CHAPTER 5

GEOLOGY

FIGURES

CHAPTER 5 GEOLOGY

5.1 Topographical and Geological Outline of Project Area

5.1.1 General

The physical features of Peru are strongly influenced by the Andes Mountain Range which runs through widely at the middle part of the country in a northwest-southeast direction with more than 4,000 m in altitude. The Andes in Peru are composed of three mountain ranges called from the Pacific Ocean side toward the Atlantic Ocean side, the Cordillera Occidental, the Cordillera Oriental and the Sub-Andes Mountain Range. With the Andes as the skeleton, the topography in Peru may be classified into eight regions. Namely, from the Pacific Ocean side they are the Cordillera de la Costa, the Llanura y depresiones costaneras, the Cordillera Occidental, the Valles y regiónes interandinas, the Cuenca del Titicaca, the Cordillera Oriental, the Cordillera Subandina, and the Llanura Amazonica.

Due to the fact that the topography of Peru is governed by a tectonic mountain range (the Andes) with a predominant trend in the northwest-southeast direction, the rivers of the country may be divided into three characteristic groups. They are,

1. The group of rivers having their headwaters on the western slopes of the Cordillera de la Costa and flowing into the Pacific Ocean, which have short watercourse but steep river gradients.
2. The group of rivers eroding the slopes between the mountain ranges extending in the northwest-southeast direction to form deep, eroded canyons and flowing either to the north or to the south to flow into the Amazonas River.

3. The group of rivers of the Llanura Amazonica with their headwaters on the eastern slopes of the Sub-Andes Mountain Range which have vast drainage areas, gentle river gradients, and long watercourses.

The Yangas Project area is located approximately 600 km to the north of Lima, and belongs to the Cordillera Occidental Region of the previously mentioned topographical divisions. The Yangas River has its headwaters at an elevation of approximately 4,000 m in the Cordillera Occidental, and is one of the tributaries of the Marañon River which flowing to the north deeply eroding the valley between the Cordillera Occidental and the Cordillera Oriental, and which in its turn is a tributary of the Amazonas River. The latitude and longitude of the project area are roughly 6°50' south and 78°10' west, respectively.

5.1.2 Topography

The Yangas River with a length of approximately 50 km and a catchment area of 1,250 sq.km flows down the eastern slopes of the mountain range bending and turning in a generally north-northeasterly direction and flows into the Marañon River.

The Yangas River is one of the major tributaries of the Marañon River along with the Llaucano River and the Crisnejas River and is a fairly rapid stream during the rainy season. The topography in the vicinity of the riverhead of the Yangas River is of a peneplain of very gentle undulations. Numerous dolines which come from karstic limestone of diameters ranging between several tens of meters to several kilometers can be recognized in this hilly area. On a 1/100,000-scale topographic map, these sinkholes are seen to be distributed in the form of a number of belts, and the arrangement of these belts shows a trend parallel to the Andes, that is, in the northwest-southeast direction.

The valleys of the Yangas River and its tributaries are generally of V-shape with numerous places where the mountain slopes on both sides are steep, and the topography is in a stage of maturity in which erosive action is most severe. Several terraces are formed at the riverside and those of large scale have heights of 40 to 50 m. There are also numerous large scale of land slides at low parts of mountain slopes within the catchment area.

5.1.3 Geology

The geology of Peru is composed of sedimentary rocks, igneous rocks and metamorphic rocks belonging to the age from the Precambrian Period to the Tertiary Period and deposits of the Quaternary Period which cover these rocks.

They are grouped in the three divisions of the Coastal Belt Region, the Andes Geotectonic Region, and the Sub-Andes Region from the standpoint of geological structure. The Yangas Project area belongs to the Andes Geotectonic Region of the above, which is mainly composed of rocks of the Mesozoic Era. The rocks distributed in this area are mainly sedimentary rocks such as sandstone, slate and limestone, of which limestone has the widest distribution.

Sedimentary rocks distributed throughout the Andes area are in general extremely folded, added to which faults are prominently developed, and these geologic structures show a northwest-southeast trend. However, compared with the eastern part of the Andes, the geologic structure of the project area is comparatively monotonous.

In other words, it may be said, as for the geological features of the area, that the extremely thick sedimentary rocks (limestone and clastic rocks) are widely distributed while the distribution of igneous rocks is very limited, and that these sedimentary rocks are influenced by the northwest-southeast trend of the geological structure to be arranged in relatively orderly rows so that the geology governs the topography to a great extent.

The geologic formations which are distributed in the Yangas River basin belong to the Cretaceous Period of the Mesozoic Era and are mostly composed of limestone and sandy to clayey limestone with intercalations of coarse to fine-grained quartziferous sandstone and slate. Of these rocks, the limy rocks distributed over a wide area, except for the sandstone and slate which are found only in limited sites, are by their lithological characters susceptible to karstic erosion, and according to photogeological interpretation, numerous dolines have been formed at the top of mountains. Meanwhile, the solution caves of diameters ranging from several tens of centimeters to several meters are found at many places along the Yangas River during the reconnaissance and watersprings were noticed from several of these caves. As for the branch valleys which join the Yangas River carving the surrounding mountains, there was no water flowing in spite of the rainy season and, in general, the ground in the vicinity presents an extremely dry appearance.

5.2. Topography and Geology of Project Damsite

The damsite first proposed was the No. 3 site, which had been recommended by MOTLIMA S.A.. But since it was found as a result of reconnaissance by this Survey Mission that this is not necessarily a good damsite from the viewpoint of topography and geology, the Mission was forced to widen the scope of investigations to the No. 1 and No. 2 sites farther upstream. These sites are called the No. 1, No. 2 and No. 3 sites from the upstream side, and as a result of brief comparison studies of the reservoir efficiency (refer to 7.2) the No. 1 site was selected as the project damsite.

Regarding the topography and geology of the No. 2 and No. 3 sites, they are mentioned in the alternative sites in the next sub-section.

5.2.1 Topography

The Yangas River flows down in a nearly straight line around this site. The width of watercourse is approximately 40 m at the damsite with the mountains on both banks closing in on the site, but both upstream and downstream this point the valley width is spread with flood plains widely distributed on both banks. Unlike other places, the damsite has no terraces and the both banks of 100 m high present steep cliffs with slopes of 70° - 80° and the shape of valley is a symmetrical U-shape. There are few gullies in the mountain masses at the abutments of the dam with precipices continuing parallel to the river flow, but approximately 200 m downstream from the damsite there are large gullies joining the river at both banks.

5.2.2 Geology

(1) Surface Deposits

As stated in the section on topography, the mountains on both banks form steep cliffs with sound bedrock exposed widely and only very slight overburdens of soil are seen at the ridges. Regarding river deposit, due to the cliffs closing in on both banks, it is considered to be not so deep at the damsite.

(2) Foundation Bedrock

The rock distributed here is limestone. Bedding planes are developed in the bedrock at intervals of 1 to 2 m, the strike and dip of the strata are N30° W, 20 - 50°NE. Accordingly, the strike of the strata crosses with the river flow at an angle of 60° and the dip is toward the downstream side. The limestone presents brownish-gray to grayish-black colors and is a slightly impure limestone in terms of lithological character, while there are no intercalations of other rocks.

Although there are numerous cracks due to stratification and jointing, so far as seen at the damsite, the contact of the strata are tight and there is no water spring from the bedrock while the evidences that the rock has been subjected to karstic erosion such as solution cave cannot be seen. Furthermore, there are no especially remarkable faults or sheared zones to be recognized.

5.2.3 Geology and Watertightness of Reservoir Area

The rocks comprising the reservoir area are limestone, sandstone, slate and their alternations. Of these, limestone is distributed on the river side from the damsite to approximately 2 km upstream, while upstream from there, sandstone, slate and their alternations are distributed and the proportions of the limestone and the other clastic rocks covering the reservoir area are roughly equal. The strikes and dips of these rocks are not constant, but as a general trend they show N20° - 30°W and 50° - 60°NE. Accordingly, the strikes cross with the river flow at angles of 50° - 60° and dips toward the downstream direction.

The lithological character of the limestone, as described in 5.2.2, "Geology," is, in most cases, slightly argillaceous, and is a rock of uniform quality with little intercalation of other rocks. The sandstone presents white color, hard and dense, and is a quartziferous sandstone of medium grade. The slate presents gray to grayish black color, fine-grained and dense, and is a relatively hard rock in which development of bedding is seen.

With respect to the watertightness of the reservoir, as stated in 5.1.3, "Geology," limestone generally is liable to be subjected to karstic erosion, and in areas in which limestone is distributed, it is not rare for karstic topography featured by dolines, underground tunnels, etc. to be developed. The site in question is no exception, and according to photogeological interpretation, there are numerous dolines, although outside of the reservoir area, which have been formed in the rock of the mountain mass of the same horizon as the reservoir area, while also, there are numerous places which are thought to be points of water springing in the reservoir area. Based on the above, it can well be expected that there might be a considerable number of solution caves in the mountain masses of the reservoir area where limestone is distributed, and in regard to the watertightness of the reservoir, careful investigations and studies will hereafter be required. However, there is flowing water in the Yangas River even in dry season. This phenomenon indicates that at least at the riverbed bedrock level within the reservoir, there is underground water throughtout the year. Therefore, the point is, even though it may be assumed that all surface and underground water within the reservoir area is presently flowing into the Yangas River, when it happens there exist caves connecting to other river basin or to the downstream of the damsite between the elevation of the present ground water level and the high water level of the reservoir or when the materials presently filled these caves should be flushed away as water pressure increases with storage of water, there is a probability of forming new leakage paths and causes very serious problems. Investigations to solve these problems will require great expense, much labor and a long period of time, but this investigations is one of the most important problem for this Project.

5.2.4 Views from Engineering Geological Standpoints

The problem with this site lies in its geologic conditions rather than topographic conditions. Topographically, the river flows down in a straight line, the width is very narrow, while there are few gullies in the mountain masses at the dam abutments with steep cliffs continuing along the river channel, and the conditions are rather on the good side for construction of a dam.

Geologically, there are few surface deposits such as of talus and river gravel and there are no evidences to be seen of the limestone comprising the foundation bedrock having been strongly subjected to karstic erosion such as springing of water or formation of caves. However, as descirbed in 5.2.3, "Geology and Watertightness of Reservoir Area," the limestone which is distributed over a fairly wide area from the damsite to the reservoir area is accompanied by a very serious problem of securing watertightness. In fact, although there are many reservoirs which have been built in limestone areas in various parts of the world depending on lithological character, there are also many cases in which construction was abandoned because of watertightness of the dam foundation and reservoir area could not be guaranteed. There have also been cases in which leakages of large quantities of water occurred after completions of the dams to impair water storage capabilities so that the desired aims were not achieved or huge sums of expense were required for treatment. Considering from the geologic standpoint of this site, as there is some possibilities for leakage to occur from the dam foundations and reservoir area due to karstic action, thorough investigations and studies must be carried out in regard to watertightness.

5.2.5 Geology of Appurtenant Structures

Emphasis was placed on investigating the topographies and geologies of the damsite and the reservoir area in this field reconnaissance. Therefore, with respect to the geologies of the waterway and the powerhouse, outlines are given here mainly based on the results of photogeological interpretations.

The geology along the waterway route is composed of limestone and non-limy rocks (quartzite, sandstone, slate and their alternations) of the Mesozoic Era and gneiss of the Paleozoic Era, of which the greater part consists of limestone.

Regarding the limestone-distributed area, as described in the sections on the damsite and the reservoir area, it is a problem whether or not there exist caves. Therefore, thorough investigations will be necessary to be made in the stage of feasibility study on the caves, and water springs along the route of the waterway tunnel. It is understood from the geology observed during this reconnaissance that the strength, weathering and sheared zones in the limestone around the tunnel route are not so bad as to impair the project. Geological problems such as to be hindrances to tunnel excavation cannot be recognized in particular in the non-limestone area of bedrock also.

Four locations were compared and studied as powerhouse sites, and other than Site No. II, they are located in the non-limesonte bedrock areas, while regarding the type of powerhouse, those other than Site No. IV are planned to be surface types. In case of a surface-type powerhouse, the bedrock existing will not bring special difficult problem as the foundation of a powerhouse, but in case of an underground powerhouse, there will correspondingly be a necessity for thorough investigations to be made. Especially, in case the powerhouse is to be located in a limestone area, the existence of caves and water springs will be a great problem as with the damsite, and it is needless to say that thorough investigations are necessary for the powerhouse site also if the Project should be undertaken.

5.3 Alternative Damsites

5.3.1 Damsite No. 2

This site is located approximately 2.5 km downstream of the proposed damsite (Damsite No. 1). The Yangas River in this vicinity flows down in a roughly straight line. The width of the watercourse at the dam axis is approximately 50 m, there are terraces of average height of approximately 10 m at the river banks with the widths of the terraces being approximately 10 m on the left bank and approximately 50 m on the right bank. The slope of the left bank at the dam axis is approximately 35°, while at the right bank the slope is approximately 60° from the riverbed to ground height of approximately 50 m above which the slope is at an angle of approximately 35°. Therefore, the shape of the valley is a non-symmetrical V-shape.

The topography of the reservoir area is that of widening immediately upstream of the damsite, with inflows from large gullies from both banks, there are formed flood plains, terraces and alluvial fans. Also, there is a large-scale landslide topography seen at the left bank upstream of the damsite.

(1) Surface Deposits

The mountainsides at the damsite are wholly covered by exposed sound bedrock and there is almost no overburden. Regarding river deposits, since the riverbed is narrowed at the damsite, and it is a rapid portion of the river, the deposits will be comparatively small.

(2) Foundation Rock

The bedrock consists of limestone. The limestone at this site is of gray to grayish-black color with impure rock generally predominant and bedding and jointing cracks are developed. The strikes and dips of the strata are in the ranges of N20° - 30° W and 50° - 60° SW with the strikes crossing the river flow roughly perpendicularly and with the dips being toward the upstream side. The important geological phenomena at this site are that there is water springing from the bedrock at the dam axis and its neighborhood, and that solution can be recognized at bedding planes, joints and cracks developed in the bedrock. It can be assumed there are caves in deep parts of the mountain and a problem is posed with respect to watertightness of the foundation. Locations of prominent springing are seen at the bedrock of the lower part of the mountainside at the right bank of the dam axis (quantity of spring water, approximately 1 ton/min), the part of terrace at the right bank (quantity, approximately 500 ℓ/min) and halfway of the left side steep cliff approximately 200 m downstream of the dam axis (quantity, approximately 3 ton/min).

5.3.2 Damsite No. 3

This site is located approximately 5 km downstream from the proposed damsite. It is situated immediately upstream of the conjunction with the Yangas River and the Grande River at the right bank and with the Catalina canyon at the left bank.

The Yangas River which flows down in a roughly straight line from the No. 2 site makes a big bend immediately upstream of the No. 3 site and changes its course to an east-west direction. The width of the river is approximately 100 m at the dam axis and there are wide terraces of heights from 40 to 50 m at the left bank. The gradients of slope of the both banks are around approximately 60° with the form of the valley being a wide U-shape symmetrical. The width of the valley in the reservoir area is increased at the upstream of the conjunctions with said tributaries immediately above the damsite with wide terraces formed at the river banks. Along the said tributaries there are continuous distributions of wide landslides.

(1) Surface Deposits

Similarly to Damsite No. 2, bedrock is exposed over the entire mountain slopes on both banks and there is almost no overburden of soil. The thickness of river deposits in the river channel is assumed to be more than 10 m while the thicknesses of the terrace deposits widely distributed at both banks are around 40 m. Therefore, the thickness of the loose deposit layer at the dam site combining with the present river deposits and the terrace deposits will be more than 50 m at the highest point.

(2) Foundation Rock

The rock distributed here is limestone. The lithological character and the degree of development of cracks are not very much different from the limestone at Damsite No. 2. The strikes and dips of the strata are in the ranges of N20° - 30° W and 60° - 70° SW with the strikes crossing the river flow at angles of approximately 50° while the dips are toward the upstream side. Springing of water from the ground is seen at the riverbed of the dam axis and at the terrace approximately 250 m upstream from the dam axis (quantity of spring water for both, approximately 2 ton/min each), and there is the same problem as for the No. 2 site in regard to watertightness of the foundation. Another problem of the dam foundation at this site is the thin ridge of the mountain on the right bank, which is moreover steep, and it is assumed that loosening of the rock foundation has progressed to fairly deep parts.

5.3.3 Views of Alternative Damsites from Engineering Geological Standpoints

(1) Damsite No. 2

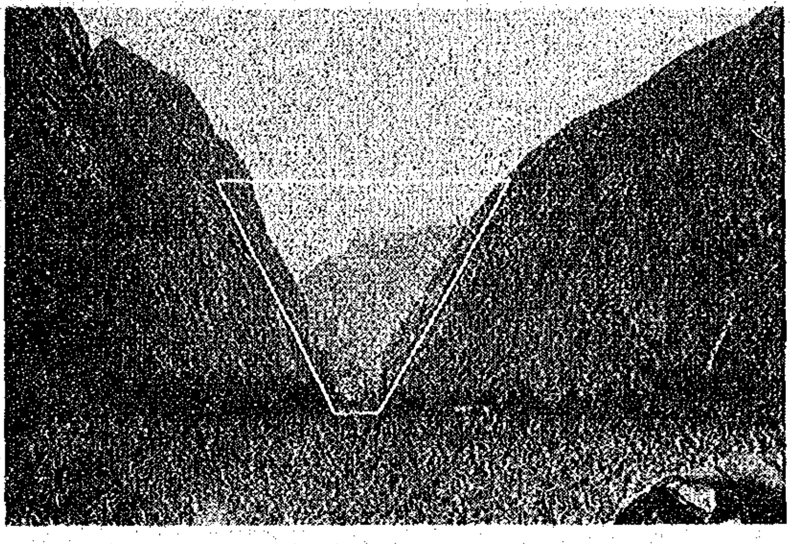
As a damsite, this location involves a problem related to geological conditions rather than topographical conditions. That is, there are large quantities of spring water from several places in the bedrock at this site, while there are many traces of solution having occurred in the cracks developed in the bedrock due to ground water and surface water, from which it is fully assumable that caves exist in the foundation, and there is a big problem of the watertightness of the dam foundation and the ground in the reservoir area.

Next, in the reservoir area, there are large scale landslides seen on the left bank immediately upstream of the damsite. If, after storage of water, there should be such an occurrence as land sliding throughout the entire landslide area, the safety of the dam and repercussions due to impact waves of the stored water would be of very dangerous, and even if crumbling should be held to a partial extent, there would be large quantities of soil flowing into the reservoir, and the storage capacity of the reservoir will be lowered to a great extent.

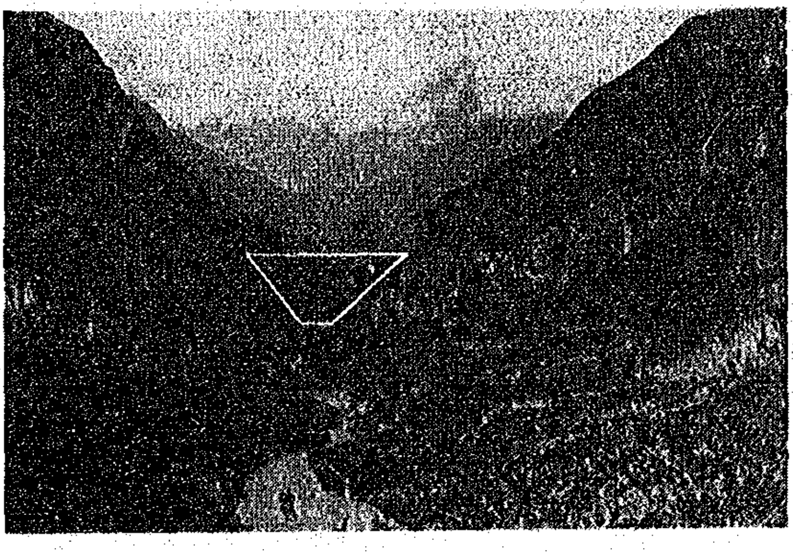
(2) Damsite No. 3

This site involves problems as a damsite, not only in the geologic conditions of the foundation, but also in regard to sedimentation within the reservoir.

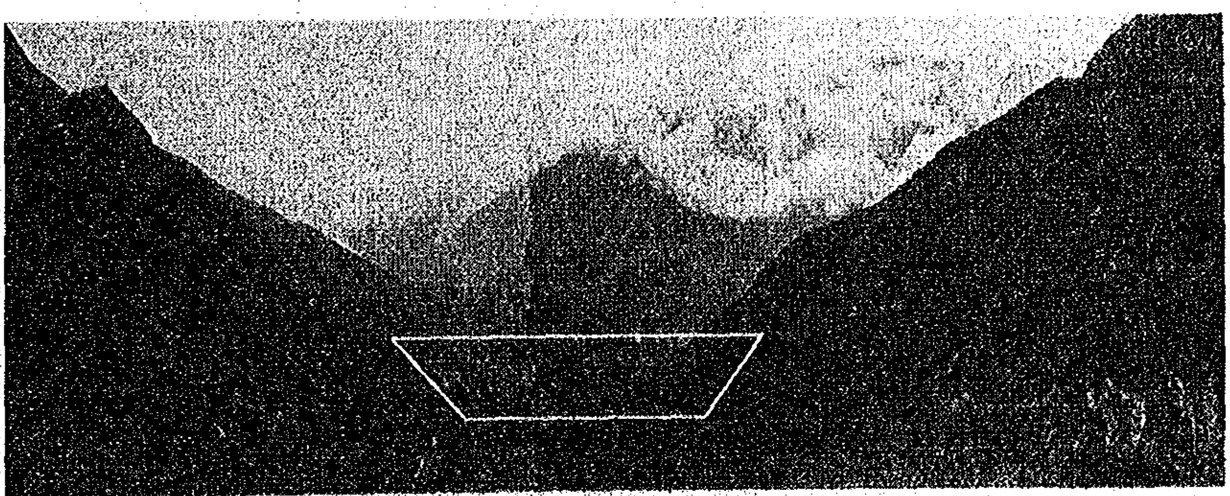
Topographically, the river width is reaching 200 m at the dam axis, which is far wider than the valley width of the proposed damsite. Geologically, the rock distributed at this damsite is limestone as at Damsite No. 2 which is susceptible to karstic action, and with regard to watertightness of the dam foundation, it is assumed there will be a similar problem. Another problem which is to be mentioned is that there is enormous amount of deposit and terrace at the dam axis which should be removed for construction, and it will impair the economics of this site completely.



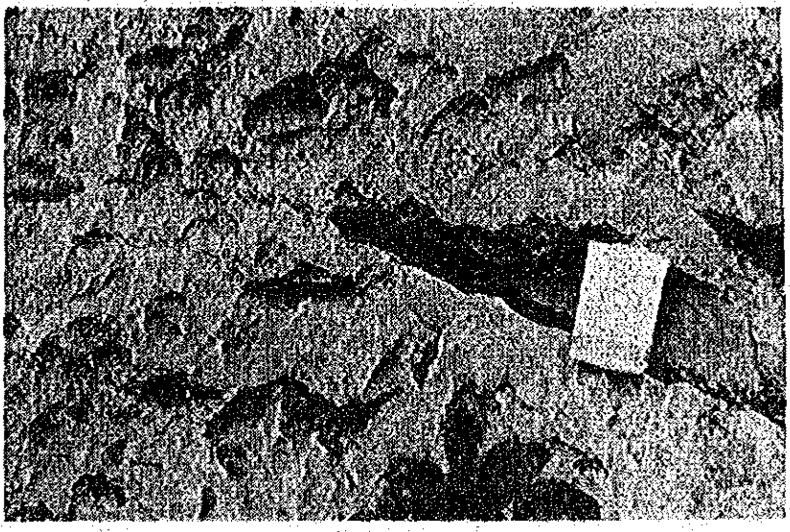
Selected Dam Site
(No.1 Dam Site)
from upstream



No.2 Dam Site
from upstream

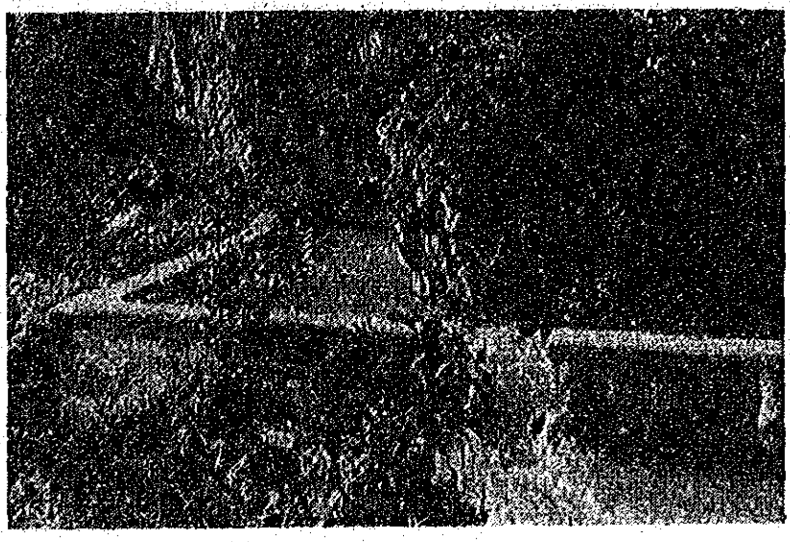


No.3 Dam Site from upstream

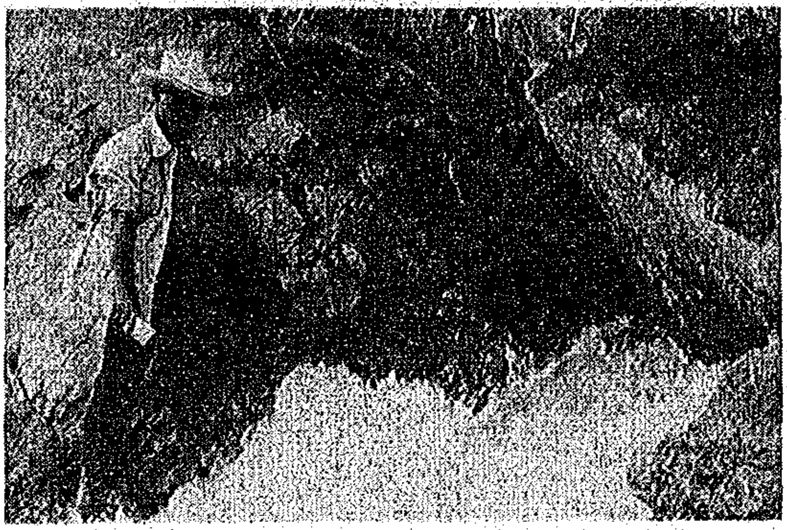


Deep limestone crack
at No.2 Dam Site

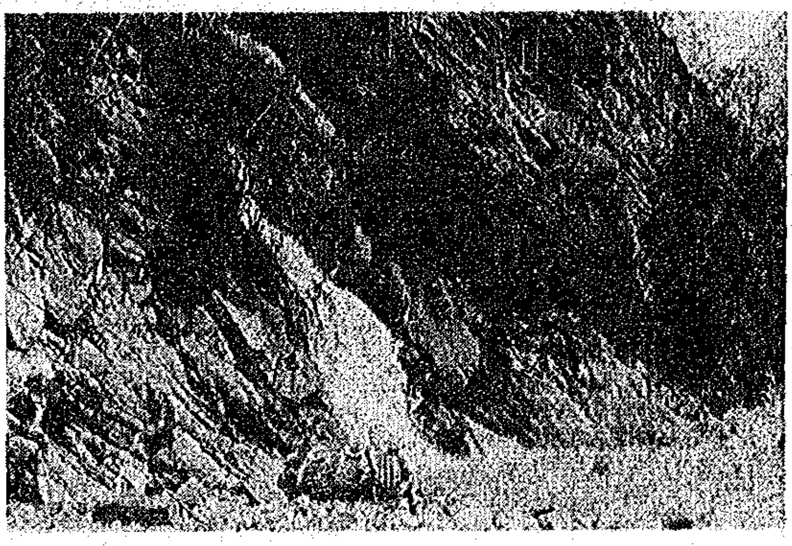
Spring water
at No.2 Dam Site



Deep river deposit
at No.3 Dam Site

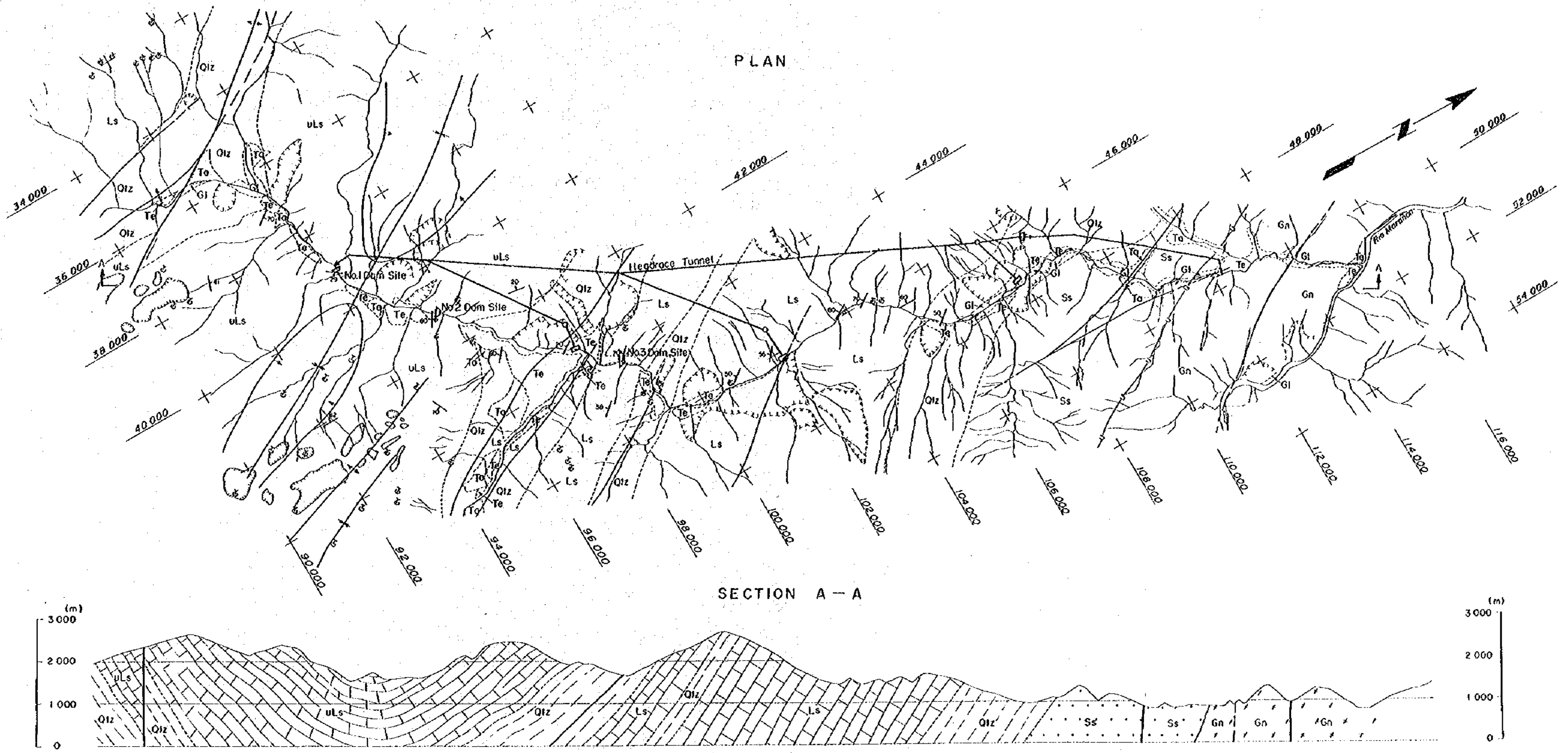


Spring water
at downstream
of No.3 Dam Site



Spring water
at downstream
of No.3 Dam Site

Deep limestone cracks
at No.3 Dam Site



LEGEND

Quaternary	Ta	Talus deposit.
	Gl	River deposit.
	Te	River terrace deposit.
Cretaceous – middle	uLs	Upper limestone.
Cretaceous – lower	Qtz	Alternation of Quartzite, Sandstone and Slate.
	Ls	Lower limestone.
Jurassic	Ss	Sandstone and Slate.
Precambrian	Gn	Gneiss.

- Geologic boundary. (conformity)
- Unconformity. (u open to younger formation)
- Fault. (; assumed)
- Axis of anticline.
- Axis of syncline.
- Strike and dip of strata.
- Spring.
- Sinkhole.
- Landslide.
- Gracial cirque.
- Section

Note;

1) Geological structure lines shown on the geological map are chiefly on the basis of the interpretation of aerial photographs.

2) Classification, strike and dip of strata, and location of spring, landslide and gracial cirque are based on the date collected by reconnaissance survey along the Rio Yangos as well as interpretation of aerial photographs.

0 5000m

Fig. 5-1 Geologic Map

CHAPTER 6

HYDROLOGY

FIGURES

TABLES

CHAPTER 6 HYDROLOGY

6.1 Hydrologic Data

As shown in Fig. 6-1, there are presently no observation station for precipitation and runoff, required in hydrologic analysis, within the Yangas River Basin except for one runoff gaging station of far upstream. However, there are precipitation and runoff observation stations in the Llaucano River Basin adjacent to the Yangas River while there are several precipitation observation stations in the Crisnejas River Basin also. The periods of observation, as shown in Table 6-1, are all very short being around 10 years except for rainfall records for Cajamarca City which are available for approximately 40 years. Furthermore, for many of the stations, there are periods during which observations are lacking.

The frequencies of observation are daily for both runoff and precipitation, while further, data on hourly precipitation have been obtained at Brillantana Observation Station. Runoff data have been calculated based on records of automatic water gages.

As described above, data required for carrying out hydrologic analysis are greatly lacking for the present.

6.2 Meteorological Conditions

The Yangas Project is situated in northern Peru on the Amazonas side of the Andes Mountain Range and is subjected to typical meteorological conditions of the Andes mountain area.

In general, the weather of the Andes mountain area is governed by the stationary Pacific Ocean high atmospheric pressure with its center off the coast of Chile, the Humboldt Current which flows north along the Pacific Ocean coast to close to the border between Ecuador and Peru, and the atmospheric pressure situations of the Amazonas area, and normally, there are no moving Highs and Lows pressures.

In the summer (from October to April), a heat Low, with its center in the Amazonas area and overlying a wide range to the eastern slopes of the Andes Mountain Range, is formed and violent ascending air currents are born. Meanwhile, the seasonal winds from the high atmospheric pressure over the Pacific Ocean towards this low pressure become strong and blow against the western slopes of the Andes. Furthermore, a frontogenesis extends from the Pacific Ocean to the mountain area. Due to the influences of these phenomena, the weather of the Andes Mountains is much cloudy in the summer and a rainy season sets in.

On the other hand, in the winter (from May to September), the Amazonas region is covered by the Atlantic Ocean high atmospheric pressure and with the Pacific Ocean high atmospheric pressure with its center off the coast of Chile, the pressure gradient becomes gentle between these two high pressures as a whole, and the seasonal winds are weakened, clear weather prevails, and it becomes the dry season.

As mentioned above the climate of the Andes mountain area is distinctly divided into a rainy season and a dry season, with extremely stable cycles.

However, in extremely rare cases, the Humboldt Current is overwhelmed by a warm current flowing to the south along the coast. Such a phenomenon occurred in March of 1925 and there are records that high temperatures and heavy rains occured in the northern coast of Peru, but it is not clear what the influences were brought on the mountain area and especially the vicinity of the Yangas River. The watershed of the Yangas Project area is situated at a altitude around 4,000 m which belongs to rather low side of the Andes Mountain Range, and therefore there is no snow and ice in summer.

The difference in quantities of precipitation for the dry and rainy seasons is prominent similarly to other mountain areas as shown in Fig. 6-2, while influenced by topography, there are large variations according to site.

As shown in Fig. 6-3, temperatures are more or less the same throughout the year. Annual mean temperatures, maximum mean temperatures and minimum mean temperatures examined in terms of relations with altitudes of observation stations are as given in Fig. 6-4. It is generally said that temperature becomes 0.6°C lower with every 100-m rise in altitude, and the relations between the annual mean temperatures and the observation station altitudes in Fig. 6-4 are close to this featuar. The temperatures at the Yangas Dam (El. 1,550 m) are estimated from this figure to be the following:

Annual mean temperature	19°C
Mean maximum temperature	25°C
Mean minimum temperature	15°C

As for humidity, it is generally high being 60 to 90% for the whole Project area, and at Celendín City, it is between 70 to 85% throughout the year as shown in Fig. 6-3.

6.3 River Run-off at Project Site

6.3.1 Method of Estimation

It is extremely difficult to estimate the daily and monthly runoffs necessary for examination of the Yangas Project because of the almost complete lack of both runoff data and precipitation data for this drainage area. Therefore, the average precipitation within the Yangas Basin was firstly estimated employing existing data of neighboring basins, and by multiplying this by the runoff coefficient estimated from the neighboring basins, the runoff was estimated.'

For checking whether or not the precipitation data used for this analysis would have changed during the observation period with variations in observation methods and surrounding conditions, the double mass curve technique was applied. And it was found that all data have linear relations and there exist ample reliabilities as shown in Fig. 6-5.

However, the greater part of the precipitation data is available for only an observation period of the recent ten years (1964 - 1973) and the data are insufficient for long-range estimation. Therefore, it was attempted to extend the period using the 40-year data at Cajamarca to be related, but since the correlations between Cajamarca data and other stations ones were poor at 0.12 - 0.73, this method could not be applied. Acordingly, it was decided to investigate only the trend of precipitation for the recent 10 years from the 40 years data of annual precipitation of Cajamarca station.

As methods of estimating the average precipitation within a basin there are generally the Thieseen Method and the Isohyetal Method, but in this Report, the Elevation Method (tentative term) was added and studies were made using these three methods.

As for runoff data of neighboring basins, those of Llaucano Derivación Gaging Station (C.A. = 341.0 sq.km) and Corellama Gaging Station (C.A. = 597.1 sq.km) further downstream in the Llaucano River Basin are available. Although the latter station has a catchment area close to that of the Yangas Project site (637.1 sq.km), there is big influence of irrigation water intake and it is judged inappropriate to apply its data to the Yangas site. Therefore, the upstream Llaucano Derivación Gaging Station was adopted as the runoff gaging station for estimation.

6.3.2 Examination of Precipitation Cycle

Fig. 6-6 shows the annual precipitation for each year and the variation in five-term moving average values at Cajamarca for the 40 years from 1934 through 1973. According to this, the precipitation cycle at Cajamarca is approximately 30 years and it is seen that the recent 10 years (1964 - 1973) used for calculating runoff corresponds to a dry period. In effect, in contrast to the average annual precipitation of 747 mm for the 40 years, the average annual precipitation of the recent 10 years is 690 mm, or approximately 8% less, while the figure for 1968 is 441 mm, the minimum for the 40-years period.

Since therefore the data used are understood to be of a dry period, the results of runoff calculations will be on the low side for a long-range forecast. However, it was decided not to make any modification in this Report for conservative purpose.

6.3.3 Thiessen Method

This is a method which is used most generally and it is possible to determine monthly runoffs of each year with ease.

Using the Thiessen Method and dividing the Yangas Basin as shown in Fig. 6-7 and determining the area controlled by each of the observation stations, the ratio thereof, are as indicated below.

Observation Station	Area Controlled	Ratio
Celendín	193.0 sq.km	0.303
Brillantana	249.4	0.391
Hda Negritos	33.5	0.053
La Llica	161.2	0.253
Total	637.1	1.000

The aggregate of the monthly average precipitations of the above 4 observation stations for the recent 10 years multiplied by the respective drainage area ratios is the monthly average precipitation of the Yangas Basin. As shown in Table 6-2, this value ranges betwen 17.8 mm and 146.1 mm for each month, and the average annual precipitation for the most recent 10 years is 1,020 mm.

Using the same method, the average precipitation was obtained for the basin of Llaucano Derivación Gaging Station (C.A. = 341 sq.km) on the adjacent Llaucano River for which runoff data are available. The monthly runoff coefficient was calculated from this area precipitation values and the observed runoff data of the Gaging Station. It was between 0.199 and 0.760 depending on the month. Applying this runoff coefficient without modification to the Yangas basin, the monthly average runoffs of the Yangas River can be obtained from the previously mentioned monthly average precipitations and the catchment area (637.1 sq.km) at the Yangas Dam site. The results, as shown in Table 6-3, are 2.15 cu.m/sec - 22.93 cu.m/sec as monthly average runoffs and 10.15 cu.m/sec as annual average runoff.

6.3.4 Isohyetal Method

Based on the data of precipitation observation stations in areas surrounding the river basin, and taking into consideration topography, wind direction, etc., it is possible to plot an isohyetal map for the Yangas River Basin. Since the amount and distribution of precipitation differs every month, an isohyetal map is drawn for each month. Typical isohyetal maps are indicated in Fig. 6-8 and Fig. 6-9. Determining the areas surrounded by isohyetal lines for each month from the maps and by multiplying by average precipitations of the respective areas, it is possible to calculate the average monthly precipitation within the basin.

From there on, the method of obtaining the runoff of the Yangas River is the same as with the Thiessen Method. The results, as shown in Table 6-4, are 1.65 cu.m/sec to 21.77 cu.m/sec for monthly average runoff and 9.46 cu.m/sec for annual average runoff.

6.3.5 Elevation Method

The distribution of precipitation in the area concerned is thought to vary in a complex manner depending on topography, wind direction and other factors. Among these, the relation between the elevation of each observation station (which is considered to be simplest parameter among the topographic factors) and precipitation (divided into rainy and dry season) was examined. And a linear relationship was found to exist between altitude and rainfall as shown in Fig. 6-10.

In the rainy season (from October to April), precipitation (total for the 7 months of the rainy season) increases by approximately 25 mm for every 100-m rise in elevation, while in the dry season (from May to September), there is some difference between the Llaucano River Basin and the Crisnejas River Basin (Cajamarca River Basin and Condebamba River Basin), and precipitation (total for the 5 months of the dry season) increases by approximately 12 mm for the former and approximately 5 mm for the latter with each rise of 100 m in elevation.

The coefficient of correlation between elevation and precipitation is 0.77 to 0.90 so that it cannot be said there is a perfect correlation, but it is thought the relation between topography and precipitation can be roughly grasped by this method.

Since elevations in the Yangas River Basin are in a relatively wide range of 1,500 to 4,000 m, it was decided to utilize the above relation to calculate precipitation of the Yangas River Basin by selecting representative precipitation observation station data, correcting them to obtain typical precipitations of each elevation band and multiplying these by area percentages of the respective elevation bands of the basin and aggregating the results. This is tentatively called here the Elevation Method.

The elevation bands, typical elevations, typical observation stations, correction factors and the drainage area ratios of the Yangas River Basin and the basin of the Llaucano Derivación Gaging Station for this Elevation Method are given in Table 6-5. By using this method, it is possible to compute monthly runoffs of the Yangas River Basin through calculation of precipitation in the basin, and similarly applying the runoff coefficients of runoff gaging stations without modification. The results of such calculations, as shown in Table 6-6, are 1.77 to 23.29 cu.m/sec with annual average runoff being 9.69 cu.m/sec.

The merit of this method is that topographic factors are taken into account to some extent and that, similarly to the Thiessen Method, monthly runoffs of each year are easily obtained. The defect of this method is that precipitation distribution characteristics and discharge characteristics from outside the Yangas River Basin are applied without modification to the basin.

6.3.6 Comparison of Estimation Methods

The 10-year average monthly precipitations and runoffs of the project site calculated by the three methods (Thiessen Method, Isohyetal Method and Elevation Method) described in the preceding sections are summed up in Tables 6-3, 6-4 and 6-6. Excerpting the annual average precipitations and annual average runoffs at the Yangas Dam site from these tables, they are the following:

Calculation Method	Annual Average Precipitation (mm)	Annual Average Runoff (cu. m/sec)
Thiessen Method	1,020	10.15
Isohyetal Method	963	9.46
Elevation Method	1,095	9.69

In effect, compared with the Elevation Method, the annual average precipitation is smaller for the two other methods. This is thought to be due to the big effect that applying the observation station data of low elevation and small precipitation (even though they are close-by to the Project site) to the Project site which is at a high elevation in case of the two methods.

However, runoff coefficient used for calculating runoff from precipitation is the unmodified runoff coefficient of the Llaucano Gaging Station computed from the precipitation obtained by the same method and actually measured runoff. Therefore, the tendency is strong for the precipitation error caused to be offset when computing runoff.

As for the values of annual average runoff, all values are smaller compared with the value of 10.31 cu.m/sec which is converted using drainage area ratio for the Project site from the average runoff at the Llaucano Derivación Gaging Station. This is due to the fact that the precipitation of the Project site basin being estimated to be smaller than for the Llaucano Gaging Station basin.

For such reasons that there are no large differences between the calculation results by the three methods, and that it has been clarified that the Elevation Method has a good correlation for the dry season which carries great weight in a power development scheme, the Elevation Method was adopted for the study to follow.

6.3.7 River Run-off

The calculations of monthly or daily runoffs of each year required for establishment of the power development scheme are performed using the following equations in accordance with the Elevation Method:

$$Q'_{Y,M} = Q_{Y,M}\left(\frac{A'}{A}\right) \cdot \left(\frac{R'_{Y,M}}{R_{Y,M}}\right) \qquad \text{monthly runoff}$$

$$Q'_{Y,M,D} = Q_{Y,M,D}\left(\frac{A'}{A}\right) \cdot \left(\frac{R'_{Y,M}}{R_{Y,M}}\right) \qquad \text{daily runoff}$$

where,

$Q'_{Y,M}$, $Q_{Y,M}$: monthly runoffs of Yangas site and Llaucano Derivación Gaging Station in M month of Y year.

A', A : catchment areas of Yangas site and Llaucano Derivación Gaging Station

$R'_{Y,M}$, $R_{Y,M}$: monthly precipitations of Yangas site and Llaucano Derivación Gaging Station (determined by Elevation Method)

$Q'_{Y,M,D}$, $Q_{Y,M,D}$: daily runoffs of Yangas site and Llaucano Derivación Gaging Station on D day, M month of Y year.

In other words, the monthly runoff or daily runoff of the Yangas site is equal to the observation data of Llaucano Derivación Gaging Station multiplied by the respective drainage area ratios and monthly precipitation ratios. The reason for using the monthly precipitation ratio for calculating daily runoff is that precipitation and runoff do not correlate to each other in a single day.

6.4 Flood Discharge

6.4.1 Method of Estimation

There are neither precipitation observation stations nor adequate gaging stations in the Project basin, while hourly records over a long term are lacking for adjacent river basins, and it is difficult to estimate the flood discharge of the Project site with high accuracy.

Here, a study is made by the method of calculating peak flood discharge using the Rational formula and triangular unit hydrograph analysis, based on the data of neighboring precipitation observation stations, and also by the method of calculating daily flood discharge from the data of neighboring runoff gaging stations.

In order to obtain flood discharge with high accuracy, it will be necessary for observation stations for precipitation and runoff capable of hourly recording to be provided at several places as well as continuing recording at the existing stations for an even longer period.

6.4.2 Probable Point Precipitation

The probable point precipitation was determined from the records of maximum daily precipitation (19 - 51 mm) data for each year during a 15-year period for Cajamarca using the Gumbel Method. The results are as shown in Fig. 6-11, and the values are arranged as following.

Return Period (years)	Probable Point Precipitation (mm)
100	70
500	83
1,000	90
10,000	110

With regard to data of other observation stations, they were not used for the reasons that their observation periods are all shorter than that of Cajamarca, while those for relatively long periods are not very much different from the above results.

6.4.3 Probable Area Precipitation

It is inconceivable for the previously mentioned point precipitation to occur uniformly throughout the catchment area. In fact, on investigation of rainfall distribution in the neighboring Llaucano River Basin, the ratio of point precipitation to area precipitation was a maximum of 0.80. Meanwhile, in "Design of Small Dams," U.S. Bureau of Reclamation, the ratio of point precipitation to area precipitation in mountainous areas is given as a function of the catchment area, and according to this method, the ratio becomes 0.85 for the approximately 640-sq.km catchment area of the Yangas site. It is decided here to use the latter figure which gives values on the conservative side, and probable area precipitation is determined by multiplying probable point precipitation obtained in 6.4.2 by 0.85.

Return Period (years)	Probable Area Precipitation (mm)
100	60
500	71
1,000	77
10,000	94

6.4.4 Peak Flood Discharge by Rational Formula

The formulae generally used for calculating flood discharge are the following:

$$Q = \frac{1}{3.6} \cdot f \cdot r_t \cdot A \quad \text{.................. (Rational formula)}$$

$$r_t = \frac{R_{24}}{24} \left(\frac{24}{T}\right)^{2/3} \quad \text{.................. (Mononobe formula)}$$

$$\left.\begin{aligned} T &= \sum_i \left(\frac{L_i}{w_i}\right) \quad \text{......................} \\ w_i &= 72\left(\frac{H_i}{L_i}\right)^{0.6} \quad \text{......................} \end{aligned}\right\} \text{(Rziha formula)}$$

where

Q : peak flood discharge

f : runoff coefficient

r_t : average rainfall intensity within flood arrival time

R_{24} : probable area precipitation

A : catchment area

T : flood arrival time

w_i : flood propagation velocity

L_i : distance of watercourse section

H_i : elevation difference between watercourse sections

Calculating the river bed gradient of the Yangas River dividing it into three sections and using these formulae, the flood arrival time will be 2.92 hours. For the runoff coefficient during floods, the value of 0.80 generally used is adopted. The peak flood discharge for each return period was obtained as below adding base flow of 30 cu.m/sec. And also, probable daily runoff could be calculated by using T = 24 hours for the above equations.

Return Period (years)	Probable Peak Flood Discharge (cu.m/sec)	Probable Daily Runoff (cu.m/sec -day)
100	1,500	380
500	1,650	450
1,000	1,900	480
10,000	2,300	580

6.4.5 Peak Flood Discharge Using Triangular Unit Hydrograph Method

Hydrographs are prepared from the probablo area precipitation listed in 6.4.3 in accordance with above mentioned "Design of Small Dams". The following formula is used for introducing cumulative discharge from cumulative precipitation:

$$Q = \frac{(P - 2.674)^2}{P + 10.695}$$

where

P : cumulative precipitation

Q : cumulative discharge

This equation has been formulated so that the runoff coefficient will be raised with increase in cumulative precipitation (see Fig. A-4, Design of Small Dams).

The result of calculations of 100-year probable flood discharge based on the above method, as shown in Fig. 6-12, is that maximum flood is reached at 4 hours after start of precipitation, and that there is sudden decrease during next 4 hours, following which there is a gradual decrease. It is also possible to obtain the daily amount of this flood discharge by measuring the area within the total hydrograph from the same Figure.

The probable flood discharges other than the 100-year return period when they are assumed to be proportional to area precipitation will be the following. In this case also, 30 cu.m/sec is added as base flow.

Return Period (years)	Probable Peak Flood Discharge (cu. m/sec)	Probable Daily Flood Discharge (cu. m/sec)
100	1, 100	360
500	1, 300	420
1, 000	1, 400	450
10, 000	1, 700	550

6.4.6 Daily Flood Discharge

The maximum daily runoff for each year is extracted from the daily runoff data for 10 years period at Llaucano Derivación Gaging Station and the probable daily flood discharge is obtained using the Gumbel Method. The result is as shown in Fig. 6-13, and converting this to the catchment area of the Yangas site, the results are as given below. The conversion rate in this case is 637.1 sq.km/341.0 sq.km = 1.868

Return Period (years)	Probable Daily Flood Discharge (cu. m/sec)
100	290
500	360
1, 000	400
10, 000	500

6.4.7 Summary

On arranging the results of calculations by the above three methods for 10, 000-year probable flood discharge, the following are obtained:

		Peak Flood Discharge (cu. m/sec)	Daily Flood Discharge (cu. m/sec -day)
(1)	Rational Method	2, 300	580
(2)	Triangular Unit Hydrograph Analysis Method	1, 700	550
(3)	Daily Flood Discharge		500

The differences in the results of (1) and (2) are to be due to the difference with regard to runoff coefficient. In other words, the difference lies in the fact that whereas a uniform 0.80 is used as the runoff coefficient in (1), it is varied in (2) between 0.03 and 0.96 depending on cumulative precipitation. The difference between (2) and (3) is due to the data used, precipitation in one and runoff in the other.

As the design flood discharge to be employed for designing spillway, 2,300 cu.m/sec, the 10,000-year probable flood discharge according to the Rational Method is used which is on the conservative side in consideration of effects on the downstream area and of the lack of data.

6.5 Evapotranspiration Loss

With regard to evaporation, the data measured for periods of 4 to 20 years at seven observation stations in the vicinity of the Yangas River were used. The observation data, as given in Table 6-7, show annual evaporation to range between 660 and 1,590 mm for an average of 1,130 mm.

Evaporation from a free water surface generally is proportional to the difference between the vapor pressure possessed by the water itself and the vapor pressure of the atmosphere contacting to the water, and varies according to wind speed and altitude (atmospheric pressure).

The equation by Carl Rohwer is used to estimate the evaporation from the abovementioned observation data.

$$E = 181\ (e_w - e_a)(1 - 0.0005\ B)(1 + 0.60\ w)$$

where

E : annual evaporation (mm)

e_w : saturated vapor pressure at water surface temperature (temperature here assumed to be equal to temperature of atmosphere) (mmHg)

e_a : vapor pressure of atmosphere (saturated vapor pressure at temperature of atmosphere multiplied by relative humidity) (mmHg)

B : atmospheric pressure at the location concerned (mmHg)

w : average wind speed (m/sec.)

Now, the respective values are calculated from the beforementioned observation station data as indicated in Table 6-8. In effect, e_w and e_a are obtained from annual mean temperature and annual average relative humidity, and B is obtained as standard atmospheric pressure from the altitude of the observation station, and $X = (e_w - e_a)(1 - 0.0005B)$ is calculated. The relationship between the value of X and the data of annual evaporation, as shown in Fig. 6-14, are more or less proportional ones.

Meanwhile, if the Rohwer equation result are plotted as shown in Fig. 6-14 with average wind speed as the parameter, it is indicated that average wind speed at the measurement time was between 2 to 3 m/sec for the above data. Although there are no measurement data on average wind speed, the above value may be judged to be reasonable judging from the average wind speeds data at 7.00 AM 1.00 PM and 7.00 PM. Therefore, it was decided to adopt the line for w = 2.5 m/sec shown in Fig. 6-14.

The following values are used for estimation of the evaporation at the Yangas site: elevation, 1,550 m; average air temperature, 19°C (see 6.2); relative humidity, 70%[average value for observation stations 75.1% (Table 6-8) but 70% on the conservative side taken]; average wind speed 2.5 m/sec. Based on these values, the following calculations are made:

$$e_w = 16.5 \text{ mmHg } (19°C)$$

$$e_a = 16.5 \times 70 \div 100 = 11.5 \text{ mmHg}$$

$$B = 630 \text{ mmHg (El. 1550 m)}$$

$$w = 2.5 \text{ m/sec}$$

$$\therefore \quad E = 181 (e_w - e_a)(1 - 0.0005B)(1 + 0.60w)$$

$$= 181 \times (16.5 - 11.5) \times (1 - 0.0005 \times 630) \times (1 + 0.60 \times 2.5)$$

$$\doteqdot 1,530 \text{ mm}$$

Since this value has been calculated based on evaporation data by evaporimeters, this value must be converted to evaporation from the reservoir surface. As the conversion factor, the generally used 0.70 was used. Consequently, the annual evaporation from Yangas Reservoir will be the following:

$$H_E = 0.70E = 1,070 \text{ mm}$$

The actual water loss due to evapotranspiration from the reservoir is expressed using the following equation:

$$H_L = H_E - (1 - \alpha) H_R$$

where

H_L : water loss through evapotranspiration (mm)

α : annual runoff coefficient in case of no reservoir

H_R : annual precipitation (mm)

H_E : annual evaporation (mm)

Putting the abovementioned annual evaporation, the annual average runoff coefficient of 0.436 calculated in Table 6-6, and the annual precipitation of 1,095 mm into the equation,

$$H_L = 1,070 - (1 - 0.436) \times 1,095$$

$$= 450 \text{ mm}$$

that is, the annual water loss through evapotranspiration from the reservoir is to be 450 mm.

For a more accurate estimation of the loss through evapotranspiration from the reservoir, it will be necessary for air temperature, water temperature, humidity, atmospheric pressure and evaporation to be measured at the Project site.

6.6 Sedimentation

It is rather difficult to make a reasonable estimation since neither data for measured sedimentation at the Yangas site nor sedimentation data at other sites are available. However, it is presumable that weathering products which comedown to the reservoir will be large in quantity since there are comparatively many land slides in the Yangas Basin, and there are prominent ups and downs in the terrain and the vegetation index is low. And also considering the fact that moving and transportation of weathered products will be prominent due to steep river gradients, it is understood there will be a relatively large amount of sedimentation.

The results of actual measurements of sedimentation in reservoirs in the Western part of the United States arranged in the form of comparison with drainage area are as shown in Fig. 6-15. According to this figure, specific sedimentations (cu.m/sq.km/year) are scattered by as much as about 10 times depending on the reservoir even with equal reservoir areas, but generally speaking, there is a trend for specific sedimentation to be reduced the larger the drainage area. The upper limit of specific sedimentation in case of a drainage area equal to that of the project site (640 sq.km) is 800 cu.m/sq.km/year.

Applying some margin to the above figure in consideration of the conditions abovementioned, the specific sedimentation of the project site was estimated to be 1,000 cu.m/sq.km/year. Therefore, the total sedimentation for a period of 100 years will be the following:

$$1,000 \text{ cu.m/sq.km/year} \times 637.1 \text{ sq.km} \times 100 \text{ year} \doteqdot 64 \times 10^6 \text{ cu.m.}$$

However, by installing a suitable equipment of sand flushing, the reservoir can be operated in a manner not to be a hindrance to power generation.

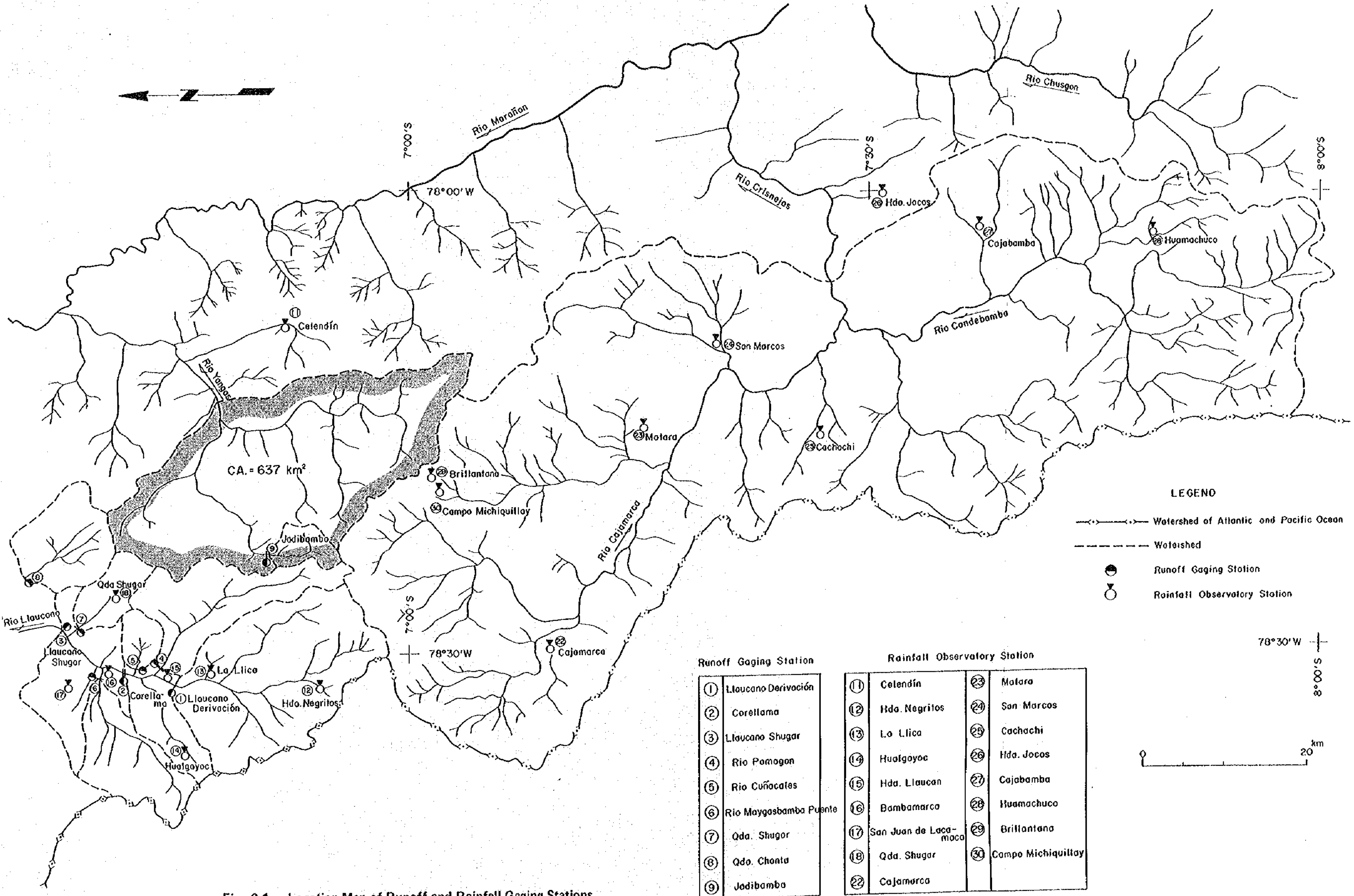


Fig. 6-1 Location Map of Runoff and Rainfall Gaging Stations

Fig. 6-2 Monthly Average Precipitation

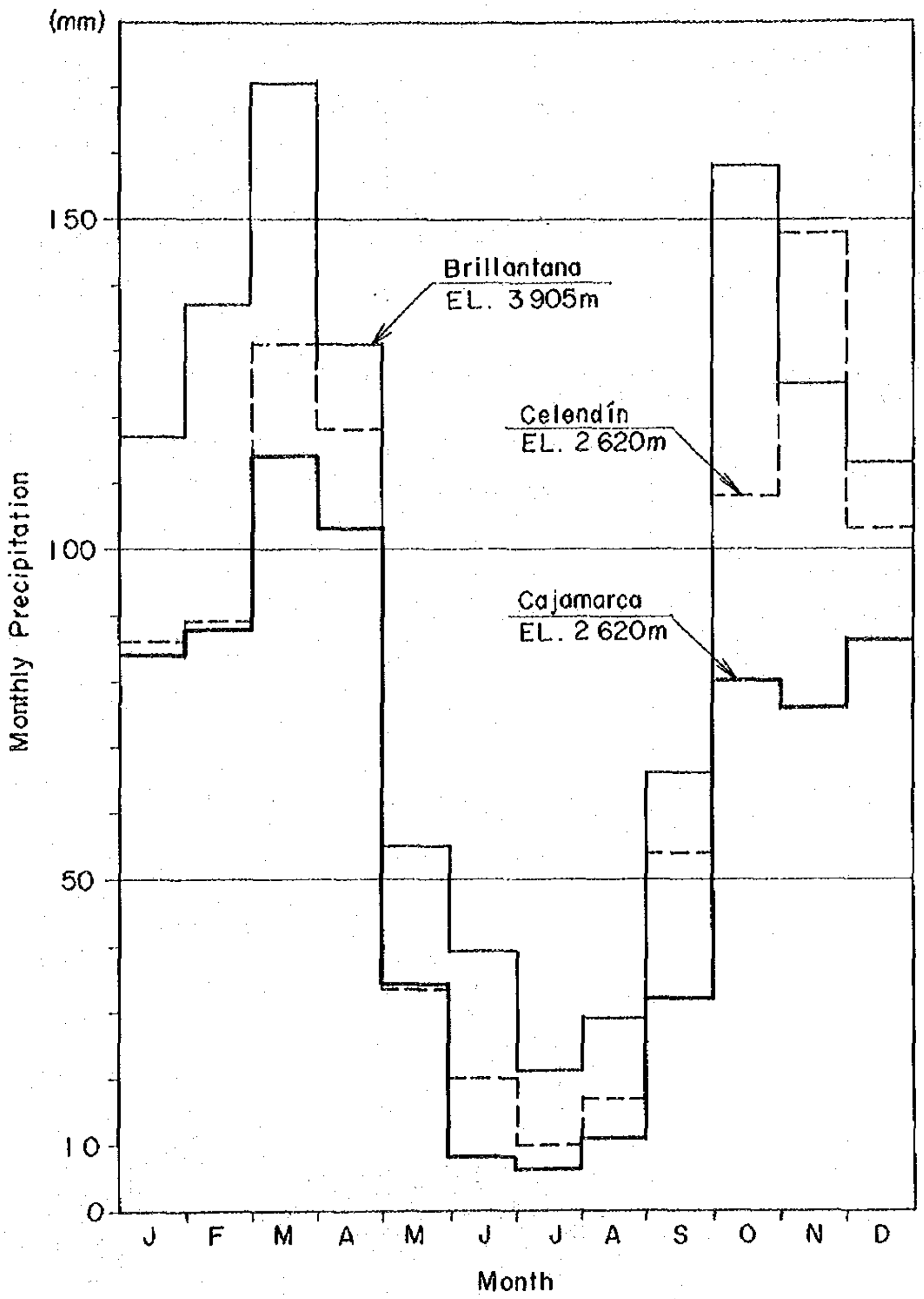
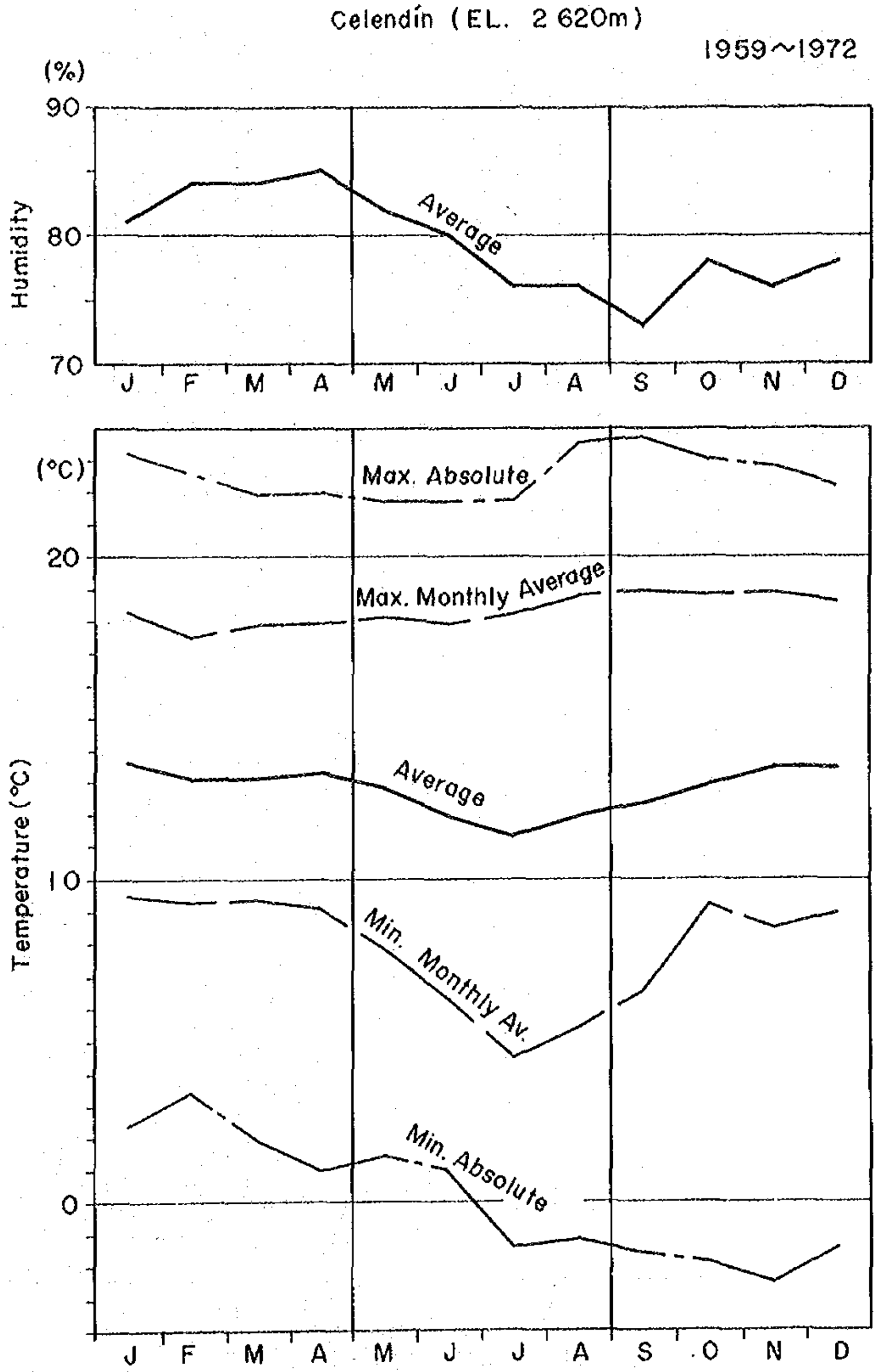


Fig. 6-3 Monthly Temperature and Humidity



after Estudio de Las Características Climatológicas, SENAMHI, 1974

Fig. 6-4 Altitude Vs Temperature

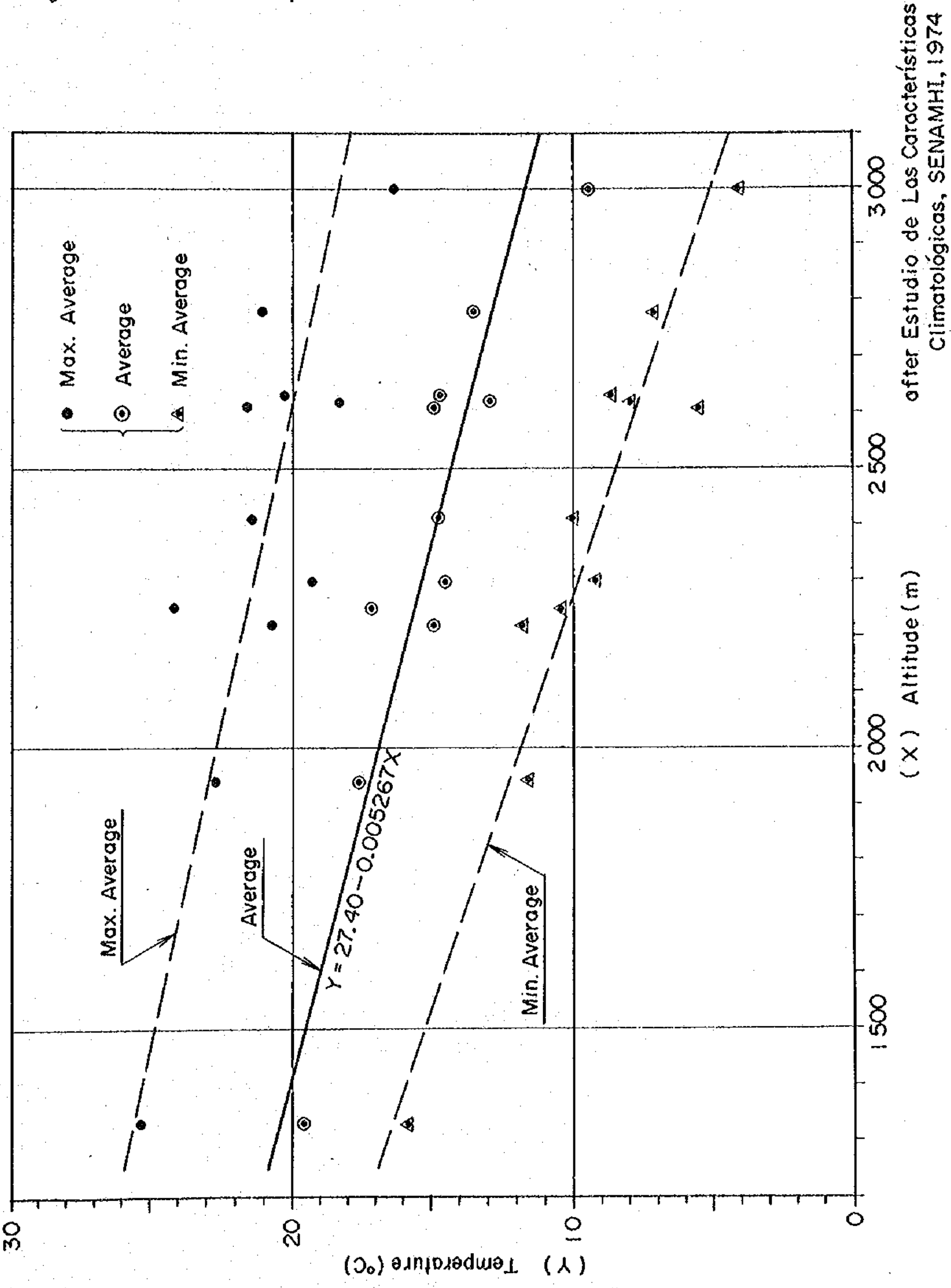


Fig. 6-5 Doble Mass Curves

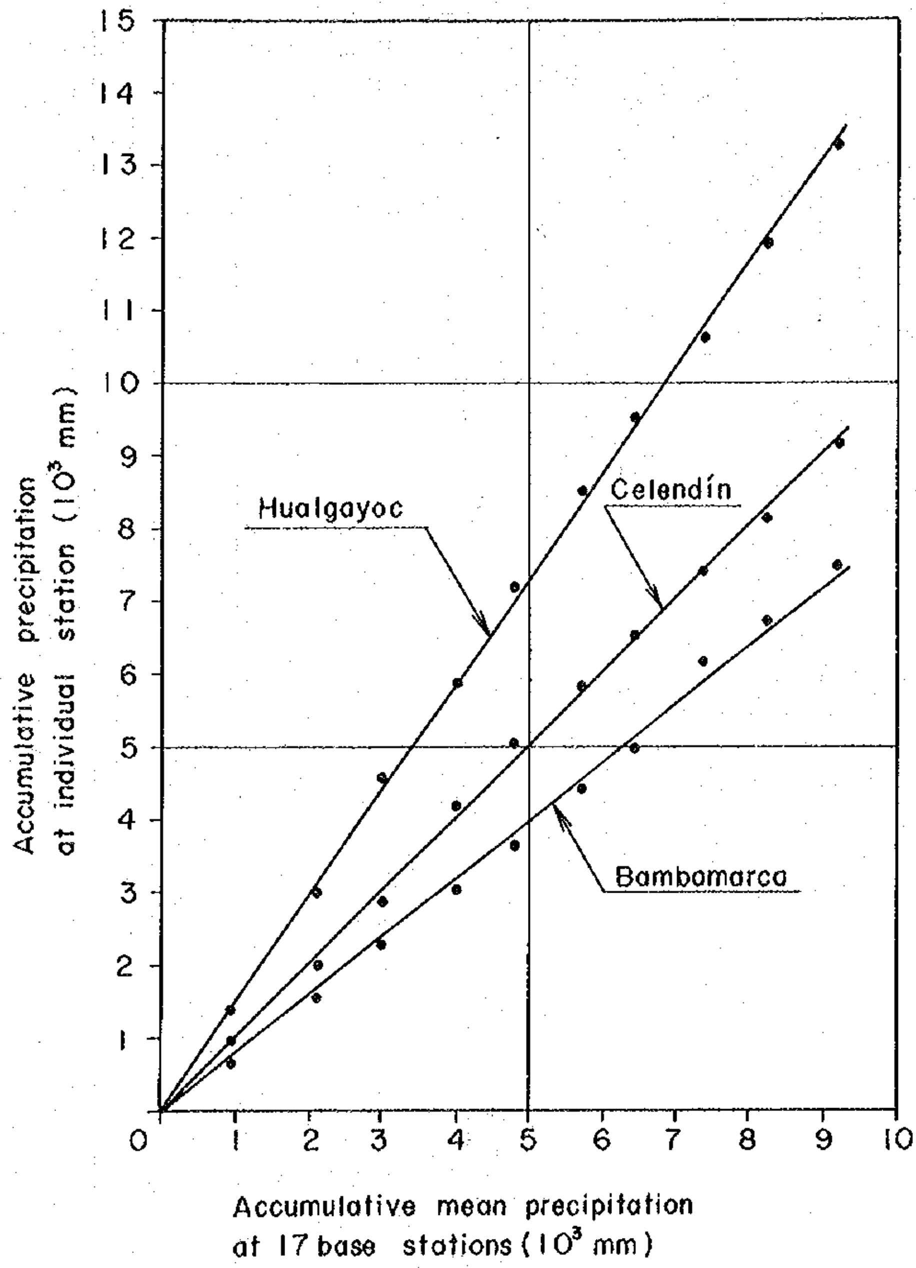


Fig. 6-6 Rainfall Period of Cajamarca

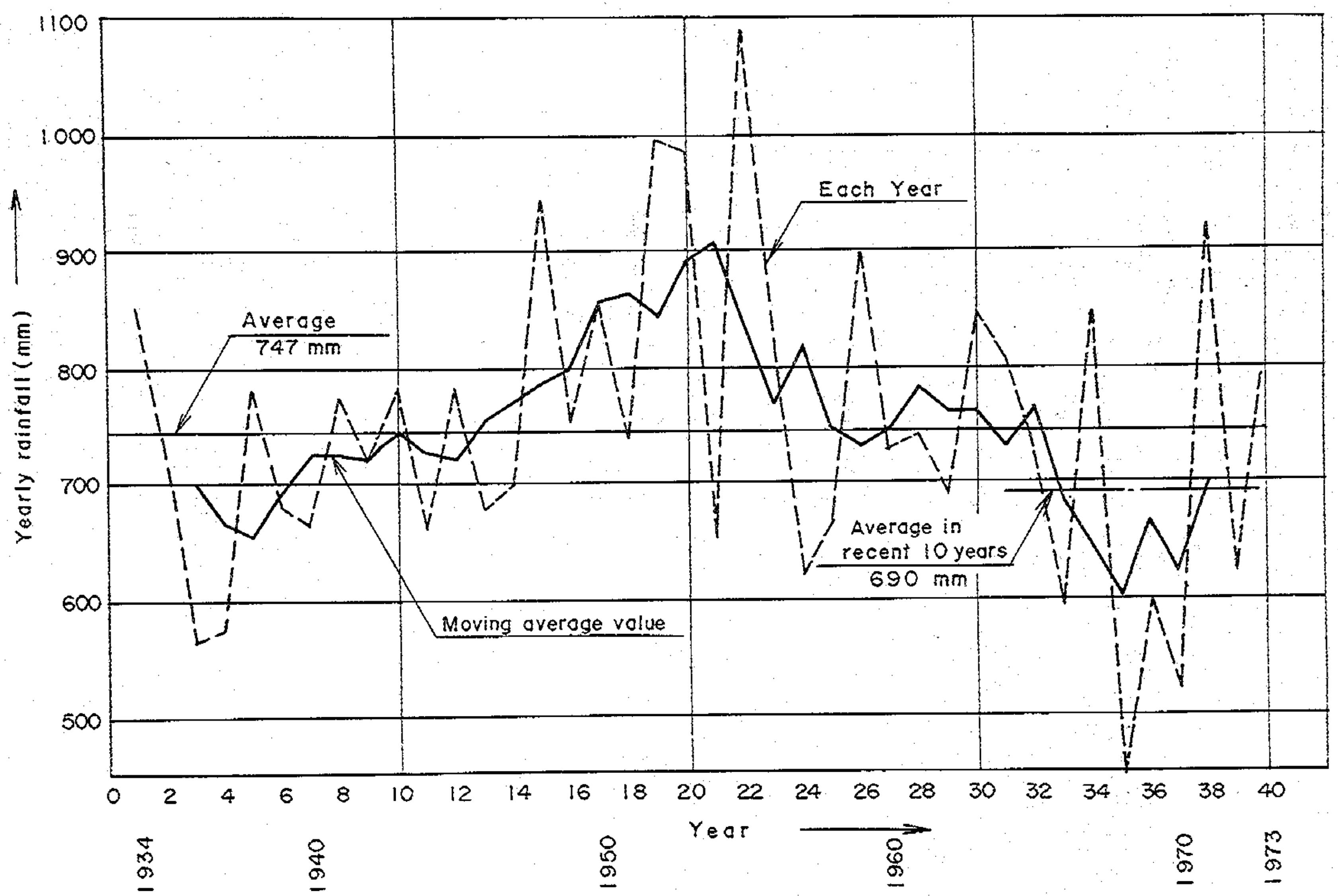


Fig. 6-7 Thissen Diagram

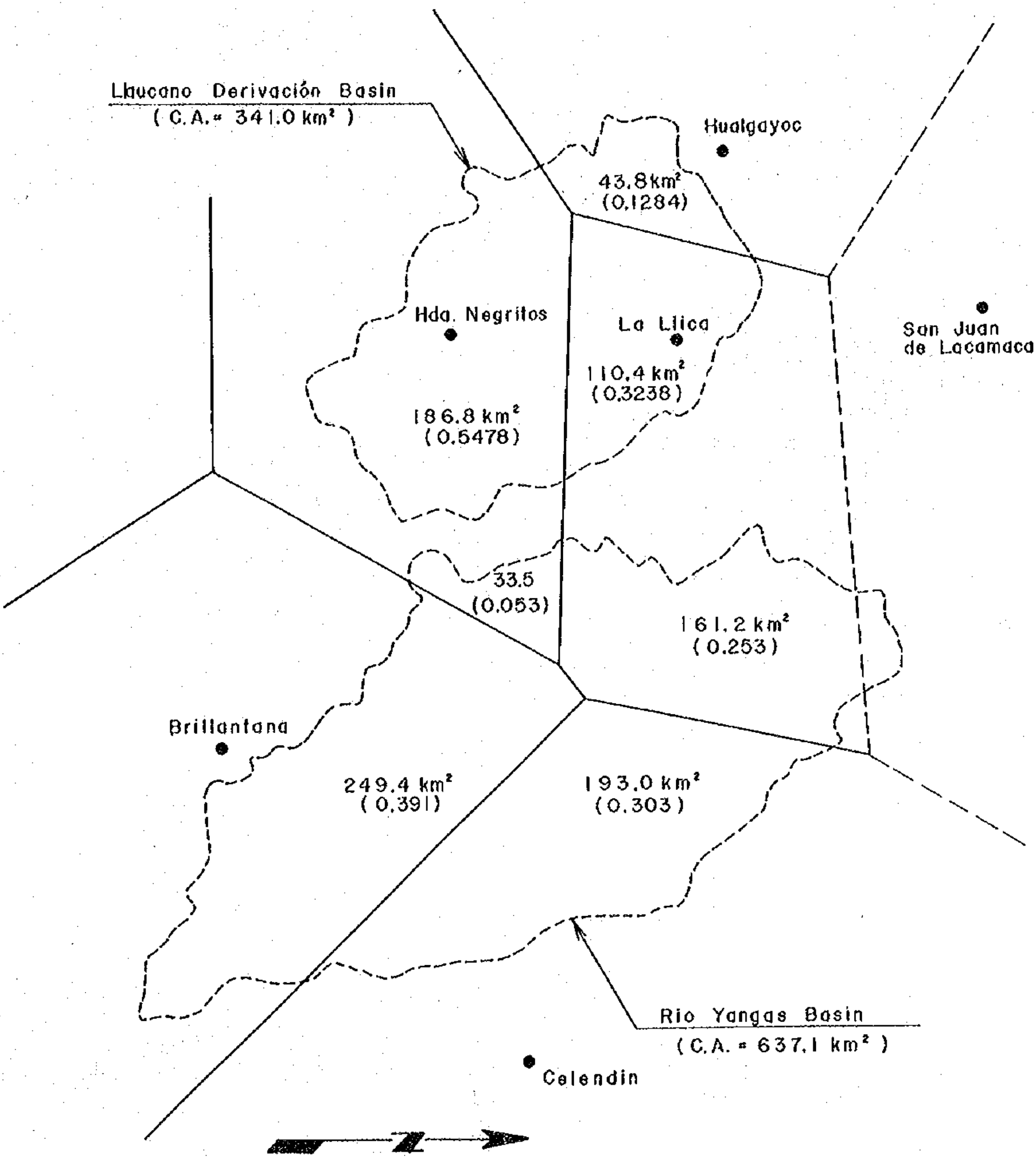


Fig. 6-8 Isohyetal Map (Mar.)

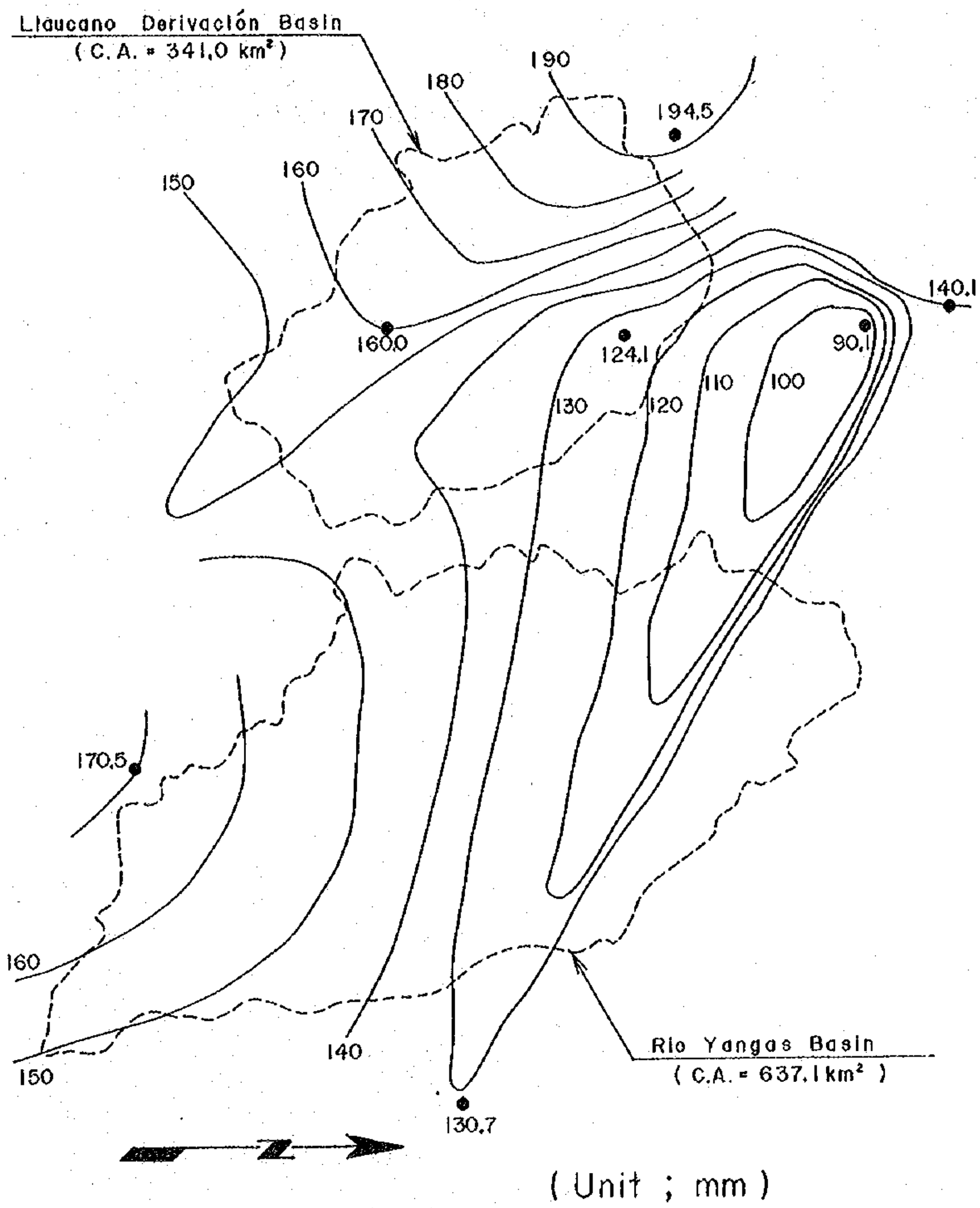


Fig. 6-9 Isohyetal Map (Aug.)

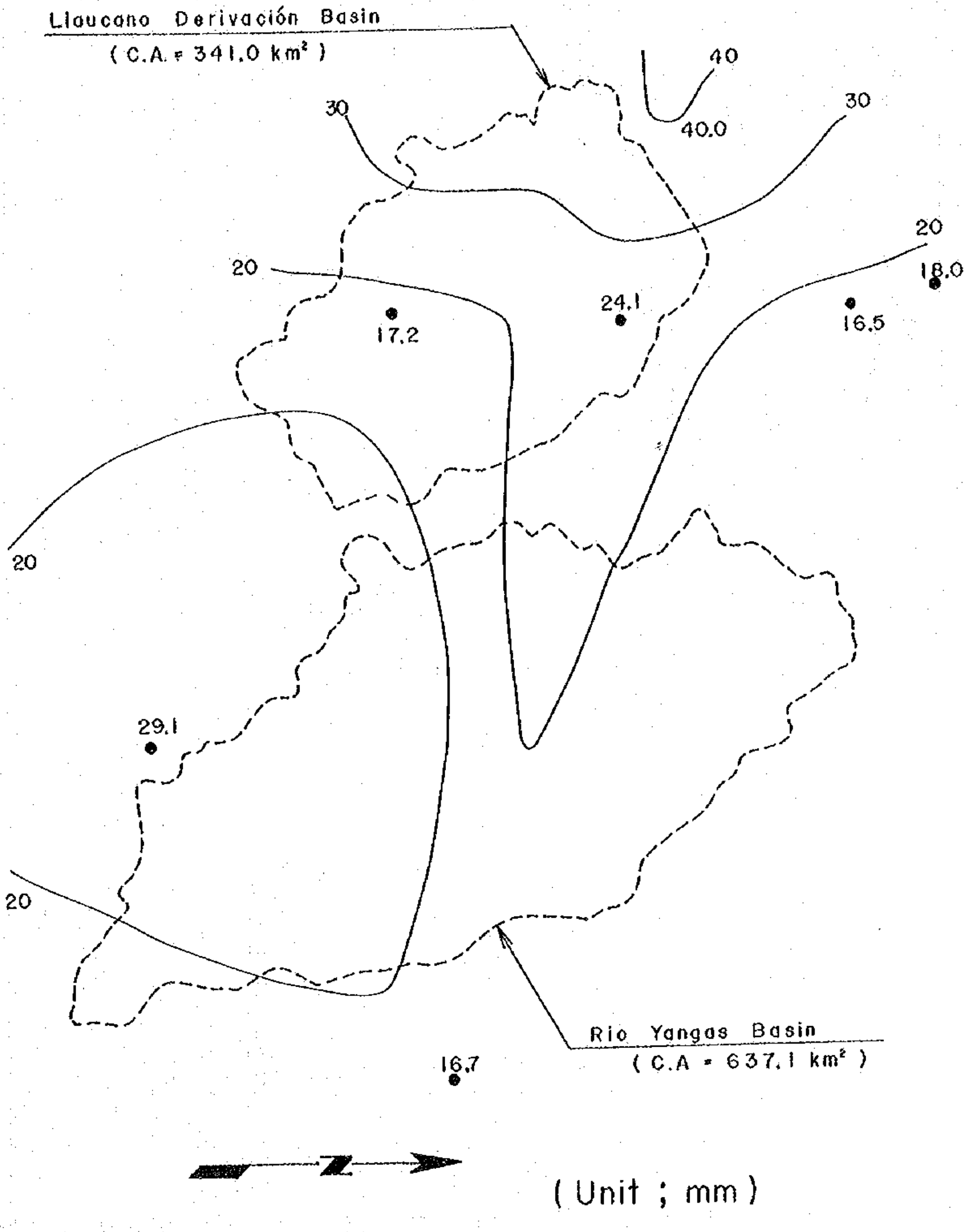


Fig. 6-10 Altitude Vs Rainfall

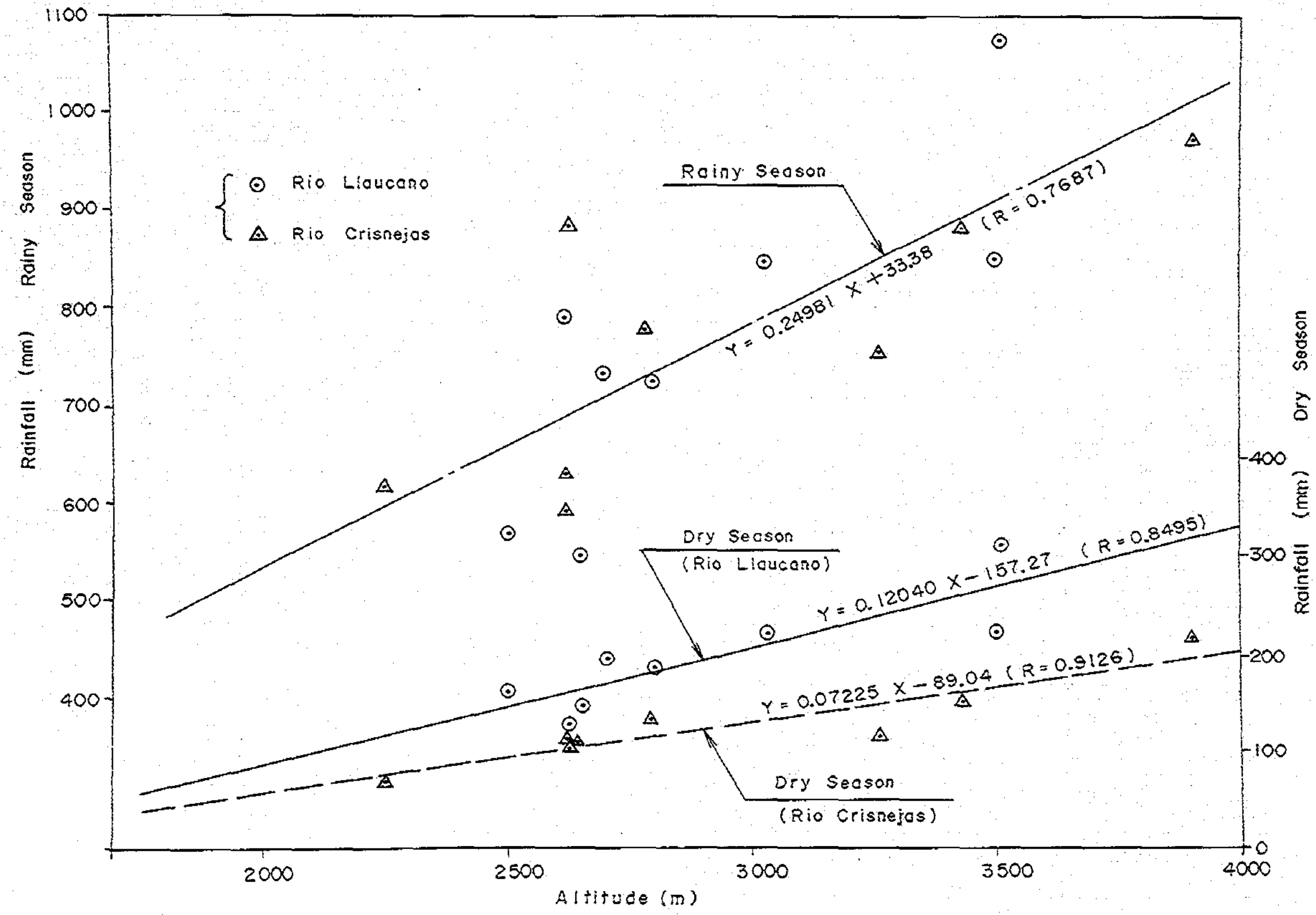


Fig. 6-11 Probable Point Rainfall (Cajamarca)

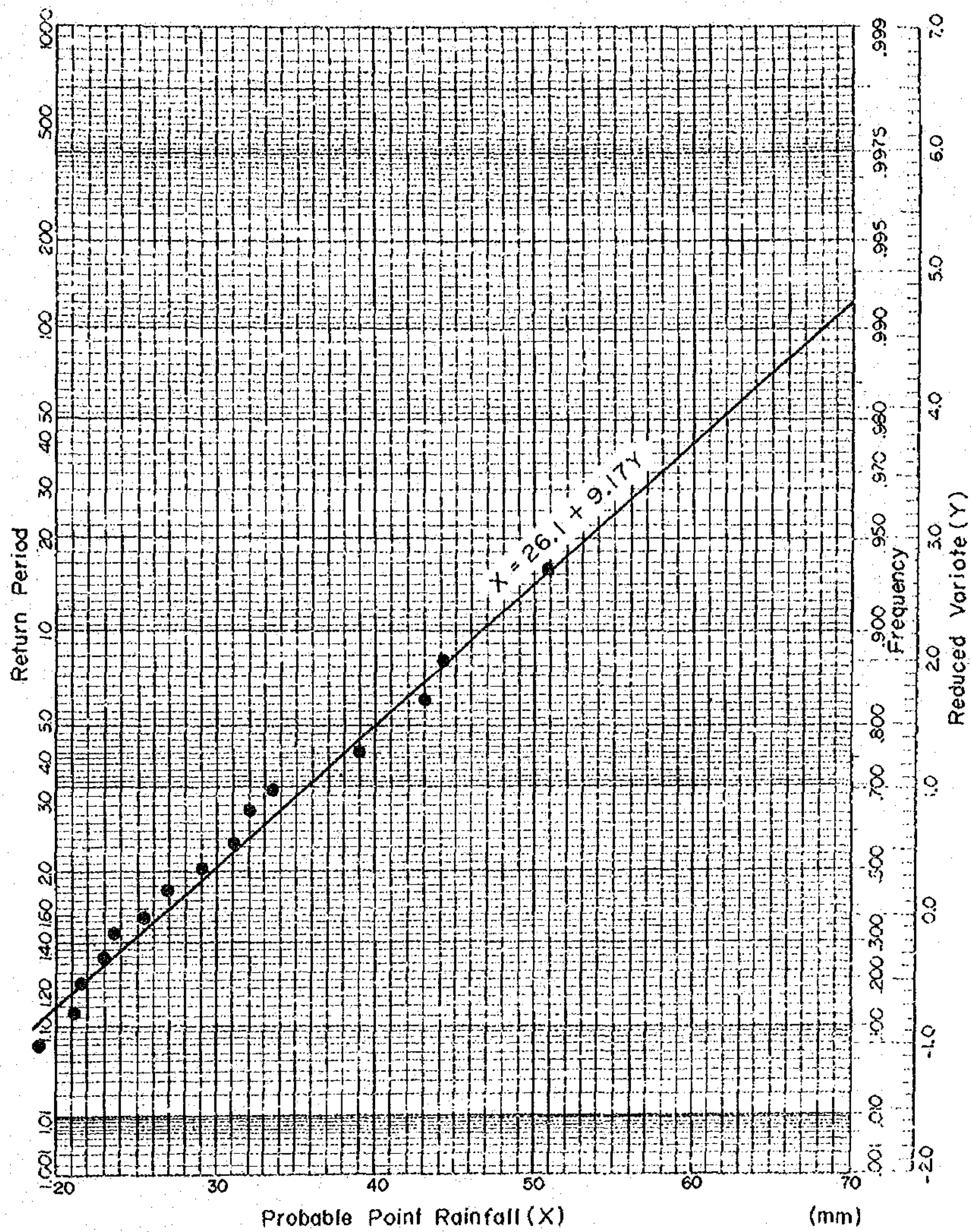


Fig. 6-12 Triangular Hydrograph Analysis

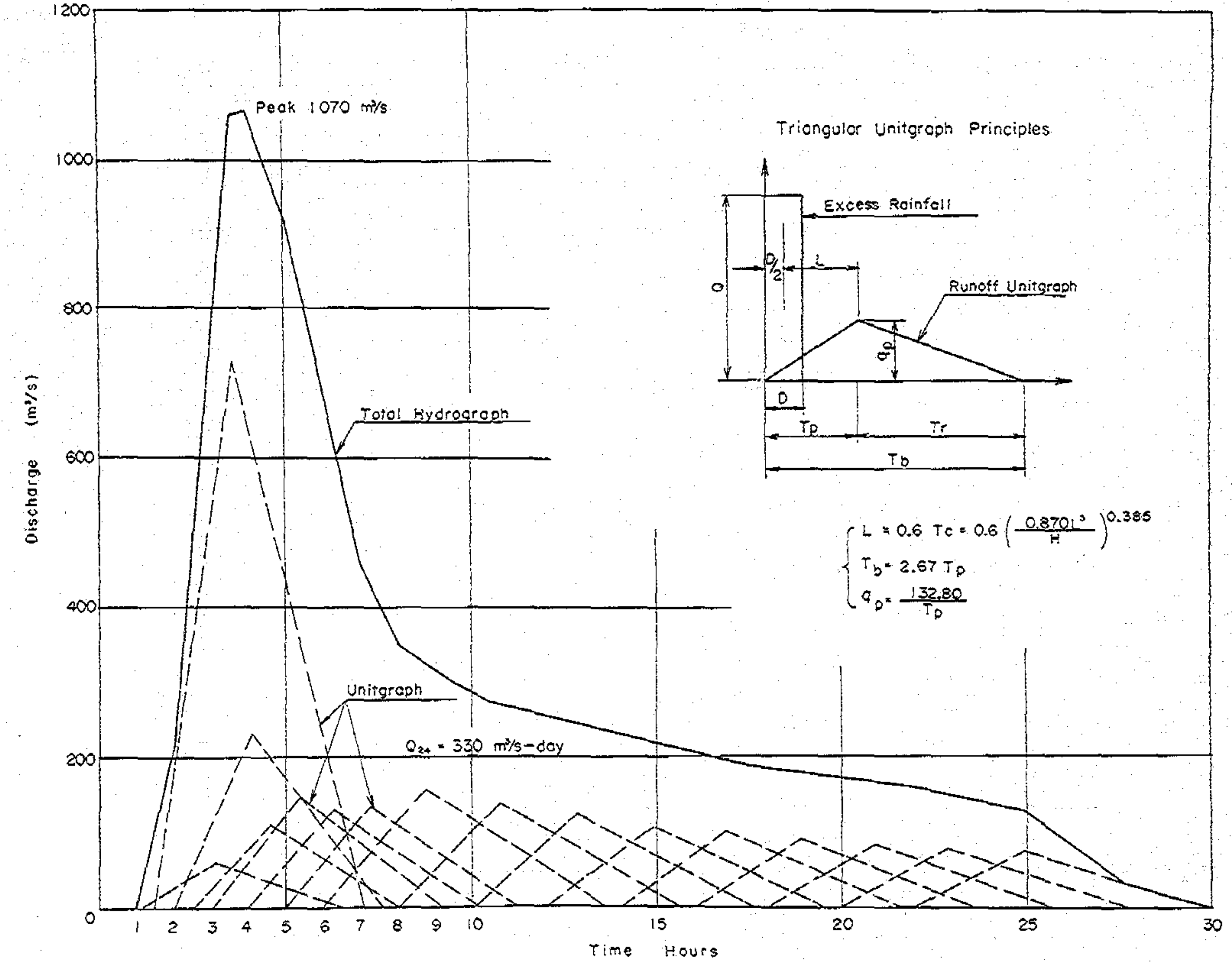


Fig. 6-13 Probable Daily Runoff (Llaucano Derivación)

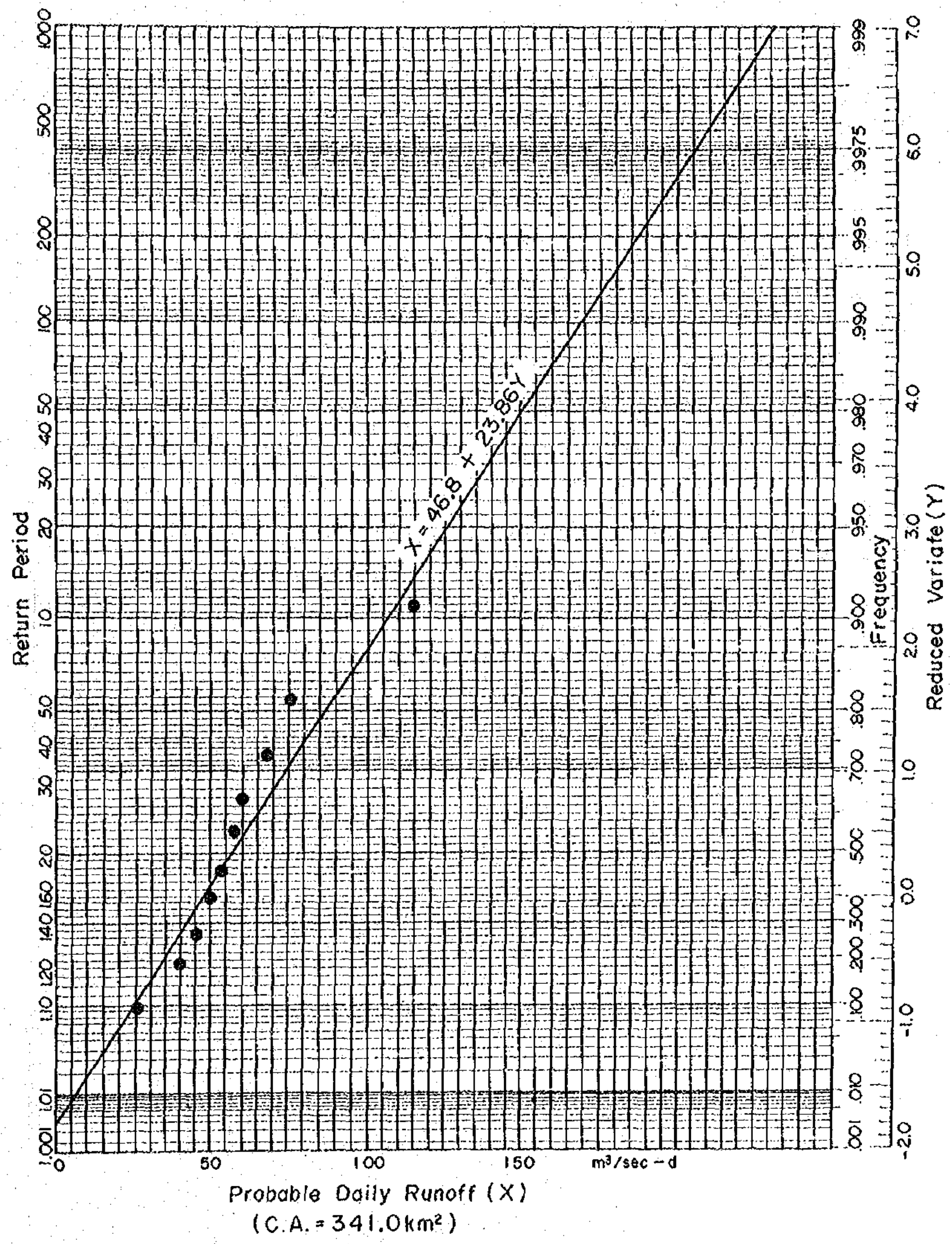


Fig. 6-14 Observed Evaporation Data and " Rohwer Equation Lines"

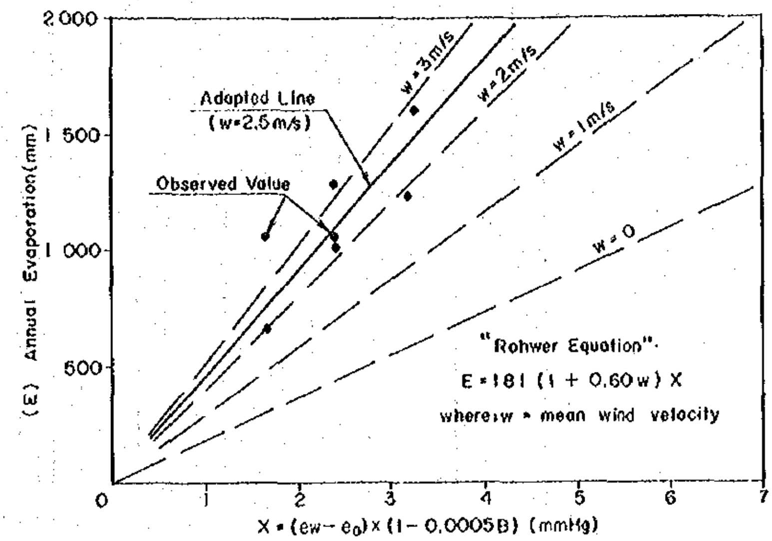


Fig. 6-15 Measured Rates of Reservoir Sedimentation of the Western United States

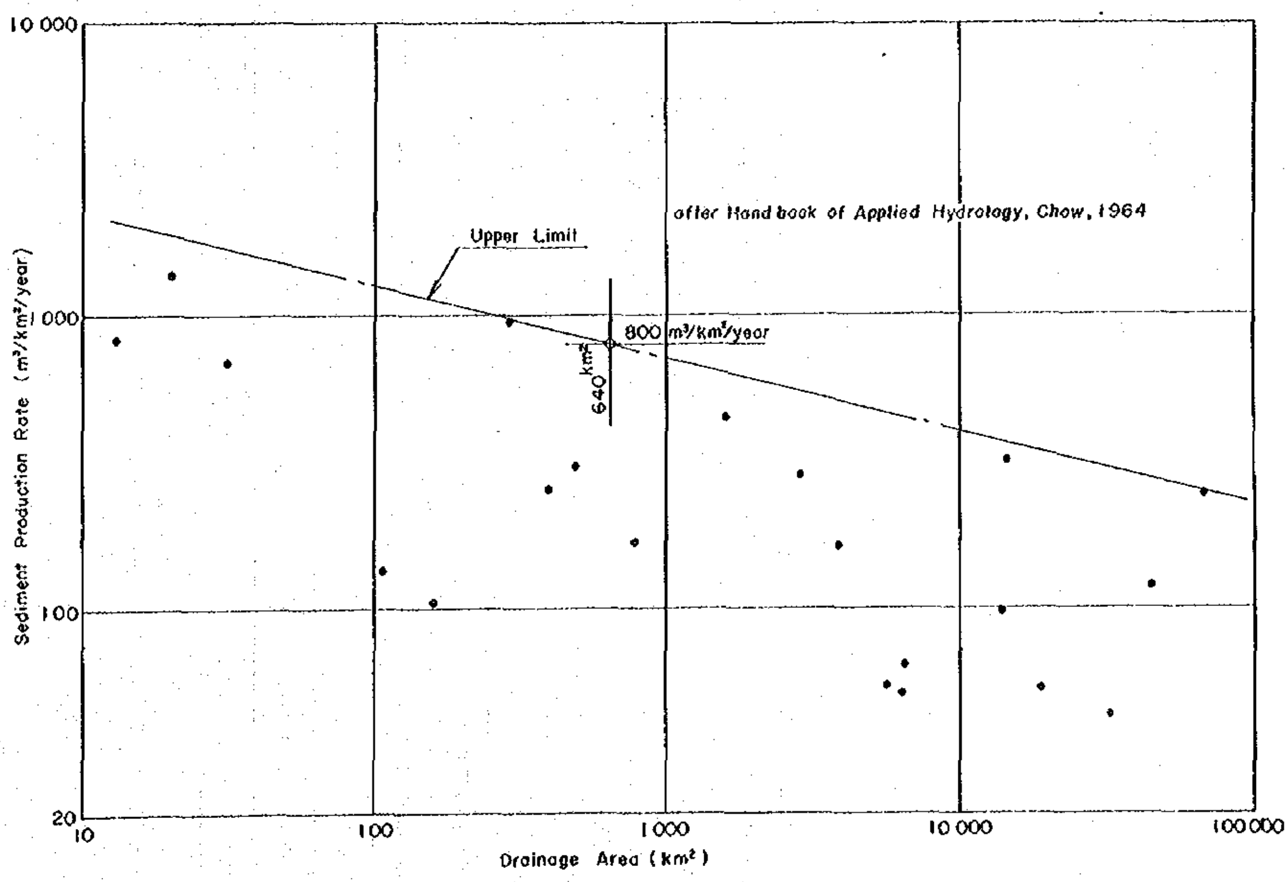


Table 6-1 (b) Rainfall Observatory Station and Existing Data
(Monthly Record)

No.	NAME OF STATION	RIVER	LOCATION Lat (s)	Long (w)	Altitude(m)	CATCHMENT AREA (km²)	'61	'62	'63	'64	'65	'66	'67	'68	'69	'70	'71	'72	'73
1	Llaucano Derivación	Llaucano	6°45'	78°32'	2 570	341.0			Feb.										Sep.
2	Corellama	〃	6°42'	78°31'	2380	597.1		Sep.											Sep.
3	Llaucano Shugar	〃	6°38'	78°28'	2 320	897.6			Feb.					Jan.					
4	Rio Pomagon	〃	6°44'	78°31'	2 520	157.4			Feb.				Dec.						
5	Rio Cuñacales	〃	6°42'	78°31'	2500	26.6			Feb.			Nov.							
6	Rio Maygasbamba Puente	〃	6°40'	78°32'	2550	119.8								Jan.					Oct.
7	Qda. Shugar	〃	6°38'	78°28'	2780	52.0								Jan.		(Dec)			Oct.
8	Qda. Chonta	〃	6°35'	78°25'	2900	—			Feb.				Dec.						
9	Jadibamba	Yangas	6°52'	78°24'	3550	30.8						Sep.	Jun.						Oct.

Table 6-1 (a) Runoff Gaging Station and Existing Data

No.	NAME OF STATION	RIVER	LOCATION Lat.(s)	Long(w)	Altitude(m)	'58	'59	'60	'61	'62	'63	'64	'65	'66	'67	'68	'69	'70	'71	'72	'73
11	Celendín	Yangas	6° 52'	78°09'	2 620							Apr.(Jun)									Aug.
12	Hda. Negritos	Llaucano	6° 54'	78°33'	3 500				Jul.	(May)			May Nov.								(Aug) Oct.
13	La Llica	〃	6°48'	78°31'	2 800						Jan.								(Oct.)		Oct.
14	Hualgayoc	〃	6° 45'	78°37'	3 510				Jul. (Dec)				(Sep)	(Jul. Aug.)				(May)			Sep.
15	Hda. Llaucan	〃	6° 45'	78°32'	2 650						Oct.										Nov.
16	Bambamarca	〃	6°40'	78°32'	2 500				Jul.	(Oct)	(Oct)		(Aug)	(Aug. Sep.)						(Apr.)	(Aug) Oct.
17	San Juan de Lacamaca	〃	6°38'	78°32'	2 700				Jul.										(Oct)	(Oct)	May
18	Qda. Shugar	〃	6° 42'	78°26'	3 030				Jul.												Aug.
22	Cajamarca	Crisnejas	7° 08'	78°28'	2 621	Jun.															Nov.
23	Matara	〃	7° 15'	78°16'	2 624						Sep.		(Aug)		(Jun) (Aug)	(Jun. Jul.)					Jan.
24	San Marcos	〃	7° 20'	78°11'	2254							Jan.	(Jun. Jul.)								Oct. (Sep.)
25	Cachachi	〃	7°27'	78°16'	2400					Jan.	Apr.	Jun. Jan.	Aug.		(Feb. Mar)			(Dec.)			Jan. (Sep)(Dec)
26	Hda. Jocos	〃	7° 31'	78°00'	2 630							Jul.									Aug. (Jun)
27	Cajabamba	〃	7° 37'	78°03'	2 787						Oct.		Dec. Jan.								Oct. (Sep.)
28	Huamachuco	〃	7° 49'	78°03'	3260					Dec. May	Oct.	(May)								May	
29	Brillantana	〃	7° 01'	78° 19'	3904.5								Aug.								Jun.
30	Campo Michiquillay	〃	7° 02'	78°20'	3 430						Aug.										Jun.

Table 6-2 Mean Monthly Precipitation in Yangas River
Estimated by Thiessen Method

Unit; mm

Month	Celendín		Brillantana		Hda. Negritos		La Llica		Yangas River
	1	1'	2	2'	3	3'	4	4'	1'+2'+2'+4'
Jan.	85.7	26.0	134.8	52.8	105.1	5.5	80.2	20.3	104.6
Feb.	88.5	26.8	136.9	53.6	105.6	5.6	86.7	21.9	107.9
Mar.	130.7	39.6	170.5	66.7	160.2	8.4	124.1	31.4	146.1
Apr.	117.9	35.7	130.9	51.2	130.3	6.9	100.9	25.5	119.3
May	33.6	10.2	54.6	21.4	68.4	3.6	43.3	11.0	46.2
Jun.	19.8	6.0	38.9	15.2	34.3	1.8	26.1	6.6	29.6
Jul.	10.0	3.0	21.2	8.3	22.9	1.2	20.8	5.3	17.8
Aug.	16.7	5.1	29.1	11.4	17.2	0.9	24.1	6.1	23.5
Sep.	52.9	16.0	66.2	25.9	53.1	2.8	63.8	16.1	60.8
Oct.	108.1	32.7	158.3	62.0	143.4	7.5	136.4	34.5	136.7
Nov.	148.2	44.9	125.4	49.1	101.8	5.4	113.0	28.6	128.0
Dec.	102.9	31.2	112.6	44.1	81.7	4.3	80.0	20.2	99.8
Total	915.0	277.2	1179.4	461.6	1024.0	53.9	899.4	227.6	1020.3
Coefficient	0.3029		0.3914		0.0526		0.2531		1.0000

notice; 1 , 2 , 3 , 4 are the data observed at respective station.
1', 2', 3', 4' are the calculated area precipitation.

Table 6-3 Mean Monthly Streamflow at Yangas Dam Site Estimated by Thiessen Method

Month	Llaucano Derivación G.S. (Llaucano River)			Yangas Dam Site	
	Area Precipitation (R)	Runoff (Q)	Runoff Coefficient (d)	Area Precipitation (R')	Runoff (Q')
	mm	cu. m/sec		mm	cu. m/sec
Jan.	101.5	6.83	0.529	104.6	13.15
Feb.	105.8	8.14	0.546	107.9	15.51
Mar.	153.0	12.85	0.660	146.1	22.93
Apr.	124.4	12.44	0.760	119.3	22.29
May	59.9	5.01	0.657	46.2	7.22
Jun.	33.1	2.34	0.537	29.6	3.91
Jul.	24.0	1.56	0.511	17.8	2.16
Aug.	22.3	1.09	0.384	23.5	2.15
Sep.	60.5	1.58	0.199	60.8	2.97
Oct.	141.4	4.10	0.228	136.7	7.41
Nov.	109.7	7.37	0.511	128.0	16.07
Dec.	88.3	3.22	0.286	99.8	6.80
Total	1,023.6	(5.52)	(0.499)	1,020.3	(10.15)

$$\begin{cases} \alpha = \dfrac{86.4}{A} \cdot \dfrac{Q}{R} \cdot D \\ Q' = \alpha \cdot \dfrac{A'}{86.4} \cdot \dfrac{R'}{D} \end{cases}$$

A = 341.0 sq. km　　D; days of month

A' = 637.1 sq. km

Table 6-4 *Mean Monthly Streamflow at Yangas Dam Site Estimated by Isohyetal Maps*

Month	Llaucano Derivación G.S. (Llaucano River)			Yangas Dam Site	
	Area Precipitation (R)	Runoff (Q)	Runoff Coefficient (d)	Area Precipitation (R')	Runoff (Q')
	mm	cu. m/sec		mm	cu. m/sec
Jan.	106.0	6.83	0.506	96.5	11.61
Feb.	106.8	8.14	0.541	100.6	14.32
Mar.	154.0	12.85	0.656	137.8	21.49
Apr.	128.5	12.44	0.736	120.4	21.77
May	59.6	5.01	0.660	43.7	6.86
Jun.	34.1	2.34	0.522	24.4	3.12
Jul.	26.0	1.56	0.472	15.8	1.77
Aug.	25.2	1.09	0.356	19.5	1.65
Sep.	62.8	1.58	0.191	59.3	2.78
Oct.	138.8	4.10	0.232	122.0	6.73
Nov.	113.2	7.37	0.495	126.0	15.40
Dec.	96.3	3.22	0.263	96.6	6.03
Total	1,051.3	(5.54)	(0.469)	963.2	(9.46)

Table 6-5 Elevation Bands, Typical Stations and Data for Elevation Method

Band	Elevation	Typical Elevation	Typical Rainfall G. S. (EL.)	(Wet Season)	(Dry Season)	Yangas River Drainage Area (qu. km)	Yangas River Ratio (ai)	Llaucano Derivacíon G.S. Drainage Area (qu. km)	Llaucano Derivacíon G.S. Ratio (ai)
	m	m							
I	- 2750	2500	Bambamarca (2500)	1.157	0.896	101.9	0.160	6.8	0.020
II	2750 - 3250	3000	Oda. Shugar (3030)	0.926	0.936	127.4	0.200	49.4	0.145
III	3250 - 3750	3500	Hda. Negritos (3500)	1.072	1.209	244.0	0.383	153.1	0.449
IV	3750 -	3900	Brillantana (3905)	1.039	1.474	163.8	0.257	131.6	0.386
Total				(Oct. - Apr.)	(May - Sep.)	637.1	1.000	341.0	1.000

$$R_A = \sum_{i=I}^{IV} ri \times \alpha i \times ai$$

R_A : Area Rainfall

r_i : Point Rainfall at gaging station i

Table 6-6 Mean Monthly Streamflow at Yangas Dam Site
Estimated by Elevation Method

Month	Llaucano Derivación G.S. (Llaucano River)			Yangas Dam Site	
	Area Precipitation (R)	Runoff (Q)	Runoff Coefficient (d)	Area Precipitation (R')	Runoff (Q')
	mm	cu. m/sec	mm	mm	cu. m/sec
Jan.	119.5	6.83	0.449	109.6	11.71
Feb.	122.3	8.14	0.472	114.3	14.21
Mar.	169.3	12.85	0.596	157.9	22.39
Apr.	134.3	12.44	0.704	134.6	23.29
May	76.6	5.01	0.514	67.7	8.28
Jun.	45.6	2.34	0.390	39.4	3.78
Jul.	29.0	1.56	0.423	27.1	2.73
Aug.	29.0	1.09	0.295	25.2	1.77
Sep.	75.5	1.58	0.159	68.0	2.66
Oct.	153.1	4.10	0.210	144.8	7.23
Nov.	117.6	7.37	0.476	112.4	13.15
Dec.	99.9	3.22	0.253	94.3	5.67
Total	1,171.7	(5.52)	(0.436)	1,095.3	(9.69)

$$\begin{cases} \alpha = \dfrac{86.4}{A} \cdot \dfrac{Q}{R} \cdot D \\ Q' = \alpha \cdot \dfrac{A'}{86.4} \cdot \dfrac{R'}{D} \end{cases} \qquad \therefore \; Q' = \frac{A'}{A} \cdot \frac{R'}{R} \cdot Q$$

A = 341.0 sq.km D : days of month

A'= 637.1 sq.km

Table 6-7 Observed Evapolation Data

Unit; mm

	Santa Cruz (1940m)	San Marcos (2254)	Contumaza (2300)	Chota (2410)	Celendín (2620)	Hda Jocos (2630)	Cajamarca (2750)	Average
Jan.	73.9	118.4	70.3	80.6	37.4	88.0	99.1	82.4
Feb.	72.9	111.7	53.3	75.7	52.3	77.0	67.7	69.3
Mar.	62.1	78.0	35.4	60.5	39.0	68.0	98.1	64.8
Apr.	91.2	106.7	53.8	62.7	44.6	78.8	94.2	76.1
May	95.3	129.2	84.4	73.8	40.8	103.0	97.1	90.7
Jun.	97.4	145.9	100.4	85.4	51.4	116.2	104.1	100.7
Jul.	95.2	162.3	120.4	115.5	73.3	146.4	110.8	117.7
Aug.	96.4	173.5	125.2	121.7	77.7	163.6	116.4	124.7
Sep.	92.2	165.0	101.7	114.2	54.2	147.7	106.7	112.5
Oct.	96.2	138.2	95.8	103.9	61.4	105.1	110.2	102.2
Nov.	88.0	134.9	93.4	83.2	71.3	96.8	104.2	93.1
Dec.	93.2	129.8	78.4	86.6	59.0	93.8	120.0	94.4
Total	1,054.0	1,593.6	1,012.5	1,063.8	662.4	1,284.4	1,228.6	1,128.6

after Estudio de Las Características Climatológicas
,SENAMHI, 1974

Table 6-8 Analysis of Evaporation

Observatory Station 1	Altitude 2 (m)	Average Temp. 3 (°C)	Relative Humidity 4 (%)	Annual Evaporation (E) 5 (mm)	e_w (mm Hg) 6	ea (mm Hg) 7	B (mm Hg) 8	X 9
Cascas	1,330	19.6	73.4	-	-	-	-	-
Santa Cruz	1,940	17.6	77.2	1,054	15.1	11.7	601	2.38
San Juan	2,224	14.9	85.3	-	-	-	-	-
San Marcos	2,254	17.2	68.4	1,594	14.7	10.1	578	3.27
Contumaza	2,300	14.5	72.8	1,013	12.4	9.0	575	2.42
Chota	2,410	14.7	81.3	1.064	12.5	10.2	566	1.65
Cajamarca	2,750	13.8	62.5	1.230	11.8	7.4	543	3.21
Celendín	2,620	12.9	79.4	662	11.2	8.9	552	1.67
Hda. Jocos	2,630	14.7	73.3	1,284	12.5	9.2	551	2.39
Cajabamba	2,783	13.5	77.7	-	-	-	-	-
Granja Porcon	3,000	9.4	75.0	-	-	-	-	-
Average			75.1	1,129				

Note; 3 , 4 , 5 are observed data

6 is satulated vapor pressure of average temperature (3)

7 equal 6 x 4 ÷ 100

8 is standard atomospheric pressure at altitude 2

9 equal $(e_w - e_a) \times (1-0.0005\ B)$

Relations between E (5) and X (9) are plotted in Fig. 6-14

CHAPTER 7

POWER DEVELOPMENT

FIGURES

TABLES

CHAPTER 7 POWER DEVELOPMENT

7.1 Outline of River Basin

The Yangas River is one of the tributaries of the Marañon River, the source of the Amazonas, and is a rapid current with an average river gradient of 1/15, length of river channel of approximately 50 km from its fountainhead at elevation of 4,000 m in the Andes Mountains to its junction with the Marañon River at approximately 700 m of elevation, and according to a 1/200,000-scale topographic map, its total drainage area is approximately 1,250 sq.km.

The basin may be divided into three sections in accordance with elevation. They are the plateau area of elevation of 4,000 m to 3,400 m (consisting of glacial and karstic topographies, with relatively plentiful precipitation, gentle slopes and abundant vegetation so that there is high water retentivity), steep slopes between elevation of 3,400 m and 1,500 m (steep-sloped land having some amount of agricultural fields and grasslands or bare rocks, but with poor water retentivity and steep river gradient), and a gently-sloped area from 1,500 m to 700 m (partly including a steeply-sloped area, but mostly with gentle slopes and adjacent to the Amazon tropical jungle area).

The Yangas River Basin, as shown in Fig. 6-1, is adjacent to the Llaucano River Basin on its northwestern side and to the Crisnejas River Basin on its southwestern side. These rivers are all tributaries of the Marañon River, and the Yangas River Basin is not directly in contact with the watershed between the Pacific Ocean and the Atlantic Ocean. Villages and cultivated area are mostly concentrated in the highlands around an elevation of 3,000 m, and can be seen rarely below 1,500 m.

7.2 Selection of Damsite

The damsite which had been proposed in previous studies (Site No. 3 partly mentioned in 5.2) is located at the Junction of the Yangas River and the Grande River, and according to an old 1/200,000-scale map it had been considered that it would be possible to obtain a storage capacity of approximately 50×10^6 cu.m by constructing a dam of height of 30 m. However, as a result of preparation of the new 1/20,000-scale topographic map and of field reconnaissance, it became questionable whether it is a damsite advantageous as said in previous report and the survey area had to be enlarged for selection of the optimum damsite.

In consideration of catchment area, effective head, water storage efficiency, geology, etc., the three sites below are conceivable as indicated in Fig. 7-2.

Damsite No. 1	The Platanal site, the conjunction of the Jadibamba River and the Sendamal River
Damsite No. 2	The Trapiche Viejo site approximately 2.5 km downstream from Damsite No. 1
Damsite No. 3	The Hda. Yangas site approximately 4.5 km downstream of Damsite No. 2 (the site selected in the MOTLIMA Report)

Upstream of the above three sites, the river is bifurcated into two big branches and it is not economical to construct two dams or otherwise the drainage area is greatly reduced, while the downstream, effective head is reduced and the storage efficiency lowered because of narrow river width, and there are no suitable sites for a dam from engineering stand point.

On comparison of the storage efficiency of the above three sites, within the scope of a realistic dam height (below about 120 m), as shown in Fig. 7-2, Damsite

No. 1 has a storage capacity of two to three times compared with the No. 2 and No. 3 damsites at equal dam heights, while the river width at No. 1 dam site is about 1/1.5 - 1/3 of No. 2 and No. 3, and as a result, the storage efficiency (ratio of effective storage capacity to dam volume) of No. 1 is approximately 10 times those of No. 2 and No. 3. With respect to the No. 3 site which had been considered to be the optimum damsite in the previous studies, since there is a difference in elevation of approximately 40 m between the riverbed and the Yangas deposited delta, the effective height of the dam would be reduced by that amount, and the dam height and volume to secure the required storage capacity would become enormous.

The drainage areas for the No. 2 and No. 3 sites are increased according to the extents they are located downstream from Damsite No. 1, but the increases are slight, and when the decreases in head are considered, they are thought not to be of such magnitudes as to reverse the overwhelming differences in storage efficiency.

Regarding the influences on society in the vicinities such as submersion of cultivated land due to construction of a reservoir, these are extremely small and there are almost no differences between the three.

And, as described in Chapter 5, marked water springs are seen at the riverbeds of the No. 2 and No. 3 damsites and it is anticipated that great technical difficulties will be encountered in constructing high dams and storing water at these two sites.

As a result of the above considerations, and based on the viewpoints of the characteristics of the sites concerned such as topography and geology, as well as the economics, Site No. 1 at Plantanal is selected to be the most suitable.

7.3 Study of Development Scale

In selecting the dam height and power house location, it is necessary to find the optimum development scale of which the ratio (B/C) between the annual benefit (B) determined by the accompanying reservior regulation capacities available discharge, effective head, etc., and the annual cost (C) determined by construction cost of structures and operation and maintenance cost would be a maximum. For this purpose considering four dam heights (storage capacities) of A to D (25, 60, 80 and 100 m) and the four powerhouse locations of I to IV at each elevation of 1,350, 1,070, 900 and 800 m, the B/C ratio for a total 16 combinations from A-I to D-IV were studied. The particulars for the respective combinations are as indicated in Table 7-1. Combinations other than this range are inconceivable from the actual topographic condition.

Calculations were made for these combinations according to the criteria indicated below.

(1) Topographic maps ..... 1/20,000-scale topographic maps made available by INIE

(2) River runoff ..... as mentioned in Chapter 6

(3) Basic output and energy production ..... nine-year average for dependable output and power energy (see 7.4)

(4) Alternative thermal power plant ..... as described in Chapter 8

(5) Load factor ..... 60% as mentioned in Chapter 4

(6) Construction cost ..... as described in 7.7 (provided that power transmission facilities are not included)

(7) Interest rate 10%

From the results of calculations, as shown in Table 7-1 and Fig. 7-4, the economics (B/C) will generally be improved the larger the scale (the higher the dam height, the greater the head, and the larger the installed capacity), but when the scale is enlarged above a certain point, the economics are conversely impaired. This is mainly due to topographic factors. That is, even generation scale is increased, the fixed costs such as access roads do not change, and on the other hand, when dam height would be too high the storage efficiency becomes worse, while when the waterway would be too long the head gained by it will not be so much as the riverbed gradient is gentle at the downstream area.

Consequently, as indicated in Table 7-7 and Fig. 7-4, the plan B-III with dam-height of 80 m, effective head of 620 m, and effective output of 50 MW, the benefit-cost ratio (B/C), which were calculated applying the basic data of 8.1. and 8.2., is a maximum of 1.09, and therefore, this is the optimum scheme in case of load factor of 60%.

7.4 Power Generation

For the amount of power generation, the mass curve indicated in Fig. 7-5 is prepared based on the damsite inflow described in Chapter 6, and the energy generated for each month is calculated from the available discharges and effective heads on the basis of optimum operation of reservoir, and the 9-year average was considered to be the energy generation.

The evapotranspiration loss from the reservoir is not considered in the calculations owing to the fact that the annual water loss by it at the damsite of 450 mm (mentioned in 6.5), when applied to the reservoir area of 1.3 sq.km at the medium water level, means an annual water loss of 585,000 cu.m which is only a very small quantity of 0.2% of average annual inflow.

As for the effective head to be used in calculations, taking the intake water level at the medium water level for the available drawdown of the reservoir, a head of 612 m was used.

As a result, the power generation was calculated to be 346 GWH (9-year average) as indicated in Table 7-2.

That is, on the precondition that there will be a thermal power plants within the system, it was assumed that in the rainy season the Yangas station would be operated at a high load factor (60% or higher) within the limits of maximum available discharge reducing the operation hours of abovementionad thermal power plants. Therefore, the annual average plant factor becomes approximately 80%.

Meanwhile, the energy loss in transmission line from the power station to the substation at the Michiquillay Mines calculated from the electrical resistance of the transmission line of total length of 55 km and abovementioned plant factor is annually 25 GWH. The energy used for station service and losses due to transformers are assumed to be approximately 4 GWH. Consequently, the total energy loss will be 29 GWH (approximately 8% of power generation), and deducting this value from the energy production of 346 GWH, the balance of 317 GWH will be the salable energy at Michiquillay Substation.

7.5 Large-Scale Development

As stated in Chapter 4, it will be appropriate for the load factor of around 60% for the power station to be constructed in the 1980s for power supply to the area concerned, and in 7.3, power generating facilities were considered which could secure a load factor of 60% even in the dry season. However, in the 1990s and thereafter, with increases in both demand and peak demand, it is anticipated that the incorporation of a power station with a supply capability at load factor of around 30% even in the dry season would be economical from the viewpoint of the system as a whole.

Therefore, as one proposal, a peak power station of load factor of about

30% on the Yangas River was studied.

The optimum development scale in case of load factor of 30% can be obtained by selecting a plan of which the benefit-cost ratio (B/C) will be the maximum as in 7.3. By changing the load factor from 60% to 30%, the maximum available discharge of each plan will be doubled and the benefit by kilowatt doubled, but when the reservoir capacity is constant, the benefit by kilowatt-hour would not be increased so much. Meanwhile, the construction cost will be increased by the amounts of increase in the inner diameter of the tunnel and in generating capacity. As the result of this calculations, the highest B/C was for a modification of plan B-III in 7.3. The particulars are as given in Table 7-3, and maximum available discharge will be 19 cu.m/sec and maximum output 100 MW. The benefit-cost ratio (B/C) is 1.33, and compared with B/C of 1.09 for the plan of load factor of 60% and output of 50 MW, the economics are found to be slightly improved.

However, in case of load factor of 30% the peak duration time will be shortened, and careful consideration of the trend in the demand and supply situation must be paid in pushing ahead with such a plan.

7.6 Preliminary Design

Preliminary design of power generation facilities was performed as indicated below in accordance with the optimum plan described in 7.3. The preliminary design drawing is as shown in Fig. 7-6.

(1) With regard to the type of the dam, since the transverse profile of the river at the damsite is a prominent V-shaped valley, an arch dam would be conceivable, but because the geologic conditions are not necessarily favorable, and in consideration of disposal of sediment flowing into the reservoir after start of operation, a concrete gravity dam was selected for the purpose of this

prefeasibility study. The height of the dam will be 82 m and the volume of the dam 240,000 cu.m.

In consideration of the geologic conditions, it was planned that grouting to prevent leakage of the storage should be carried out very carefully, but depending on the results of future detailed geologic surveys, there might be a possibility that an even more large-scale foundation treatment would be necessary. The dam will be provided with a spillway capable of safely discharging the design flood of 2,300 cu.m/sec and 3 gates will be installed.

The dead water storage below the low water level of 1,528 m of the reservoir is 5,000,000 cu.m, which would be filled by sedimentation in approximately 8 years. Therefore, after the sedimentation plane reaches the low water level, effective storage capacity must be secured by operating suitable facilities of sand flushing.

(2) The inner diameter and route of the headrace tunnel were selected for most economical passage of the maximum available discharge of 9.5 cu.m/sec. The headrace tunnel is a circular pressure tunnel with an inner diameter of 2.30 m and total length of 15 km, having work adits at three places along the way. Regarding to draining of sediment flowing into the headrace tunnel, a detailed examination must be made at the stage of the feasibility study.

It will be possible from the standpoint of topography and also economics that a vertical-shaft surge tank is provided at the connection point between the headrace tunnel and the penstock.

The penstock is to be a surface-type steel pipe and high-tensile steel capable of withstanding maximum internal pressure of 83 kg/sq.cm is desirable. Since the penstock would be very long as 1,460 m, it is to be bifurcated at close to the powerhouse and the two branches connected to the respective turbines.

(3) With regard to the powerhouse, a surface type was selected, since from the standpoints of topography and geology, it is possible to select a location where there would be no danger by land slide at the time of earthquakes, while improvement in economy by making the powerhouse an underground type cannot be expected.

For turbines and generators, considering the fact that the scale of the electric power system in the northern part of Peru is comparatively small, it was decided that two units of unit capacity of 25 MW should be installed. Since the effective head is high, the turbines to be used are dual-jet Peltons.

It will be possible to secure an adequate area for a switchyard and construction yard in the vicinity of Bombón where the powerhouse is to be located.

(4) Concerning to access roads, the existing road of approximately 15 km branching from the highway between Cajamarca and Celendín and reaching to Huasmín is to be improved, while for the approximately 50 km from Huasmín to the powerhouse via the damsite, a new road is to be constructed.

(5) In order to connect Yangas Power Station with the electric power system for the northern part of Peru, a transmission line is to be newly constructed from Yangas Power Station to a substation planned to be provided near the Michiquillay Mines.

This transmission line is of 66-kV single circuit considering the projected maximum output of 50 MW of Power Station and in keeping with the voltage used in the existing power system. The conductor of 330-sq. mm ACSR will be economical from the standpoint of corona and power loss. The transmission capacity will be 70 MW with the length of the line being approximately 55 km long.

The route of the transmission line is shown in Fig. 7-1, and the drawing of a standard steel tower for this transmission line is shown in Fig. 7-7.

The route of the transmission line is in a mountainous area from elevation of 900 m to 3,750 m, and in design of the route, considerations were given to lowering of insulation capacity of air and to wind pressure.

7.7 Construction Cost

The construction cost was estimated based on the following criteria in accordance with the preliminary design of the preceding section.

(1) The date of estimation of construction cost is taken to be in July 1974, and variations in the foreign currency exchange rates thereafter are not taken in account.

(2) It is deemed that the materials procurable in Peru such as cement, aggregates, reinforcing steel and fuel, and the greater part of labor costs are all be paid for with domestic currency, while other materials, construction machinery and electrical equipment which can be imported are to be budgeted for payments with foreign currency.

(3) The construction cost is expressed entirely in terms of domestic Peruvian currency (S/o). As the exchange rate with international currency at this time, the official rate of $1.00 = S/o 43.38 in July 1974 is used.

(4) The construction cost was estimated based on recent performances in construction of hydroelectric power stations, other construction costs, and materials costs, in Peru, neighboring South American countries and in Japan, and the estimates were made taking into account the regional conditions.

(5) The construction cost includes the costs of civil works and electrical equipment installation works, engineering and administration costs required for supervision of work, contingencies, and interest during construction.

(6) Taking into consideration the accuracy of the topographic map (scale, 1/20,000) which served as the basis for calculation of work quantities, contingency funds in the construction cost are 15% and 10% for civil works and electrical equipment installation works respectively.

(7) The construction period, as indicated in 7.8, is considered to be 5 years from start of construction of access roads to start of power generation, and the interest during construction is included assuming an interest rate of 10% per annum during the period.

The construction cost calculated based on the above criteria, as shown in Table 7-4, is 2,988 million S/o including construction cost of 2,873 million S/o of power generation facilities including a switchyard and 115 million S/o of construction cost of the transmission line. Of this amount, 998 million S/o would be the domestic currency requirement and 1,990 million S/o (≑ $43.6 million) the foreign exchange requirement.

7.8 Construction Schedule

The construction schedule for the Yangas Project, as shown in Fig. 7-8, must start with basic surveys such as geologic investigations, topographic surveying and hydrologic data collection, followed by preparatory work such as a feasibility study, raising funds, detailed designing and contracting of work, after which construction would be commenced, and it is estimated that approximately 8 years will be required as the entire period up to commissioning of the power station. Of this period, the construction period from start of work of access roads to start of power generation would be 5 years as stated above. It is considered that the schedule as a whole will be governed by the construction schedule for the long headrace tunnel. It might be possible to study to shorten the abovementioned construction period at the stage of feasibility study, raising funds, detailed design and award of contract for construction, but since the damsite geology consists of limestone, it is conceivable an unexpectedly long period of time may be required for geological studies of the dam site.

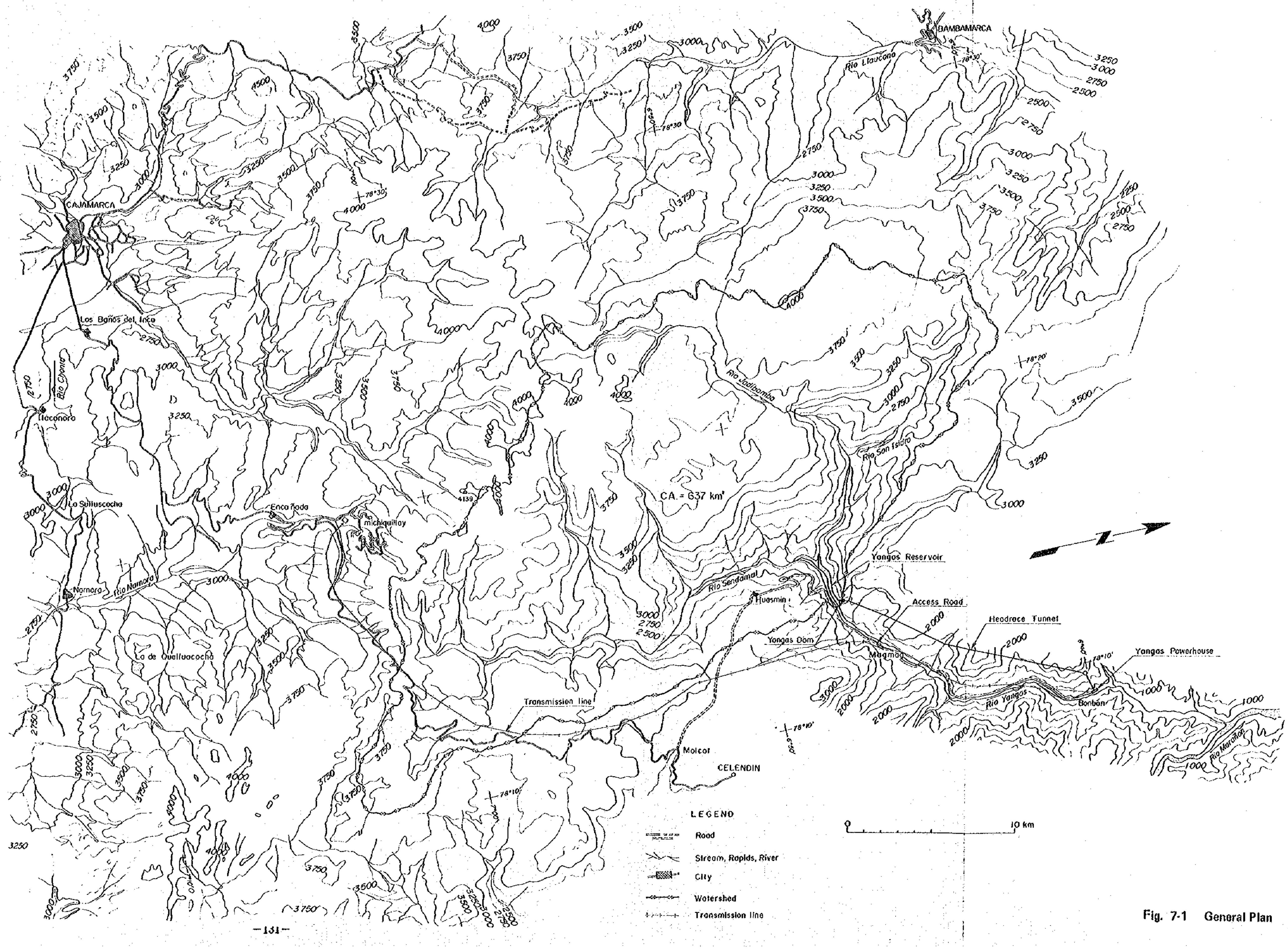


Fig. 7-1 General Plan

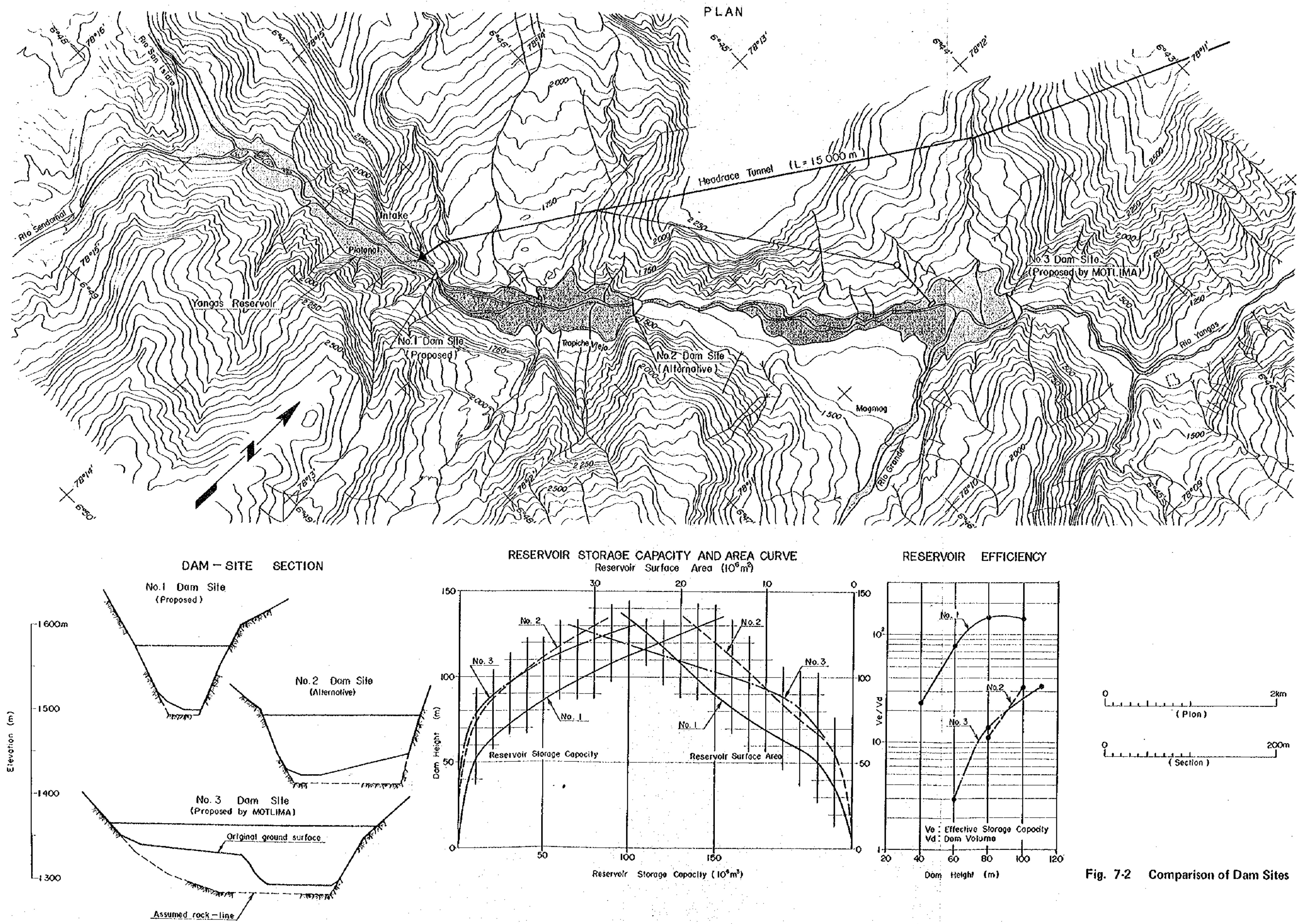


Fig. 7-2 Comparison of Dam Sites

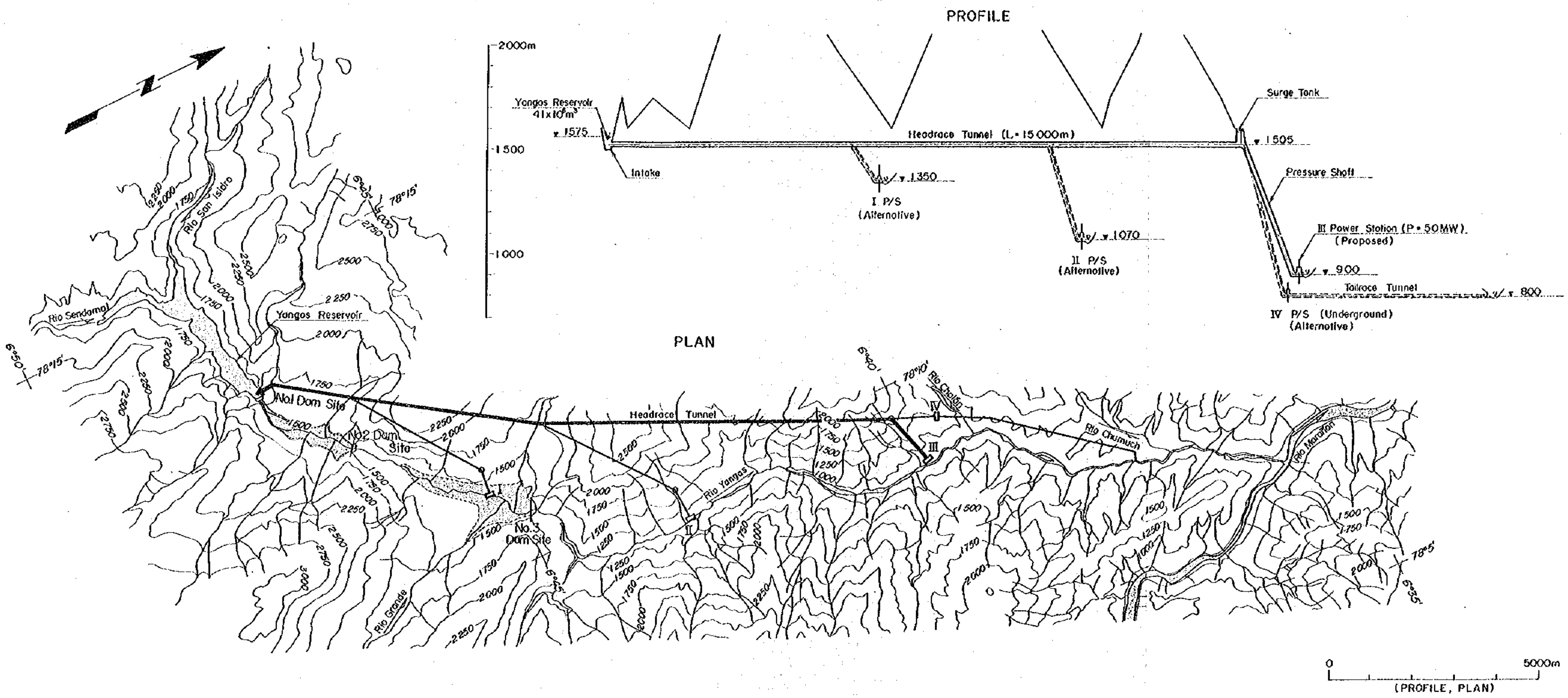
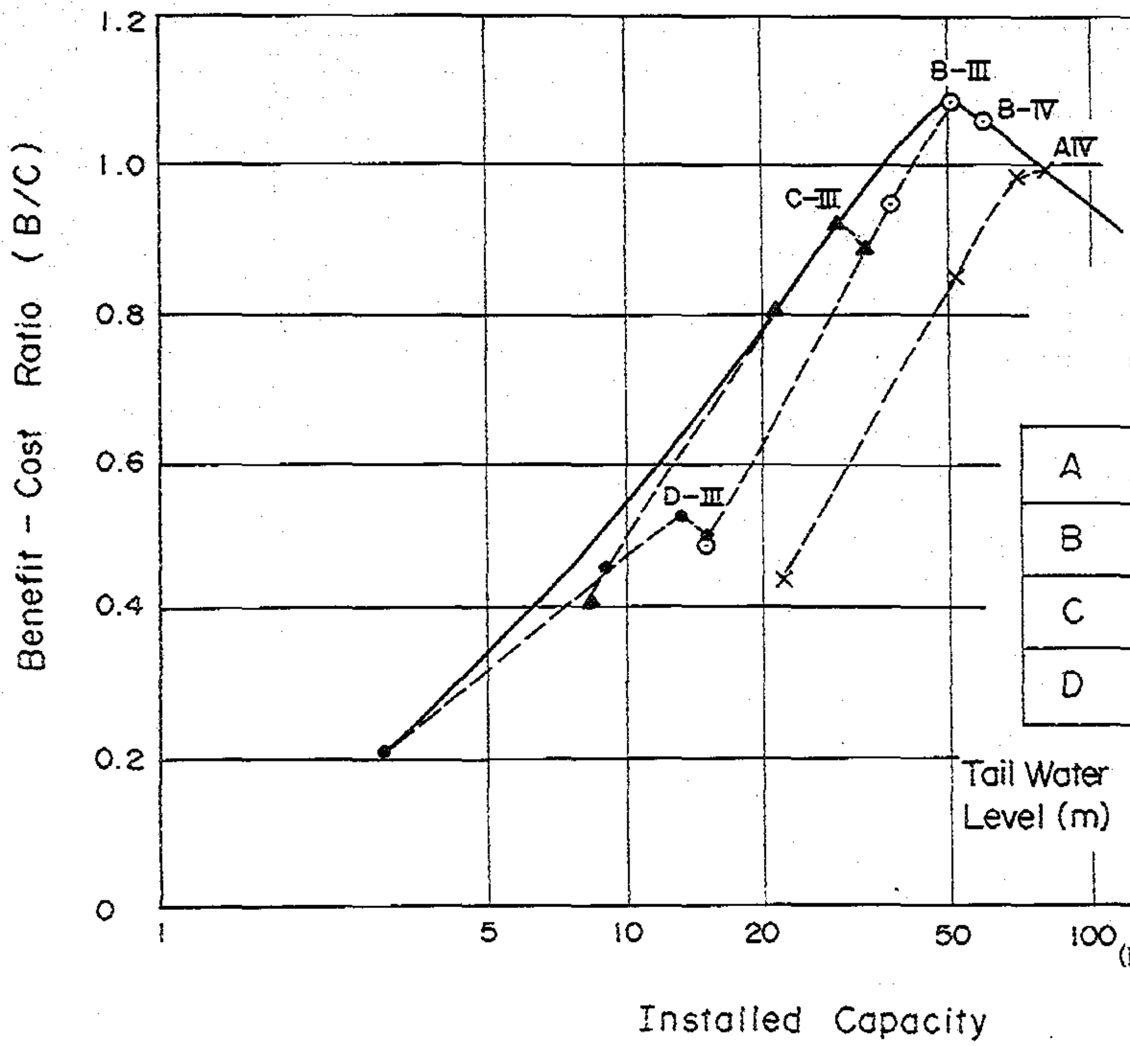


Fig. 7-3 Plan of Development

Fig. 7-4 Installed Capacity and Benefit Cost Ratio



Installed Capacity (MW)

	I	II	III	IV	Height of Dam (m)
A	22	51	69	80	100
B	15	37	50	58	80
C	8.4	21	28.5	33	60
D	3	9	13	15	25

	I	II	III	IV
Tail Water Level (m)	1 350	1 070	900	800

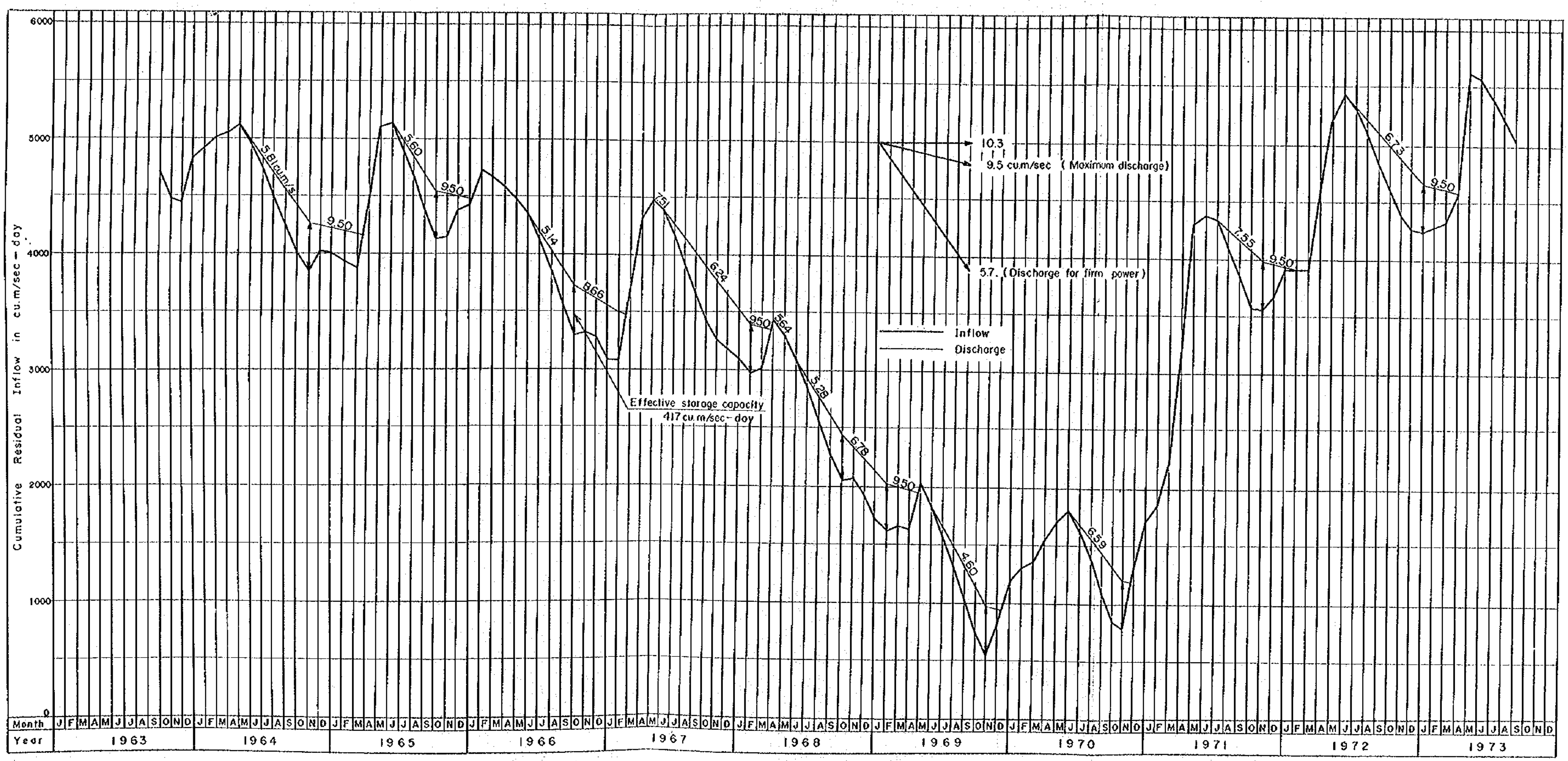


Fig. 7-5 Mass Curve

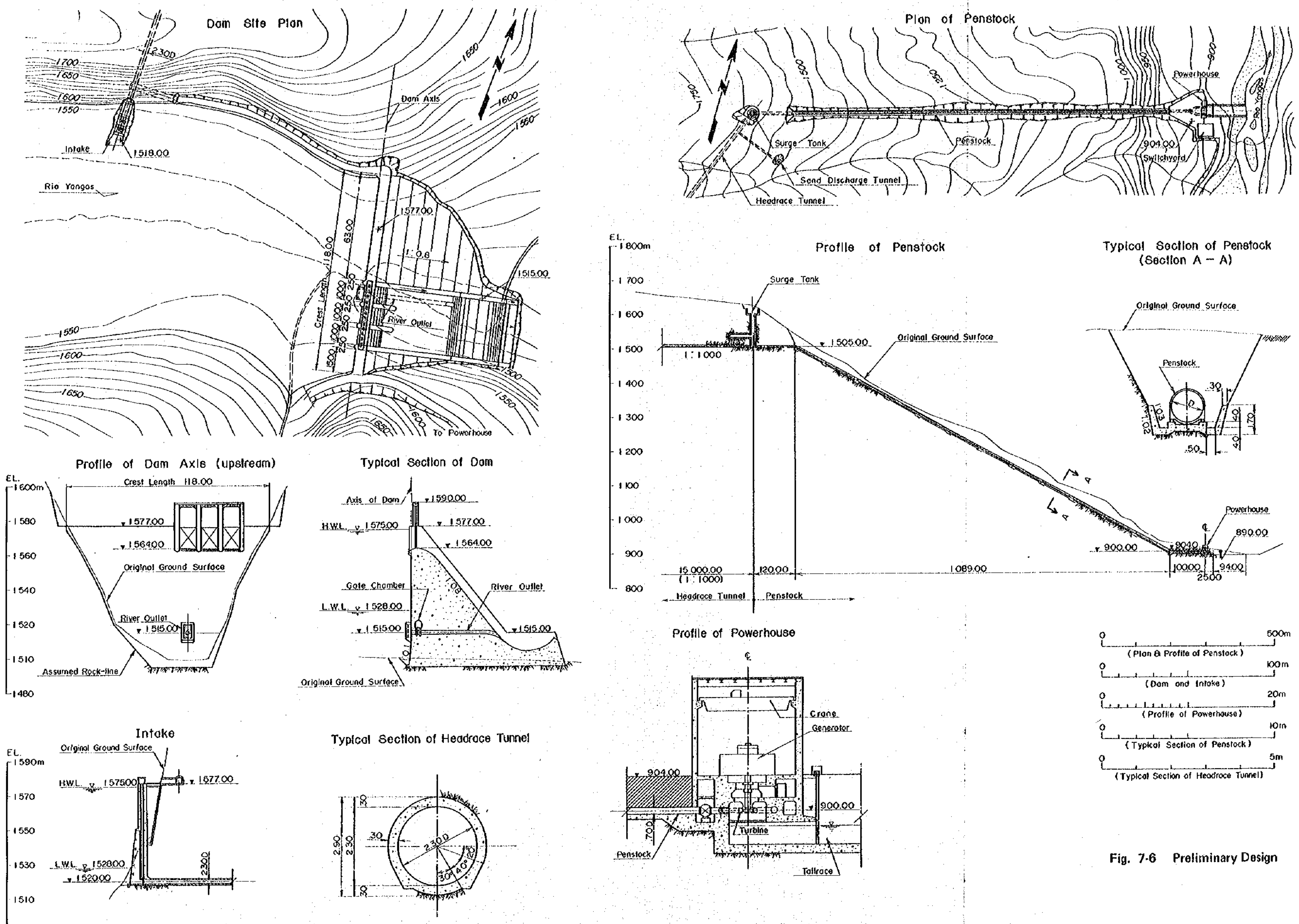


Fig. 7-6 Preliminary Design

Fig. 7-7 Transmission Steel Tower

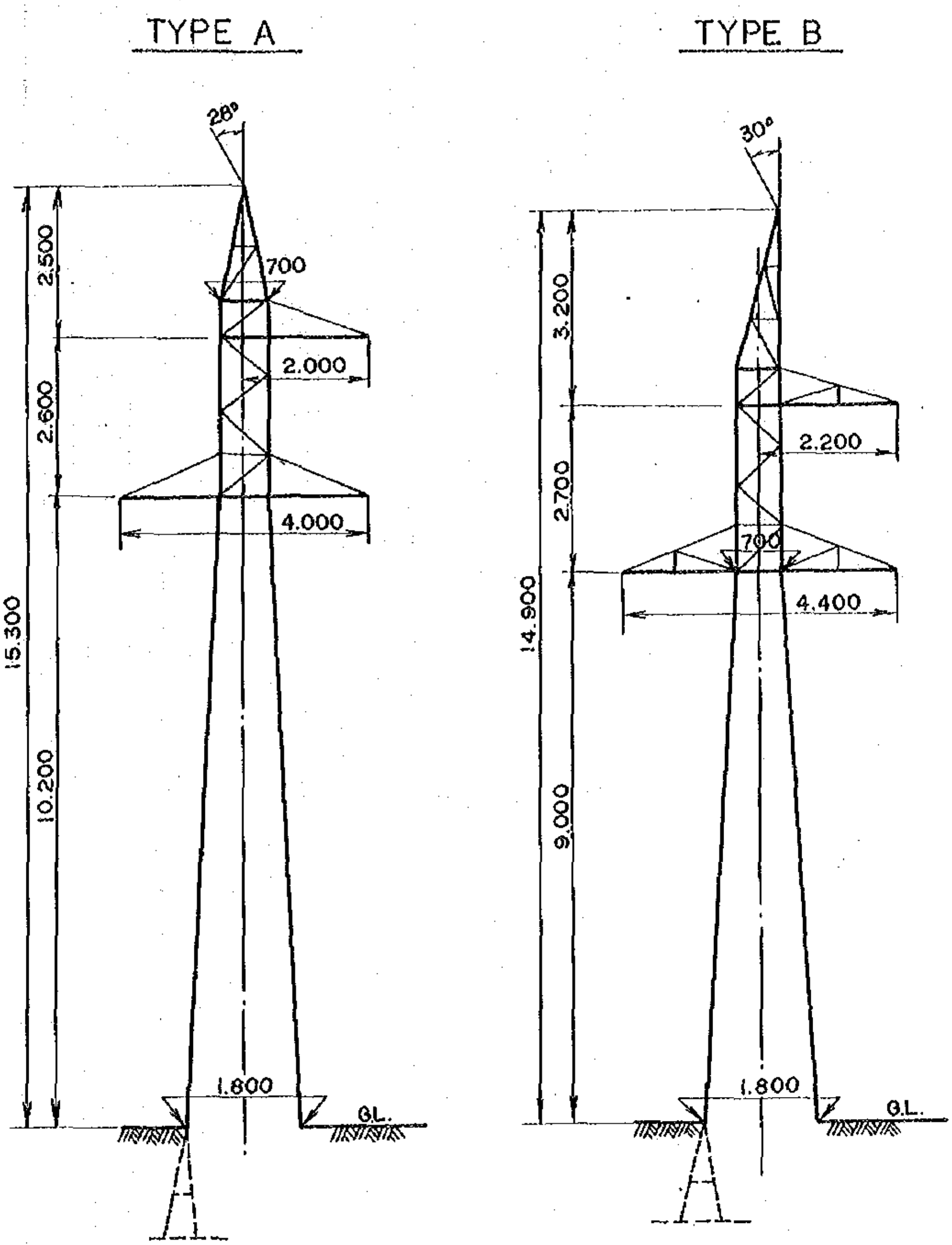


Fig. 7-8 Construction Schedule

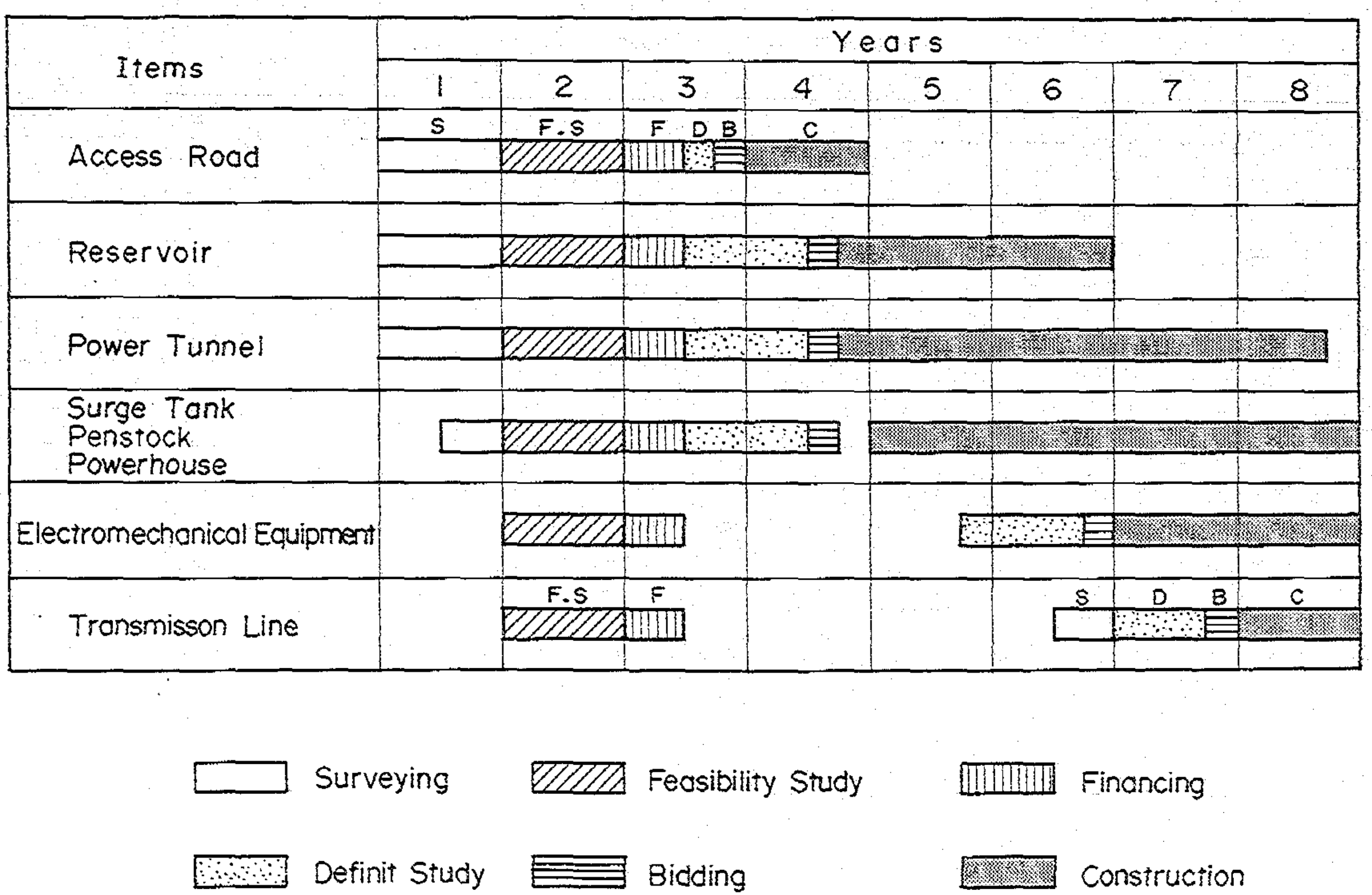


Table 7-1 Comparison of Plan of Development

Item	Unit	A-I	A-II	A-III	A-IV	B-I	B-II	B-III	B-IV	C-I	C-II	C-III	C-IV	D-I	D-II	D-III	D-IV
Height of Dam	m	100				80				60				25			
H. W. L.	"	1,595				1,575				1,555				1,520			
Available drawdown	"	67				47				27				12			
Effective Storage Capacity	10^6cu.m	71				36				10				1.7			
Inner Diameter of Conduit	m	2.50				2.30				1.90				1.90			
Maximum Discharge	cu.m/se	13.0				9.5				5.5				2.5			
Annual Discharge	10^6cu.m	317				238				147				72			
Tailwater Level	m	1,350	1,070	900	800	1,350	1,070	900	800	1,350	1,070	900	800	1,350	1,070	900	800
Total Length of Conduit	"	6,400	11,170	16,460	21,260	6,400	11,170	16,460	21,260	6,400	11,170	16,460	21,260	6,400	11,170	16,460	21,260
Net Head	"	200	465	625	725	185	460	620	720	180	450	610	710	150	425	590	690
Installed Capacity	MW	22	51	69	80	15	37	50	58	8.4	21	28.5	33	3	9	13	15
Annual Energy Production	GWH	142	340	460	534	99	251	346	400	61	155	211	246	27	78	109	127
Construction Cost	Mio s/.	3,004	3,667	4,246	4,891	1,844	2,392	2,873	3,411	1,247	1,678	2,003	2,423	1,053	1,421	1,736	2,147
Annual Cost (C)	"	333	407	471	543	205	266	319	379	138	186	226	269	117	158	193	238
Annual Benefit (B)	"	146	355	466	542	101	253	347	401	60	152	206	240	25	73	103	119
B/C	-	0.44	0.87	0.99	1.00	0.49	0.95	1.09	1.06	0.43	0.82	0.91	0.89	0.21	0.46	0.53	0.50

Table 7-2 Power Generation

Unit: GWH

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1964	36.7	34.4	36.7	35.5	22.4	21.7	22.4	22.4	21.7	22.4	35.5	36.7	348.5
1965	36.7	33.1	36.7	35.5	36.7	20.9	21.6	21.6	20.9	36.7	35.5	36.7	372.6
1966	36.7	30.7	26.4	26.5	24.9	19.3	19.9	19.9	19.3	33.5	32.4	33.5	323.0
1967	33.5	33.1	36.7	35.5	29.1	23.4	24.1	24.1	23.4	24.1	23.4	24.1	334.5
1968	24.1	34.4	36.7	21.1	20.4	19.8	20.4	20.4	19.8	26.2	25.4	26.2	294.9
1969	26.2	33.1	36.7	35.5	17.8	17.2	17.8	17.8	17.2	17.8	35.5	36.7	309.3
1970	36.7	33.1	36.7	35.5	36.7	24.7	25.5	25.5	24.7	25.5	35.5	36.7	376.8
1971	36.7	33.1	36.7	35.5	36.7	32.0	29.2	29.2	28.3	29.2	35.5	36.7	398.8
1972	36.7	34.4	36.7	35.5	36.7	25.2	26.0	26.0	25.2	26.2	25.2	26.0	359.8
Average	33.8	33.3	35.6	32.9	29.0	22.7	23.0	23.0	22.3	26.8	31.5	32.6	346.5

Table 7-3 Comparison with Large Scale Development

Item	Unit	Selected Plan	Alternative Plan
Installed Capacity	MW	50	100
Annual Energy Production	GWH	346	440
Load Factor	%	60	30
Height of Dam	m	80	
H. W. L.	"	1,575	
Available drawdown	"	47	
Effective Storage Capacity	10^6cu·m	36	
Tailwater Level	m	900	
Total Length of Conduit	"	16,460	
Inner Diameter of Conduit	"	2.30	2.90
Maximum Discharge	cu·m/sec	9.5	19
Annual Discharge	10^6cu·m	238	304
Construction Cost	Mio s/.	2,873	3,568

Table 7-4 Summary of Construction Cost

Unit ; million S/o

I Generating Facilities	
1. Direct Costs	
Access Road	241.9
Construction Facilities	142.0
Dam	594.4
Headrace Tunnel with Intake and Surge Tank	505.6
Penstock	116.8
Powerhouse and Switchyard	32.8
Gates and Valves	27.4
Electromechanical Equipment	241.7
Contingency	273.4
Sub total	2,176
2. Engineering and Administration	218
3. Interest during Construction	479
Total	2,873
II. Transmission Line	
1. Direct Costs	101
2. Engineering and Administration	10
3. Interest during Construction	4
Total	115
Grand Total	2,988