	2	21		5960
22	16	2	14		7100
23	13	2	11		7810
24	10	2	8		8050
25	360	86	274		7860
26					7240
27					6260
28					5190
29					4190
30					3340
31					2610
32					1990
33					1510
34					1110
35					780
36					544
37					358
38					224
39					160
40					121
41					100
					96680

TABLE 4. COMPUTATION OF FLOOD FREQUENCY FOR EL TIGRE

Dr. A. 81 km^2; L 17.5 km; Slope 4.6%; n 0.06

T_c equals $1.28\left(\frac{10 \times 17.5 \times 0.06}{\sqrt{4.6}}\right)^{0.6}$ equals 3.3 hours

Est. average monthly rainfall - 272 mm; runoff - 214 mm; % runoff - 79

Q_p equals 0.26 A I_r equals 21.1 I_r equals 21.1/3.3 I equals 6.4 I

Freq., Recur. Int., Years	2.3	5	10	25	50	100	1000
Basin Rainfall, 3.3 hrs., mm	48	56.6	63.8	72.7	79.3	85.8	107.5
Direct Runoff, %	42	44.5	46.5	49.0	50.8	52.5	58.3
Direct Runoff, I, mm	20.2	25.2	29.7	35.6	40.3	45.0	62.8
Q_p, cms	129	161	190	228	258	288	402

TABLE. 5. COMPUTATION OF FLOOD FREQUENCY FOR GACHETA

Dr. A. 700 km^2; L 42 km; Slope 2.6%; n 0.06

T_c equals $1.28\left(\frac{10 \times 42 \times 0.06}{\sqrt{2.6}}\right)^{0.6}$ equals 6.6 hours

Est. Average monthly rainfall - 157 mm; runoff - 115 mm; % runoff - 73.4

Q_p equals 0.26 A I_r equals 182 I_r equals 182/6.6 I equals 27.6 I

Freq., Recur. Int., Years	2.3	5	10	25	50	100	1000
Basin Rainfall, 6.6 hrs., mm	41.6	47.6	52.6	58.7	63.3	67.8	82.9
Direct Runoff, %	42.0	43.8	45.3	47.0	48.2	49.5	53.8
Direct Runoff, I, mm	17.5	20.9	23.8	27.5	30.5	33.6	44.6
Qp, cms	483	577	658	760	843	928	1230

TABLE 6. COMPUTATION OF FLOOD FREQUENCY FOR EL AGUILA

Dr. A. 1230 km^2; L 57 km; Slope 2.2%; n 0.06

T_c equals $1.28\left(\frac{10 \times 57 \times 0.06}{\sqrt{2.2}}\right)^{0.6}$ equals 8.4 hours

Est. average monthly rainfall - 165 mm; runoff - 122 mm;
% runoff - 74

Q_p equals 0.26 A I_r equals 320 I_r equals 320/8.4 I equals 38.1 I

Freq., Recur. Int., Years	2.3	5	10	25	50	100	1000
Basin Rainfall, 8.4 hrs., mm	44.4	50.8	56.1	62.6	67.6	72.4	88.4
Direct Runoff, %	41.0	42.8	44.2	46.2	47.5	48.9	53.3
Direct Runoff, I, mm	18.2	21.5	24.8	28.2	32.1	35.5	47.1
Q_p, cms.	695	828	945	1110	1220	1350	1800

TABLE 7. COMPUTATION OF FLOOD FREQUENCY FOR FARALLONES

Dr. A. 301 km^2; L 22.5 km; Slope 3.5%; n 0.06

T_c equals $1.28\left(\frac{10 \times 22.5 \times 0.06}{\sqrt{3.5}}\right)^{0.6}$ equals 4.2 hours

Est. average monthly rainfall - 173 mm; runoff - 129 mm;
% runoff - 74.5

Q_p equals 0.28 A I_r equals 84.3 I_r equals 84.3/4.2 I equals 20.1 I

Freq., Recur. Int., Years	2.3	5	10	25	50	100	1000
Basin Rainfall, 4.2 hrs., mm	38.7	44.3	48.9	54.6	58.9	63.1	77.1
Direct Runoff, %	39.8	41.4	42.6	44.3	45.3	46.7	50.7
Direct Runoff, mm	15.4	18.2	20.8	23.2	26.7	29.5	39.1
Q_p, cms	310	366	419	487	536	592	786

INFORME DE

MR. DWIGHT E. NUNN

September 1, 1972

Mr. Carlos S. Ospina
Ingetec
Apartado Aereo 5099
Bogotá, Colombia

Dear Mr. Ospina:

In accordance with your request there is transmitted herewith my final report of probable maximum precipitation estimates for the Guavio development. The report was prepared for Ingetec and for use by Mr. Snyder in his hydrological engineering development of the (Probable Maximum Flood Estimates) spillway design flood criteria for the three projects, Gacheta, El Aguila and El Tigre sites.

The probable maximum precipitation estimates presented in the report are the same values that were furnished periodically to Mr. Snyder as the studies progressed. The various factors conducive to extreme rainfall conditions in the area of the projects is not predicated on the results of one and only one precise combination of meteorological events. However, the primary precipitation threat, for the relative small watersheds above the projects would consist of a close spacing of intense thunderstorms and showers under conditions of extreme development of the intertropical convergence zone weather. The analyses and estimates of the probable maximum precipitation values (in millimeters) for the designated areas of the three Guavio projects are presented in enclosed report titled "Probable Maximum Precipitation Estimates for Guavio Projects," May 1972. A copy of this report has been furnished Mr. Franklin F. Snyder.

Sincerely yours,

DWIGHT E. NUNN
Meteorologist

Enclusure: (4 cys.)
PMP Report dtd. May 1972

cc w/enclosure:
Mr. Franklin F. Snyder

PROBABLE MAXIMUM PRECIPITATION ESTIMATES

FOR GUAVIO PROJECTS

COLOMBIA, SOUTH AMERICA

REPORT TO
INGETEC
BOGOTA, COLOMBIA

Prepared by
DWIGHT E. NUNN
Hydrometeorologist

May 1972

PROBABLE MAXIMUM PRECIPITATION ESTIMATES

FOR GUAVIO PROJECTS

COLOMBIA, SOUTH AMERICA

Dwight E. Nunn
Hydrometeorologist

INTRODUCTION

The probable maximum precipitation estimates presented herein are for three dam sites located in the headwaters of Rio Guavio east-northeast of Bogotá, Colombia, South America. The probable maximum precipitation estimates are derived for the Ingetec and for use by Franklin F. Snyder in determining the spillway design flood estimates for the Guavio project dam sites. The hypothetical optimum upper limit conditions producing extreme rainfall representing the probable maximum precipitation are estimated for the designated watersheds of the (1) Río Gacheta, above the Guavio damsite, drainage basin of 700 square kilometers, (2) the Rio Gacheta above the El Aguila damsite, drainage basin of 1230 square kilometers, and (3) the Rio Negro above the El Tigre damsite of 81 square kilometers. The probable maximum precipitation estimates determined herein are considered to be the theoretically greatest depths of precipitation for a given duration that are reasonably possible over the designated area. These hypothetical rainfall estimates for the above designated watersheds, of the upper tributaries of the Rio Guavio, are based on optimum parameters of severe weather phenomena that are representative and consistent with the South American region north of the equator. A comparison of climatological characteristics and variations associated with rainfall and storm mechanisms, supplemented by meteorological investigations, serves as a basis for establishing the upper limit of the most severe conditions representative of the rainfall situation considered "reasonably characteristic" of the upper Rio Guavio region. The optimum meteorologic conditions are based primarily on analyses of air mass properties (convergence, effective precipitable water, temperature dewpoints, winds), synoptic and upper air situation prevailind during storm situations in northern South America, topographical features (elevation, ground slopes), season of occurrence and geographical location of the basin.

ACKNOWLEDGEMENT

The hydrometeorological data and the geographical features of the study area furnished by Ingetec and through Mr. Franklin F. Snyder by Ingetec is gratefully acknowledged. Appreciation is also expressed to Mr. Snyder for his consultation throughout the study of the projects.

METEOROLOGICAL CHARACTERISTICS

Several meteorological factors enter into the climatic control and precipitation variations of northern South America. The most important are (1) the relative position of the sub-tropical centers of high pressure, (2) the equatorial zone of convergence, (3) the intertropical front, (4) the prevailing winds, and (5) the effects of local topography, each of which is discussed as follows:

(1) The sub-tropical high pressure belt in the northern hemisphere reaches its most southern position in the period of December through February. In and along the border of this high pressure area the air is subsiding, which is not conducive to heavy precipitation, particularly in eastern Colombia. This is indicated by the monthly precipitation records where the rainy season usually lasts from May through October with a decisive peak about the middle of the period.

(2) The equatorial zone of convergence is conducive to intense precipitation as contrasted to the sub-tropical belts of high pressure where the air is descending and divergent. The true equatorial air is ascending and convergent. The ascending currents reach a maximum in the afternoon, or are delayed in the more mountainous areas until evening when convection due to heating is greatest. This results in heavy thunderstorm rains throughout the entire belt of convergence and shows its effects in eastern Colombia during the rainy season from May through October. Humidity is usually high throughout the equatorial zone of convergence. Temperatures are high, but the range is not great and the extremes which are recorded in the sub-tropical belts of high pressure do not occur. The supply of moisture (maritime-tropical air origin) results from the easterly trade winds crossing over large water areas of the Atlantic Ocean and over low interior lands with numerous rivers and swamps. In general, the lofty barriers of the Andes cuts off the moist surface air circulation of the Pacific from the interior of the continent and eastern Colombia.

(3) The intertropical front or intertropical convergence zone is the region or zone of rain shower activity, cumulonimbus, well developed clouds, and high humidity between the trade winds of

the northern and southern hemispheres. The rain squalls and severe local thunderstorms occur frequently late in the evening or night, but may develop at any hour of the day or night. Over mountainous land areas the intertropical front is not as readily defined as in the low coastal areas of South America since its origin is to be found in the moisture differences between the two air masses. This lack of major colliding air streams is made up by land heating effects, which are highly conducive to convergence. The front and convergence zone moves northward in the spring and southward in the fall. This movement produces a double annual maximum in precipitation from the intertropical front passage in several locations in Colombia such as headwaters o Rio Guavio. The exceptions due to location are found in northern fringes of the intertropical convergence zone of the mountainous regions.

(4) The prevailing winds over the major portion of northern South America are from the easterly quadrant, between southeast and northeast. One of the exceptions is the west slopes of the Pacific coast of Colombia. Here the prevailing winds are from the western quadrant, although, at the higher levels, the predominant flow is also from the easterly quadrant. Wherever the higher elevations interrupt the prevailing air stream, local climatic variations are prevalent particularly in the amount of precipitation. The Choco province of Colombia has the heaviest rainfall in South America with the maximum occurring in September-October when the diverted southeasterlies and southwesterlies attain their maximum strength and persistency. However, the predominantly easterly winds over the eastern slopes of Colombia are moisture laden and are considered the most favorable for precipitation over the areas of eastern Colombia.

(5) Orographic lifting by windward mountain slopes is extremely conducive to precipitation in the tropical region. The Eastern Cordillera range is very effective in that regard and the prevailing easterlies, which cross the valleys and streams of northeastern South America, results in heavy rains along the eastern slopes. Similarly, the Western Cordillera in Colombia interrupts the westerlies of that region, causing heavy rains along the western slopes. Whenever higher elevations interrupt the prevailing air streams, local variations are to be expected in the amount of precipitation received. The mountain barriers may reduce or cut off the supply of moisture laden air to the leeward mountain

slopes as in the intermountain valleys of Colombia. However, the windward slopes on interior basins, as in the case of the upper Rio Guavio or similar areas, the mountain slopes may supply the slight rise which is necessary to trigger convection in unstable air. The orographic effects are influential on the mean annual precipitation and play some role in the probable maximum storm precipitation. The heat source over the ground varies more with the greater differences in elevation than over large masses of lowlands. Thus, under favorable conditions higher elevations stimulate convection that occurs earlier in the day, and is more intense, indicating the importance of local winds and topography in releasing precipitation. During the midle and latter part of the day local winds tend to ascend the heated mountains and set up thunderstorms over the lower elevations, as is evident in the storms of record.

The precipitation shows maxima at the time of the fall and spring equinoxes, although the precipitation in local showers may occur in any month of the year. The rainy season corresponds with the advance of the equatorial zone of convergence northward while the dry season corresponds to a southward advance of the North Atlantic belt of high pressure. Humidity is high during the wet season and moderate to low during the dry season. There is very little variation in the mean monthly temperature throughout the year, although there is a greater temperature range during the dry season than the rainy season. The flow and temperature distribution in the atmosphere at high levels, from about 6 to 12 km, in the tropics is complicated although it influences the weather at the surface. Air at this level that is cooler than normal tends to make the air column unstable and to set up regions of convection and widespread precipitation. Coldness aloft is characteristic of easterly waves but is not restricted to these disturbances. These cold upper lows frequently have an association with a polar trough.

METEOROLOGY OF SIGNIFICANT PRECIPITATION CONDITIONS

The probable maximum precipitation in any area is limited, among other things, to the available water vapor content of the air supplied to the region.The thunderstorm activity may be active on any one day within any one or all of the three areas of the Guavio project, feeding primarily on moisture within the basin and producing local small area rainfall. This situation is not necessarily conducive to producing depths of rainfall of probable maximum storm (Probable Maximum Flood) magnitude when the continuing supply of moisture is limited, or, considering a hypothetical assumption, that the inflow is physically cut off, the shower activity within a basin would only prevail until all the instability of the atmosphere has been discharged. To support the probable maximum precipitation estimates, an inflow of moisture accompanying the easterly winds into the basins during the storm is essential. The essential controls assumed for the probable maximum storm situation over the individual project basin (Gacheta, El Aguila and El Tigre) are: (1) the rate at which moisture may be carried into the basins, and (2) the efficiency with which this moisture may be processed into precipitation. For purposes of estimating extreme precipitation for the basins it is necessary to know the role played by thunderstorms. Not that the thunderstorms per se are all important, but the occurrence and severity of thunderstorms during most large rainstorms are indicative of characteristics of stability conditions and degree of vertical development. Therefore, the occurrence of severe thunderstorm (convergence) indicates the degree of similarity of Colombia extreme rainfall phenomena to that of other parts of the world.

Generally, the most rain that might be expected as a result of the undisturbed easterly trade winds is moderate shower activity. However, persistent winds from a southeasterly direction are especially conducive to high rainfall values if the trade winds inversion is displaced with vertical motion coupled with unstable moisture laden air. Potential for pricipitation is enhanced by varying degrees depending upon the nature and source of the disruption of moisture laden winds, whether it be a shearline, intertropical convergence, surface trough, trough aloft, easterly wave or significant circulation disturbances. Severe storm precipitation potential exists along the eastern slopes of Colombia when increased convergence, acting on sufficiently unstable moist air, results in vertical development of clouds. The overall meteorological situations pertinent to the upper Rio Guavio region and conducive to formulating precipitation conditions were discussed in the preceding topic of meteorological characteristics.

The rainfall in the vicinity of Guavio project varies significantly from one location to another, depending on geographical locations, in relation to moisture inflow, elevations, and ground slopes. The mean annual precipitation estimates for the basins under study are:

Stream	Drainage Area km^2	Average Annual Rainfall, mm
Rio Guavio above Gacheta dam site	700	1880
Rio Guavio above El Aguila dam site	1230	1980
Rio Negro above El Tigre dam site	81	3260

GENERAL PARAMETERS CONSIDERED IN DEVELOPING PROBABLE MAXIMUM PRECIPITATION

The primary rain threat to the region is a close spacing of intense thunderstorms and showers under conditions that favor extreme development of intertropical convergence zone weather. The general meteorological conditions assumed to exist which could produce the level of precipitation estimated as probable maximum are:

(1) A strong intertropical convergence zone (ITC) centered over or near the basin.

(2) Strong (above average for region) persisting winds from the southeast at low level and extending upward several kilometers, in association with the ITC.

(3) High moisture content from the Atlantic Ocean.

(4) Cooler than normal air, above 7 km. to tropopause.

(5) The characteristic instability of inflow air intensified by (3) and (4).

The meteorological processes most likely to be prevalent during a probable maximum rainstorm are explained in the following paragraphs:

<u>Orographic control, moisture and wind</u>. Wind data from stations east of the Cordillera Oriental of the Andes indicate that precipitation storms winds may vary from northeast (45 degrees) clockwise through southeast (155 degrees). In the present study, it was assumed that easterly (ENE to SE) winds prevailed. However, the surface winds in the lower elevations of the basin would funnel up the valleys. The slopes facing these wind directions are potentially full windward slopes in the intense rain area. Slope correlations with slope rainfall were derived on the assumption that vertical motion resulting from effective ground slope intensifies precipitation where sufficient moisture is available. It was also assumed that the moisture supply is drastically reduced by intervening mountains or up-wind barriers which reduce the opportunity for heavy rain. The generalized effective up-wind slopes

together with elevations contours were utilized in the evaluations of effective precipitation. Total precipitation was broken down into convergent rainfall and rainfall due to effective ground up-slope (orographic precipitation). The following storms in the vicinity of the eastern slopes of Colombia were correlated: 5 August 1961, 18 and 29 June 1961, 22 July 1961, 3 June 1962 and 19 July 1962, with particular emphasis on the 19 July 1962 storm. These meteorological parameter evaluations are also appropriate for the Guavio projects.

Convergence. The portion of storm rainfall resulting directly from convergence for storm and point rainfall in Colombia was determined by use of the limited available vapor pressure data (converted to dewpoint) and dewpoint records. The estimated dewpoints were considered to be moist adibatic (converted to precipitable water) which correspond to the various relatively high intense rainfall periods. It was also assumed, for these small areas, that most of the effective precipitation would occur in 24 hours or less. The maximum point rainfall in each storm was expressed as a C-Factor. The C-Factor at a given station is defined as the ratio of the rainfall (R) to the available precipitable water (Wp) in the atmosphere (C=R/Wp). Precipitable water (Wp) for a storm is derived from the 1000 mb sea level 12 hour dewpoint values, assuming saturated pseudoadiabic atmosphere under critical storm conditions, adjusted for inflow barrier and station elevation. The convergent maximum C-Factor values for the larger rainfall amounts in the mountains' eastern slopes ranged from very low values up to 15, adjusted for 24-hour duration, with the variation depending upon the maximum point storm rainfall estimate and the estimated representative dewpoints. An estimated C-Factor of 13.5 was selected which corresponds to the C-Factor selected for the eastern slopes of Colombia in previous studies. The adopted C-Factor of 13.5 is compatible with the 13.5 used for Hawaii, the Panama Canal, for small areas within the Rio Atrato, Colombia, and for western U.S. areas.

Orographic Precipitation. The rainfall adjusted for orographic precipitation requires consideration of two factors: (1) depletion of moisture with ground elevation and (2) intensification of up-wind slopes assuming the same representative dewpoint. The potential for short-duration intense local rains is not restricted by topographic effects as much as long-duration general rains. In the more intense storms, such as the probable maximum storm, persisting winds can govern the location of rain centers often in the foothills or valleys where little additional orographic lift acts as a trigger in the rain formation. For the relative short-duration intense precipitation such lifting can be helpful and increase the rain on steep windward slopes, but the atmospheric processes are more important. The orographic effects are very influential on the mean annual precipitation and would play some role in the probable maximum precipitation. The storms mentioned in the preceding discussion were analysed by use of their respective, effective up-wind slopes (determined in the direction of the effective wind from the topographic contours). The assumed available water and the estimated C-Factor were used in determining convergence rainfall for various locations. The difference between the computed convergence rainfall and the actual rainfall was assumed to be caused by the ground slope. These orographic values were plotted on arithmetical graph paper with the effective ground slope as the ordinate and the orographic rainfall in percent of the convergent rainfall as the abscissa. A mean curve was constructed through the points. The graphical resulting analysis corresponded closely to the Hawaii effective slope curve and other studies, and was considered to be representative of the Colombia area slopes under severe storm conditions. The storms of Colombia mentioned in the preceding discussion, and references 2 and 3, indicate that the effective up-wind slopes would be essentially the same as the estimated ground slope effects on rainfall as presented in references 8, 9 and 10. For convenience the adopted orographic rainfall depth resulting from up-slope effects were determined and converted to percent of the convergent rainfall. The results and the adopted values are presented in Table 1.

TABLE 1

OROGRAPHIC INTENSIFICATION DUE TO EFFECTIVE UP-SLOPE

Effective Ground (Up-wind) Slope	Orographic Rainfall in Percent of Convergence Precipitation
0.04	2
0.05	5
0.06	7
0.07	10
0.08	14
0.10	20
0.12	26
0.14	29
0.16	31
0.18	32
0.20	33
0.22	34
0.24	35

Maximum dewpoint. For the representative Rio Guavio project basins, maximum dewpoint reduced to sea level was estimated from vapor pressure records and was correlated for geographical consistency by use of the extreme sea level dewpoints for periods of record at 14 locations in South America north of the equator. Many of these recorded dewpoints were located a considerable distance from the area. In order to check the reasonableness of the representative maximum 12 hour 1000 mb dewpoint estimate for the area, contour lines of these values were drawn over the area of northern South America, giving due consideration to an easterly wind flow crossing the large land mass to the east of the basin. The 12 hour 1000 mb maximum dewpoint (Dp) of 24.5 degree centigrade was selected as representative of the conditions assumed to prevail during the probable maximum precipitation storm over the river basins of the project areas. The adopted 24.5 degree centigrade sea level dewpoint represents a moist column of precipitable water of 77.6 millimeters for a saturated pseudo-adiabatic atmosphere. Precipitable water (Wp) values vary inversely with ground elevations or inflow barrier elevations; that is, as ground elevations increase the precipitable water decreases. The precipitable water for the selected sea level dewpoint temperature available at various elevations is presented in Table 2.

TABLE 2

PRECIPITABLE WATER ABOVE DESIGNATED ELEVATIONS

Ground Barrier Elevation in Meters	Precipitable Water (Wp) in Millimeters
Sea Level	77.6
500	67.1
1000	57.7
1500	49.1
2000	41.7
2500	35.3
3000	29.5
3500	24.4
4000	20.1
4500	16.4
5000	13.3

The use of the above optimum Wp values in developing the Probable Maximum Precipitation estimates for the Rio Guavio project basins are explained in subsequent discussion.

Depth-Area. The depth-area relations and typical isohyetal patterns for areas other than point rainfall were derived by use of rainfall data from storms of record in Colombia with particular emphasis on the 19 July 1962, 17 April 1965, and relationships determined in previous studies (Ref. 2 and 3). The depth-area-duration relationships shown on Plate 1 are designated in percentage of storm center rainfall amounts considered to represent 1 square kilometer (assumed as point rainfall). The curves on Plate 1 represent the maximum average depth of rainfall over the indicated area as determined by planimetering the isohyetal pattern. The adopted 24-hour depth-area curve quite closely reproduces the actual areal distribution of the 1962 and 1965 storms, as well as several storms in Hawaii and Panama. In order to develop a time distribution breakdown of the probable maximum 24-hour index point rainfall, mass curves of observed storms (same as Refs. 2 and 3) were converted into percent of the total rainfall and the values plotted against time. The time distribution curves for point rainfall were derived by analyses of the 24-hour storm data with the depth-area relations obtained in Colombia, Panama, Hawaii and other storm studies. This was required to obtain increments of rainfall for periods less than 24 hours. The adopted probable maximum depth-area-duration curves expressed in percent of the 24-hour index rainfall are shown on Plate 1.

Hypothetical Storm Isohyetal Pattern. The elliptical isohyetal pattern shown by dashed lines on Plates 2 through 4 resulted from the studies made of isohyetal patterns of observed storms occurring in Colombia and in other locations. The hypothetical pattern converted to percent of point-index rainfall closely approximates the Colombia April 1960 and July 1962 storms adjusted to a common elevation and inflow barrier. Storms with high rainfall values are generally elliptical in shape (unless topographical influences are pronounced), however a wide variety of patterns might be selected with equal validity. It was determined from the study of the few available storms of record in northern South America adjusted to a common inflow and elevation that the ratio of the major to minor axis was about 2.0. This is essentially the same as storm patterns used with critical storms on the U.S. mainland and Hawaii. The hypothetical elliptical storm pattern in percent of point rainfall superimposed by the dashed line shown on Plates 2, 3 and 4, was constructed to correspond to depth-area relationships represented by the 24 hour curve of Plate 1. Use of the elliptical pattern is explained in subsequent discussions and illustrated in Plates 2, 3 and 4.

SUMMARY OF PROCEDURES FOR DETERMINING PROBABLE MAXIMUM PRECIPITATION

Procedure I. Preparation of the Probable Maximum Precipitation (PMP) estimates for the Guavio projects, utilizing the parameters discussed in the preceding paragraphs involves the following steps:

(1) The 24-hour point PMP index rainfall (R) is computed by use of the formula, R=CWpS; where C is assumed to equal 13.5; Wp (Precipitable Water) is obtained from Table 2, corresponding to a location elevation shown on contour map, ref. 1; and S (effective ground slope) is obtained from Table 1, corresponding to effective up-slope for a particular location determined from contour map, Ref. 1. The results of the index point rainfall determined for the various square kilometer locations for the watersheds above the Gacheta, El Aguila and El Tigre damsites are shown as the top figure in the grids on Plates 2, 3 and 4, respectively.

(2) An overlay of the hypothetical elliptical pattern shown on Plates 2, 3 and 4 has been placed critically (max. depth area) over the respective project basins on which the 24-hour index point rainfall values for each grid have been plotted as determined in step (1). The 24-hour PMP values are the product of the corresponding percentage values of the elliptical pattern and the index point values shown as the lower figure in the respective grids on the plates. The product represents grid point 24-hour PMP values associated with the selected storm center and are used in construction of the PMP isohyetal pattern.

(3) Construct an isohyetal pattern using the plotted computed grid PMP values obtained in step (2) above as a guide. The isohyetal pattern represents the areal distribution of the PMP. The isohyetal patterns thus developed for the designated tributary areas for the designated projects are shown on Plates 2, 3 and 4.

(4) The weighted basin average 24-hour probable maximum rainfall amount over each of the basins, or any subarea of a basin, is obtained by planimetering the isohyets. Time distribution of 24-hour PMP values into 2-hour time units is obtained from the percent-

age curves shown on Plate 1, read at the point corresponding to the area of the project basin. PMP values are obtained by multiplying the percentage amounts indicated by the dashed lines on Plate 1 for the various time periods by the average 24-hour PMP value. Incremental amounts are determined by subtraction.

(5) Arrange the incremental rainfall values determined in step (4) in a sequence favorable to production of critical runoff.

Procedure II

(1) The alternative Procedure II, same as Procedure I step (1) above.

(2) Determine the weighted basin average index 24-hour rainfall by use of the grid point index rainfall (shown in this study as the top figure in the grid on Plates 2, 3 and 4), using either a weighted grid system or constructing lines of equal grid index rainfall. From the determined weighted basin average index rainfall enter Plate 1 corresponding to the area of the project basin, multiply the percentages obtained for the various time periods shown by solid lines by the basin average index 24-hour rainfall to obtain average depth and time distribution of the PMP within the basin.

(3) Arrange the incremental PMP rainfall values determined in step (2) in a sequence favorable to production of critical runoff. The arrangement for the 24-hour PMP may be accomplished by dividing storm rainfall into 2-hour or any required increments.

PROBABLE MAXIMUM PRECIPITATION ESTIMATES FOR DESIGNATED GUAVIO PROJECTS

The probable maximum precipitation (PMP) estimates for the watershed above the Gacheta, El Aguila and El Tigre damsites are based on the analyses indicated in the report. The rainfall values were determined by Procedures I and II, explained in the preceding paragraphs and resulted in essentially the same probable maximum precipitation estimates.

The average 24-hour PMP values, Procedure I, obtained by planimetering the associated isohyets within the basins above the reservoir sites resulted in an insignificant reduction (less than 10 percent) of the values determined by Procedure II. The weighted basin average index rainfall values, Procedure II, resulted in an approximate value of index rainfall for: (1) Rio Guavio above Gacheta damsite drainage basin 700 km^2, with the basin index rainfall of 550 mm; (2) Rio Guavio above El Aguila damsite drainage basin 1230 km^2, with the basin index rainfall of 575 mm; (3) Rio Negro above El Tigre damsite drainage basin 81 km^2, with an index rainfall of 614 mm.

The associated PMP pertinent data and isohyets for each of the designated basins are shown on Plates 2, 3 and 4. The PMP estimates are tabulated in Tables 3, 4 and 5 for the basin indicated in the Table heading. For this study the 24-hour PMP estimates were divided into appropriate time increments as shown in Tables 3, 4 and 5 for use in developing the probable maximum flood for each project damsite. The incremental rainfall values shown are to be arranged in sequence most favorable to produce critical runoff.

TABLE 3

PROBABLE MAXIMUM PRECIPITATION ESTIMATE
STORM CENTERED CRITICAL OVER THE RIO GUAVIO ABOVE
THE GACHETA DAMSITE

(Drainage Area 700 km^2)

PROBABLE MAXIMUM PRECIPITATION IN MILLIMETERS

Time in Hours	Rio Guavio above Gacheta Damsite (700 km^2)	
	Σ	Δ
0		
1	65	65
2	107	42
3	137	30
4	161	24
5	184	23
6	204	20
7	220	16
8	236	16
9	259	13
10	261	12
11	273	12
12	284	11
13	295	11
14	305	10
15	315	10
16	324	9
17	333	9
18	341	8
19	349	8
20	357	8
21	364	7
22	371	7
23	378	7
24	384	6
		384

Reference Plate 2 for Isohyetal Pattern.

TABLE 4

PROBABLE MAXIMUM PRECIPITATION ESTIMATE
STORM CENTERED CRITICAL OVER THE TOTAL
AREA OF THE RIO GUAVIO
ABOVE EL AGUILA DAMSITE

(Drainage Area 1230 km^2)

PROBABLE MAXIMUM PRECIPITATION IN MILLIMETERS

Time in Hours	Rio Guavio above El Aguila damsite (1230 km^2)	
0	Σ	Δ
1	60	60
2	100	40
3	124	24
4	147	23
5	169	22
6	185	16
7	199	14
8	213	14
9	226	13
10	238	12
11	250	12
12	262	12
13	272	10
14	282	10
15	291	9
16	299	8
17	307	8
18	315	8
19	323	8
20	331	8
21	339	8
22	346	7
23	353	7
24	360	7
		360

Reference Plate 3 for Isohyetal Pattern.

TABLE 5

PROBABLE MAXIMUM PRECIPITATION ESTIMATE
STORM CENTERED CRITICAL OVER THE RIO NEGRO
ABOVE THE EL TIGRE DAMSITE

(Drainage Area 81 km^2)

PROBABLE MAXIMUM PRECIPITATION IN MILLIMETERS

Time in Hours	Rio Negro above El Tigre Damsite (81 km^2)	
0	Σ	Δ
1	110	110
2	166	56
3	210	44
4	249	39
5	284	35
6	314	30
7	334	20
8	352	18
9	370	18
10	387	17
11	403	16
12	419	16
13	433	14
14	447	14
15	460	13
16	472	12
17	484	12
18	494	10
19	504	10
20	513	9
21	522	9
22	530	8
23	538	8
24	545	7
		545

Reference Plate 4 for Isohyetal Pattern.

REFERENCES

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9. Unpublished - "Standard Project Storm Determinations, Hawaiian Islands," 1962, Corps of Engineers.

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11. "Estimation of Maximum Floods"; Technical Note No. 98; WMO-No. 233. TP.126; World Meteorological Organization; 1969.

12. Various Hydrological and Meteorological data collected, processed and furnished by Ingetec.

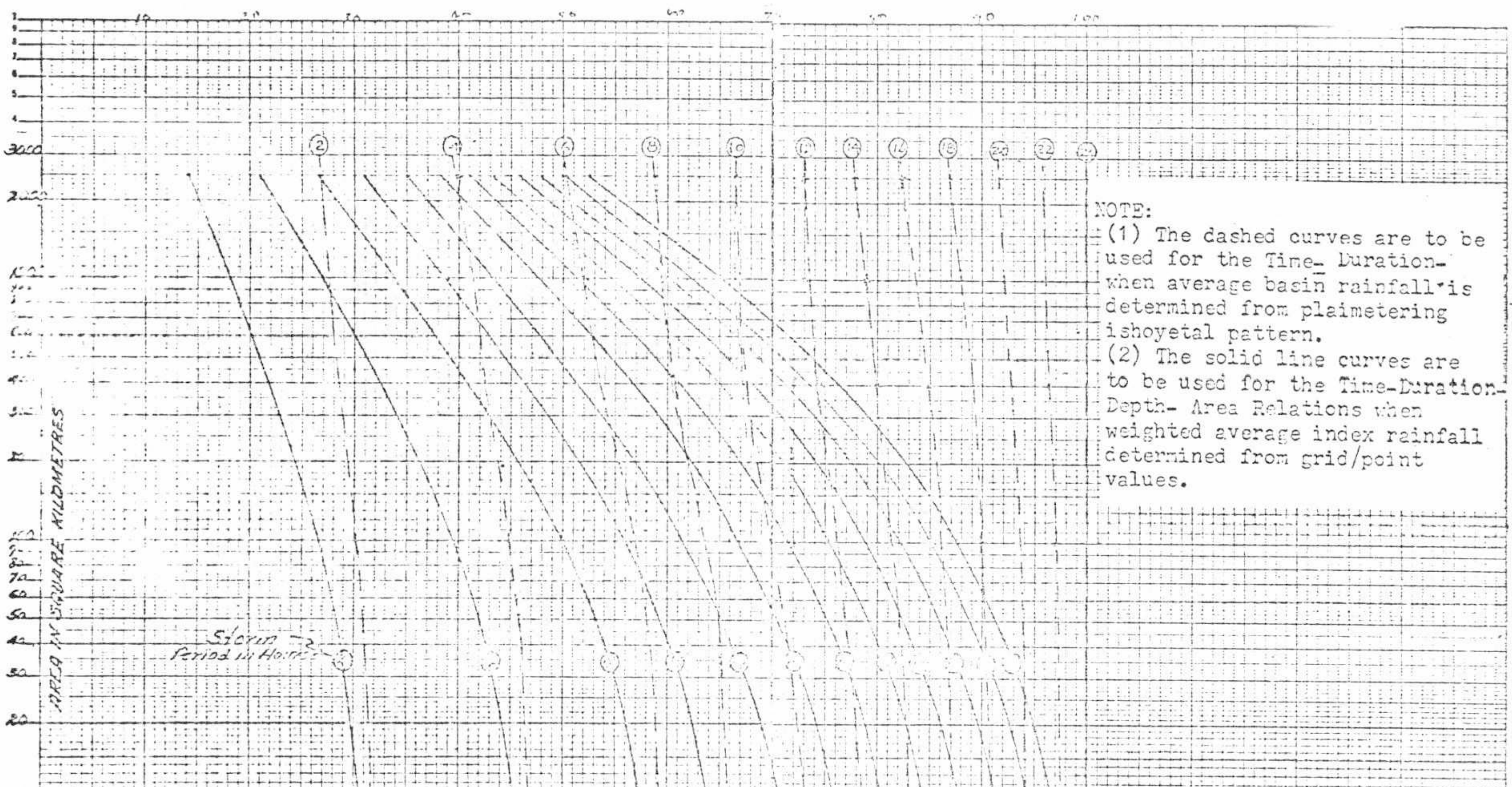
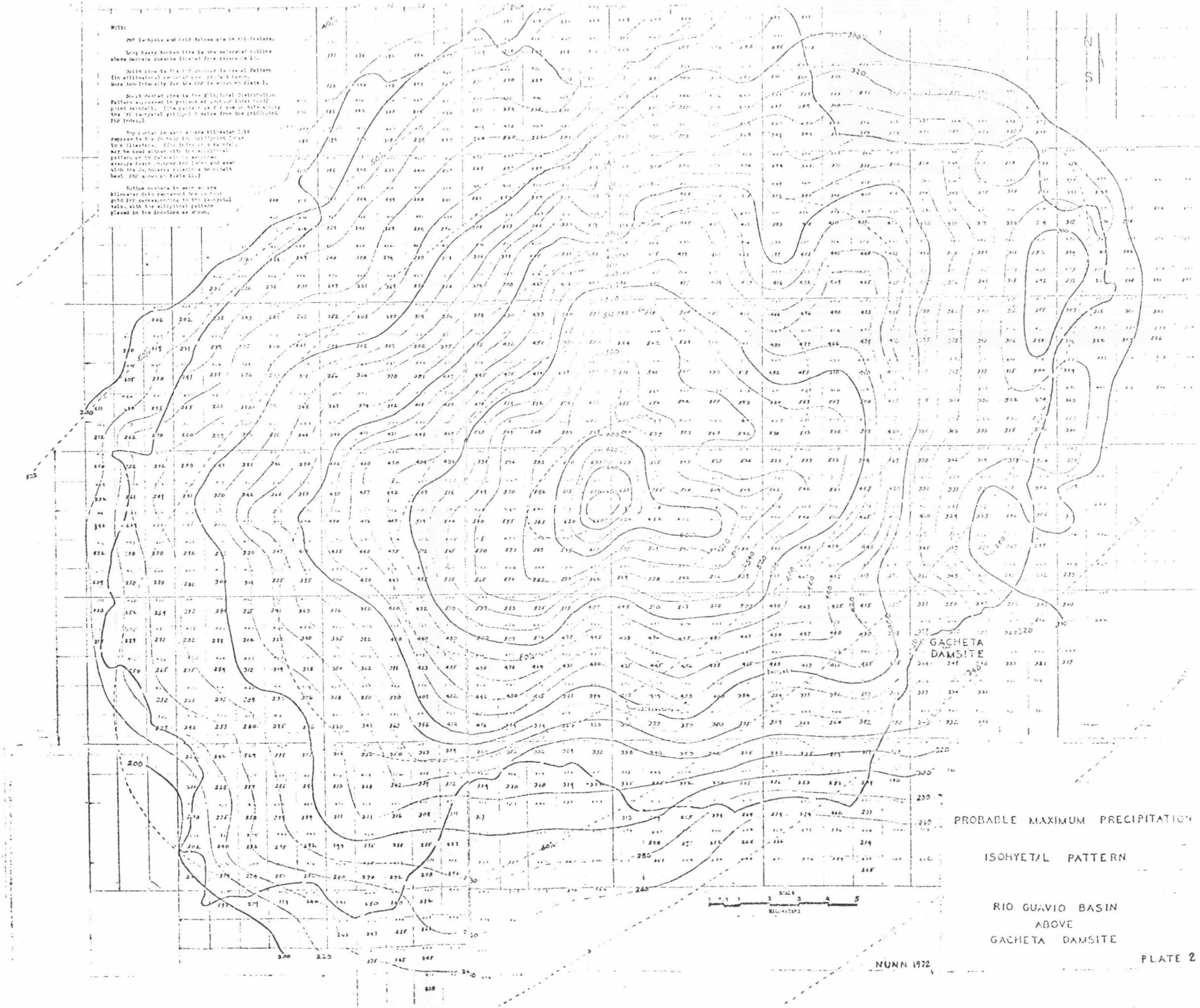

NOTE:
(1) The dashed curves are to be used for the Time- Duration- when average basin rainfall is determined from plaimetering ishoyetal pattern.
(2) The solid line curves are to be used for the Time-Duration-Depth- Area Relations when weighted average index rainfall determined from grid/point values.
PROBABLE MAXIMUM PRECIPITATION
DEPTH- AREA- DURATION
GUAVIO PROJECTS
COLOMBIA, SOUTH AMERICA
Nunn,1972
AREA IN SQUARE KILOMETRES
Storm Period in Hours
PERCENT OF 24-HOUR PRECIPITATION
0
10
20
30
40
50
60
70
80
90
100

PLATE 1


GACHETA DAMSITE
NUNN 1972
PROBABLE MAXIMUM PRECIPITATION
ISOHYETAL PATTERN
RIO GUAVIO BASIN
ABOVE
GACHETA DAMSITE
PLATE 2

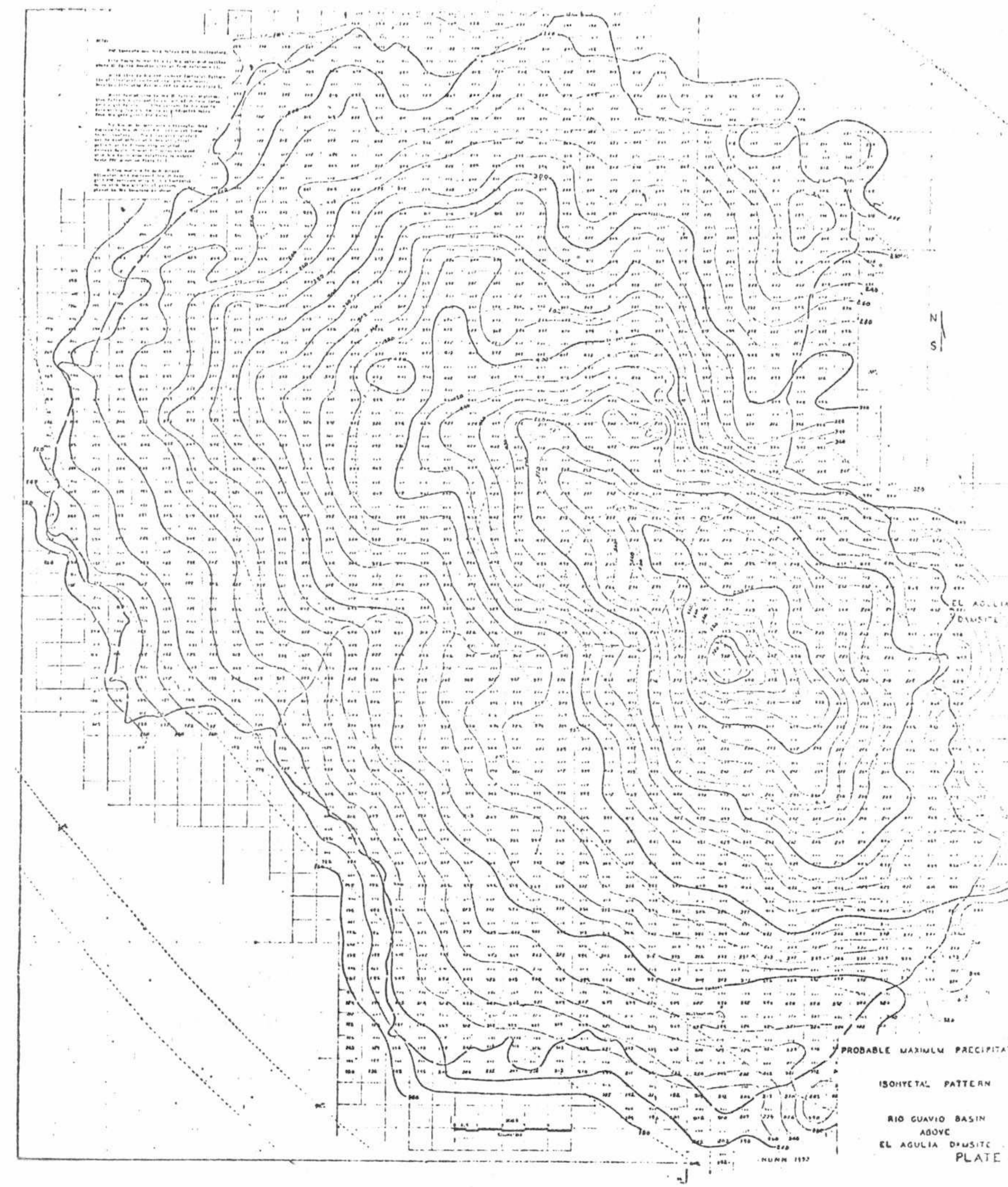
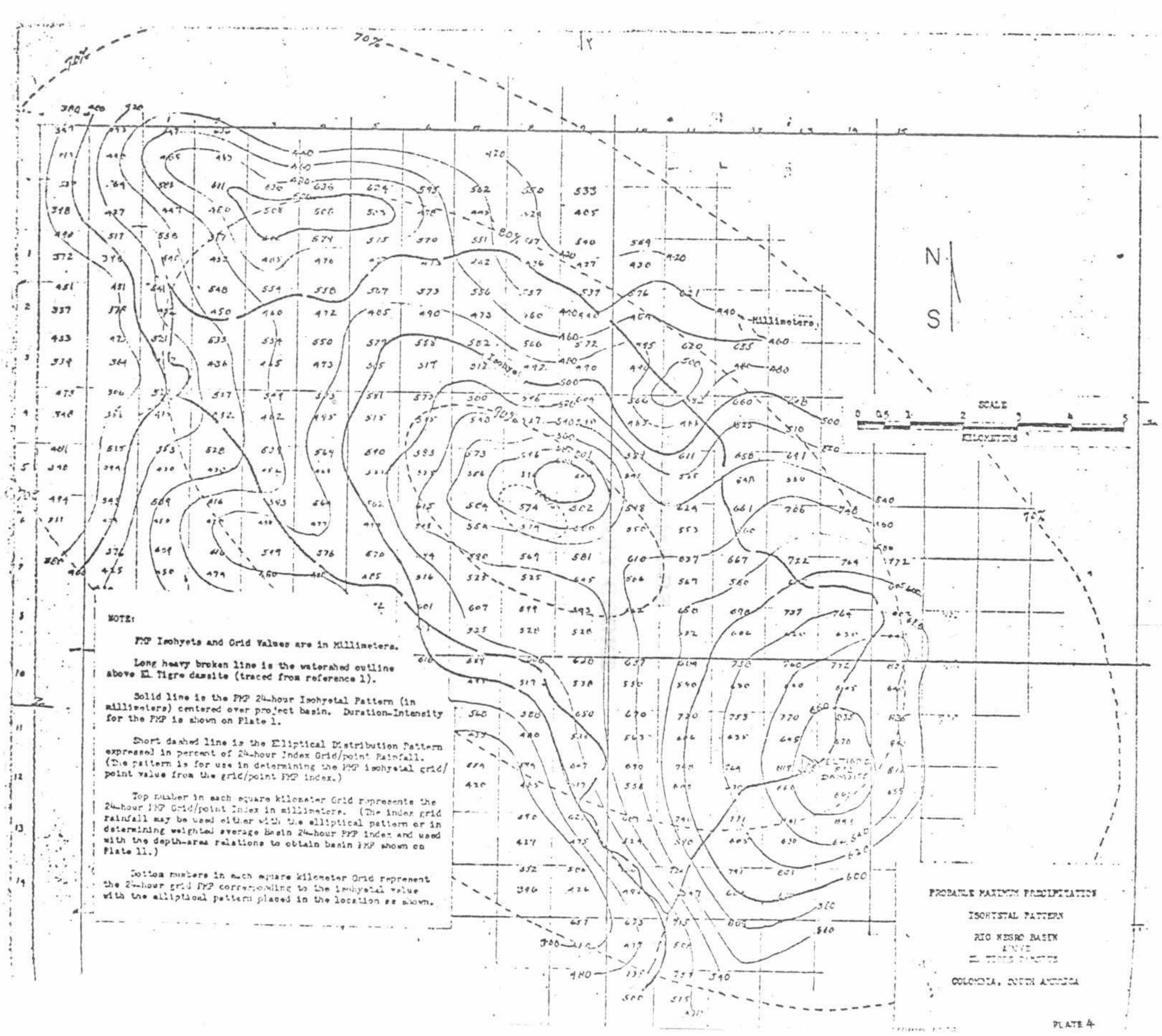

N
S
EL AGUILA
DAMSITE
PROBABLE MAXIMUM PRECIPITATION
ISOHYETAL PATTERN
RIO GUAVIO BASIN
ABOVE
EL AGUILA DAMSITE

PLATE 3


70%
80%
90%
N
S
Millimeters
SCALE
KILOMETERS
NOTE:
PMP Isohyets and Grid Values are in Millimeters.
Long heavy broken line is the watershed outline above El Tigre damsite (traced from reference 1).
Solid line is the PMP 24-hour Isohyetal Pattern (in millimeters) centered over project basin. Duration-Intensity for the PMP is shown on Plate 1.
Short dashed line is the Elliptical Distribution Pattern expressed in percent of 24-hour Index Grid/point Rainfall. (The pattern is for use in determining the PMP isohyetal grid/point value from the grid/point PMP index.)
Top number in each square kilometer Grid represents the 24-hour PMP Grid/point Index in millimeters. (The index grid rainfall may be used either with the elliptical pattern or in determining weighted average Basin 24-hour PMP index and used with the depth-area relations to obtain basin PMP shown on Plate 11.)
Bottom numbers in each square kilometer Grid represent the 24-hour grid PMP corresponding to the isohyetal value with the elliptical pattern placed in the location as shown.
PROBABLE MAXIMUM PRECIPITATION
ISOHYETAL PATTERN
RIO NEGRO BASIN
COLOMBIA, SOUTH AMERICA
PLATE 4

INFORME DE

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Bogotá, August 8, 1974

Dr. HERNANDO QUIJANO
Ingetec Ltda.
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Bogotá - Colombia

GUAVIO PROJECT
SPILLWAY
H. Quijano Letter of
14 June 74

Dear Hernando :

I have been acquainted with the proposal of tunnel spillways for Guavio. I have reviewed your letter and enclosed drawings of an open channel spillway.

You mention that the tunnel spillways have been criticized. I agree that open channel spillways are preferred. However, tunnel spillways are satisfactory and used where significantly lower in cost. The type of tunnel spillway proposed for Guavio does not have the elbow and high velocity problem such as for Infiernillo, Hungry Horse, Sultan and other tunnel spillways using diversion tunnels. The discharge from the proposed Guavio tunnel spillways is well clear of the dam.

On 3 August the spillway was reviewed by JBC, CSO, Gonzalo Duque, Jesús Sierra, Alberto Marulanda and yourself. A memo by Jorge Acosta suggested a study to determine if high surcharge might be appropriate, since the dam is in such a narrow canyon and the reservoir surface area is large. It was decided to study a 2 gate single tunnel with a high surcharge.

On 8 August a single tunnel scheme was presented in a meeting attended by JBC, CSO, Gonzalo Duque and yourself. The tunnel was 14 m ID circular concrete lined, on 5% grade. Velocity would be 30 m/second. Two 16 m wide by 14 m high gates were proposed.

Elevations were :

1640	Top of dam
2	Freeboard on PMF
1638	Max. ws for PMF
8	Surcharge for PMF
1630	Top of gates
14	Height of gates
1616	Gate sill

This arrangement would handle the PMF of 8000 m^3/sec peak and 348.000.000 m^3 volume by discharging a max. of 4250 m^3/sec and storing 90.000.000 m^3 in the 8 m surcharge zone. The tunnel would change from a flow to a pressure tunnel, a satisfactory condition common to diversion tunnels.

Gates. For the high surcharge and operation of tunnel for full flow, which would occur only for floods approaching the PMF, radial gates, in the open position, would be partially submerged. This could be considered acceptable. If it were decided not to accept such a possible operation, wheel gates could be used. Wheel gates of such size are common in Canadian practice; they are often counterweighted; and, they cost more than radial gates.

Gate operation. A two gate spillway requires careful operation procedures. A satisfactory operation would be to keep gates open when the reservoir level is below the gate sill, and lower them for filling reservoir near end of the wet season.

Sincerely yours,

Barry Cooke

JBC/eg
cc: Carlos Ospina