10.3.3 El Siete No. 1 Auxiliary Dam Site

A fairly large amount of sedimentation is expected at the No. 1 regulating reservoir and it is thought that frequent sand flushing will be necessary. It is planned that an auxiliary dam (concrete gravity dam, crest length 78 m, height approximately 21 m) be constructed to continue power generation even during sand flushing.

Three trenches totalling 37.15 m in length were excavated at this damsite, as shown in Dwg.-10 and Table-10.1.

(1) Topographical Conditions

No. 1 auxiliary dam site is located approximately 0.9 km upstream of No. 1 damsite (at the upstream end of the No. 1 regulating reservoir). The middle portion of the No. 1 dam reservoir is a V-shaped gorge, but this damsite is located at an area upstream of the gorge where the valley width increases and terraces are formed at an area of the left-bank side in the vicinity.

At the damsite, as shown in the section in Dwg.-10, the right bank is sloped at 50 to 60°, and the left bank is sloped approximately 30° above two steps of river terraces (relative heights 1 m and 10 m). The settling basin appurtenant to the No. 1 auxiliary dam is to be located on the terraces at the left-bank side.

(2) Basement Rock Types and Lithologies

The dam foundation consists of the K_{1B} Formation composed mainly of a siliceous shale with intercalations of calcareous shale and chert overlain by Quaternary terrace deposits, slopewash, and river deposits.

Basement rock outcrops in the dam axis vicinity are generally seen in large numbers at the right-bank side, but cannot be recognized at the terraces on the left-bank side. The black siliceous shale and the grayish white calcareous shale are generally hard rocks with beds of 2 to 5 cm developed, while fissibility is also recognized in part of the siliceous shale. The strata of the damsite's right bank show a strike (approximately NS) roughly parallel to the dam axis and vertical dip.

Joints with directionality in particular cannot be seen in the basement rock.

(3) Overburden

1) Terrace Deposit

The terrace deposit at the damsite is an unconsolidated deposit consisting mainly of pebbles and cobbles, with sand and silt filling the interstices.

This deposit is distributed at the left bank of the damsite and although the thickness is unconfirmed, it is estimated to be 10 m at the dam foundation and the settling basin as judged based on the topographical characteristics.

2) Slopewash

Slopewash at the damsite consists of angular fragments of chert or siliceous shale, and silt filling the interstices of these angular fragments. This slopewash is distributed overlying the basement and terrace deposit.

This deposit is not found at the foundation of the dam, but is thought to be distributed in a thickness not more than several meters in the vicinity.

3) River Deposit

The river deposit at the damsite mainly consists of cobble gravels, with sand and silt filling the interstices. The gravels contained are mostly subrounded sandstone, shale, basalt, and chert. It is estimated that this deposit is not more than 2 or 3 m thick at the dam foundation.

(4) Geological Engineering Assessments

The geological engineering assessments of this auxiliary dam site based on the topographical and geological conditions disclosed by the various investigations made so far as described below.

1) Dam Foundation Excavation

A concrete gravity dam of crest elevation 1,465 m is planned at this site (present riverbed surface elevation approximately 1,450 m). For the foundation of a dam of this scale there should be adequate bearing power of the ground if it is of shales with comparatively little weathering such as observed at the river bank. Accordingly, it would be necessary to excavate and remove river bed sand and gravel, terrace deposits, and weathered portions of basement rock surface layer for the dam foundation, and as a result, the maximum excavation depths from the ground surface may exceed 10 m at places.

2) Dam Foundation Watertightness

Water pressure tests have not yet been performed at this site. However, the high water level at this auxiliary dam will be low at El. 1,460 m (a height of approximately 10 m from the present river bed), and as the basement may be considered to be impervious where fresh, it is thought that adequate dam foundation watertightness can be secured with excavation of not more than 2 or 3 m from the basement surface. It may however become necessary for water barriers to be provided with grout curtains at places.

3) Supplemental Investigations

It will be necessary for drilling investigations, including water pressure tests, to be carried out to ascertain the ultimate excavation line of the dam foundation and the necessary to provide water barriers.

10.3.4 El Siete No. 1 Headrace Tunnel Route

The El Siete No. 1 Project headrace is to be a pressure tunnel (finished inside diameter 3.4 m) approximately 3,150 m long from an intake provided immediately upstream of the left-bank abutment of the No. 1 dam to a surge tank also at the left-bank side of the Rio Atrato.

Surface geological surveys using rough topographical maps (prepared by JICA in 1981) of approximately 1/25,000 scale, and photogeological interpretations of the El Siete No. 1 headrace tunnel route and photogeological interpretations have been made up to this time, and the results are summarized in Dwg.-06.

(1) Topographical Conditions

This headrace tunel, as shown in Dwg.-06 and Dwg.-07 (H_2-H_2), is planned to run under the mountain body at the left bank of the Rio Atrato which flows east to east-southeast as a whole, the maximum cover of the tunnel being approximately 500 m, with the sections at the two ends of the tunnel, its cover being less than 100 m totalling 200 m in length. Three small ravines are passed under at the first-half section of the tunnel, and the covers at these points are 170, 200, and 200 m, respectively, while the remaining part of the tunnel will generally pass under a ridge.

According to photogeological interpretations made up to this time, there is no especially devastated area (landslide, large collapse, other) to be seen in the area through which the tunnel will pass.

(2) Rock Types and Lithologies

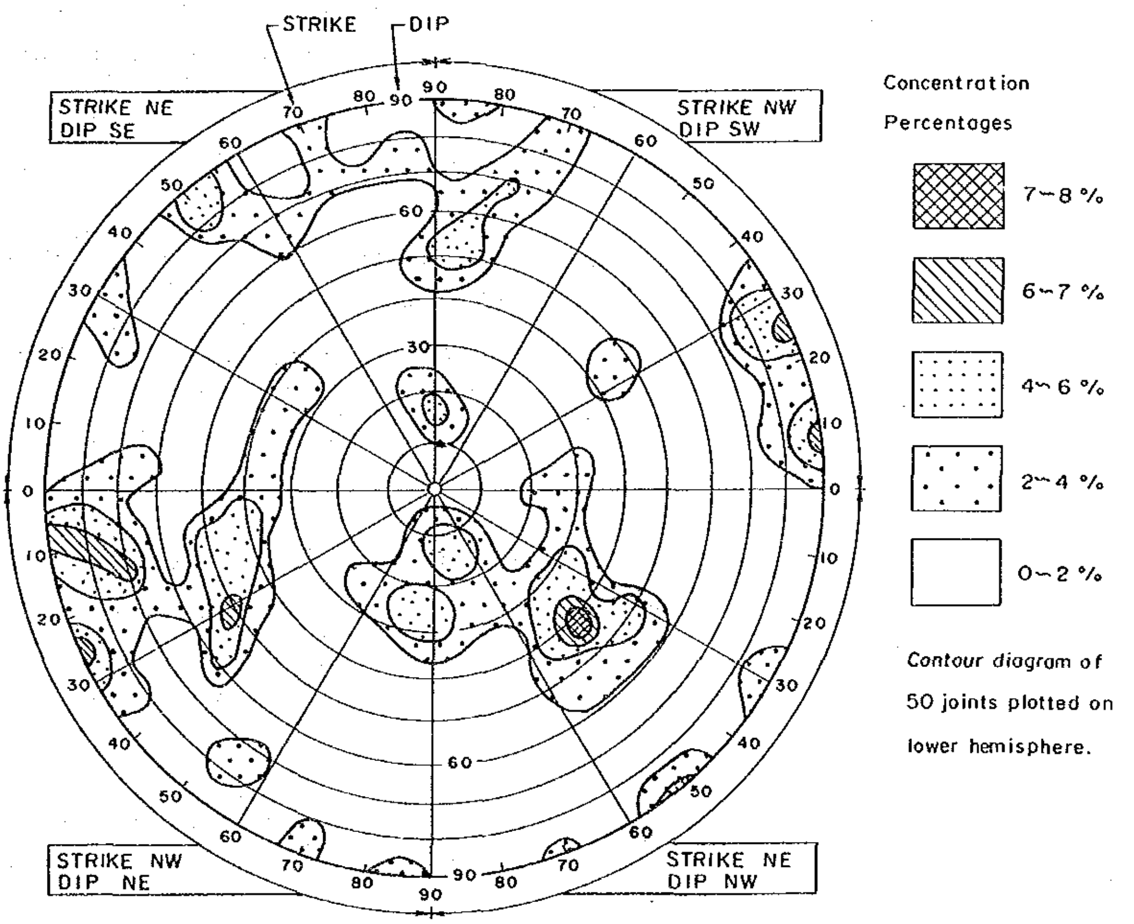
According to the results of investigations up to this time, this tunnel is estimated to pass through besalt of the K_{2A} Formation at the upstream two-thirds, with the remaining section going through alternations of conglomerate, sandstone, and shale of the K_{2B} Formation with partial besalt interbedding.

The basalt (the K_{2A} Formation) distributed at the upstream part of the headrace tunnel has partial intercalation of basaltic tuff breccia. These rocks, so far as seen at outcrops along the Rio Atrato, are hard with few cracks where fresh, and there is hardly any bedding recognizable. At the No. 1 dam site, bedding of basaltic tuff breccia with strike of N20°W, and dip of 85°N is observed. Further downstream, along the Rio Atrato, there is also a flow structure of strike N50°E and dip 40° - 50°NW in part of the basalt.

The conglomerate, sandstone, shale alternations distributed at the downstream section of the headrace have individual 5 to 50 cm thick layers which are repeated to result in stratifications, while basalt is intercalated in these strata in thicknesses of several to several tens of meters. These rocks, so far as seen along the Rio Atrato, Qda. Sta Isabel (Ravine), etc., are hard with few cracks where fresh. The strikes of these strata are in the NS direction, roughly orthogonal to the tunnel, while dips are mostly to the east and abruptly inclined.

The predominant directions of joint planes in the surroundings of the headrace tunnel route, as shown in Fig.-10.2, are strike of N40°E and dip of 40°NW, strike of N10°E and dip of 80°NW.

Fig. 10-2 Contour Diagram of Joints in El Siete No.1 Waterway Alignment



(3) Faults and Lineaments

Faults were not confirmed at the No. 1 headrace tunnel route in field geological survey made to this time. In the photogeological interpretations made of this route three lineaments were discerned at the vicinities of the ravines in the upstream area, of which are that goes through Qda. Nieve (NS direction) possesses continuity. It was not ascertained whether this air photo lineament was formed by a fault.

(4) Groundwater

Surface water can be seen in Qda. Nieve crossing the No. 1 headrace tunnel route and other ravines in the vicinity of the route up to higher than EL. 1,600 m, and from the fact that trees flourish in some parts of high elevation, it is surmised that a comparatively shallow groundwater table exists conforming to the topography.

(5) Geological Engineering Assessments

1) Rocks distributed along the El Siete No. 1 headrace tunnel route are all hard and dense at parts that are fresh, while parts of cover of 200 m or more make up about 70 percent of the total tunnel length, so that it is considered that there are no geological conditions to present any particular problems during excavation.

2) It is considered that the rock at sections where cover for the tunnel is thin such as the vicinities of the intake and surge tank will have been subjected to some influence of weathering from the ground surface, and therefore, care will need to be exercised.

3) According to photogeological interpretation made up to this point, there are three lineaments intersecting the tunnel, but their causes are not presently known. Should these lineaments happen to be faults, considered together with the fact that it is expected the groundwater table along the route exists at shallow depths from the ground surface, it is possible that pressurized spring water will be encountered at these faulted parts during tunnel excavation.

4) It will be necessary for further geological mapping to be done with improved precision regarding the No. 1 headrace tunnel route.

10.3.5 El Siete No. 1 Auxiliary Connection Tunnel Route

The El Siete No. 1 Project auxiliary connection tunnel is to connect from the intake at the left bank upstream of the No. 1 auxiliary dam to the No. 1 headrace tunnel planned at the left-bank side of the downstream No. 1 dam. It will be a non-pressure tunnel, approximately 860 m long (finished inside diameter 3.4 m).

Up to now, geological mapping has been done on the El Siete No. 1 auxiliary connecting tunnel route using 1/1,000 topographical maps, together with photogeological interpretations.

(1) Topographical Conditions

The auxiliary connection tunnel, as shown in Dwg.-06 and Dwg.-07 (H_1-H_1), is planned under the mountain body at the left-bank side of the Rio Atrato with a maximum tunnel cover of approximately 60 m. At Qda. Sta Lucia (Ravine) this cover will be approximately 20 m. The tunnel, other than passing under Qda. Sta Lucia at a location midway along the route, runs mainly under the foot of a steeply sloped mountainside.

(2) Rock Types and Lithologies

According to the results of investigations made up to the present, it can be estimated that the first half of this tunnel will pass through alternation of chert and limestone of the K_{1B} Formation, with the remainder passing through basalt of the K_{2A} Formation. The two formations appear to be in conformity according to observations of outcrops at the left-bank side of the Rio Atrato althought there is some shearing seen in the vicinity of the boundary between the two formations.

Rocks distributed along this tunnel route are all hard at fresh portions.

Bedding planes of the alternation of chert and limestone intersect the tunnel almost perpendicularly with a strike of N5°W-N20°E, with steep dips in the downstream direction in most cases. The beds are from 2 to 40 cm thick. Basalt is generally massive in this section.

With regard to the degrees of weathering, because the tunnel route's topography is rugged and there are numerous outcrops of relatively hard and fresh rocks, it is thought that the rock will be mostly fresh along the tunnel route.

(3) Faults and Lineaments

There are four faults in the north-south direction with sheared zone widths of 10 to 30 cm found in surface geological surveys in the vicinity of Qda. Sta Lucia as shown in Dwg.-07 and Dwg.-08. There is also an air photo lineament extending along the ravine in a north-south direction which was recognized in an aerial photograph.

The tunnel crosses with the previously-mentioned boundary between the K_{2A} and K_{2B} Formation, and that boundary portion has been slightly sheared and is schistose 2 to 3 m in width on the K_{2B} Formation side at outcrops on the left-bank side of the Rio Atrato.

(4) Groundwater

As surface water can be seen in Qda. Sta Lucia and every one of its feeder ravines up to higher elevations, it is thought that a shallow groundwater table exists along the tunnel route.

(5) Geological Engineering Assessments

1) On summation of the investigation results obtained to this point, in addition to intersecting with the boundary between the K_{1B} and K_{2A} Formations which is slightly sheared in a width of 2 to 3 m, the possibility exists of it crossing with at least three minor faults (all sheared in width of 20 to 30 cm) near Qda. Sta Lucia as shown in Dwg.-08.

   The tunnel will pass about 20 m directly below Qda. Sta Lucia which has a relatively large quantity of surface water, but so far as seen at the ground surface in the vicinity, outcrops that can be observed are fresh. There is however, the possibility that the tunnel will cross with the previously-mentioned weakness line, and when excavating in the vicinity of this site it will be necessary for care to be exercised as a certain amount of water springing may occur.

2) Other than the described poor geological conditions it is expected that fresh, hard rock is distributed along the greater part of the tunnel route.

10.3.6 El Siete No.1 Surge Tank, Penstock, Powerhouse and Tailrace Site

It is planned for the El Siete No.1 surge tank (bottom elevation 1,423 m) to be approximately 43 m high (inside diameter 7.8 m), with a connecting penstock approximately 1,300 m long (inside diameters from 3.4 to 1.25 m), the greater part being surface type, and a surface type powerhouse (discharge water level, 1,071 m) located on the left bank of the Rio Atrato at a ground elevation of approximately 1,090 m. A tailrace tunnel (total length 185 m, inside diameter 3.6 m) would be at the left-bank side of the Rio Atrato leading to the No.2 intake dam to be provided upstream of the No.1 powerhouse.

Besides surface geological survey of the abovementioned projected structures and their surroundings made with aerial photographs enlarged to 1/2,000 scale and a rough 1/25,000 scale topographical map (mapped by JICA in 1981), investigations have been made up to this time through 16 trenches totalling 173.40 m in length, 2 pits totalling 3.65 m in depth, seismic prospecting with 3 traverse lines totalling 770 m in length, and 3 measuring points in vertical electric prospecting[1/].

1/ The electric prospecting measuring points for the project area were distant from the presently planned structures so that this investigation result is not discussed herein, but is described in Appendix-III-3(2).

(1) Topographical Conditions

The surge tank site was selected at a ridge extending roughly to the east, and the penstock route selected along a ridge branching southwest to west-southwest from the surge tank site.

The penstock route, as shown in Dwgs.-11 and -12, is at a ridge, sloped gradually as a whole at approximately 15°, and the penstock will cross a ravine (Qda. Aguila) immediately before reaching the powerhouse. The penstock route is at a location where the ridge is wide, but is narrow at the middle and further below. There are small slope collapses scattered at midheight of this ridge.

The powerhouse site is located on a flat area of relative height 50 m at the left-bank side of the Rio Atrato. The tailrace tunnel of the No.1 powerhouse, as previously described, will pass the left-bank side of the Rio Atrato to the No.2 intake dam site to connect to the No.2 headrace tunnel. The cover for this tailrace tunnel will be approximately 40 to 50 m.

(2) Basement Rock Types and lithologies

Mainly alternation of sandstone and shale of the Upper Cretaceous K_{2B} Formation with conglomerate and basalt intercalated here and there are distributed from the surge tank site to the powerhouse site as shown in Dwgs.-11 and 12. Regarding the tailrace tunnel, the one-third section on the powerhouse side consists of sandstone-shale alternations with basalt distributed at the remainder. The bedding planes of these strata generally show strikes orthogonal to the penstock and dips to the upstream side (NS, 50° - 70°E).

Fresh, hard parts of the basement rock are seen at the ravine in the vicinity of the penstock route, but according to trench investigations at the foot of the slope, that part of the penstock route is soft to a depth of at least 2 to 3 m.

The previously mentioned ravine (Qda. Aguila) in the vicinity of the powerhouse site shows fresh and hard alternation of sandstone and shale at its bottom.

Fresh, hard, and massive basalt is exposed at the left-bank slope of the Rio Atrato, forming a cliff along the tailrace route.

(3) Weathering

The surface portion of the basement rock forming the ridge down which the penstock will pass has generally been subjected to strong weathering. The strongly weathered portion of the sandstone-shale alternations has changed in color to brown, and rock fragments have mostly been softened to a degree that they can be crushed with the fingers, while strongly weathered portions of basalt present a condition of sound basalt in the form of gravels remain in soft weathered rock which has been changed to a brown color.

At the ridge where the penstock route is to pass, the slope is gradual, and there is a narrow section, and it is estimated that weathering has generally penetrated deeply. According to the results of investigations made up to now and topographical properties, the strongly weathered portion of the basement is estimated to be less than 5 m thick from the surge tank site to approximately EL. 1,250 m of the penstock route, while from approximately EL. 1,250 m of the penstock route to EL. 1,100 m the strongly weathered portion would generally be around 5 m thick, but between EL. 1,160 m and 1,140 m where the ridge narrows, it is estimated to be about 20 m thick. The strongly weathered layer at the powerhouse site and the tailrace route is thought to be less than 5 m thick.

(4) Faults and Lineaments

Faults were not found in investigations up to the present.

An air photo lineament extending northwest-southeast was found at the lower part of the penstock route in photogeological interpretations, and this crosses with the lower part of the penstock (vicinity of approximately 200 m upstream of the powerhouse), but surface geological surveys made up to now have not confirmed whether this was formed by faulting.

(5) Groundwater

The ravines at either side of the penstock route ridge (Qda. Aguila, Qda. San José) show surface water up to parts of high elevations at which trees grow thickly. It is therefore estimated that groundwater exists at relatively shallow depths from the ground surface conforming to the topography, except for that area where the ridge is narrow.

The tailrace tunnel is expected to pass below the groundwater table.

(6) P-wave Velocity

Seismic prospecting by the refraction method (Three traverse lines) was carried out at the lower part of the El Siete No.1 penstock and its surroundings. The results are as given in Table-10.5, Dwg.-12, and Appendix-III-3(1), and ground P-wave velocities can be classified according to 4 or 5 layers. The first and second layers show P-wave velocities under 0.7 km/s, and are thought to correspond to topsoil and slopewash. The velocities of 1.1 to 1.4 km/s of the third layer are thought to be comparable to slopewash, terrace deposits, and residual soil. The velocites of 1.6 to 1.9 km/s of the fourth layer to strongly weathered rock. The velocities of 2.2 to 4.2 km/s of the fifth layer to slightly weathered rock to fresh rock.

(7) Overburden

Overburden at the surge tank site and penstock route is generally thin, and as prevously described, strongly weathered bedrock changed to a residual soil condition is distributed from the ground surface to shallow area of the ridge.

A loose deposit consisting mainly of angular fragments of basalt, sandstone, etc., and silt is seen distributed at the powerhouse site and its surroundings.

There is very little overburden distribution at the tailrace tunnel route.

(8) Geological Engineering Assessments

1) There is a possibility that the bedrock at the surge tank site has been strongly weathered from the ground surface to a depth of about 5 m. A cut slope of approximately 20 m high will appear at the ground surface of the surge tank site and since strongly weathered rock will be exposed at this cut slope, it will be necessary to pay attention to securing the stability of that section.

2) The thickness of the overburden and strongly weathered portion of the penstock route is estimated to be approximately 5 m at the upper part of the slope and approximately 20 m at the lower part, according to the results of trench investigations and seismic prospecting, and in view of topographical conditions. The penstock anchor block foundations will need to be secured from at least below this overburden and strongly weathered portion of rock.

   As described small slope collapses are scattered the penstock route vicinity, and it is considered that detailed investigations should be made of these in the future to fully determine their porperties. Depending on the results of these investigations it may be necessary that certain measure be taken to prevent expansion of the collapse areas.

3) Since it is planned for the powerhouse to be constructed upon excavation of approximately 20 m from the ground surface, the basement rock at the bottom of the powerhouse will be fresh, hard sandstone-shale alternations and basalt. It is estimated that adequate ground bearing power can be obtained if the structure is of the scale planned.

   A cut slope approximately 30 m high will appear in the vicinity of the powerhouse and as slopewash will be distributed at the top portion followed by strongly weathered bedrock, it will be necessary for attention to be paid to the stability of this slope.

4) It is considered that the tailrace tunnel will pass below the groundwater table, but the rock in that area is estimated to be

fresh and hard as a whole, and it is presumed from the investigation results gained up to this point, that there will be few porblematical points in relation to engineering geology.

5) When geological mapping was done on the surge tank site and the upper part of the penstock route, highly accurate topographical maps and sharp aerial photographs could not be utilized, and it will therefore be necessary for detailed reconnaissances (including investigations of collapse areas) to be carried out when topographical maps of high accuracy become available in the future.

   It will also be necessary for the bedrock conditions, including the weathered state of the basement rock, to be confirmed by drilling at the surge tank site, penstock anchor block sites, and the powerhouse site.

10.4 Geology of El Siete No.2 Project Site

The El Siete No.2 Project is a scheme for power generation of a maximum 85 MW combining the water discharged from the No.1 powerhouse with the water of the catchment area downstream of the No.1 dam, and conducting this water to the No.2 powerhouse by a headrace approximately 9 km in length at the right-bank side of the Rio Atrato and a penstock of approximately 1 km.

10.4.1 El Siete No.2 Intake Dam Site

The El Siete No.2 intake dam site would be located approximately 130 m upstream of the No.1 powerhouse, where it is planned for a concrete gravity dam with a 1,075 m crest elevation, 146 m crest length, and 35 m dam height to be constructed. A settling basin is planned to be provided at the right-bank side together with the dam. (The No.1 tailrace tunnel will cross the No.2 intake dam going through the dam body to join the No.2 headrace tunnel at the right-bank side.)

Two drillholes totalling 79.4 m in length and seismic prospecting on 4 traverse lines totalling 1,310 m in length as shown in Table-10.1 were provided at this intake dam site and its surroundings. Of these, Drillhole BD-2 (discontinued at depth of 9.5 m due to mechanical trouble) and seismic prospecting (Traverse Line SB-2, length 320 m) were at the dam axis.

(1) Topographical Conditions

The intake dam site, as shown in Dwg.-11, is to be located at a bend where the Rio Atrato changes course from south-southeast to south-southwest. The river-bed elevation there is approximately 1,055 m, and the channel is approximately 15 m wide. A terrace of relative height approximately 20 m is formed at the right-bank side of the damsite. According to the present plan, the dam crest will be slightly higher than the surface of the terrace.

The slopes at the two banks of the damsite, as shown on Section C-C in Dwg.-10, are included 40 to 50° It is estimated that an old river channel buried by a mudflow deposit as described later exists immediately below the surface of the terrace at the right-bank side.

(2) Basement Rock Types and Lithologies

Basalt (the K_{2B} Formation) at the left bank and river bed of the dam foundation and sandstone-shale alternations of the same period (the K_{2B} Formation) at the right bank are covered by a thick mudflow deposit, terrace deposits, and slopewash.

Basement rock outcrops are seen in large numbers at the right-bank side while at the left-bank side they can be observed only in very small numbers along the river bed. The sectional shape of the basement at the right bank is estimated to have a depression lower than the present river bed directly under the right-bank tableland as shown in Dwg.-10 on overall examination of the results of surface geological survey, Drillhole BD-1 (bedrock located at depth of 42 m from the ground surface), and seismic prospecting. It is thought this depression was formed by the Rio Atrato flowing more to the right bank than the present river channel before deposition of the mudflow.

The basalt making up the dam foundation presents a black color and is hard, but has cracks developed at 30 to 100 cm intervals, while the alternation of sandstone and shale consist of gray sandstone layers 1 to 5 cm thick and black shale layers 1 to 30 cm thick, the hardness being slightly inferior compared with the basalt. Lapilli tuff of thickness 1.5 m is intercalated at the boundary between the basalt and the alternation of sandstone and shale.

The strike of the alternations is roughly north-south in the vicinity of the damsite, with steep dipping to the west.

(3) Faults

Up to the present, investigations have not disclosed any faults directly passing through the dam foundation.

(4) Groundwater

The only measurements made of the groundwater table was the water level (depth 0.6 m) in the drillhole during boring of BD-2 at the right bank of the damsite. In view of the water table (depth 8.4 m) at BD-1 upstream of the dam, the conditions of ravines in the damsite vicinity, and the existence of wet ground at the surface, it is estimated that the groundwater table is at a comparatively shallow depth from the ground surface.

(5) P-wave Velocity

Seismic prospecting (4 traverse lines) by the refraction method was done at the damsite and its surroundings to determine the P-wave velocities of the mudlfow deposits and the bedrock. As a result, as shown in Table-10.5, Dwg.-10, and Appendix-III-3(1), the P-wave velocities of this site may be calssified according to 4 to 5 layers. Of these, the first layer, which is the surface layer, to the third layer indicated P-wave velocities less than 1.8 km/s and were thought to correspood to the mudflow deposit or slopewash, while the velocities of 1.7 to 2.7 km/s for the fourth layer are thought to be comparable to strongly weathered bedrock or bedrock with cracks developed, and the velocities of 2.3 to 4.6 km/s of the fifth layer to slightly weathered or fresh bedrock.

(6) Overburden

1) Mudflow Deposit

The mudflow deposit at the damsite shows variation in constitution depending on the location, but mainly contains 50 to 80 percent of subangular and/or subrounded gravels of basalt 0.5 to 30 cm in diameter in a gray, silty matrix, and is an unconsolidated deposit which also contains wood fragments in places.

It is estimated from the results of drilling (drillhole BD-1) upstream of the dam that the mudflow deposit is about 30 to 40 m thick.

This deposit shows a fairly well-compacted condition in places at the surface layer, but is estimated not to possess strength sufficient for the foundation of a concrete dam.

2) Terrace Deposit

The terrace deposit at the damsite mainly consists of cobbles and silty sand filling the interstices, and is thought to be about 7 to 8 m thick at maximum. This deposit makes up a loose stratum as a whole.

3) Slopewash

The slopewash at the damsite consists of silt containing angular fragment of basalt and sandstone 3 to 30 cm in diameter and is a deposit which is loose as a whole and is distributed at a thickness of about 2 to 3 m at this site.

4) River Deposit

The river deposit at the damsite mainly consists of basalt cobbles and pebbles with silty sand filling the interstices, but there are also boulders several meters in diameter scattered at the river bed.

This deposit at the damsite is thought to be not more than 5 m thick.

(7) Geological Engineering Assessments

1) Dam Foundation Excavation

A concrete gravity dam with a 1,075 m crest elevation is planned at this site (present river-bed elevation approximately 1,055 m). It is necessary for the foundation of a dam of this scale to be sought on bedrock with comparatively little weathered bedrock such as that found at the river banks, and for this purpose, the river-bed sand-gravel, mudflow deposit, and strongly weathered basalt would be excavated and removed. It is thought that in dam foundation excavation, parts of P-wave velocities less than 2.0 km/s in the seismic prospecting analysis section (Section C-C in Dwg.-10) should at least be excavated and removed. There may be also necessary for some degree of improvement (e.g., consolidation grouting) to be performed on the foundation rock, but this should be determined after further detailed studies are carried out.

2) Basement Watertightness

Water pressure tests have not yet been carried out for the permeability of the basement, but estimated from P-wave velocities according to seismic prospecting, it is expected that cracks are developed at least to depths of 10 to 20 m from the bedrock surface, and it will be necessary for some type of water barrier treatment to be provided.

(8) Supplemental Investigations

To ascertain the excavation line and the extent of water barrier treatment required for the dam foundation, it is necessary for geological investigations (mainly drilling and water pressure tests) to be carried out to confirm the properties, especially those of the mudflow deposit and/or bedrock at the river-bed portion of the dam and the tableland at the right-bank side of the damsite.

10.4.2 El Siete No.2 Headrace Tunnel Route

The El Siete No.2 Project headrace would be a pressure tunnel approximately 9.1 km long (inside diameter 3.6 m) from the intake at the right bank upstream of the No.2 intake dam to the No.2 surge tank at the right-bank side of the Rio Atrato.

Surface geological surveys using rough topographical maps of scale approximately 1/25,000 (prepared by JICA in 1981) and photogeological interpretations were made regarding the El Siete No.2 headrace tunnel route.

(1) Topographical Conditions

This headrace tunnel, as shown in Dwg.-06 and Dwg.-07 (Section H_2-H_2), is planned to run through the mountain body on the right-bank side of the Rio Atrato which flows roughly south as a whole.

The maximum cover for this tunnel would be approximately 400 m, while sections of cover less than 100 m, including the two end portions of the tunnel and parts of ravines along the way, total approximately 1,700 m in length (approximately 20 percent of the total length of the tunnel).

The tunnel, as shown in Dwg.-07, passes three large ravines, in its first half section, where the covers are 140 m, 70 m, and 80 m, respectively, in order from the upstream, while the latter half passes directly under the ridge as a whole.

(2) Rock Types and Lithologies

The El Siete No.2 headrace tunnel route has distributions in order from the intake toward the downstream of the K_{2B} Formation consisting of alternation of sandstone and shale, the K_3 Formation mainly consisting of diabase, and the Td Formation mainly consisting of diorite. According to the results of elementary investigations up to the present, it is estimated that these formations, K_{2B}, K_3, and Td, are distributed at the tunnel route in proportions of approximately 10 percent, approximately 60 percent, and approximately 30 percent, respectively.

The general strike and dip of the bedding planes of the K_{2B} Formation are N-S, 70°-90°E, while there are intercalations at places of tuffacious or rhyolitic rocks. The individual layer thicknesses are generally about 5 cm and the lithology of fresh parts is hard.

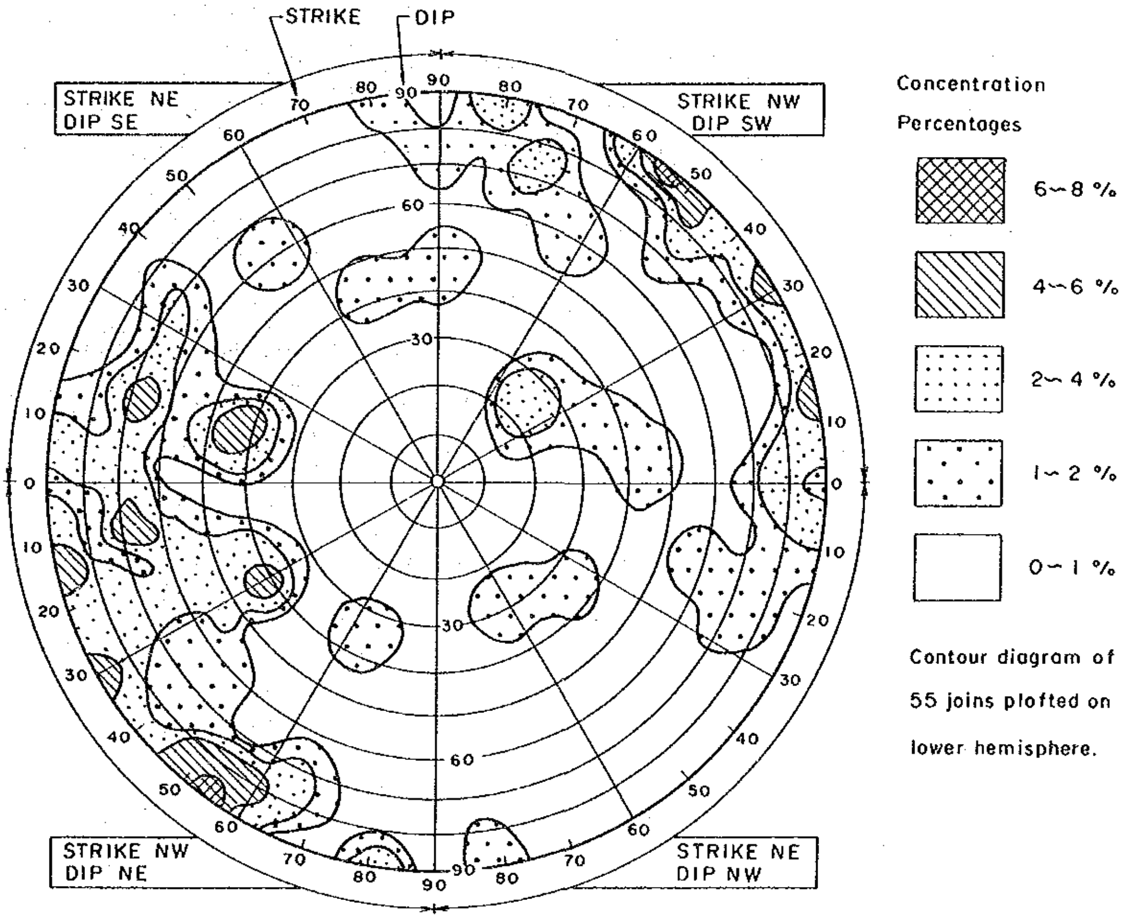
The diabase of the K_3 Formation is dark green, hard and dense rock, but at outcrops at the ground surface it is slightly weathered and somewhat cracky.

The diorite of the Td Formation is basically a dense rock and at this tunnel it will constitute a hard rock body except in the surge tank vicinity and near the boundary with the K_3 Formation.

The boundary between the diabass and diorite runs along a comparatively large ravine (Qda. El Piñon) coming in between the tunnel and the Rio Atrato at the downstream part of the tunnel, and at outcrops in the ravine, an amphibolite belt (width more than 200-300 m at the outcrop) of schistose structure caused by metamorphism is distributed along the boundary. This amphibolite belt is of deteriorated lithology when compared with the rocks on both sides, and there is a possibility of crossing with this tunnel at a point slightly downstream (on the surge tank side) of its middle.

Regarding joint planes around the tunnel route, as shown in Fig.-10.3, those of strike N55°W dip vertical and strike N30°W - N20°E and dip 49 - 90°E are slightly predominant.

Fig. 10-3 Contour Diagram of Joints in El Siete No.2 Waterway Alignment



(3) Faults and Lineaments

There are two air photo lineaments extending 1 to 2 km in the north-south direction crossing with this tunnel at the first-half section, and five in the N45 - 50°W direction at the latter-half section, but the causes of these air photo lineaments are unknown.

(4) Groundwater

As surface water is seen at ravines along the tunnel route up to high elevation areas and trees grow up to the mountaintops, it is estimated that the groundwater table along the tunnel route exists at a comparatively shallow depth from the ground surface, conforming to the topography.

(5) Geological Engineering Assessments

1) As previously described, a schistose structure is developed in the amphibolite belt at the boundary between the disbase and diorite, with a lithology thought to be brittle compared with the diorite and diabase. Tunnelling in this area will therefore require care to be exercised.

2) According to photogeological interpretations, there are seven air photo lineaments roughly orthogonal to the tunnel route existing in this tunnel area. Of these, the one at the middle section of the tunnel along Qda. Quitasuenos is at a portion of comparatively shallow cover (approximately 80 m) for the tunnel, and it will be necessary for further investigations to be made regarding the properties of this lineament.

3) More than 80 percent of the entire length of the El Siete No.2 headrace tunnel will have covers exceeding 100 m. Even should there be parts of poor quality locally as described in 1) and 2) above, the greather part of the tunnel will pass through good rock. However, since the proportion of costs of the No.2 Project to be taken up by construction of this headrace tunnel will be large, it will be necessary for further investigations to be made to determine tunnel route's geological condition in more detail.

10.4.3 El Siete No.2 Surge Tank, Penstock and Powerhouse Site

The El Siete No.2 surge tank (bottom elevation 1,040 m) is planned to be 48 m high (inside diameter 9 m) with the penstock connected to it approximately 1,045 m long (inside diameters 3.4 to 1.25 m), the greater part being constructed on the surface. The No.2 powerhouse (discharge water level El. 687 m) is planned as a surface type at the right bank of the Rio Atrato.

Drilling (2 drillholes, total length 68.68 m), standard penetration tests (9 points at 2 drillholes), trenching (5 trenches, total length 70.95 m), and electric porspecting (3 measuring points) as shown in Table-10.1 were carried out at this site at locations lower than the middle point of the penstock.

(1) Topographical Conditions

The No.2 Project surge tank site is located near the end of the main ridge extending roughly in a southward direction at the right-bank side of the Rio Atrato. The inclination angle of the slope in the vicinity is approximately 30°.

The penstock will first run along a narrow ridge branching in the southeast direction from the abovementioned main ridge, following which it will go down a flat area with a rouned, gradual slope and reach the No.2 powerhouse site.

No.2 powerhouse is planned at the foot of this slope which is inclined 35° at the right-bank side of the Rio Atrato.

(2) Basement Rock Types and Lithologies

Diabase is distributed from the surge tank site, down through the penstock area to a point before the powerhouse site. Diabase and amphibolite (the K_3 Formation) exist together in the vicinity of the powerhouse foundation, a part having been subjected to mylonitization.

Fresh parts of the diabase have joints developed at 20 to 40 cm, but are hard. Weathering has however occurred to depths of 20 to 30 m from the ground surface at the penstock route, core recovery being poor in drilling, with cores in the form of sand-gravel recovered. Three joint sets of strike N10 - 50°E and dip 60°SE, strike N60 - 90°W and dip 60 - 90°N, and strike N10°E - N60°W and dip 20 - 50°NE are seen in the diabase, and it was observed that weathering has progressed along these joints.

The amphibolite seen at the river bank in the vicinity of the powerhouse site is fresh and comparatively hard, but at Drillhole CD-1 it has been subjected to weathering to a depth of about 8 m. A schistose structure of strike in the east-west direction and dip toward the river of around 70° is recognized in this rock.

(3) Weathering

The diabase constituting the bedrock, according to the results from Drillhole CD-2 made at an elevation of approximately 815 m on a gently-sloped ridge, is extremely weathered to a depth of 23 m, core recovery is poor, and recovered cores are in the form of sand-gravel. To approximately 0.5 m from the ground surface is strongly weathered to present a residual soil condition, down to a depth of 4 m has changed to a brown color, with the constituent minerals themselves weathered and having become brittle compared with rock at lower depths.

As shown in Dwg.-14, the ridge from the surge tank site to elevations of 900 to 800 m along the penstock route is estimated to be weathered to depths of 20 to 30 m, while weathering of the slope from EL. 800 m down to the powerhouse site is shallow and is 1 to 2 m.

(4) Faults and Lineaments

Regarding faults, there is one (width of sheared zone: 20 cm) confirmed at the right bank of the Rio Atrato approximately 250 m upstream of the powerhouse site with a strike (N52°E) parallel to the Rio Atrato and a dip toward the river of 55°, and it is possible that this passes the vicinity of the powerhouse site. At Drillhole CD-1, rock of poor quality was confirmed to a depth of 20 m from the ground surface, and it is estimated that this part of poor condition is due to the influence of the previously described fault.

There have been two air photo lineaments extracted that cross with the penstock part way along its route at a flat area as shown in Dwg.-13. The causes of there lineaments have not been discerned in investigations up to this point.

(5) N-Value

The results of standard penetration tests in drillholes at the penstock and powerhouse sites, as shown in Table-10.6, are depths from the ground surface to obtain an N-value of 50 or higher of approximately 2 m at Drillhole CD-1, and approximately 7 m at CD-2.

These results indicate that even though the bedrock itself is weathered, the N-value is comparatively high.

Table-10.6 N-Value in Drillholes at No.2 Penstock Site

Hole Name	Depth in Hole (m)	N-Value (Numbers of blqw in 30 centimeters)
CD-1	0 - 0.6	14
	0.6 - 1.2	28
	1.2 - 1.8	31
	1.8 - 2.0	above 50
CD-2	0 - 1.18	1
	1.18 - 5.29	2 - 4
	5.29 - 6.32	6 - 7
	6.32 - 6.82	30
	6.82 - 7.32	above 50

(6) Electric Prospecting

Dwg.-06 shows the locations of vertical electric prospecting, and Table-10.7 the results. At the El Siete No.2 penstock route, it is estimated from comparisons with the drilled core of CD-2 that the first layer

Table-10.7 Resistivity Layers in Project Area (Vertical Electrical Sounding)

Location	Mesuring-Point	Resistivity (ohm·m) 1st layer	2nd layer	3rd layer	4th layer	Remarks
El Siete No.1 Penstock Site	S-4	110 (1)	860 (15)	19.5 (40)	3,000	Location of mesuring-points are shown in Dwg.-06.
	S-5	52 (5) 70 (15)	200 (30)	22 (62)	200	
	S-6	240 (4)	600 (8)	24 (30)	180	
El Siete No.2 Penstock Site	S-1	650 (2.5)	1,300 (9)	350 (30)	2,500	
	S-2	1,500 (5)	3,200 (11)	250 (25)	1,500	
	S-3	640 (4.5)	1,400 (7.5)	185 (28)	3,000	

NOTE : ( ) shows depth of resistivity boundary in meter.

corresponds to a completely weathered diabase, the second layer a diabase of degree of weathering less than the above, the thrid layer rock of the second layer saturated by groundwater, and the fourth layer fresh diabase, respectively.

These results indicate that weathering in this area is deep as a whole.

(7) Overburden

As overburden, only small amounts of slopewash and river deposits are seen in the neighborhood of the powerhouse site, with almost no distribution at the projected structure sites.

(8) Geological Engineering Assessments

1) It is possible that the basement of the surge tank site is strongly weathered to a maximum depth of about 20 m from the ground surface. It will be necessary for this weathering condition to be considered in designing the surge tank vertical shaft and the cut slope at the surface portion.

   To be able to proceed hereafter with detailed designing it will be necessary, for example, to confirm the basement weathering depths and the groundwater table depth by drilling.

2) As described, it appears that the basement at the part of the penstock route is weathered to a maximum depth from the ground surface of approximately 20 to 30 m. However, there is a possibility for penstock anchor block bearing to be obtained at 6 to 7 m or shallower if the results of standard penetration tests are to be referred to, and it will be necessary for the final depth of the anchor block foundations to be determined by further investigations of higher accuracy.

3) Regarding the powerhouse foundation, since it will be at a level approximately 15 m deeper than the present ground, it is estimated that the foundation will be amphibolite, partially weathered, but providing more or less adequate bearing power.

As for stability of the cut slope back of the powerhouse, care will be needed since joints and schistose structures dipped toward the river are seen.

10.5 Concrete Aggregate

An exploration of natural aggregate deposits and an investigation of soil materials as well were carried out by ICEL at this project area, but since it was decided that the dam will be a concrete gravity type as described in Chapter 12, the results of the soil tests will not be touched upon here, but attached as Appendix-III-5(5). These investigation quantities are as shown in Table-10.1, and the locations in Dwg.-06.

The quantities of concrete aggregate necessary for the El Siete No.1 and No.2 Projects are shown below.

El Siete No.1 Dam	175,000 m^3
Other structures	387,000 m^3
Total	562,000 m^3

(1) Natural Aggregate Deposits

ICEL carried out natural aggregate investigations on river deposits at the Puente Sanchez area (AF_s site in Dwg.-06) along the Rio Atrato located 1.5 km upstream of the El Siete No.1 dam site and the Quebrada La Borrasca area (Af_b site in Dwg.-06) located 2.5 km downstream of the El Siete No.2 powerhouse site. Four test pits were excavated at the Puente Sanchez area and seven pits and a trench at the Quebrada La Borrasca area, and gradation analyses made of samples collected. The results are shown in Table-10.8.

Regarding varieties, shapes, and hardnesses of gravels, according to observations, subrounded and/or subangular gravels of sandstone, shale, chert, basalt, etc. are found at the river deposit in the Puente Sanchez area. These are mostly hard gravels, but a fair amount of weathered, soft sandstone is contained, while there is also a high content of flat pieces of shale showing fissibility. The ICEL report points out that the possibility exists of sulfates being contained in the deposit at this area in excess of allowable limits.

At the Quebrada La Borrasca area, the deposit consists of subrounded and/or subangular gravels of basalt, diabase, and conglomerate, which are mostly hard. The estimated available volume of aggregate, according to ICEL, is 15,000 m^3.

Table-10.8 Gradation of Concrete Aggregate

Sampling Spot	Trench or Pit Name	Coarse Aggregate (%)				Fine Aggregate (%)							Gradation (%)	
		76.2-38.1mm	38.1-19.0mm	19.0-9.51mm	9.51-4.76mm	4.76-2.38mm	2.38-1.19mm	1.19-0.595mm	0.595-0.297mm	0.297-0.149mm	0.149-0.074mm	≦ 0.070mm	76.2-4.76mm	≦4.76mm
PUENTE SANCHEZ	AFs-13	21	19	32	27	34	18	11	16	11	5	5	62	38
	AFs-14	23	25	26	26	31	15	10	13	15	5	10	61	39
	AFs-15	25	31	26	18	20	20	13	13	10	5	18	61	39
QUEBRADA LA BORRASCA	AFb-1	25	29	24	22	20	20	14	12	8	6	18	51	49
	AFb-2B	33	26	21	20	53	26	5	5	0	0	11	81	19
	AFb-5	24	34	28	14	19	19	19	19	12	2	10	58	42
	AFb-6	8	31	29	31	24	23	18	16	6	3	10	28	62
	AFb-7	12	21	33	33	40	28	14	7	2	2	7	57	43

Considering these above investigation results, it will not be possible to economically produce high-quality concrete with aggregate from the Puente Sanchez area, as the quantity available is small, and therefore, this area is unsuitable as the principal source of aggregate. As for the aggregate of the Quebrada La Borrasca area, despite the estimated available volume being 15,000 m^3, the transportation distance is 1 or 2 km from an existing highway and more than 20 km distant from the El Siete No.1 damsite, resulting in a problem of economics.

(2) Rock Quarry for Concrete Aggregate

As stated, it is judged impossible from the standpoint of both quantity and economy to produce a large volume of high-quality concrete using natural aggregates.

In view this, the JICA Survey Mission made an approximate investigation of prospective rock quarries for concrete aggregate within a radius of about 10 km centered at the El Siete No.1 dam site.

As a result of these reconnaissances, a massive ridge located at the left bank approximately 1.5 km downstream of the El Siete No.1 dam site estimated to have a distribution of basalt which is hard, fresh and has few cracks and to have only thin overburden, was selected as a prospective quarry area (the Quebrada Copo area).

Ravines at this prospective quarry site have exposures of basalt which is fresh and hard, but has cracks at spacings of 5 to 50 cm.

JICA performed tests in Tokyo of the following items using rock samples collectd from outcrops at the prospective concrete aggregate quarry area. The sample locations are given in Appendix-III-6.

a. Petrographic observations by microscope
b. Identification of clay minerals by X-ray analysis
c. Physical properties of rock (specific gravity, absorption)

These tests were performed on three samples collected, and the results given in Table-10.9.

As a result of the tests, all three samples had bulk specific gravity on saturated surface-dry basis between 2.85 and 2.94 and absorption between 0.7 and 1.16, both being satisfactory for concrete aggregate. Expanding clay minerals or minerals liable to show alkali-aggregate reaction harmful to concrete were not discovered in petrographic observations by microscope and X-ray analysis.

(3) Geological Engineering Assessments

The natural aggregate distributed in this area is small in quantity, while there is also a possibility of problems occurring with regard to quality, and therefore, even if this aggregate is to be used, it would be in concrete for temporary facilities. In the event of use as aggregate for the project proper, it will be necessary to carry out specific gravity, absorption, soundness, abrasion loss tests.

Since high quality concrete is required for important structures such as the El Siete No.1 dam, rock collected from a quarry would be crushed and used. The mountain at the left bank 1.5 km downstream of the No.1 dam site, the Quebrada Copo area is promising as a prospective quarry site, and it will be necessary to confirm the available quantity and quality through drilling at this site with several drillholes to determine the condition of the rock with respect to weathering, hardness, and cracking, and also to perform soundness, abrasion loss, and unconfined compression tests of the rock.

Table-10.9 Physical Properties of Basalt in Quarry Site (Quebrada Copo Area)

Sample No.	Rock Name	Bulk Specific Gravity (S.S.D.)	Bulk Specific Gravity (Dry)	Absorption (%)	Microscopic Observation and X-ray Analysis
Q - 1	Augite meta-basalt	2.94	2.92	0.83	No trace of harmful mineral for concrete
Q - 2	ditto	2.95	2.92	1.16	
Q - 3	ditto	2.85	2.83	0.70	

10.6 Probability Analysis on Seismic Hazard at the Project Site

10.6.1 Seismicity Data

Seimicity data used in this study are based on those retrieved from 'The Earthquake Data File' compiled by NOAA (National Oceanic and Atmospheric Administration Environmental Data Service). Total number of the data amounts to 2711, covering a period from 1929 to 1981. In Fig. 10.6.1 are plotted all the data used in the study, whose epicentral distances from the Atrato project (5°51'N, 76°15'W) are smaller than 1,000 km. Numbers of the data in each year during the period are shown in Table-10.6.1, together with accumulative numbers from 1929. General aspects of the data such as magnitude and epicentral distance can be seen in Table-10.6.2 and also in Figs. 10.4.2 - 10.11.

10.6.2 Attenuation Models

Of previously proposed attenuation models which express peak acceleration, A (gal), in terms of earthquake magnitude, M, and hypocentral distance, R (km), or epicentral distance, D (km), five models shown below are used in this study.

$$\log A = 3.090 + 0.347M - 2 \log (R+25) \quad (1)$$

proposed by C. Oliveira[1].

$$\log A = 2.674 + 0.278M - 1.301 \log (R+25) \quad (2)$$

proposed by R. K. McGuire[2].

$$\log A = 2.041 + 1.842M - 1.6 \log D \quad (3)$$

proposed by L. Esteva and E. Rosenblueth[3].

$$\log A = 2.308 + 0.411M - 1.637 \log (R+30) \quad (4)$$

proposed by T. Katayama[4].

$$\log (A/640) = (D+40)(-7.6+1.724M-0.1036M^2)/100 \quad (5)$$

proposed by S. Okamoto[5].

For all the data described earlier, peak accelerations were calculated by using the above attenuation models, and maximum accelerations in each year - long interval were found to be as shown in Table-10.12.

10.6.3 Statistical Analysis of Maximum Accelerations

Using the maximum acceleration values estimated for each year during the period from 1929 to 1981, a probalistic model was established on the basis of the "Theory of Extreme Values" by setting an equal time interval to one year.

Although a probability function of the maximum acceleration expected at the project site is not known, it is reasonable to suppose that the function should be associated with the third type asymptotic distribution defined by

$$P(x) = \exp[-[(w-x)/(w-u)]^{k}] \qquad (6)$$

where w is an upper limit of a variable, k is a shape parameter, u is a characteristic value, and x is a random variable taken as logarithm of the maximum acceleration during a year-long interval, expressed as

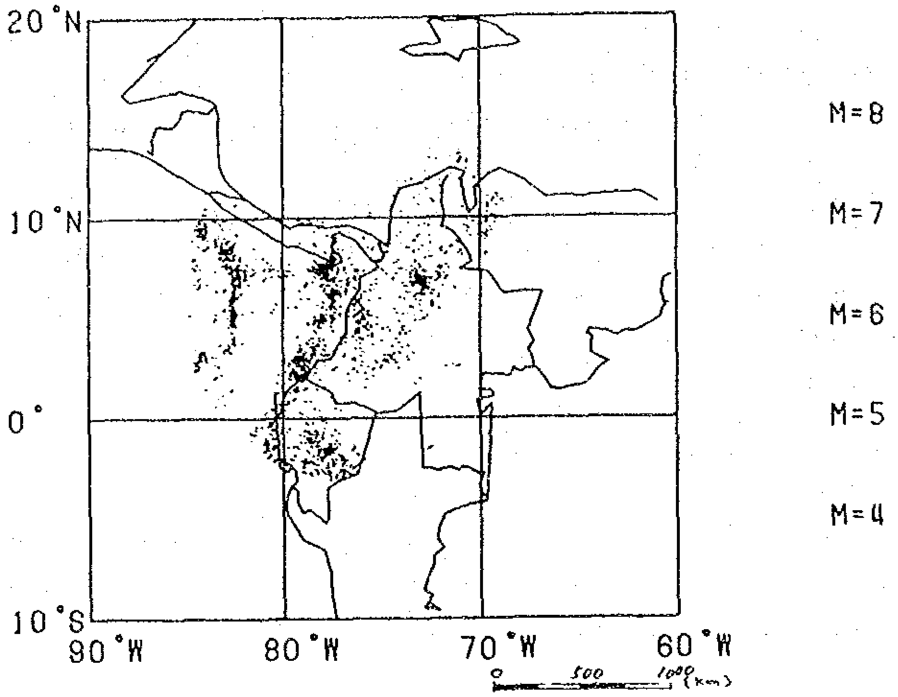
$$x = \log A_{max} \qquad (7)$$

The previously mentioned maximum acceleration values are plotted in Figs. 10-12 - 10-16. Plotting position of each maximum value was calculated by

$$p(m) = (N-m+1)/(N+1) \qquad (8)$$

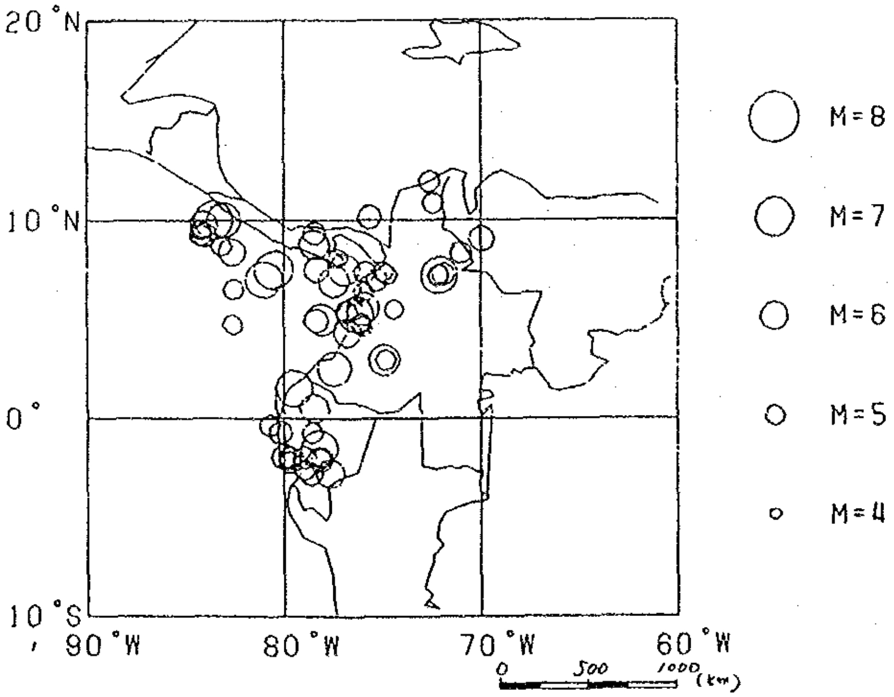
where N (=53) is the total number of the time interval and m is the order of the value from the largest one. In these figures, regression curves estimated for the third asymptotic distribution function are also shown by solid lines, from which the maximum acceleration for any return period can be evaluated. Table-10.13 shows the maximum accelerations expected at the site for different four return periods of 50, 100, 200 and 500 years.

Fig. 10-4 Seismicity of All Data in 1929 - 1981



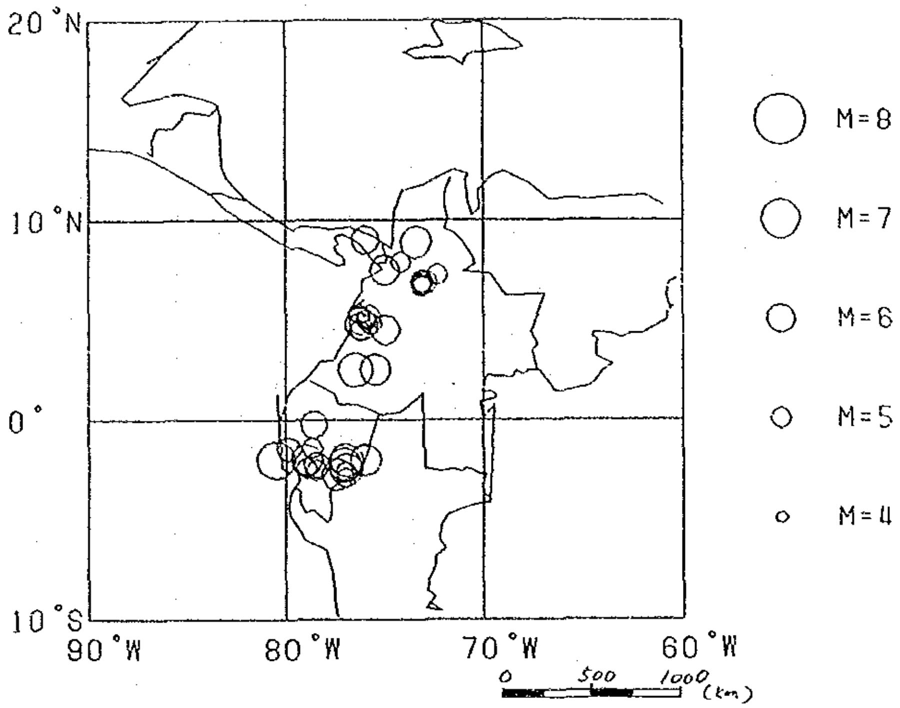
Total Number of Plots in the area of Δ ≦ 1000.0 (km) is 2407.

Fig. 10-5 Distribution of Focul Depth 50 ≦ D < 100 km (M≧5) in 1929 - 1981



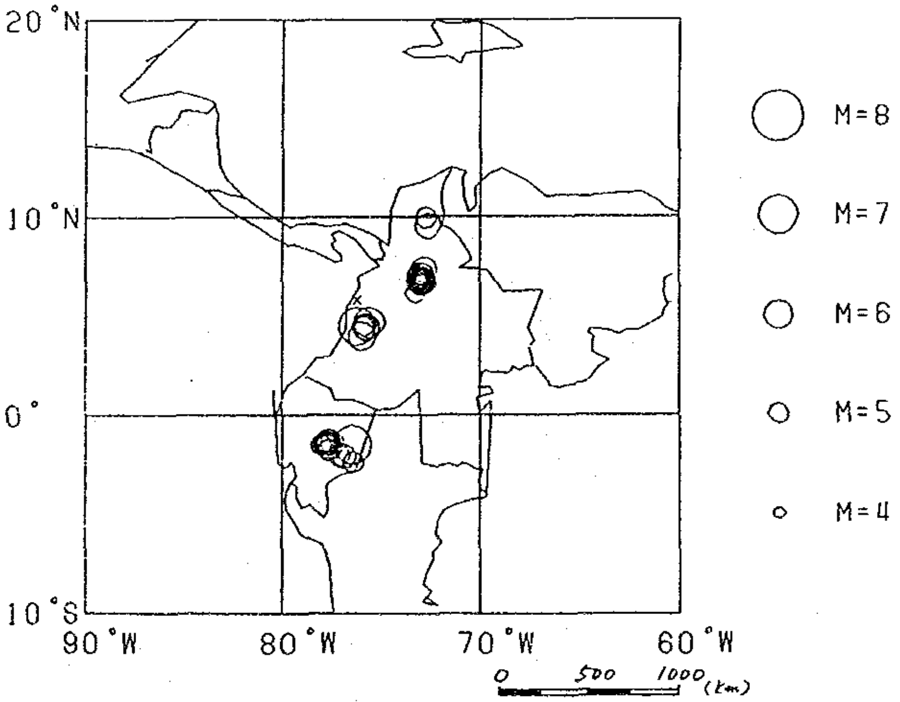
Total Number of Plots is 59.

Fig. 10-6 Distribution of Focul Depth 100 ≦ D < 150 km (M≧5) in 1929 - 1981



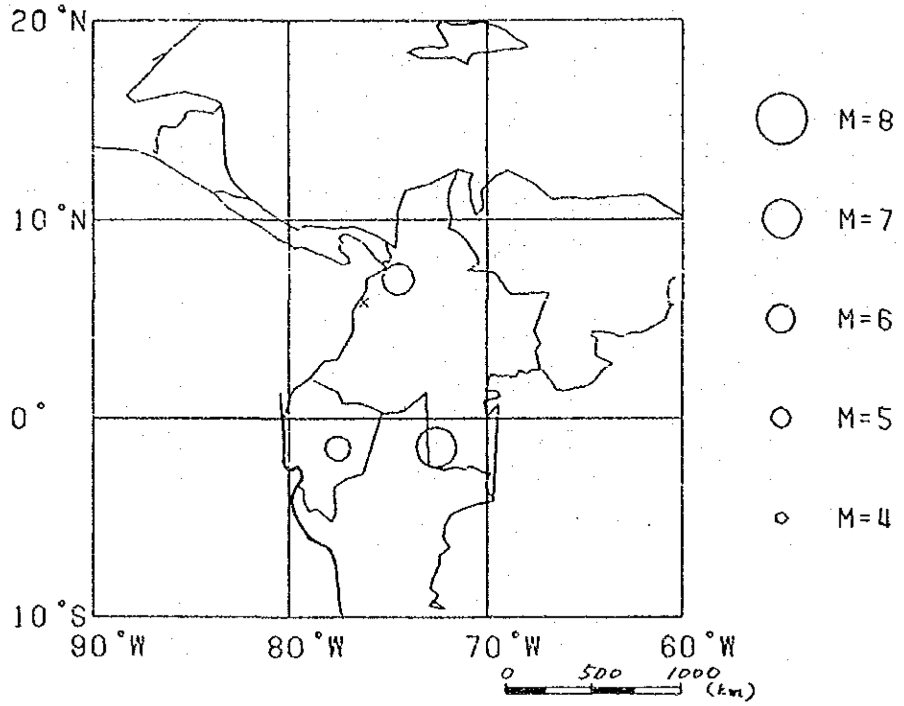
Total Number of Plots is 36.

Fig. 10-7 Distribution of Focul Depth 150 ≦ D < 200 km (M≧5) in 1929 - 1981



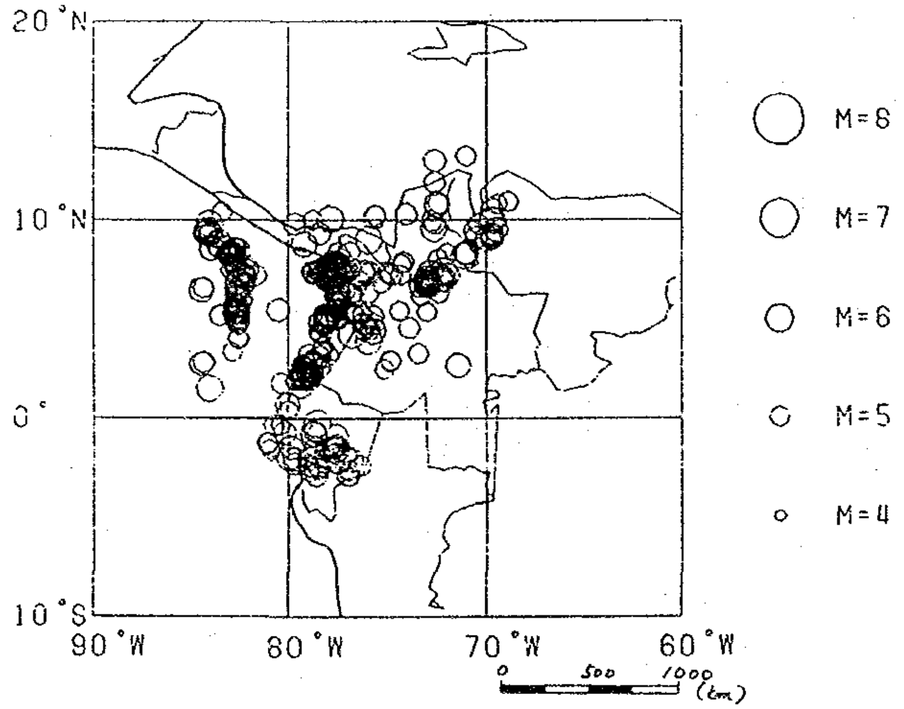
Total Number of Plots is 84.

Fig. 10-8 Distribution of Focul Depth D ≥ 200 km (M≥5) in 1929 - 1981



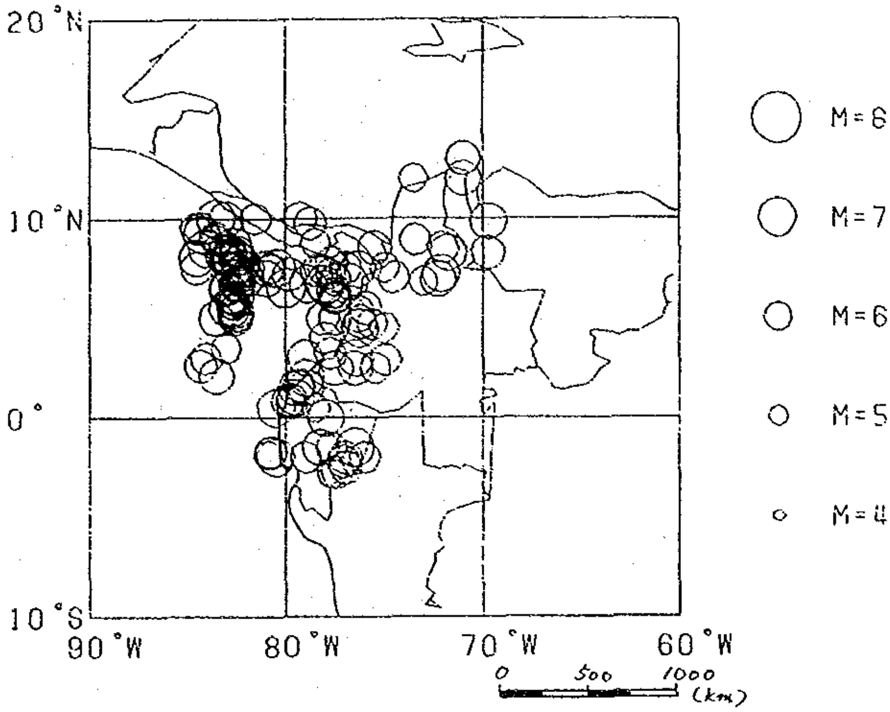
Total Number of Plots is 3.

Fig. 10-9 Seismicity of Magnitude 5≤M<6 in 1929 - 1981



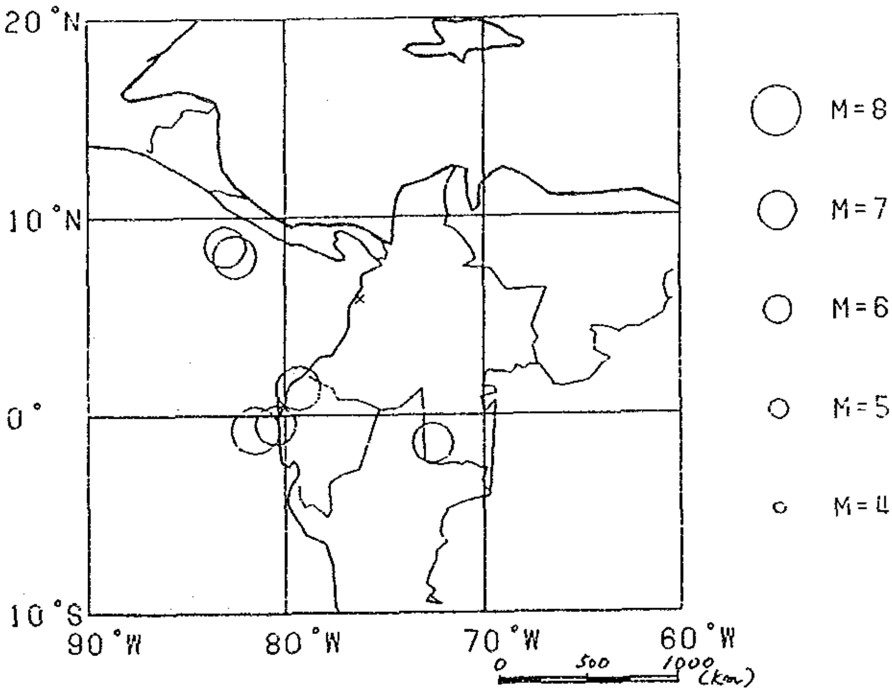
Total Number of Plots is 407.

Fig. 10-10 Seismicity of Magnitude 6≦M<7 in 1929 - 1981



Total Number of Plots is 142.

Fig. 10-11 Seismicity of Magnitude 7≦M<8 in 1929 - 1981



Total Number of Plots is 6.

Fig. 10-12 Maximum Acceleration [Oliveira, C Eq.(1)]

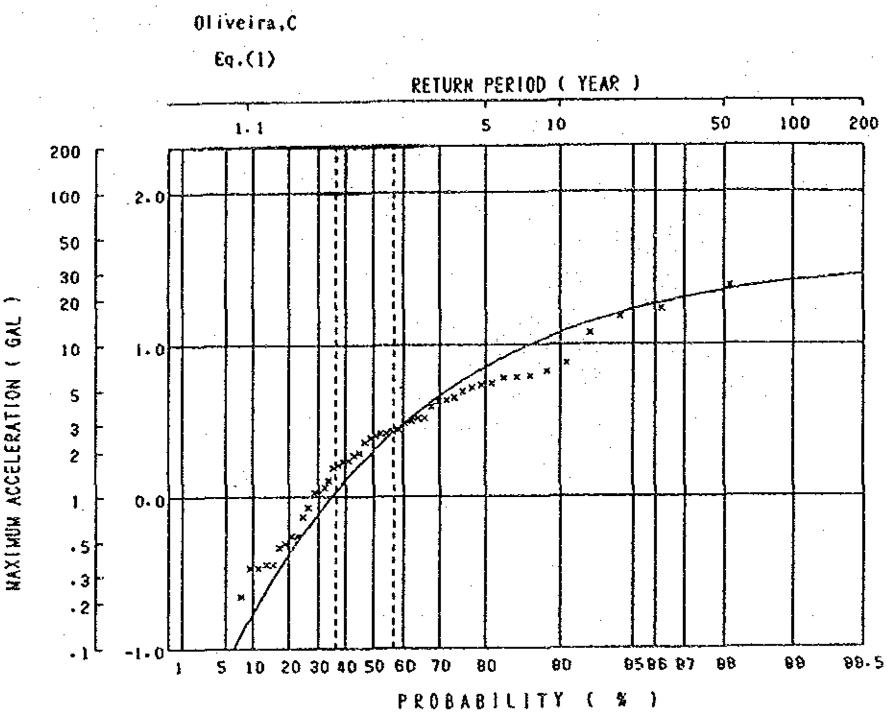


Fig. 10-13 Maximum Acceleration [McGuire, R.K. Eq.(2)]

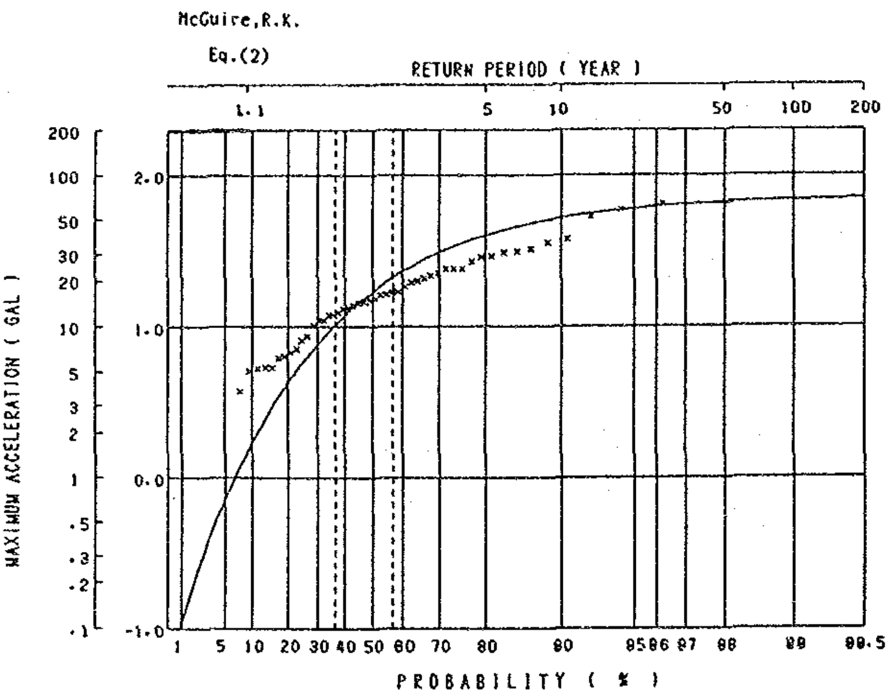


Fig. 10-14 Maximum Acceleration [Esteva, L. α Rosenblueth, E. Eq.(3)]

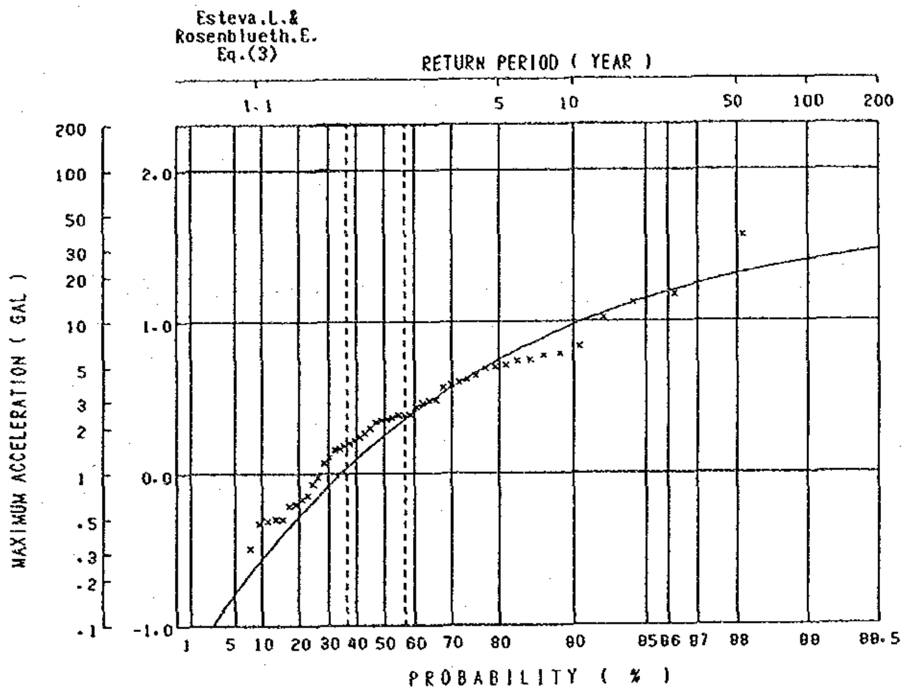


Fig. 10-15 Maximum Acceleration [Katayama. T. Eq.(4)]

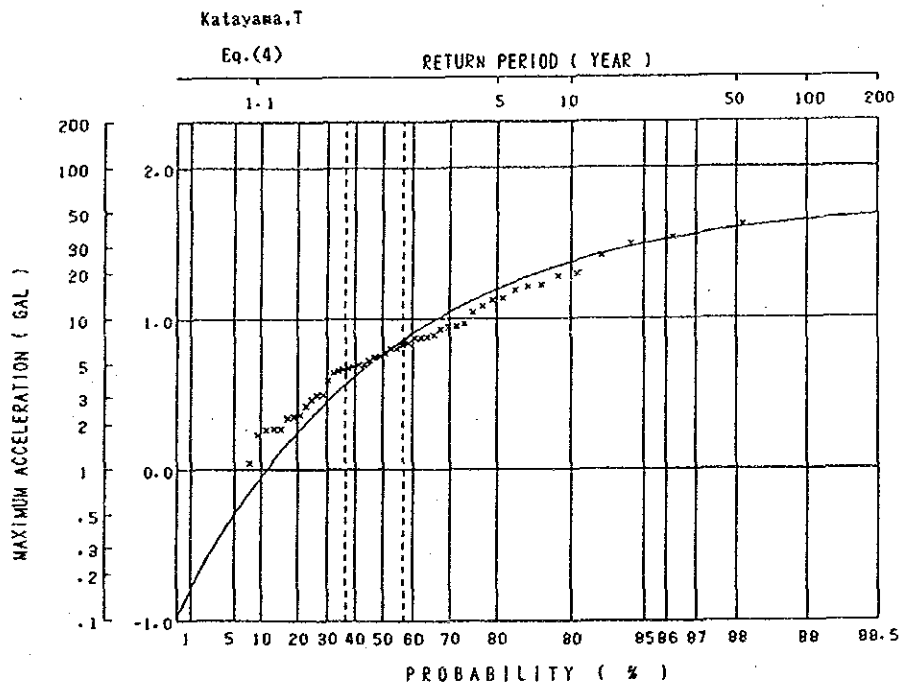


Fig. 10-16 Maximum Acceleration [Okamoto. S. Eq.(5)]

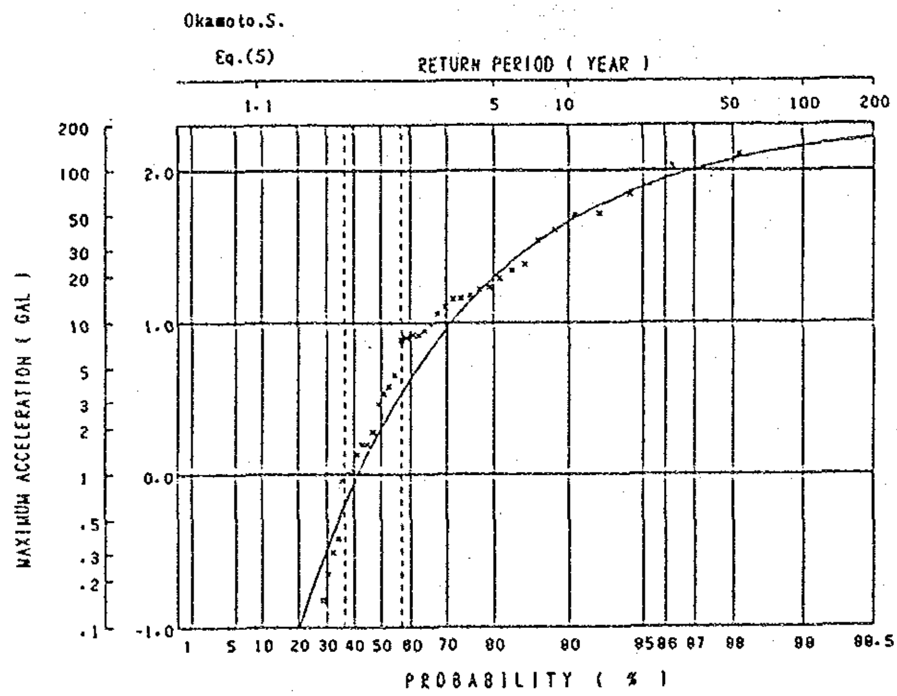


Table-10.10 Number of Earthquakes in a year during the period from 1929 to 1981

Year	N	Sum of N	Year	N	Sum of N
1929	2	2	1958	15	124
1930	3	5	1959	3	127
1931	3	8	1960	6	133
1932	3	21	1961	6	139
1933	10	21	1962	8	127
1934	8	29	1963	104	251
1935	6	35	1964	129	380
1936	0	35	1965	159	539
1937	5	40	1966	151	690
1938	2	42	1967	146	836
1939	3	45	1968	100	936
1940	2	47	1969	74	1010
1941	5	52	1970	113	1123
1942	4	56	1971	73	1196
1943	8	64	1972	66	1262
1944	3	67	1973	132	1394
1945	3	70	1974	185	1579
1946	0	70	1975	103	1682
1947	0	70	1976	170	1852
1948	1	71	1977	83	1935
1949	2	73	1978	79	2014
1950	1	74	1979	212	2226
1951	3	77	1980	137	2363
1952	11	88	1981	44	2407
1953	0	88			
1954	5	93			
1955	1	94			
1956	8	102			
1957	7	109			

Table-10.11 Distribution of Magnitude and Epicentral Distance of the Seismicity Data

	0 ≦ < 50	<100	<200	<300	<400	<500	<600	<700	<800	≦1000	Total
3.0 ≦ M < 3.5	0	0	0	0	3	1	0	0	0	0	4
< 4.0	0	5	9	6	59	11	5	4	7	35	141
< 4.5	5	10	43	74	255	54	30	41	64	192	768
< 5.0	1	8	73	137	234	79	44	59	100	204	930
< 5.5	0	6	24	56	78	24	17	21	42	54	322
< 6.0	0	1	8	10	11	6	6	9	13	21	85
< 6.5	1	0	9	10	4	3	5	7	19	19	77
< 7.0	0	1	4	2	5	4	6	14	10	19	65
< 7.5	0	0	0	0	0	0	0	1	1	3	5
< 8.0	0	0	0	0	0	0	0	0	0	1	1
Total	7	31	170	295	649	182	113	156	256	548	2407

Δ : Epicentral Distance (km)
M : Magnitude

Table-10.12 Maximum Accelerations during a year from 1929 to 1981

	Oliveira,C Eq.(1)	McGuire,R.K. Eq.(2)	Esteva,L. & Rosenblueth,E. Eq.(3)	Katayama,T Eq.(4)	Okamoto,S. Eq.(5)
1929	.3	5.1	.5	1.7	.0
1930	1.6	13.2	1.6	5.3	2.9
1931	5.6	30.0	5.1	15.4	14.4
1932	3.3	20.6	3.1	8.9	3.8
1933	.5	6.7	.7	2.3	.0
1934	2.0	15.2	2.0	6.3	1.1
1935	15.1	58.5	13.4	33.9	127.4
1936	.0	.0	.0	.0	.0
1937	1.3	12.0	1.4	5.0	.4
1938	5.2	30.9	5.0	16.8	40.3
1939	.8	8.7	.9	3.1	.0
1940	.2	3.7	.3	1.1	.0
1941	5.5	28.7	4.9	13.3	11.6
1942	2.3	16.9	2.3	7.6	3.4
1943	1.6	14.6	1.8	6.7	.9
1944	1.1	10.2	1.2	3.9	.1
1945	4.3	26.8	4.1	13.7	9.9
1946	.0	.0	.0	.0	.0
1947	.0	.0	.0	.0	.0
1948	.3	5.3	.5	1.9	.0
1949	.4	5.3	.5	1.8	.0
1950	.4	5.3	.5	1.8	.0
1951	1.1	11.1	1.3	4.7	.2
1952	7.6	37.3	6.8	19.8	24.0
1953	.0	.0	.0	.0	.0
1954	.5	6.2	.6	2.2	.0
1955	.5	7.1	.7	2.6	.0
1956	6.8	35.3	6.2	18.8	21.8
1957	11.9	52.0	10.6	31.1	51.5
1958	1.2	12.4	1.5	5.9	.3
1959	.5	6.4	.6	2.2	.0
1960	3.9	23.9	3.7	11.1	8.0
1961	.7	8.1	.9	2.9	.0
1962	17.1	65.8	15.1	41.3	106.3
1963	1.7	11.0	1.5	3.1	1.3
1964	2.5	13.8	2.2	4.6	12.7
1965	2.7	16.1	2.4	5.6	16.5
1966	2.6	14.4	2.3	4.4	17.0
1967	1.7	12.1	1.6	4.5	1.9
1968	4.5	21.5	4.0	7.5	4.6
1969	6.1	24.1	6.0	7.5	8.3
1970	24.2	63.6	36.3	26.2	50.9
1971	2.5	14.8	2.2	4.8	1.0
1972	3.1	18.3	2.8	6.9	69.7
1973	3.3	20.0	3.0	8.8	15.0
1974	4.3	22.4	3.8	8.6	8.9
1975	6.3	29.1	5.5	12.2	14.2
1976	5.0	24.2	4.4	9.3	19.4
1977	3.2	19.4	2.9	7.8	7.8
1978	2.6	16.8	2.4	6.3	1.6
1979	6.1	32.3	5.5	16.3	34.7
1980	1.9	13.3	1.7	4.9	8.4
1981	2.8	16.4	2.5	5.7	1.6

unit: gal

Table-10.13 Maximum Accelerations for Three Return Periods (gal)

Model (Eq. No.)	Proposer(s)	Return Period. Tr (year)			
		50	100	200	500
(1)	C. Oliveira	23	26	29	31
(2)	R.K. McGuire	65	68	69	70
(3)	L. Esteva & R. Rosenblueth	32	40	48	58
(4)	T. Katayama	39	44	48	51
(5)	S. Okamoto	118	142	161	179

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10.7 Conclusions and Recommendations

10.7.1 Conclusions

The geological engineering findings concerning the El Siete No. 1 and No. 2 Projects obtained in the investigations just carried out may be summarized as follows:

(1) El Siete No. 1 Regulating Reservoir

There are no topographical or geological conditions liable to pose problems concerning watertightness in the El Siete No. 1 regulating reservoir area.

Regarding the slope stabilities of the mudflow deposits distributed at the reservoir rim, the deposits would need to be made objects for study, but since the greater part would be located above the reservoir high water level, it is thought there will be little possibility of large-scale collapses occurring so long as protection is provided in certain areas.

With regard to small failures of slopes around the reservoir and in upstream areas, and the problem of sedimentation of tractional load in the reservoir, it will be necessary to consider measures against sedimentation to secure the reservoir's regulating capacity.

(2) El Siete No. 1 Dam

It will be necessary for the mudflow deposit distributed at the right-bank side to be excavated and removed for the El Siete No. 1 dam foundation. Also, for the dam foundation, it will be necessary for the weathered surface portion of the basalt constituting the basement at this site to be excavated, and, a suitable water barrier treatment (for example, curtain grouting) to be provided to secure watertightness.

Since it is possible for cut slopes of mudflow deposits upstream and downstream of the right-bank abutment of the dam to collapse as a result of water impoundment, it will be necessary to consider a stabilizing measure for these slopes.

(3) El Siete No. 1 Auxiliary Dam

It is necessary for river-bed sand-gravel, terrace deposits, and weathered surface portions of basement rock to be excavated and removed for the El Siete No. 1 auxiliary dam. With regard to the watertightness of the basement, it is considered that adequate watertightness can be secured if the weathered portion of the basement surface layer is removed, but water barrier treatment (for example, curtain grouting) may be necessary at certain sections.

(4) El Siete No. 1 Headrace Tunnel

It is estimated that basalt is distributed over approximately 80 percent of the El Siete No. 1 headrace tunnel route, with alternation mostly of sandstone and shale distributed in the approximately 20 percent remaining.

The greater part of these rocks distributed along the headrace tunnel route is expected to be fresh, hard and dense, while since there are few faults estimated to cross with the tunnel, it is thought there are few geological conditions unfavorable for tunnel excavation.

(5) El Siete No. 1 Auxiliary Connecting Tunnel

It is estimated that alternation of shale and calcareous shale and alternation of chert and limestone are distributed over approximately 50 percent, and basalt over the other 50 percent of the El Siete No. 1 auxiliary connecting tunnel route. These rocks distributed along the tunnel route are fresh, hard, and dense, and it is thought that they will not pose major problems geologically in tunnel excavation.

However, the tunnel cover is thin in the vicinity of the Qda. Sta Lucia across the tunnel route, and there is a possibility of a number of minor faults crossing with the tunnel in this vicinity so that some water springing may occur during excavation, and care will need to be exercised.

(6) El Siete No. 1 Surge Tank, Penstock, Powerhouse, and Tailrace Sites

It is expected that weathering of the basement surface has occurred to lower depths the farther down along the route of the penstock from the El Siete No. 1 surge tank.

Since strongly weathered bedrock will be exposed by cutting at the surge tank site, it will be necessary to give consideration to securing the stability of the cut slope.

It is necessary for penstock anchor block foundations to be obtained at levels deeper than the strongly weathered portions of the basement.

Fresh, hard bedrock is distributed at the powerhouse foundation and ample bearing power as a foundation for the powerhouse will be obtained. However, since slopewash and a strongly weathered portion of the bedrock will exist at the cut slope, appearing as a result of powerhouse foundation excavation, it will be necessary to be careful about slope stability.

It is estimated that fresh, hard bedrock is distributed along the entire tailrace tunnel route, and it is expected there will be few geological engineering problems encountered.

(7) El Siete No. 2 Intake Dam

The mudflow deposit at the right-bank side of the El Siete No. 2 intake dam site is unsuitable for a dam foundation and will need to be excavated and removed. It will be necessary to consider excavation and removal of the strongly weathered surface portions of the basalt and alternation of sandstone and shale comprising the basement at this site, and also provide suitable water barrier treatment (for example, curtain grouting).

(8) El Siete No. 2 Headrace Tunnel

It is estimated that alternation of sandstone and shale is distributed over approximately 10 percent, diorite over approximately 60 percent, and diabase over approximately 30 percent of the El Siete No. 2 headrace tunnel route. Most of these rocks distributed at this tunnel route are fresh and hard, and although there may be localized poor condition areas it is considered that the greater part of the tunnel will pass through good quality bedrock.

However, a schistose structure is developed in the amphibolite belt at the boundary between the diabase and diorite crossing with the tunnel and tunnelling in this section will require care to be exercised.

(9) El Siete No. 2 Surge Tank, Penstock, and Powerhouse Sites

It is possible that the diabase comprising the basement of the surge tank site is strongly weathered to a maximum depth of about 20 m from the surface, and it is necessary for this factor to be considered in designing the surge tank vertical shaft and the cut slope at the ground surface.

The diabase constituting the basement of the penstock route is estimated to be weathered to a maximum of approximately 30 m from the ground surface, but is is possible that bearing ground for penstock anchor blocks can be obtained at 6 to 7 m from the ground surface or shallower.

It is thought that the powerhouse foundation will be amphibolite which is weathered in parts but possessing more or less ample bearing power. Regarding the stability of the slope behind the powerhouse, it is necessary for care to be exercised because joints dipping toward the river and a schistose structure exist in the basement.

(10) Concrete Aggregate

The natural aggregate distributed in this area is small in quantity, while there is also the possibility of problems arising regarding quality. Therefore, even if this aggregate is to be used, it would be in concrete for temporary facilities.

Since high quality concrete is required for important structures such as the El Siete No. 1 dam, rock collected from a quarry would be crushed and used. The mountain at the left bank 1.5 km downstream of the No. 1 dam site is promising as a prospective quarry site (Quebrada Copo Area).

10.7.2 Recommendations

The conclusions given herein are based on preliminary investigations. To proceed hereafter to the stage of definite design, it is thought further investigations such as those shown below are necessary at the individual projected structure sites, and it is recommended they be implemented.

a. Geological mapping of individual structure sites utilizing highly accurate topographical maps (scale about 1/500 to 1/1,000).

b. Geological mapping of waterway routes utilizing highly accurate topographical maps (scale about 1/5,000).

c. Drillings and adits shown in Table-10.14 and Dwg.-50.

Table-10.14 Proposed Additional Subsurface Investigation Works

Project	Site		No.	Length (m)	Direction	Remarks
EL SIETE NO.1	Auxiliary Dam and Sedimen-tation Baisn	Drillhole	AD-101	20	Ver.	Water pressure test should be performed
			102	20	Ver.	
			103	25	Ver.	
			104	20	Ver.	
			105	20	Ver.	
			106	15	Ver.	
			107	15	Ver.	
			Total: 7 holes 135m			
	Dam	Drillhole	AD-10	50	Ver.	Water pressure test should be performed.
			11	30	45° N18°E	
			12	40	Ver.	
			13	30	Ver.	
			14	40	Ver.	
			15	40	Ver.	
			Total: 6 holes 230m			
		Adit	AA-1	50	N20°E	Plate loading test and rock shear test should be performed.
			-2	30	S20°W	
			Total: 2 adits 80m			
	Surge Tank	Drillhole	AD-201	60	Ver.	Standard penetration test should be performed.
	Penstock		202	15	Ver	
			203	15	Ver.	
			204	15	Ver.	
			205	20	Ver.	
	Powerhouse		206	40	Ver.	
			207	20	Ver.	
	Surge Tank – Powerhouse		Total: 7 holes 185m			

Project	Site		No.	Length (m)	Direction	Remarks
EL SIETE NO.2	Intake Dam	Drillhole	BD-101	20	Ver.	Water pressure test should be performed.
			102	30	45°N80°E	
			103	25	Ver.	
			104	50	Ver.	
			105	45	Ver.	
			106	20	Ver.	
			Total: 6 holes 190m			
	Surge Tank	Drillhole	CD-3	60	Ver.	Standard penetration test and lateral loading test should be performed.
	Penstock		4	15	Ver.	
			5	15	Ver.	
			6	15	Ver	
			7	20	Ver.	
	Power-house		8	20	Ver.	
			9	20	Ver.	
	Surge Tank – Penstock		Total: 7 holes 165m			
	Penstock	Pit	BT-15	5	Ver.	Plate loading test should be performed.
			16	10	Ver.	
			Total: 2 pits 15m			
NO.1 AND NO.2	Quarry Site	Drillhole	QD-1	50	Ver.	
			QD-2	30	Ver.	
			QD-3	30	Ver.	
			Total: 3 holes 110m			

CHAPTER II. STUDIES OF DEVELOPMENT TYPE AND DEVELOPMENT SCALE

CHAPTER 11 STUDIES OF DEVELOPMENT TYPE AND DEVELOPMENT SCALE

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FIGURE LIST

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CHAPTER 11. STUDIES OF DEVELOPMENT TYPE AND DEVELOPMENT SCALE

11.1 Basic Principles and Basic Conditions

11.1.1 Introduction

The basic development concept for the Atrato River Hydroelectric Power Development Project is proposed as the El Siete No. 1 and No. 2 projects in the Master Plan Report. The scheme is for the construction of a dam on the Atrato River in the vicinity of the confluence with the tributary Toro River to provide a regulating pond, drawing water at the left-bank side of the dam. The water is to be conducted by a headrace to a No. 1 powerhouse provided at the left-bank side of the Atrato River at point upstream of Girardot Gauging Station to generate electric power. After power generation, the discharge is to flow directly into the headrace of the No. 2 power station combining the inflow from the remaining catchment area drawn at the No. 2 intake dam constructed on the Atrato River immediately upstream of the No. 1 powerhouse with the discharge from the No. 1 powerhouse. This water is to be conducted by a headrace to the No. 2 powerhouse to be provided at the right bank of the Atrato River at point downstream of the confluence with the Piñon River to generate electric power.

Further downstream, although not included among the objects of investigation this time, there are plans for an El Once Power Station to be constructed.

Upon examination of the present power generation scheme, it was decided to carry out the studies below using the results of hydrological, meteorological, topographical, and geological investigations based on the described basic development concept implemented after preparation of the Master Plan Report.

- Comparison studies of regulating pond type run-of-river type, and reservoir-type development schemes

- Sedimentation countermeasures in regulating pond type development

- Selection of El Siete No. 1 dam axis

- Selection of El Siete No. 1 dam height

- Study of powerhouse location and headrace route of each power station

- Study of maximum available discharge of each power station

Since the basic development concept of this scheme is for both the No. 1 and No. 2 projects to be regulating pond type peak power stations with daily regulating capacity given the dam of the El Siete No. 1 Project, it was decided that the two power stations would be put together and evaluated as one project.

11.1.2 Basic Conditions

(1) Project Area

In the basic development concept of the Master Plan, the project area is to be a section of approximately 18.4 km on the main stream of the Atrato River from a point downstream of the confluence with the tributary Toro River at El. 1,450 m, and a point downstream of the confluence with the tributary Rio Piñon at El. 687 m.

The average river-bed gradient in this section is 1/24, indicating a swift stream, but upstream of this is an average of approximately 1/60, and downstream approximately 1/50, showing comparatively gentle gradients.

Other than the previously mentioned point downstream of the Toro River confluence, there is no suitable site for a reservoir or regulating pond.

Consequently, regardless of the type of development, to expand the project area further upstream or downstream from the basic development concept would greatly impair the Project's economics.

Therefore, in this chapter's study, the project area is to be identical to the area of the basic development concept in the Master Plan. This area is outside the range of influence of the El Once project cited as a downstream development site in the Master Plan.

(2) Sedimentation Inside Regulating Pond

As described in Paragraph 5, Chapter 8, a sediment inflow of 1.6 x 10^6 m^3 is expected annually when a regulating pond or reservoir is provided at the El Siete No. 1 dam site on the Atrato River. This volume of sediment is enough to bury the regulating pond even before one year has elapsed.

It was therefore decided to consider a system incorporating a regulating pond which would flush out deposited sediment.

(3) Firm Discharge and Maximum Available Discharge

A firm discharge was selected as the discharge condition in making a comparison study of the development project.

The firm discharge flowing in at the El Siete No. 1 dam site is 12.3 m^3/s determined by catchment area ratio from runoff data of Sanchez Gauging Station located immediately upstream, while at the El Siete No. 2 dam site it is 14.3 m^3/s.

Firm discharge is the discharge secured for 95 percent of the year (346 days), and is the probable discharge which is the basis for formulating hydro project plans at ICEL and ISA.

The maximum available discharges of the El Siete No. 1 and No. 2 projects were based on discharges permitting operation at 50 percent load factor applying firm discharges to system load, in the case of a regulating pond type, discharges corresponding to double the firm discharges.

These values are applied in common to all alternatives in the study of development type, but in the case of a reservoir type, the firm discharge applied is the above-mentioned discharge plus the discharge supplemented from the reservoir.

(4) Evaluation Criteria for Determining Optimum Scheme

The evaluation criterion in determining the scale of development in the case of the precondition that a hydroelectric power station will be built is described.

Although the financial and economic evaluations of this project described in Chapter 16 are one measure for evaluation, it is not a means to measure the optimum utilization of water power resources at the project site, and comparisons with projects in sectors other than electric power are made, with market interest rates and with the opportunity cost of capital.

In general, economic evaluation of a hydroelectric power station is done comparing the product of the project-electricity-with the cost of producing electricity with an alternative power generating facility having the same supply reliability, and when the cost of the hydroelectric project is lower, the hydroelectric project is considered to be economically feasible.

Regarding the alternative power generating facility, the present state of the electric power system and the future power supply structure are considered, and it is important to establish the evaluation criterion assuming a power generating facility having the possibility of being constructed as a truly alternative facility where the project under study did not exist.

The Survey Mission investigated the present state of Colombia's electric power system in this sense and selected the Cartagena coal-fired thermal power facility scheduled to be developed as the alternative thermal power generating facility.

The feasibility of the Cartagena coal-fired thermal power project has already been established by CORELCA and may be considered the most reasonable evaluation criterion as an alternative facility for a hydroelectric power development project.

In designating this Cartagena coal-fired thermal power generation project as the alternative power generation facility, correction factors to bring it to the same power supply reliability level as a hydroelectric project were considered and as a result the following were obtained:

1) Preconditions (Assuming Coal-Fired Thermal of 300-MW Unit)

- Construction cost per kW (1984 chronological table price)	:	US$897.5
- Service life	:	30 yr

- Station service ratio	:	6%
- Fault outage ratio	:	5%
- Periodic outage ratio	:	10%
- Operation and maintenance cost (excluding fuel cost)	:	3%
*- Related 220-kV transmission line	:	US$66/kW/km
- Coal price delivered at Cartagena Power Station	:	US$47.0/ton
- Calorific value of coal	:	6,900 kcal/kg
- Thermal efficiency at power station outlet	:	35.0%
- Plant utility factor	:	70%
- Annual energy production	:	1,840 kWh
- Discount factor, i	:	12%/yr

* Cartagena-Ternera-Sabana Larga 86.4 km

2) KW and KWh Values

The evaluation criteria used to decide hydroelectric power plant scale are as follows:

kW : US$154.7/kW

kWh : US mill 22.3 kWh (firm)

US mill 18.4 kWh (secondary)

Note : See Appendix-IV for calculation results

It will suffice for the point that should be observed in evaluating a hydro resource at this project site to be only the evaluation of the value of electricity, which is the product of the project, and additional benefits such as flood control and irrigation need not be considered. The evaluation criterion for this Project is that the optimum scale will be the scale of a hydro project where benefit of the hydro resource as seen through cost comparison with the alternative thermal facility will be maximum.

In this sense, it will be appropriate for the optimum scale to be decided by the maximum benefit (B-C) deducting cost from benefit, and not evaluation by benefit-cost ratio (B/C).

11.2 Study of Development Type

11.2.1 Comparisons of Regulating Pond Type, Run-of-River Type and Reservoir Type

(1) Regulating Pond Type Scheme

The scheme is for the regulating pond created by El Siete No. 1 Dam to be given the capacity for daily regulation, providing firm peak operation.

The regulating pond capacity to regulate 12.3 m^3/s firm discharge described in 11.1 above, will be 540 x 10^3 m^3.

The high water level with which this capacity can be secured would be at El. 1,450 m, with the low water level at 1,440 m. The dam height for this purpose will be 55 m, and crest length 207 m.

The El Siete No. 1 auxiliary dam described in 11.2.2, would be combined with this scheme. The purpose of the auxiliary intake dam is for use when flushing out sediment deposited at the El Siete No. 1 dam. Discharge for power would be intaked at the auxiliary intake dam for about 90 days annually.

The outlines of the regulating pond type scheme are given in Table-11.1.

(2) Run-of-River Type Scheme

In case of a run-of-river type scheme, the El Siete No. 1 regulating pond type scheme would be eliminated, and the No. 1 auxiliary intake dam would serve as the intake dam for the run-of-river type scheme.

The intake water level can therefore be at El. 1,460 m, 10 m higher than in the regulating pond type scheme. For inflow intake from the tributary Toro River and Santa Lucia Ravine, it will be necessary to provide a Toro River branch waterway and a Santa Lucia intake waterway, which are unnecessary in the case of the regulating pond type scheme.

Maximum available discharge would be 25 m^3/s when determined from the discharge for an annual plant utility factor of approx. 80 percent.

Regarding the handling of sediment inflow, a sand flush gate would be provided at a crest of the intake dam to flush away sediment deposited in front of the intake.

The outlines of the run-of-river type scheme are given in Table 11.2.

(3) Reservoir Type Scheme

In view of the prevailing topography and geology, the same dam axis for the regulating pond type scheme would be selected when El Siete No. 1 Dam is to be for a reservoir type scheme.

The discharge duration at the projected site is comparatively stable as described in Chapter 8. Were a reservoir to be provided, there would be fewer opportunities to discharge from the spillway so that it must be judged that the greater part of the sediment inflow will be trapped in the reservoir.

Accordingly, it was assumed that an annual sediment inflow of 1,566 x 10^3 m^3 would be deposited during the entire 50-year service life considered for a hydro power station, to amount to 78 x 10^6 m^3, and the low water level was taken to be at El. 1,548.00 m, considering the static draft head of the intake at the sediment surface.

The following two cases were studied based on the preconditions described.

1) Scheme for Physically Maximum Height Dam in View of Topographical and Geological Conditions of Damsite

This is a scheme for high water level at El. 1,580 m, with available drawdown of 32 m and an effective storage capacity of 99 x 10^6 m^3.

This storage capacity would enable firm discharge to be increased to 17.5 m^3/s and consequently, maximum available discharge would be 35 m^3/s.

The maximum available discharge supplement duration of the reservoir would be 33 days.

The dam height would be 188 m, the dam volume 3.2 x 10^6 m^3, therefore, volume of effective storage capacity of the reservoir is 30.9 times in comparison with the volume of the dam concrete.

2) Scheme for Effective Storage Capacity of 36.3 x 10^6 m^3 Corresponding to 5 Percent of Annual Inflow

5 percent is the minimum value at which efficiency as a reservoir can be expected, and the high water level in this case would be at El. 1,561 m.

The firm discharge would be 15 m^3/s and the maximum available discharge 30 m^3/s.

The continuous maximum available discharge supply of the reservoir would be 14 days.

The dam height would be 166 m, the dam volume 2.4 x 10^6 m^3, therefor effective storage capacity of the reservoir is 15 times in comparison with the volume of the dam concrete.

In both scheme, the hamlet of El Siete would be submerged, while it would also be necessary to relocate approximately 10 km of the national road.

There are also various other problems such as the risk of collapses at terraces formed by river deposits and hills around the reservoir due to seepage from the reservoir. The outlines of these two cases of the reservoir schemes are given in Table-11.3.

(4) Comparison Results

In comparisons regarding the economics of the daily regulating pond, run-of-river, and reservoir type schemes, the energy cost (overall) with the daily regulating pond type will be USmill28/kWh with the annual surplus benefit (B-C) of US$17,434 thousand and the benefit-cost ratio (B/C) of 1.558.

In comparison, the energy cost with the run-of-river type will be USmill23/kWh with the annual surplus benefit (B-C) of US$9,792 thousand and the benefit-cost ratio (B/C) of 1.384.

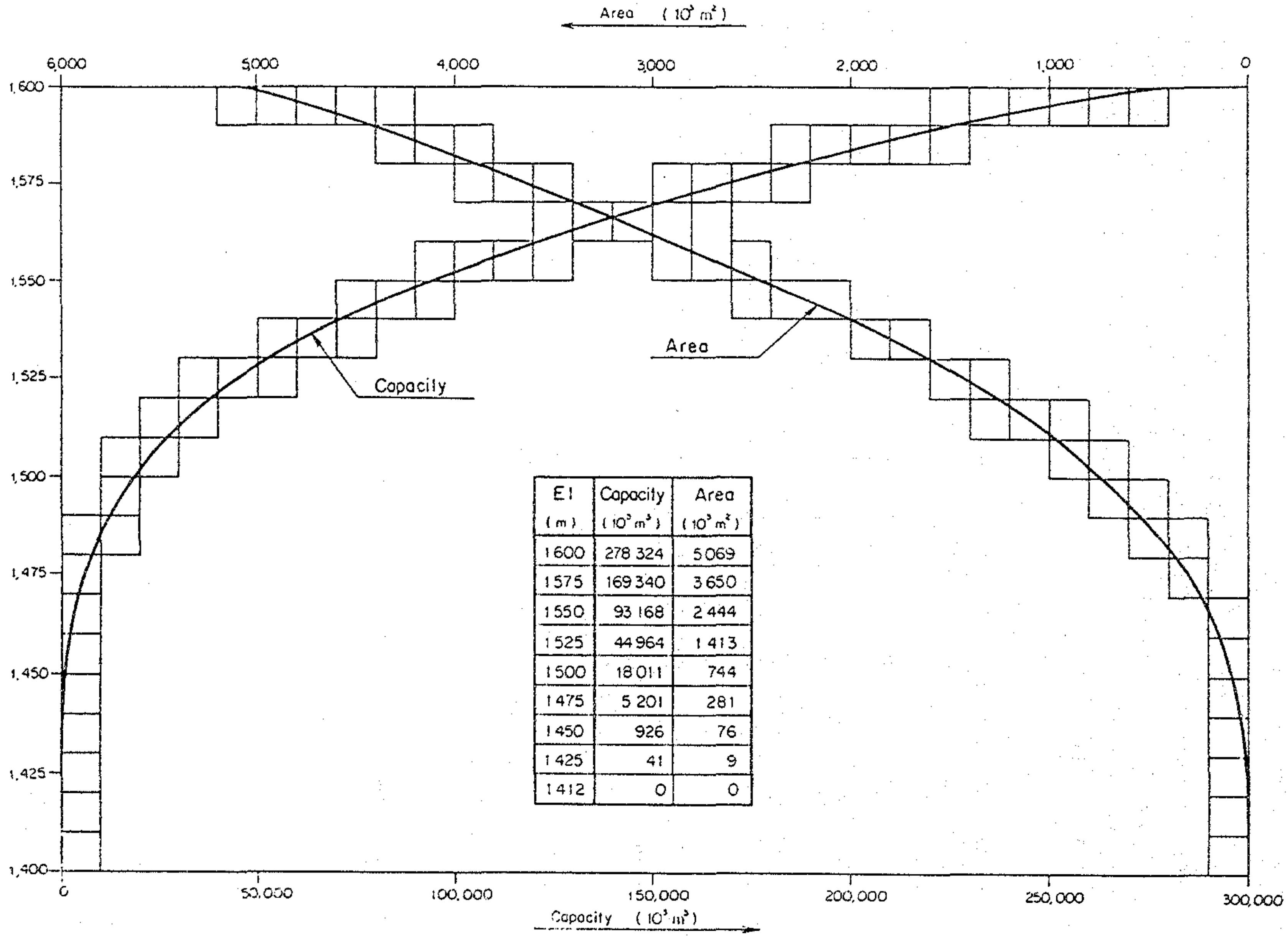
From the basis of these results, the energy cost in the case of the run-of-river type is USmill23/kWh, seemingly cheaper than daily regulation, but secondary energy would be increased because of the lack of regulating capacity, while the kW value for only firm discharge can be counted on (50 percent reduction compared with regulating pond type) thereby reducing benefit (B). Hence, the economics in terms of the annual surplus benefit (B-C) and the benefit-cost ratio (B/C) would be inferior compared with the regulating pond type.

With the reservoir type, the energy cost will be USmill63/kWh with a dam height of 188 m and USmill54/kWh with a dam height of 166 m for a benefit-cost ratio (B/C) of 0.792 and 0.864, respectively, indicating that the costs would be higher than for the alternative thermal.

On overall judgment of these points, the regulating pond type scheme should be adopted as the development type.

The results of the comparison study are given in Table-11.4.

Fig. 11-1 Area Capacity Curve of El Siete No.1 Reservoir



El (m)	Capacity (10^3 m^3)	Area (10^3 m^2)
1600	278 324	5 069
1575	169 340	3 650
1550	93 168	2 444
1525	44 964	1 413
1500	18 011	744
1475	5 201	281
1450	926	76
1425	41	9
1412	0	0

Table-11.1 Outline of Development Plans of Regulating Pond Type

Description	Unit	No.1 Project	No.2 Project	Total
Catchment Area				
Main area/ Sub area	km^2	256.3/-	297.9/-	
Total	"	256.3	297.9	
Annual Inflow	10^6m^3	725	843	
Reservoir				
High water level	m	1,450.00	1,070.00	
Low water level	"	1,440.00	-	
Available draw down	"	10.00	-	
Gross storage capacity	m^3	926,000	-	
Effecitve storage capacity	"	540,000	-	
Main Dam				
Type		C.G	C.G	
Hight x Crest length	m	55 x 207	35 x 146	
Volume	m^3	143,000	60,000	
Auxiliary Dam				
Type		C.G	-	
Hight x Crest length	m	21.5 x 148		
Volume	m^3	36,000		
Headrace Tunnel				
Type x Number of tunnels		Pressure x 1	Pressure x 1	
Diameter x Length	m	3.40 x 3,145	3.60 x 9,109	
Connection Tunnel			-	
Diameter x Length	m	3.30 x 858		
Penstock Line				
Number of lines		1 - 2	1 - 2	
Diameter x Length	m	3.40 - 1.25 x 1,301	3.60 - 1.25 x 1,045	
Powerstation				
Type of turbine x Number of unit		VP x 2	VF x 2	
Development Plan				
Intake water level	m	1,445.00	1,068.50	
Tail water level	"	1,071.00	687.00	
Gross head	"	374.00	381.50	
Loss of head	"	21.00	24.00	
Effective head	"	353.00	357.50	
Maximum discharge	m^3/s	25.00	28.00	
Installed capacity	MW	75.0	85.0	160.0
Annual energy production	GWh	508.0	588.3	1,096.3
Firm energy	"	319.8	376.5	696.3
Secondary energy	"	188.2	211.8	400.0
Firm output	MW	73.8	80.8	154.6

Table-11.2 Outline of Development Plans of Run-Off River Type

Description	Unit	No.1 Project	No.2 Project	Total
Catchment Area				
Main area/ Sub area	km^2	235.7/19.8	297.9/-	
Total	"	255.5	297.9	
Annual Inflow	10^6m^3	722	843	
Reservoir				
High water level	m	1,460.00	1,070.00	
Low water level	"	-	-	
Available draw down	"	-	-	
Gross storage capacity	m^3	-	-	
Effecitve storage capacity	"	-	-	
Main Dam				
Type		C.G	C.G	
Hight x Crest length	m	21.5 x 148	35 x 146	
Volume	m^3	36,000	60,000	
Auxiliary Dam				
Type		C.G	-	
Hight x Crest length	m	5 x 20		
Volume	m^3	500		
Headrace Tunnel				
Type x Number of tunnels		Pressure x 1	Pressure x 1	
Diameter x Length	m	3.40 x 3,753	3.60 x 9,109	
Connection Tunnel			-	
Diameter x Length	m	1.80 x 1,000		
Penstock Line				
Number of lines		1 - 2	1 - 2	
Diameter x Length	m	3.40 - 1.25 x 1,301	3.60 - 1.25 x 1,045	
Powerstation				
Type of turbine x Number of unit		VP x 2	VF x 2	
Development Plan				
Intake water level	m	1,460.00	1,068.50	
Tail water level	"	1,071.00	687.00	
Gross head	"	386.00	381.50	
Loss of head	"	23.70	24.00	
Effective head	"	365.30	357.50	
Maximum discharge	m^3/s	25.00	28.00	
Installed capacity	MW	77.9	85.0	162.9
Annual energy production	GWh	546.9	610.7	1,157.6
Firm energy	"	329.7	376.5	706.2
Secondary energy	"	217.2	234.2	451.4
Firm output	MW	38.0	43.6	81.6

Table-11.3 Outline of Development Plans of Reservoir Type

Description	Unit	Reservoir Type Case 1			Reservoir Type Case 2		
		No.1 Project	No.2 Project	Total	No.1 Project	No.2 Project	Total
Catchment Area							
Main area/ Sub area	km^2	256.3/-	297.9/-		256.3/-	297.9/-	
Total	"	256.3	297.9		256.3	297.9	
Annual Inflow	10^6m^3	725	843		725	843	
Reservoir							
High water level	m	1,580.00	1,070.00		1,561.00	1,070.00	
Low water level	"	1,548.00	-		1,548.00	-	
Available draw down	"	32.00	-		13.00	-	
Gross storage capacity	m^3	188	-		124	-	
Effecitve storage capacity	"	99	-		36.3	-	
Main Dam							
Type		C.G	C.G		C.G	C.G	
Hight x Crest length	m	185 x 565	35 x 146		166 x 512	35 x 146	
Volume	m^3	3,200,000	60,000		2,400,000	60,000	
Auxiliary Dam		-					
Type							
Hight x Crest length	m						
Volume	m^3						
Headrace Tunnel							
Type x Number of tunnels		Pressure x 1	Pressure x 1		Pressure x 1	Pressure x 1	
Diameter x Length	m	4.00 x 2,650	4.20 x 9,109		3.70 x 2,680	3.90 x 9,109	
Connection Tunnel							
Diameter x Length	m	-	-		-	-	
Penstock Line							
Number of lines		1 - 2	1 - 2		1 - 2	1 - 2	
Diameter x Length	m	4.00 - 1.50 x 1,635	4.20 - 1.50 x 1,045		3.70 - 1.50 x 1,635	3.90 - 1.50 x 1,045	
Powerstation							
Type of turbine x Number of unit		VP x 2	VF x 2		VP x 2	VF x 2	
Development Plan							
Intake water level	m	1,569.30	1,068.50		1,556.70	1,068.50	
Tail water level	"	1,071.00	687.40		1,071.40	687.20	
Gross head	"	494.50	381.10		482.30	381.30	
Loss of head	"	23.80	21.30		23.50	22.90	
Effective head	"	474.70	359.80		461.80	358.40	
Maximum discharge	m^3/s	35.00	38.00		30.00	33.00	
Installed capacity	MW	141.2	116.2	257.4	117.7	100.5	218.2
Annual energy production	GWh	802.6	682.8	1,485.4	743.7	651.9	1,395.6
Firm energy	"	616.7	520.5	1,137.2	514.6	452.6	967.2
Secondary energy	"	185.9	162.3	348.2	229.1	199.3	428.4
Firm output	MW	140.8	106.7	247.5	117.5	91.3	208.8

Table-11.4 Comparison Study of Development Type of Atrato Project

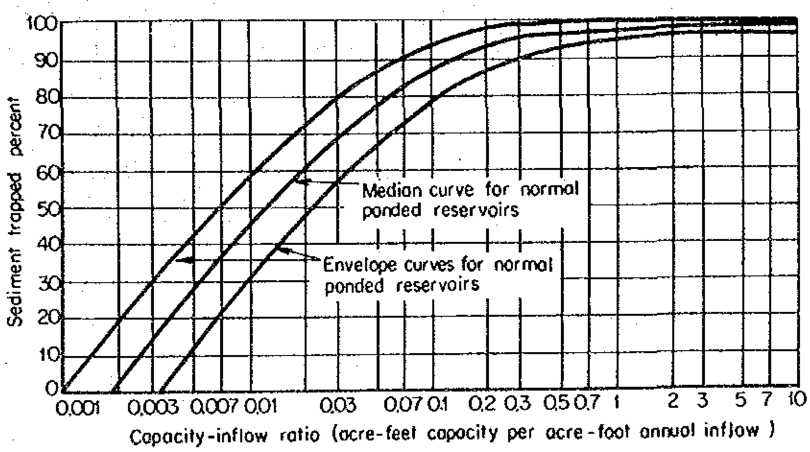
Case	Type of Development	Name of Project	Installed Capacity (MW)	Annual energy production (GWh)	Construction Cost (10^3 US$)	Annual benefit (10^3 US$)	Annual Cost (10^3 US$)	Cost of energy (US mil/KW)	Annual surplus benefit (10^3 US$)	Benefit-cost ratio
(1)	Daily regulating	El Siete No.1	75	508.0	134,740	21,354	16,169	32.81		
		No.2	85	588.3	114,771	24,056	13,773	24.13		
		Total	160	1,096.3	249,511	45,410	29,941	28.15	15,468	1.517
(2)	Run-off	El Siete No.1	77	546.9	97,493	16,093	11,699	22.05		
		No.2	85	610.7	114,771	18,153	13,773	23.25		
		Total	162	1,157.6	212,264	34,246	25,472	22.68	8,774	1.344
(3)	Reservoir Dam height 188m	El Siete No.1 (HWL=1480)	141	802.6	624,701	38,218	74,964	96.28		
		No.2	107	687.2	132,821	30,538	15,939	24.07		
		Total	248	1,485.3	757,522	68,756	90,903	63.09	-22,147	0.756
	Dam height 166m	El Siete No.1 (HWL=1461)	118	743.7	487,142	33,247	58,457	81.03		
		No.2	91	651.9	124,091	27,397	14,891	23.55		
		Total	209	1,395.6	611,233	60,644	73,348	54.18	-12,704	0.826

11.2.2 Sedimentation Countermeasures of Regulating Pond

The estimated sediment yield of the Atrato River is very large at 6,075 $m^3/km^2/yr$ as described in 8.5 and it would be impossible to cope with this considering a sedimentation capacity within the regulating pond. As shown in Fig. 11-2, due to trap efficiency, there would be fewer opportunities to discharge from the spillway the larger that storage capacity is made, and a large proportion of inflowing sediment would settle. As a result, it would be necessary for the storage capacity to be made even larger to constitute a vicious cycle and ultimately, it would become necessary to have a 78 x 10^6 m^3 sedimentation capacity as mentioned in 2.1 of this chapter.

To secure the sedimentation capacity, a dam 134 m in height would be necessary and it would not be possible to maintain the economics of the Project.

Fig. 11-2 Sediment Trap Efficiency



In this project therefore, it must be possible for the sediment of No. 1 dam to be flushed out by a flushing gate provided at the bottom of No. 1 dam, and when the river runoff at the regulating pond exceeds a given level, the flushing gate is to be fully open to lower the water level, the sediment in the regulating pond to downstream of the dam by the flowing storaged water to return the interior of the regulating pond to the condition of a natural stream, sediment coming down from the upstream area being directed downstream without deposition.

While the water level of the regulating pond is being lowered to remove sediment it would not be possible for water to be intaked from the intake, and therefore, water is to be intaked at the No. 1 auxiliary intake dam so that power generation will not be stopped because of sediment flushing from the regulating pond.

The No. 1 auxiliary intake dam would be provided approximately 850 m upstream of El Siete No. 1 Dam near the end of the No. 1 regulating pond backwater. The run-of-river system is to be adopted for the intake method.

A study of the El Siete No. 1 regulating pond sediment flushing operations was made to ascertain the effect of the interconnected operation of El Siete No. 1 Regulating Pond and the No. 1 auxiliary dam. The results are given in Table-11.5.

Table-11.5 Operation Study on Sediment Flushing at El Siete No.1 Regulating Pond

Description	Runoff at El Siete No.1 Auxiliary Intake Dam at the time of Sediment Flushing is started				
	Unit	20 m^3/sec	25 m^3/sec	30 m^3/sec	35 m^3/sec
Annual Inflow	m^3/s-day	8,393.28	8,393.28	8,393.28	8,393.28
Annual available intake	"	6,573.27	7,421.30	7,892.96	8,146.02
Annual Sediment	10^3m^3	1,557	1,557	1,557	1,557
Annual times of sediment flushing	time	16	17	13	9
Annual number of days of sediment flushing	day	181	96	51	20
Maximum sedimentation at the time flushing is started	10^3m^3	10	28	66	82
Annual waste discharge from flushing	m^3/s-day	352.69	366.66	351.24	319.87
Actual annual available discharge	m^3/s-day	6,220.58	7,054.64	7,541.72	7,826.15

It was determined that when starting flushing at the No. 1 dam with 25 m^3/s inflow at the auxiliary dam site the water level lowering frequency for sediment flushing would be 17 times per year, the maximum sedimentation at the time flushing is started would be less than 30,000 m^3 and, as the river-bed gradient above and below the main dam is steep at 1/20 and less, sediment flushing at the dam is very possible with the natural flow of river water. The annual waste discharge from flushing would be 367 m^3/s day (32 x 10^6 m^3), less than 5 percent of the available annual intake and not an amount to impair the economics of the Project.

As a result of the study on sediment flushing at El Siete No.1 dam, deposited sand materials in the reservoir will be eliminated by the excess water more than 25 m^3/s of the natural river flow of Rio Atrato by means of the sand flush gates operation 17 times a year, which was clarified by a mode of simulation.

Since, the reservoir area where the sand material would be accumulated is about 150 m in width and 400 m in length, the deposited material can be eliminated by bulldozer if necessary.

It is necessary to construct the auxiliaries intake dam by the reason why both El Siete No.1 and No.2 power plant can continue generating operation while the sand flushing gates opening are kept at a reservoir water level corresponding to the original river bed in the reservoir. For this it is possible for both El Siete No.1 and No.2 power plants to generate energy about 368 GWH.

11.2.3 Selection of El Siete No. 1 Dam Axis

For the El Siete No. 1 Dam axis, in addition to the dam axis (downstream dam axis) selected in the Master Plan, an alternative dam axis was selected approximately 100 m upstream of the downstream dam axis, and a comparison study was made of the economics.

In this case, with the downstream dam axis, the regulating capacity required can be secured with high water level at EL. 1,450 m. With the upstream dam axis it would however be necessary to secure the regulating capacity required with a high water level at EL. 1,453 m.

The dam height would be the same 55 m for both, but whereas the volume of the downstream dam would be 143 x 10^3 m^3, that for the upstream dam would be 165 x 10^3 m^3.

The results of the study comparing the two dams are given in Table-11.6.

Table-11.6 Study of Economic Comparison on El Siete No.1 Dam Axis

Description	Unit	Master Plan Axis (Down-stream)	Alternative Plan Axis (Up-stream)
Reservoir			
HWL	m	1,450	1,453
LWL	m	1,440	1,443
Draw down depth	m	10	10
Total storage capacity	10^3m^3	926	880
Effective storage capacity	10^3m^3	540	540
Dam height	m	55	55
Crest length	m	207	210
Volume	10^3m^3	143	165
Construction cost of El Siete No.1 P.S.	10^3US$	134,740	137,930
El Siete No.1 P.S.			
Annual benefit: (B)	"	22,276	22,465
Annual cost: (C)	"	16,169	16,552
B - C		6,107	5,913
B/C		1.378	1.357

It was therefore decided to select the downstream dam axis for El Siete No. 1 Dam.

11.2.4 Selection of No. 1 Auxiliary Intake Dam Axis

El Siete No. 1 Auxiliary Intake Dam is a facility to be used as an alternative to the No. 1 intake facilities during sediment flushing operations at the No. 1 regulating pond.

The dam axis was selected at a point approximately 350 m downstream from the confluence of the Sanchez River and the Atrato River, approximately 1 km upstream from the El Siete No. 1 dam axis, that is, at the end of the No. 1 regulating pond backwater, where the river is comparatively wide and there is open flat river terrace space where facilities such as the intake dam, intake, settling basin, etc., can be laid out rationally from the viewpoints of purpose and function.

11.2.5 Determination of Maximum Scale of El Siete No. 1 Dam (Regulating Pond)

In examination of the optimum scale for El Siete No. 1 Dam, a comparison study was made setting up a minimum-scale dam as the basic-scale, and then setting up an alternative scale.

In effect, the original ground around the regulating pond was considered, the maximum extent of available drawdown permissible as a dam for daily regulation was taken to be 10 m, and the three alternatives of the basic-scale minimum-scale dam (high water level, EL. 1,450 m), high water level of EL. 1,452.5 m and high water level of EL. 1,455 m, were compared and studied to secure the 540,000 m^3 regulating capacity necessary for the Project at this available drawdown.

The results of the study are given in Fig. 11-3 and Table-11.7.

Fig. 11-3 Study on Optimum Dam Height

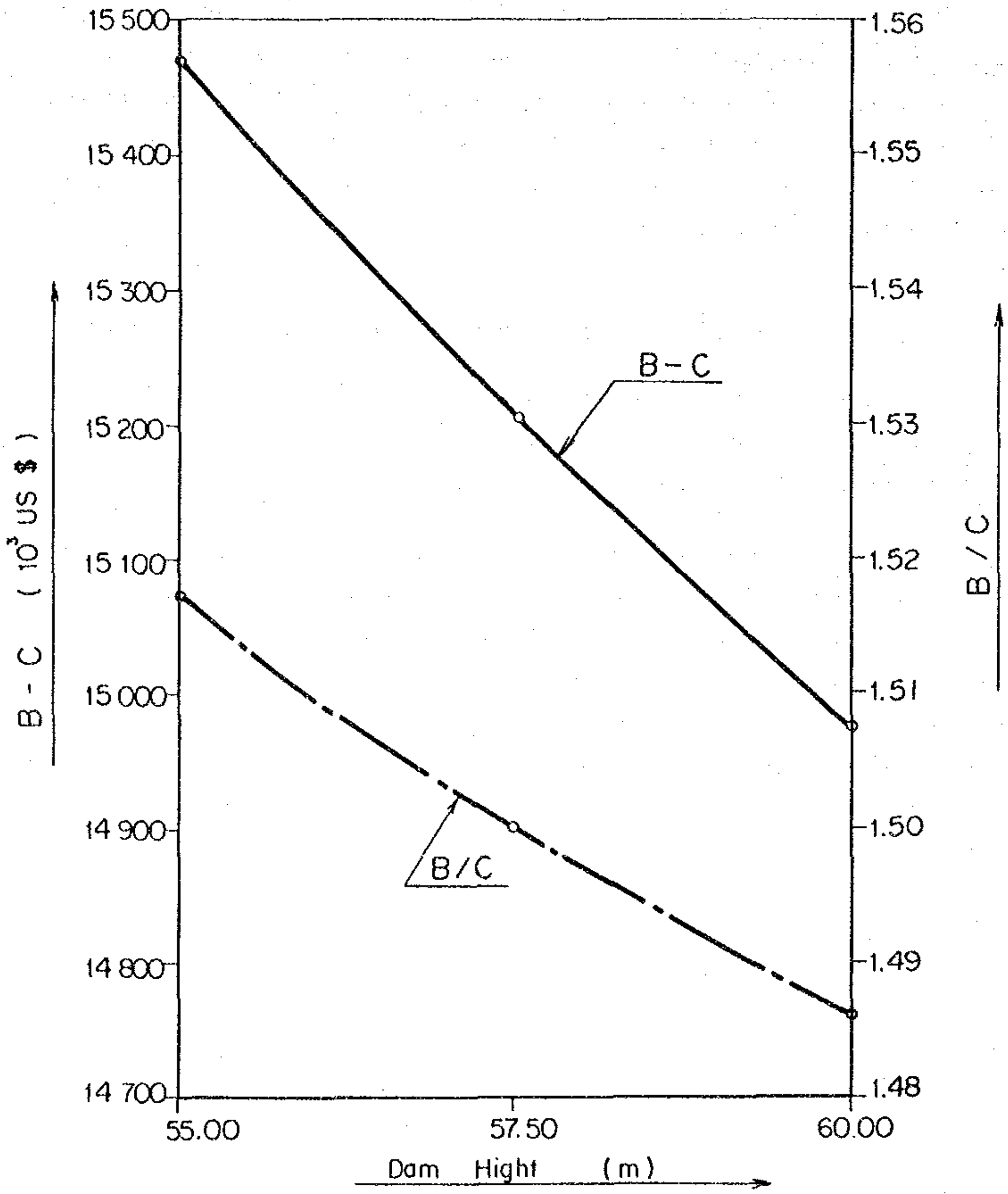


Table-11.7 Study of El Siete No. 1 dam height in daily control reservoir type

Case	(HWL.) m	Name	Dam height m	Dam Volume 10^3 m^3	Storage Capacity 10^3 m^3	Max. dis-charge m^3/S	Head m	Max. out-put MW	Annual energy product GWh	Const-ruction cost 10^3 US$	Annual benefit 10^3 US$	Annual cost 10^3 US$	Cost of energy US$ mil /KWh	Annual surplus benefit B - C	B/C
1	1450.	El Siete No. 1	55.0	143	540	25	353.0	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
		No. 2	35.0	60		28	357.5	85.0	588.3	114,771	24,056	13,773	24.13	10,283	
		Total						160.0		249,511	45,410	29,941	28.15	15,468	1.517
2	1452.5	El Siete No. 1	57.5	170	540	25	356.5	75.7	513.1	138,692	21,566	16,643	33.44	4,923	
		No. 2	35.0	60		28	357.5	85.0	588.3	114,771	24,056	13,773	24.13	10,283	
		Total						160.7	1,101.4	253,463	45,622	30,416	28.47	15,206	1.500
3	1455.	El Siete No. 1	60.0	189	540	25	359.5	76.4	518.1	142,148	21,747	17,058	33.94	4,689	
		No. 2	35.0	60		28	357.5	85.0	588.3	114,771	24,056	13,773	24.13	10,283	
		Total						61.4	1,106.4	256,919	45,803	30,831	28.72	14,972	1.486

As is clearly indicated in the study results, the Project's economics would be impaired as the size of the dam is increased, and therefore the minimum-scale dam of high water level at EL. 1,450 m and 55 m dam height was selected as the optimum scale.

11.2.6 Studies of Powerhouse Locations and Headrace Routes

El Siete No. 1 Dam would be for regulating pond type development with a high water level at EL. 1,450 m, the location being most reasonable topographically and geologically with the dam axis (downstream dam) at the Sanchez district. In this paragraph, therefore, the results of comparison studies for powerhouse locations and their waterway routes are described.

In this case, the section from the Sanchez district, the confluence of the Atrato River and the Toro River to the Piñon district where river-bed level of EL. 685 m was taken to be the project area.

The river-bed gradient in this section is 1/24 for a very swift stream, and especially, the section between Sanchez and Girardot is in the form of cascades.

On the other hand, from the topographical and geological viewpoint, it is not appropriate for the head of the section from the El Siete No. 1 dam site to the Piñon district to be developed in a single stage. It was therefore decided that power generation would be by dividing the head into two stages with El Siete No. 1 Power Station selected at the Girardot district as the first-stage power plant and El Siete No. 2 power Station at the Pinon district as the second-stage power plant.

For the location of El Siete No. 1 Powerhouse considering the topographical and geological conditions, the connection to the El Siete No. 2 power station, intake of water from the remaining catchment area, three sites were selected - the vicinity of the confluence of the Atrato River and Santa Isabel Ravine selected in the Master Plan (called Site 1M), the vicinity of the confluence of the Atrato River and Aguila Ravine selected in field investigations of the present Feasibility Study (called Site 1F), and a site 700 m downstream of the confluence of the Atrato River and the Girardot River considered an alternative for Site 1F (called Site 1G) for comparative study.

As for the selection of the El Siete No. 2 Power Station site upstream of the Atrato River and Girardot River confluence, there is no merit in shortening the headrace length of the El Siete No. 2 Project by short-cutting Atrato River menders there, and as it would be necessary to construct an intake dam on the Girardot River in addition to on the Atrato River, it would be uneconomical.

On the other hand, for El Siete No. 2 Power Station, in view of the topographical and geological conditions, the limit to drawing the discharged water (maximum 25 m^3/s) from the No. 1 power station and the inflow of the remaining catchment area (maximum 3 m^3/s) and conducting by headrace tunnel would be to the Piñon district. Therefore, the No. 2 powerhouse site facing the Atrato River it will be possible to select the site chosen in the Master Plan (called Site 2M) and shown in Fig. 11-2, and the site chosen through field investigations in the present Feasibility Study (called Site 2F) shown in Fig. 11-4.

As described, there are three possible sites for the El Siete No. 1 powerhouse, (Site 1F, Site 1M, and Site 1G) and two possible sites for the El Siete No. 2 powerhouse, (Site 2F and Site 2M)and the following six combinations of Nos. 1 and 2 sites can be made.

(1) 1F-2F Site

As shown in Fig. 11-4, El Siete No. 1 Power Station would be located at the confluence of the Atrato River and Aguila Ravine.

At this location, the discharge water level would be at EL. 1,071 m with a total head of 374 m obtained from the intake level of the El Siete No. 1 regulating pond at EL. 1,445 m

The surface type penstock length would be 1,301 m, shorter than for the 1G and 1M site.

For El Siete No. 2 Power Station, the intake water level would be at EL. 1,068.5 m with discharge into the Atrato River at 687 m for a total head of 381.5 m. The surface type penstock would be 1,043 m, shorter than the 2M site.

When the cover of the mountain for the headrace tunnels connecting El Siete No. 1 Dam, the No. 1 powerhouse, and the No. 2 powerhouse and work adits are considered, the route would be as shown in Fig. 11-4.

The No. 1 and No. 2 headrace tunnels would be 3,145 m and 9,109 m long respectively.

The headrace route to the 2F Site for El Siete No. 2 Power Station had been considered as promising at the time of the Master Plan Study, but was not adopted since the surge tank site elevation could not be confirmed in the field when preparing the Master Plan Report.

(2) 1M-2F Site

As shown in Fig. 11-2, the El Siete No. 1 powerhouse would be provided immediately upstream of the confluence of the mainstream Atrato River and Santa Isabel Ravine. This site was selected at the time of the Master Plan Study.

Since this location is 2,100 m downstream of the previously-mentioned 1F site, the discharge water level would be at EL. 988 m, and a total head of 457 m would be obtained from the intake water level at the El Siete No. 1 regulating pond of EL. 1,445 m. However, the penstock length will be 2,186 m, approximately double that in the 1F site.

As for El Siete No. 2 Power Station, the tail water level would be at EL. 687 m, unchanged from that in the 2F site. The total head from the intake water level at EL. 980.5 m will be 293.5 m, decreased compared with the previously mentioned 1F-2F proposal by the amount of increase at the No. 1 power station.

The penstock length can also be shortened to 831 m.

However, as described in Table 11.9, the total headrace length of 15,442 m will be 732 m longer than the 14,710 m for the 1F-2F site.

This indicates that the economics will be inferior to the 1F-2F site.

(3) 1G-2F Site

As shown in Fig. 11-2, the El Siete No. 1 powerhouse would be provided at a point 700 m downstream from the confluence of the Atrato River and the Girardot River.

Since this location is 2,100 m upstream compared with the 1F site, the total head from the intake water level at EL. 1,445 m of the El Siete No. 1 regulating pond would be 341 m, and with the tail water level high, the penstock length would be 2,043 m, providing little difference from the 1M site.

However, the No. 1 headrace tunnel will be 2,511 m and 636 m shorter than in the 1F site.

For El Siete No. 1 Power Station, the intake water level would be at EL. 1,098.5 m so that the total head to the tail water level of EL. 687 m would be 411.5 m, higher than in the case of the previously mentioned 1F-2F site.

The No. 2 penstock will be 1,161 m, or 118 m longer than in the 1F-2F site.

The No. 2 headrace tunnel length would be 8,983 m and the total waterway length would be 14,778 m, as shown in Table-11.9, and although shorter than in the 1M-2F site, it would be little different from the 1F-2F site.

(4) 1F-2M, 1M-2M, and 1G-2M Site

Regarding the three sites 1F, 1M, and 1G for the El Siete No. 1 powerhouse, the comparisons of combinations with 2F and 2M for the No. 2 powerhouse were carried out to determine which would be better.

The previously mentioned 1F-2F, 1M-2F and 1G-2F sites are of the same powerhouse particulars as El Siete No. 1, and there is only the difference of the El Siete No. 2 powerhouse changing from Site 2F to Site 2M.

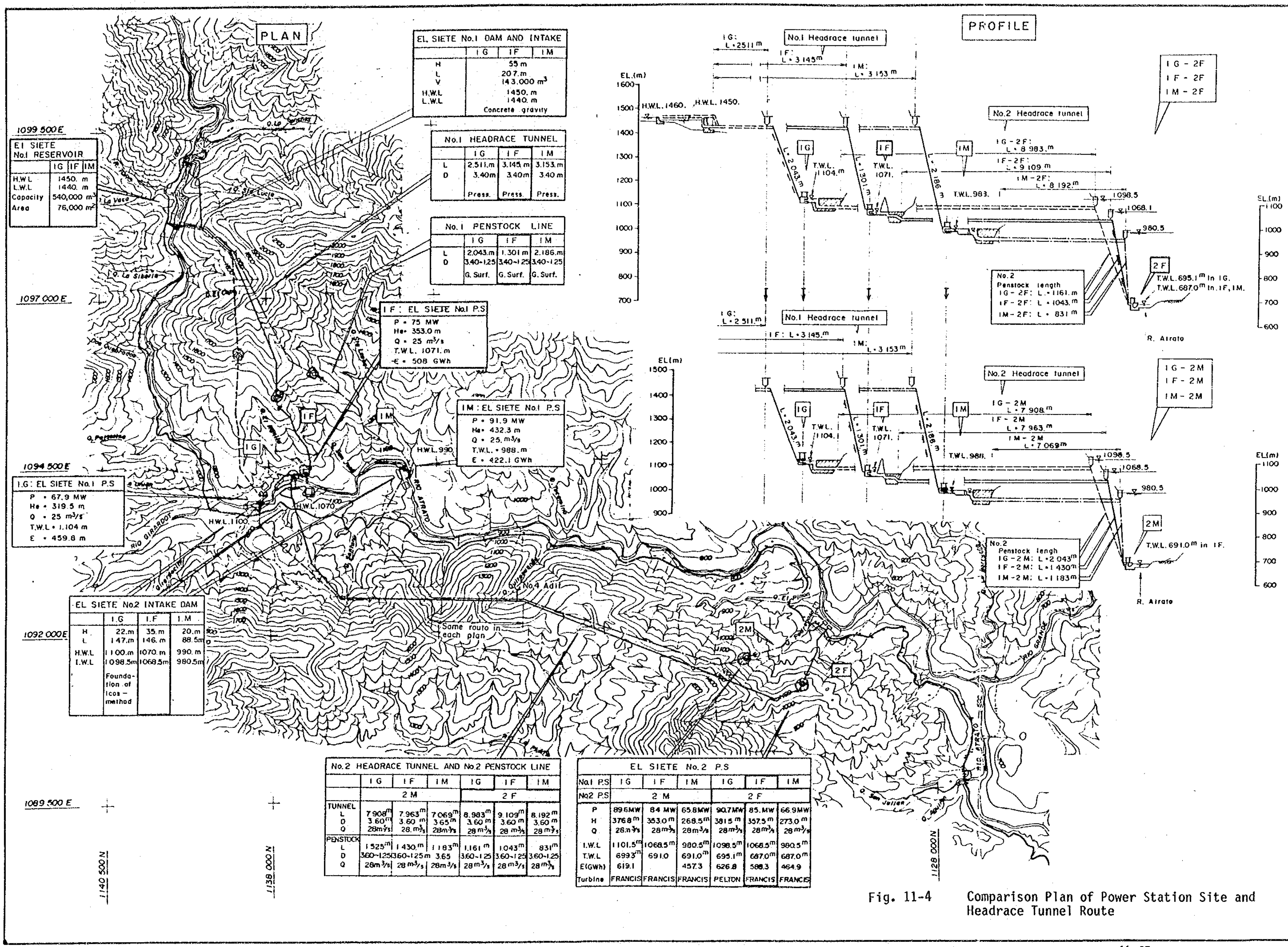
(5) Study Results

The results of comparisons made on the various powerhouse locations, namely, the six cases of 1F-2F, 1M-2F, 1G-2F and 1F-2M, 1M-2M, 1G-2M, are as given in Table-11.8.

According to these results, the 1F-2F site, or that of providing El Siete No. 1 Power Station at the confluence of the Atrato River and Aguila Ravine with the El Siete No. 2 powerhouse at the downstream 2F site in the

Pinon district, will be of minimum power energy cost, with both the annual surplus benefit (B-C) and the benefit-cost ratio (B/C) being maximum.

The above is followed in order by the site of 1M-2F, 1M-2M, and 1M-2M (Master Plan site), and it was therefore decided to select the 1F-2F site.



El Siete No.1 Reservoir

	IG	IF	IM
H.W.L	1450. m		
L.W.L	1440. m		
Capacity	540,000 m³		
Area	76,000 m²		

EL SIETE No.1 DAM AND INTAKE

	IG	IF	IM
H	55 m		
L	207.m		
V	143,000 m³		
H.W.L	1450. m		
L.W.L	1440. m		
	Concrete gravity		

No.1 HEADRACE TUNNEL

	IG	IF	IM
L	2.511.m	3.145 m	3.153.m
D	3.40m	3.40m	3.40 m
	Press.	Press.	Press.

No.1 PENSTOCK LINE

	IG	IF	IM
L	2.043.m	1.301 m	2.186.m
D	3.40-1.25	3.40-1.25	3.40-1.25
	G. Surf.	G. Surf.	G. Surf.

EL SIETE No2 INTAKE DAM

	I.G	I.F	I.M
H	22.m	35.m	20.m
L	147.m	146.m	88.5m
H.W.L	1100.m	1070.m	990.m
L.W.L	1098.5m	1068.5m	980.5m
	Foundation of Icos – method		

No.2 HEADRACE TUNNEL AND No.2 PENSTOCK LINE

	IG	IF	IM	IG	IF	IM
	2M			2F		
TUNNEL L	7.908 m	7.963 m	7.069 m	8.983 m	9.109 m	8.192 m
D	3.60 m	3.60 m	3.65 m	3.60 m	3.60 m	3.60 m
Q	28 m³/s	28. m³/s	28 m³/s	28 m³/s	28 m³/s	28 m³/s
PENSTOCK L	1.525 m	1.430. m	1.183 m	1.161 m	1.043 m	831 m
D	3.60-1.25	3.60-1.25 m	3.65	3.60-1.25	3.60-1.25	3.60-1.25
Q	28 m³/s	28 m³/s	28 m³/s	28 m³/s	28 m³/s	28 m³/s

EL SIETE No.2 P.S

No.1 P.S	IG	IF	IM	IG	IF	IM
No2 P.S	2M			2F		
P	89.6MW	84 MW	65.8MW	90.7MW	85.MW	66.9MW
H	376.8 m	353.0 m	268.5 m	381.5 m	357.5 m	273.0 m
Q	28 m³/s	28 m³/s	28 m³/s	28 m³/s	28 m³/s	28 m³/s
I.W.L	1101.5 m	1068.5 m	980.5 m	1098.5 m	1068.5 m	980.5 m
T.W.L	699.3 m	691.0	691.0 m	695.1 m	687.0 m	687.0 m
E(GWh)	619.1		457.3	626.8	588.3	464.9
Turbine	FRANCIS	FRANCIS	FRANCIS	PELTON	FRANCIS	FRANCIS

Table-11.8 Outline of Development Plans of Alternative Power Station Sites (1)

Description	Unit	1.G - 2.F			1.F - 2.F		
		No.1 Project	No.2 Project	Total	No.1 Project	No.2 Project	Total
Catchment Area							
Main area/ Sub area	km^2	256.3/-	297.9/-		256.3/-	297.9/-	
Total	"	256.3	297.9		256.3	297.9	
Annual Inflow	$10^6 m^3$	725	841		725	843	
Reservoir							
High water level	m	1,450.00	1,103.00		1,450.00	1,070.00	
Low water level	"	1,440.00	-		1,440.00	-	
Available draw down	"	10.00	-		10.00	-	
Gross storage capacity	m^3	926,000	-		926,000	-	
Effecitve storage capacity	"	540,000	-		540,000	-	
Main Dam							
Type		C.G	C.G		C.G	C.G	
Hight x Crest length	m	55 x 207	30 x 160		55 x 207	35 x 146	
Volume	m^3	143,000	55,000		143,000	60,000	
Auxiliary Dam							
Type		C.G	-		C.G	-	
Hight x Crest length	m	21.5 x 148	-		21.5 x 148		
Volume	m^3	36,000			36,000		
Headrace Tunnel							
Type x Number of tunnels		Pressure x 1	Pressure x 1		Pressure x 1	Pressure x 1	
Diameter x Length	m	3.40 x 2,511	3.60 x 8,983		3.40 x 3,145	3.60 x 9,109	
Connection Tunnel							
Diameter x Length	m	3.30 x 812	-		3.30 x 812	-	
Penstock Line							
Number of lines		1 - 2	1 - 2		1 - 2	1 - 2	
Diameter x Length	m	3.40 - 1.25 x 2,043	3.40 - 1.25 x 1,161		3.40 - 1.25 x 1,301	3.40 - 1.25 x 1,045	
Powerstation							
Type of turbine x Number of unit		VF x 2	VP x 2		VP x 2	VF x 2	
Development Plan							
Intake water level	m	1,445.00	1,101.50		1,445.00	1,068.50	
Tail water level	"	1,104.00	695.10		1,071.40	687.00	
Gross head	"	341.00	406.40		374.00	381.50	
Loss of head	"	21.50	24.40		21.00	24.00	
Effective head	"	319.50	381.50		353.00	357.50	
Maximum discharge	m^3/s	25.00	28.00		25.00	28.00	
Installed capacity	MW	67.9	90.7	158.6	75.0	85.0	160.0
Annual energy production	GWh	459.8	626.8	1,086.6	508.0	588.3	1,096.3
Firm energy	"	289.5	401.1	690.6	319.8	376.5	696.3
Secondary energy	"	170.3	225.7	396.0	188.2	211.8	400.0
Firm output	MW	66.8	86.2	153.0	73.8	80.8	154.6

Table-11.8 Outline of Development Plans of Alternative Power Station Sites (2)

Description	Unit	1.M - 2.F			1.G - 2.M		
		No.1 Project	No.2 Project	Total	No.1 Project	No.2 Project	Total
Catchment Area							
Main area/ Sub area	km^2	256.3/-	301.8/7.7		256.3/-	297.4/-	
Total	"	256.3	309.5		256.3	297.4	
Annual Inflow	10^6m^3	725	875		725	841	
Reservoir							
High water level	m	1,450.00	982.00		1,450.00	1,100.00	
Low water level	"	1,440.00	-		1,440.00	-	
Available draw down	"	10.00	-		10.00	-	
Gross storage capacity	m^3	926,000	-		926,000	-	
Effective storage capacity	"	540,000	-		540,000	-	
Main Dam							
Type		C.G	C.G		C.G	C.G	
Hight x Crest length	m	55 x 207	30 x 165		55 x 207	30 x 160	
Volume	m^3	143,000	42,000		143,000	55,000	
Auxiliary Dam							
Type		C.G	C.G		C.G	-	
Hight x Crest length	m	21.5 x 148	5 x 20		21.5 x 148	-	
Volume	m^3	36,000	500		36,000		
Headrace Tunnel							
Type x Number of tunnels		Pressure x 1	Pressure x 1		Pressure x 1	Pressure x 1	
Diameter x Length	m	3.40 x 3,153	3.65 x 8,192		3.40 x 2,511	3.60 x 7,908	
Connection Tunnel							
Diameter x Length	m	3.30 x 812	1.80 x 130		3.30 x 812	-	
Penstock Line							
Number of lines		1 - 2	1 - 2		1 - 2	1 - 2	
Diameter x Length	m	3.40 - 1.25 x 2,186	3.65 - 1.25 x 831		3.40 - 1.25 x 2,043	3.40 - 1.25 x 1,525	
Powerstation							
Type of turbine x Number of unit		VP x 2	VF x 2		VP x 2	VF x 2	
Development Plan							
Intake water level	m	1,445.00	980.50		1,445.00	1,101.50	
Tail water level	"	985.00	687.00		1,104.00	699.30	
Gross head	"	460.40	293.50		341.00	402.20	
Loss of head	"	28.10	20.50		21.50	25.40	
Effective head	"	432.30	273.00		319.50	376.80	
Maximum discharge	m^3/s	25.00	28.80		25.00	28.00	
Installed capacity	MW	91.9	66.9	158.8	67.9	89.6	157.5
Annual energy production	GWh	622.1	464.9	1,087.0	459.8	619.1	1,078.9
Firm energy	"	391.7	298.7	690.4	289.5	396.2	685.7
Secondary energy	"	230.4	166.2	396.6	170.3	222.9	293.2
Firm output	MW	90.4	63.0	153.4	66.8	85.1	151.9

Table-11.8 Outline of Development Plans of Alternative Power Station Sites (3)

Description	Unit	1.F - 2.M No.1 Project	1.F - 2.M No.2 Project	1.F - 2.M Total	1.M - 2.M No.1 Project	1.M - 2.M No.2 Project	1.M - 2.M Total
Catchment Area							
Main area/ Sub area	km^2	256.3/-	297.9/-		256.3/-	301.8/7.7	
Total	"	256.3	297.9		256.3	309.5	
Annual Inflow	$10^6 m^3$	725	843		725	875	
Reservoir							
High water level	m	1,450.00	1,070.00		1,450.00	982.00	
Low water level	"	1,440.00	-		1,440.00	-	
Available draw down	"	10.00	-		10.00	-	
Gross storage capacity	m^3	926,000	-		926,000	-	
Effecitve storage capacity	"	540,000	-		540,000	-	
Main Dam							
Type		C.G	C.G		C.G	C.G	
Hight x Crest length	m	55 x 207	35 x 146		55 x 207	30 x 165	
Volume	m^3	143,000	60,000		143,000	42,000	
Auxiliary Dam							
Type		C.G	C.G		C.G	C.G	
Hight x Crest length	m	21.5 x 148	5 x 20		21.5 x 148	5 x 20	
Volume	m^3	36,000	500		36,000	500	
Headrace Tunnel							
Type x Number of tunnels		Pressure x 1	Pressure x 1		Pressure x 1	Pressure x [illegible]	
Diameter x Length	m	3.40 x 3,145	3.65 x 7,963		3.40 x 3,153	3.65 x 7,069	
Connection Tunnel							
Diameter x Length	m	3.30 x 812	1.80 x 130		3.30 x 812	1.8 x 130	
Penstock Line							
Number of lines		1 - 2	1 - 2		1 - 2	1 - 2	
Diameter x Length	m	3.40 - 1.25 x 1,301	3.65 - 1.25 x 1,430		3.40 - 1.25 x 2,186	3.65 - 1.25 x 1,185	
Powerstation							
Type of turbine x Number of unit		VP x 2	VF x 2		VP x 2	VF x 2	
Development Plan							
Intake water level	m	1,445.00	1,068.50		1,445.00	980.50	
Tail water level	"	1,071.00	691.00		985.00	691.00	
Gross head	"	374.00	377.50		460.40	289.50	
Loss of head	"	21.00	24.50		28.10	21.00	
Effective head	"	353.00	353.00		432.30	268.50	
Maximum discharge	m^3/s	25.00	28.00		25.00	28.80	
Installed capacity	MW	75.0	84.0		91.9	65.8	
Annual energy production	GWh	508.0	580.9		622.1	457.3	
Firm energy	"	319.8	371.8		391.7	293.8	
Secondary energy	"	188.2	209.1		230.4	163.5	
Firm output	MW	73.8	79.8		90.4	62.0	

Table-11.9 Study of Economic Comparison on Each Power Station Sites

El Siete No.1 Dam — HWL: 1,450m, LWL: 1,440m — Reservoir Capacity: $540,000m^3$, Dam Height: 55m, Crest Length: 207m

Each Power Station Sites No.1	No.2	Name	Max. Output	HWL	TWL	Effective Head	Max. Discharge	Length (m) Head-race	Surge Tank	Pen-stock	Total	Annual Energy Production	Construction Cost	Annual benefit	Annual Cost	Cost of Energy	Annual Surplus benefit	B/C	Remarks
			MW	m	m	m	m^3/s					GWh	10^3US$	10^3US$	10^3US$	US$ mil /kWh	B-C		
1G		El Siete No.1	67.9	1,450	1,104	319.5	25	2,511	40	2,043	4,594	459.8	130,250	19,328	15,630	35.04	3,698		
		No.2	90.7	1,103	695.1	381.5	28	8,983	40	1,161	10,184	626.8	126,742	25,635	15,209	25.01	10,426		
		Total	158.6			701					14,778		256,992	44.962	30,839	29.26	14,124	1.458	
1F	2F	El Siete No.1	75	1,450	1,071	353.0	25	3,147	40	1,301	4,488	508.0	134,740	21,354	16,169	32.81	5,185		
		No.2	85	1,070	687	357.5	28	9,148	40	1,043	10,222	588.3	114,771	24,056	13,772	24.13	10,284		
		Total	160			710.5					14,710		249,511	45,410	29,941	28.15	15,469	1.517	
1M		El Siete No.1	91.9	1,450	988	432.3	25	3,153	40	2,186	6,379	422.1	143,644	26,152	17,237	28.56	8,915		
		No.2	66.9	982	687	273.0	28	8,192	40	831	9,063	464.9	104,969	18.884	12,596	27.93	6,288		
		Total	158.8			705.3					15,442		248,613	45,036	29,833	28.29	15,203	1.510	
1G		El Siete No.1	67.9	1,450	1,104	319.5	25	2,511	40	2,043	4,594	459.8	130,250	19,328	15,630	35.04	3,698		
		No.2	88.9	1,103	699.3	376.8	28	7,908	40	1,525	9,473	619.1	127,968	25,322	15,356	25.78	9,966		
		Total	156.8			696.3					14,067	1,078.9	258,218	44.650	30,986	29.74	13,664	1.441	
1F	2M	El Siete No.1	75	1,450	1,071	353.0	25	3,147	40	1,301	4,488	508.0	134,740	21.354	16,169	32.81	5,185		
		No.2	84	1,070	691	353.0	28	7,963	40	1,430	9,433	580.8	115,943	23.752	13,913	24.69	9,839		
		Total	159.0			706.0					13,921	1,088.8	250.683	45,106	30,082	28.48	15,024	1.499	
1M		El Siete No.1	91.9	1,450	988	432.3	25	3,153	40	2,186	5,379	622.1	143,644	26,152	17,237	28.56	8,915		
		No.2	65.8	982	691.0	268.5	28	7,069	40	1,183	8,292	457.3	103,170	18,573	12,381	27.91	6,192		
		Total	157.7			700.8					13,671	1,079.4	246,814	44,725	29,618	28.28	15,107	1.510	

Note: Each power station sites; 1G, 1F, 1M of El Siete No.1 Power Station Sites and 2F, 2M of El Siete No.2 Power Station Sites refer to fig. 11.1: Plan of alternative power station's sites.

11.3 Study of Development Scale

11.3.1 Maximum Available Discharge of El Siete No. 1 Power Station

As a result of the studies in 11.2, the development type selected is to be the regulating pond type, with the height of El Siete No. 1 Dam at 55 m, and the high water level at EL. 1,450 m, and with the powerhouse location and headrace route as 1F-2F. Comparison studies were made of the scale of El Siete No. 1 Power Station varying the maximum available discharge of it at 1 m^3/s intervals in a range of 20 to 30 m^3/s.

Since the No. 1 and No. 2 power stations are connected by a waterway the two power staions are put togther and evaluated as one Project, when examining the maximum available discharge of the No. 1 power stastion, the discharge of 3 m^3/s taken in from the remaining catchment area at the No. 2 intake dam was added to the maximum available discharge of the No. 1 Power Station.

The results of comparison studies are shown in Fig. 11-5 and Table-11.10.

On examination of the results, the case of available discharge 25 m^3/s is of minimum energy cost with both the annual surplus benefit (B-C) and the benefit-cost ratio (B/C) maximum, and for this Project it was decided to select the maximum available discharge of El Siete No. 1 Power Station at 25 m^3/s.

Fig. 11-5 Study on Optimum Maximum Discharge

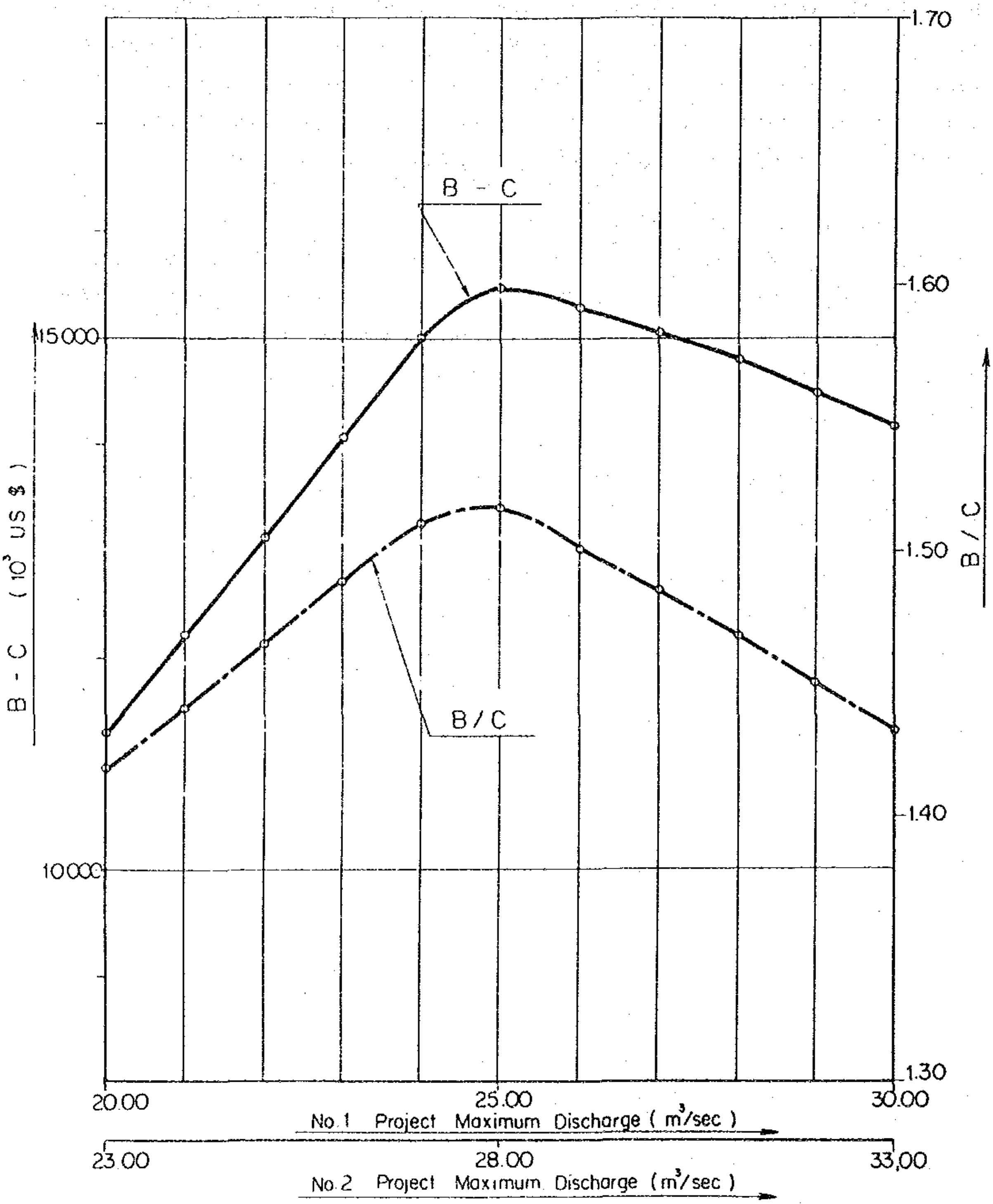


Table-11.10 Study on Optimum Maximum Discharge

Case	Name	Max. Discharge m^3/s	Installed Capacity MW	Annual Energy Production GWh	Construction Cost 10^3US\$	Annual Benefit (B) 10^3US\$	Annual Cost (C) 10^3US\$	Cost of Energy US\$ mil /kWh	Annual Surplus Benefit (B-C) 10^3US\$	Benifit-Cost Ratio (B/C)
1	El Siete No.1	20	59.4	443.3	121,908	17,808	14,629	34.02	3,179	
	2	23	69.8	526.4	103,808	20,613	12,457	24.39	8,156	
	Total		129.2	969.7	225,716	38,421	27,086	28.79	11,335	1.419
2	El Siete No.1	21	62.6	456.5	124,400	18,567	14,928	33.71	3,639	
	2	24	72.8	539.6	106,640	21,368	12,797	24.45	8,571	
	Total		135.4	996.1	231,040	39,935	27,725	28.69	12,210	1.440
3	El Siete No.1	22	65.7	470.0	127,000	19,323	15,240	33.42	4,083	
	2	25	75.9	552.0	108,700	22,095	13,044	24.36	9,051	
	Total		141.6	1,022.0	235,700	41,418	28,284	28.53	13,134	1.464
4	El Siete No.1	23	68.8	483.5	129,500	20,079	15,540	33.13	4,539	
	2	26	78.9	565.0	110,730	22,824	13,288	24.24	9,536	
	Total		147.7	1,048.5	240,230	42,903	28,828	28.34	14,075	1.488
5	El Siete No.1	24	71.9	496.6	132,150	20,828	15,858	32.92	4,970	
	2	27	82.0	577.6	112,630	23,564	13,516	24.12	10,048	
	Total		153.9	1,074.2	244,780	44,392	29,374	28.19	15,018	1.511
6	El Siete No.1	25	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
	2	28	85.0	588.3	114,771	24,056	13,773	24.13	10,283	
	Total		160.0	1,096.3	249,511	45,410	29,942	28.15	15,468	1.517
7	El Siete No.1	26	78.1	518.0	137,300	21,560	16,476	32.78	5,084	
	2	29	88.1	596.7	116,740	24,210	14,009	24.20	10,201	
	Total		166.2	1,114.7	254,040	45,770	30,485	28.19	15,285	1.501
8	El Siete No.1	27	81.3	526.0	139,950	21,728	16,794	32.91	4,934	
	2	30	91.2	604.4	118,730	24,355	14,248	24.30	10,107	
	Total		172.5	1,130.4	258,680	46,083	31,042	28.31	15,041	1.485
9	El Siete No.1	28	84.5	534.0	142,700	21,893	17,124	33.06	4,760	
	2	31	94.3	612.0	120,600	24,497	14,472	24.38	10,025	
	Total		178.8	1,146.0	263,300	46,390	31,596	28.42	14,794	1.468
10	El Siete No.1	29	87.6	541.2	145,650	22,038	17,478	33.29	4,560	
	2	32	97.4	618.8	122,400	24,626	14,688	24.47	9,938	
	Total		185.0	1,160.0	268,050	46,664	32,166	28.58	14,498	1.451
11	El Siete No.1	30	90.8	547.7	148,666	22,167	17,840	33.58	4,327	
	2	33	100.5	625.3	124,091	24,722	14,891	24.55	9,831	
	Total		191.3	1,173.0	272,757	46,889	32,731	28.76	14,158	1.433

11.3.2 Maximum Available Discharge of El Siete No. 2 Power Station

The maximum available discharge of El Siete No. 2 Power Station would be the maximum 25 m^3/s discharge of El Siete No. 1 Power Station conducted directly to the No. 2 powerhouse and adding the discharge of the remaining catchment area obtained at the No. 2 intake dam.

Consequently, deciding the maximum available discharge of El Siete No. 2 Power Station is deciding to maximum intake water at the No. 2 intake dam. The intake water at the No. 2 intake dam was varied between 0 and 5 m^3/s in this study.

In effect, comparison studies were made for available discharge of El Siete No. 2 Power Station varied between 25 m^3/s and 30 m^3/s.

The results of the studies are given in Fig. 11-6 and Table-11.11.

As a result, the generating cost of El Siete No. 2 Power Station will be a minimum at 29 m^3/s, while the annual surplus benefit (B-C) will be a maximum at 28 m^3/s and B/C maximum in the case of 27 m^3/s.

On the other hand, the case of no intake at the No. 2 intake dam, that is the case of maximum available discharge of 25 m^3/s gave the highest B/C, but B-C was smaller than in the case of 28 m^3/s.

Consequently, the maximum available discharge of the No. 2 power station was selected from the viewpoint of maximum utilization of water resources at 28 m^3/s where the annual surplus benefit (B-C) would be largest.

Power generation in El Siete No.2 by utilizing overflow water from El Siete No.1 dam in case that El Siete No.1 power plant stops is not considered by the following reasons:

(1) Possibility to get the overflow water for power generation in El Siete No.2 project is very small by the reason why the expected inspection and maintenance for the civil structure and mechanical and electrical equipment installed in El Siete No.1 project is a few times which correspond only to 3 or 4 days per year.

(2) Construction cost of a sediment basin next to El Siete No.2 Dam is very expensive by the reason why the sediment basin should be located in the under-ground due to the topographical condition of the site.

Fig. 11-6 Optimum Capacity of No.2 Intake Dam

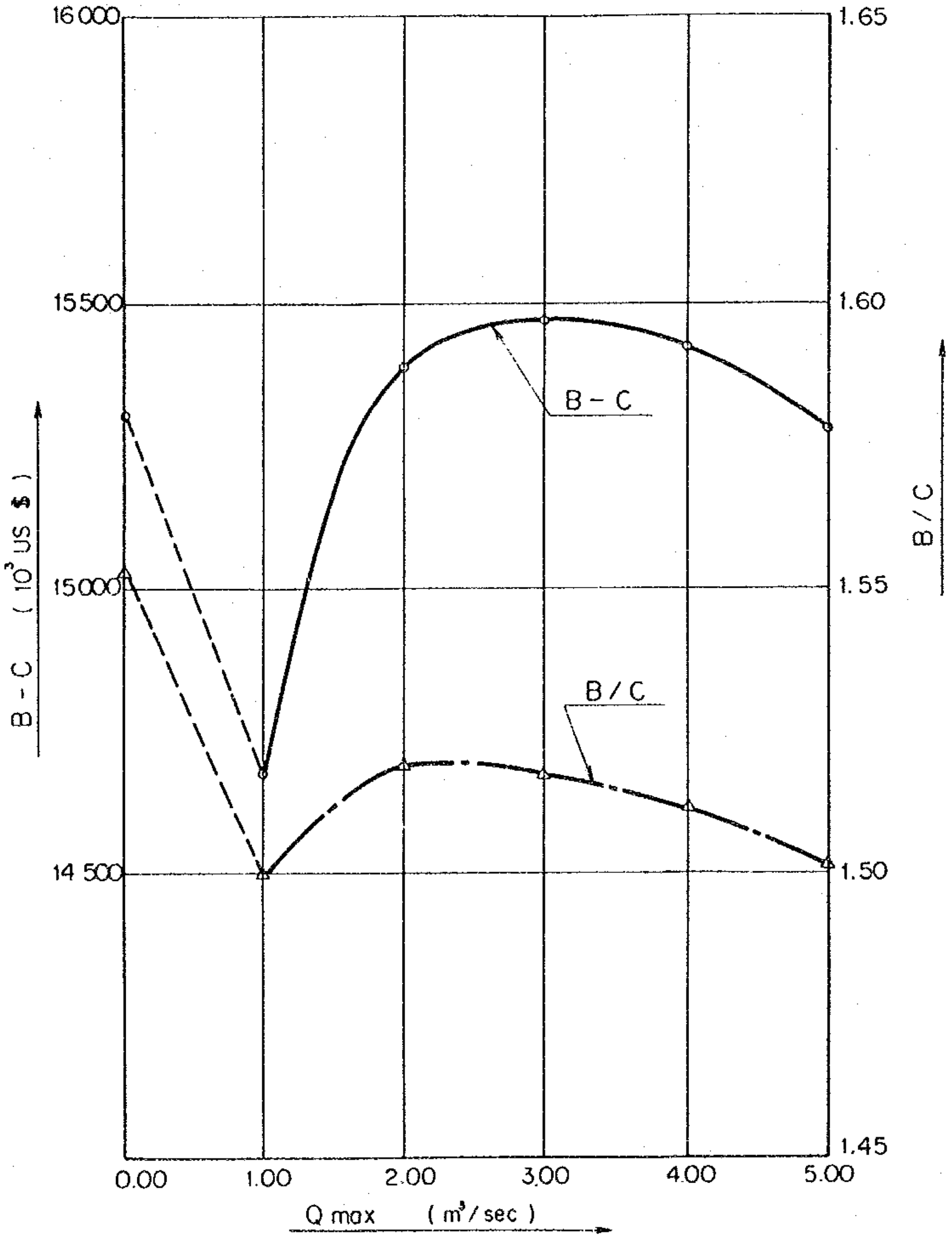


Table-11.11 Study on Optimum Maximum Discharge of No.2 Project

Case	Name	Maximum Discharge from: No. 2 Intake dam m³/sec	No. 1 Project m³/sec	Total m³/sec	Installed capacity MW	Annual energy Production GWh	Construction cost 10^3 US$	Annual benefit (B) 10^3 US$	Annual cost (C) 10^3 US$	Cost of energy US$ mil /kWh	Annual surplus benefit (B-C) 10^3 US$	Benefit-Cost Ratio (B/C)
1	El Siete No. 1			25	75.0	508.0	135,125	21,354	16,215	21.90	5,139	
	No. 2	0	25	25	75.9	513.8	95,306	21,598	11,437	22.95	10,161	
	Total				150.9	1,021.8	229,431	42,952	27,652	27.77	15,300	1.553
2	El Siete No. 1			25	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
	No. 2	1	25	26	78.9	540.7	109,612	22,642	13,153	25.08	9,489	
	Total				153.9	1,048.7	244,352	43,996	29,322	28.62	14,674	1.500
3	El Siete No. 1			25	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
	No. 2	2	25	27	82.0	566.8	112,192	23,664	13,463	24.43	10,201	
	Total				157.0	1,074.9	246,932	45,018	29,632	28.42	15,386	1.519
4	El Siete No. 1			25	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
	No. 2	3	25	28	85.0	588.0	114,771	24,056	13,773	24.13	10,283	
	Total				160.0	1,096.3	249,511	45,410	29,942	28.15	15,468	1.517
5	El Siete No. 1			25	75.0	508,0	134,740	21,354	16,169	32.81	5,185	
	No. 2	4	25	29	88.2	602.0	117,176	24,313	14,061	24.07	10,252	
	Total				163.2	1,110.1	251,916	45,667	30,230	28.07	15,437	1.511
6	El Siete No. 1			25	75.0	508.0	134,740	21,354	16,169	32.81	5,185	
	No. 2	5	25	30	91.2	609.0	119,548	24,439	14,346	24.28	10,093	
	Total				166.2	1,117.0	254,288	45,793	30,515	28.16	15,278	1.501

CHAPTER 12. PRELIMINARY DESIGN

CHAPTER 12 PRELIMINARY DESIGN

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FIGURE LIST

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CHAPTER 12. PRELIMINARY DESIGN

12.1 Study of Basic Specifications

In selecting the basic specifications of the structures comprising the El Siete No.1 and No.2 power stations, examinations were made of the type of El Siete No.1 Dam, the inside diameters of the No.1 and No.2 headrace tunnels, the inside diameters of the No.1 and No.2 penstocks, and the types of the two powerhouses. The results of the examinations are given below.

(1) Study of El Siete No.1 Dam Type

A comparison study was made regarding the El Siete No.1 dam type based on the particulars of the Project examined and decided in 11.2. The dam types normally conceivable are the concrete gravity, concrete arch, center impervious core rockfill, and concrete facing rockfill.

Items fundamental in selecting the dam type are topographical and geological conditions, flood discharge, and meteorological conditions during construction.

In the case of the El Siete No.1 Dam, it is clear in view of the topographical and geological conditions that the damsite is not suitable for arch concrete dam.

Regarding meteorological conditions, as described in Chapter 8, there is much annual rainfall, and a great many days of rain fall per year. When this is considered, the adoption of a center impervious core rockfill would incur problems in the number of days work can be done on the center core.

Consequently, it was judged that application of a center impervious core rockfill dam would be difficult.

It was therefore decided to make a comparison study of the concrete gravity and concrete facing rockfill type dams.

The design of the concrete gravity type is shown in Dwgs. 17 and 18, and the concrete facing rockfill design is shown in Dwg. 47.

The comparison study results are given in Table 12.1 below.

The concrete gravity type is US$15,040 thousand cheaper in total construction cost.

In view of these results, the concrete gravity type dam was selected for El Siete No.1. The study results are shown in Table-12.1

Table-12.1 Comparison of the El Siete No.1 Dam Type

Item	Unit	Gravity Concrete Dam (A)	Concrete Facing Rockfill Dam (B)	Difference (B) - (A)
HWL	m	1,450	1,450	
LWL	m	1,440	1,440	
Draw down depth	m	10	10	
Effective strage cap.	10 m^3	540	540	
Dam hight	m	55	57	
Crest length	m	207	207	
Dam volume	m^3	143,000 (Concrete)	326,000(Rock) 43,200(Concrete)	
Total construction cost	10^3US$	134,740 *	149,780 *	+15,040
Care of river No.1 dam	"	2,680 23,360	37,930	
Sub-total	"	26,040	37,930	+11,890
Others	"	108,700	111,850	

* No include the cost of transmission line

(2) Economic Comparison Study of No.1 and No.2 Headrace Tunnel Inside Diameters

- No.1 and No.2 Headrace Tunnels

- No.1 Headrace Tunnel

No.1 headrace tunnel is a pressure tunnel with a circular cross section and a length of 3,145 m. It was decided to study the economic nature of the inside diameter for the case of a 25 m^3/s discharge capacity selected in Chapter 11.

In general, determination of a pressurized headrace tunnel's inside diameter is made by economic comparison, seeking the minimum sum of the annual cost related to the tunnel construction cost and annual electricity charge loss related to the hydraulic gradients.

In case of the No.1 headrace tunnel, calculations were made at a pitch of 0.2 m for the inside diameter in a range of 2.8 m to 4.2 m. The results are shown in Fig. 12-1. As the figure shows, the economical inside diameter for the No.1 headrace tunnel is 3.4 m.

- No.2 Headrace Tunnel

No.2 headrace tunnel is a pressure type with a circular cross section and a length of 9,109 m. It was decided to make a study of the economics with a 28 m^3/s discharge capacity.

Calculations were made at a pitch of 0.2 m for the inside diameter in a range of 3.0 to 4.4 m. The results are shown in Fig. 12.2. The economical inside diameter is 3.6 m

(3) Economic Comparison Study of No.1 and No.2 Penstock Average Inside Diameters

- No.1 Penstock

No.1 penstock is a surface type 1,301 m long, bifurcated at its end section by a Tee-branch. The economics were examined for the case of a 25 m^3/s discharge capacity selected in Chapter 11.

The economic comparison technique was the same as in the case of the headrace tunnel.

The study results are as shown in Fig. 12-3. The optimum average inside diameter is 2.70 m.

- No.2 Penstock

No.2 penstock is a surface type 1,045 m long bifurcated at its end section by a Y-branch. The economics were examined for the case of a 28 m^3/s discharge capacity. The economic comparison technique was the same as previously outlined.

The study results are shown in Fig. 12-4. The optimum inside diameter is 2.75 m.

Fig. 12-1 Economical Diameter Diagram for El Siete No.1 Project Headrace Tunnel

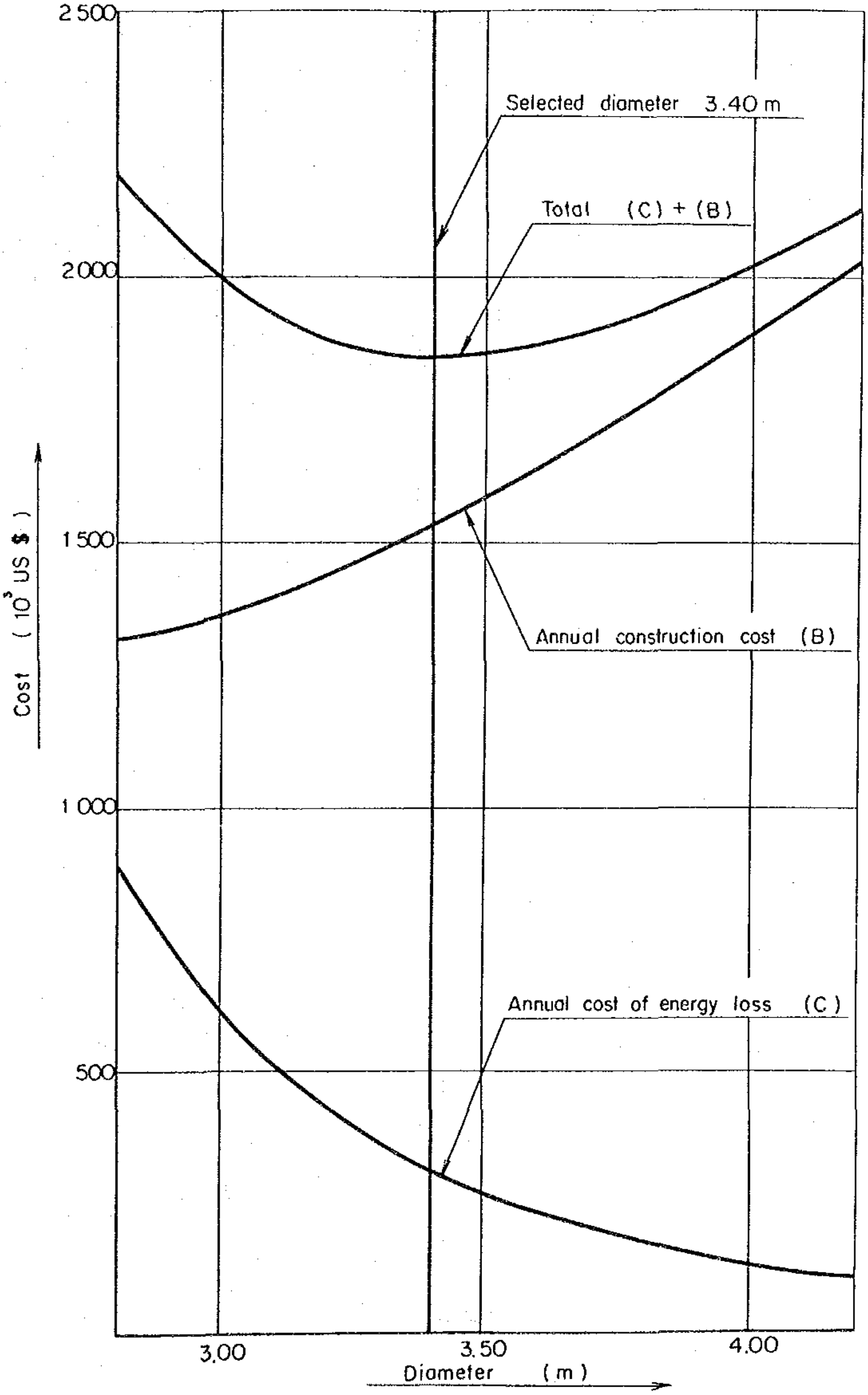


Fig. 12-2 Economical Diameter Diagram for El Siete No.2 Project Headrace Tunnel

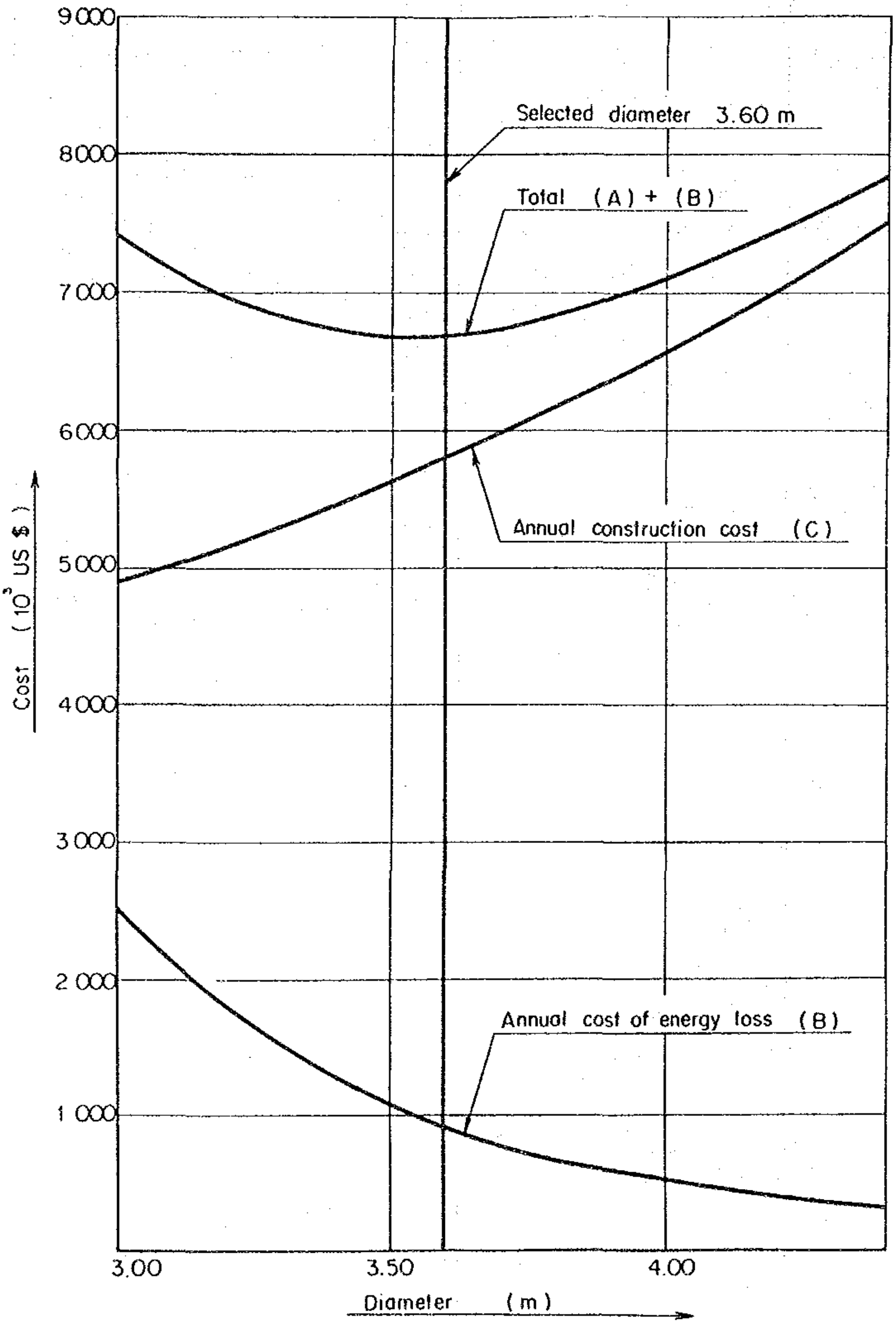


Fig. 12-3 Economical Diameter Diagram for El Siete No.1 Project Penstock Line

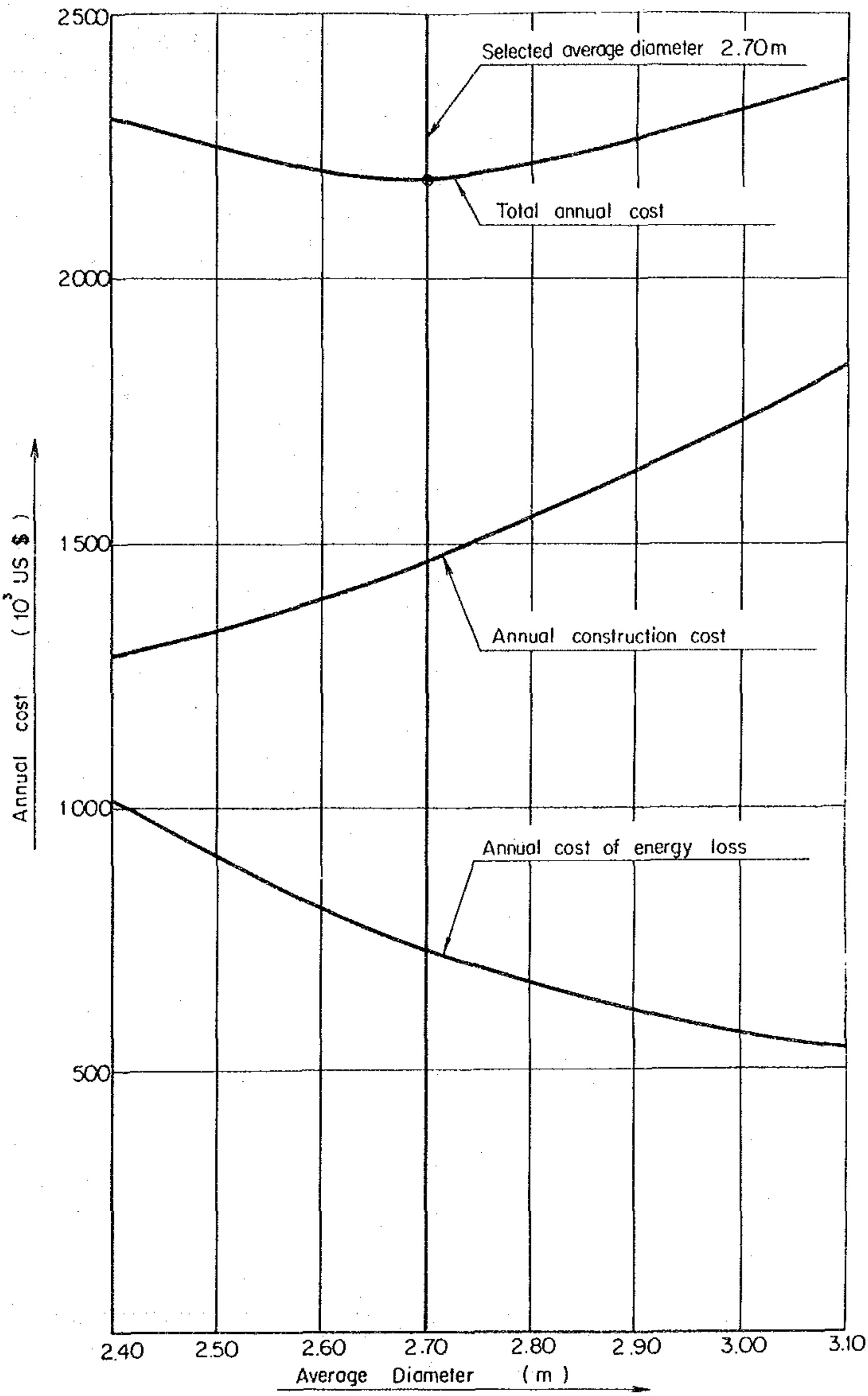
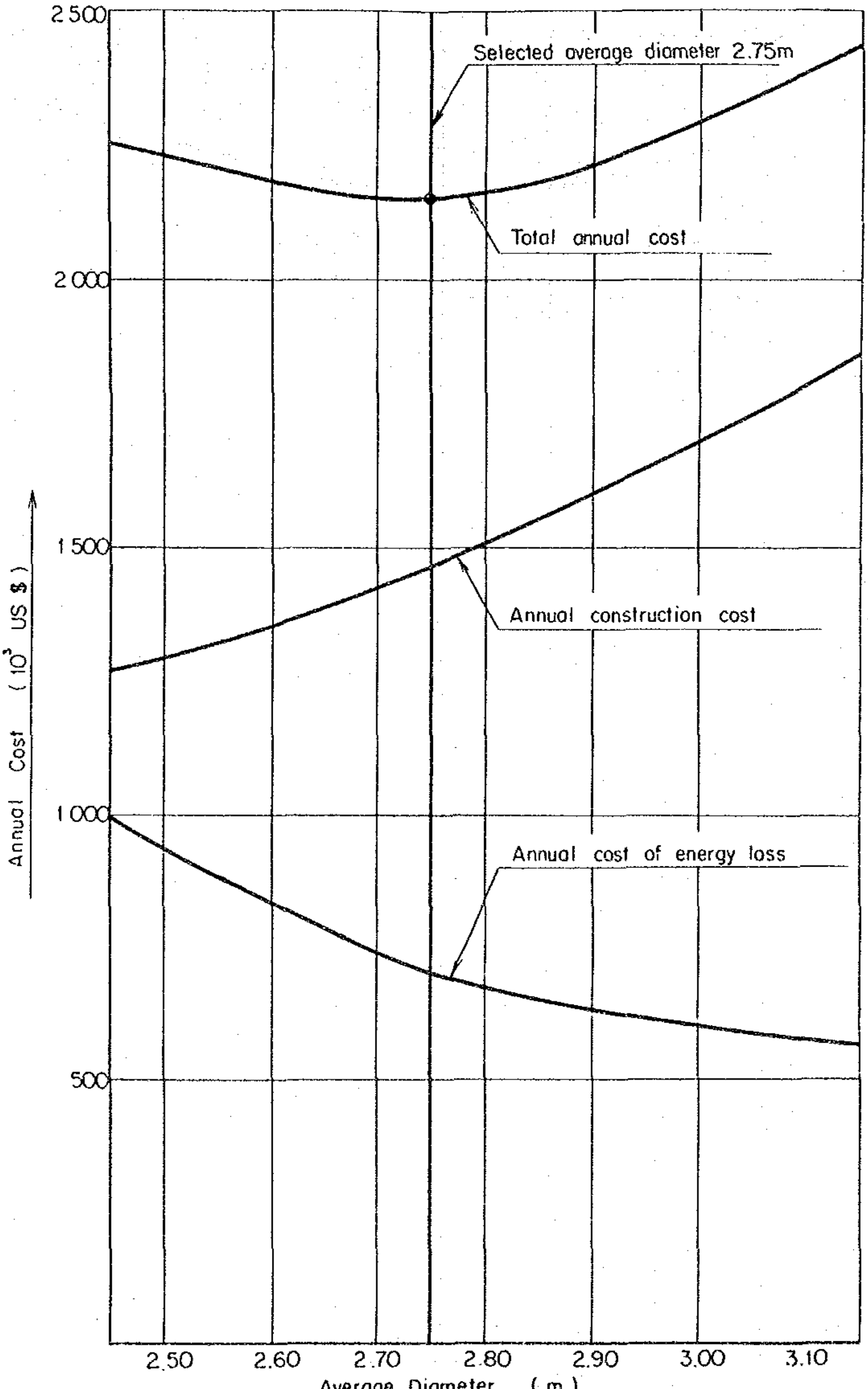


Fig. 12-4 Economical Diameter Diagram for El Siete No.2 Project Penstock Line



(4) Comparison of Powerhouse Types (Surface, Underground)

From the topographic and geological standpoints, the El Siete No.1 and No.2 Power Stations powerhouse structures can be of either surface or underground type.

The type selected would be decided by the difference in construction costs, and it is necessary to make comparisons concerning construction items such as penstock, powerhouse and tailrace.

Regarding El Siete No.1 Power Station, the layout for a surface type is shown in Dwg.-27, and the underground type shown in Dwg.-48.

In the case of the surface type it would be a simple combination of penstock, powerhouse and tailrace.

In the case of the underground type, it would be composed of an underground penstock, underground powerhouse, tailrace tunnel, access tunnel, cable tunnel and work adit.

The differences between the two would be an effective head of 353 m for the surface type and 357.5 m for underground type, with a maximum output of 75 MW and 76.2 MW, respectively.

However, with respect to total construction costs, it would be US$134,740 thousand for the surface type, and US$148,925 thousand for the underground type, for the benefit-cost ratios (B/C) of 1.378 and 1.262, respectively.

The surface type would cost US$14,185 thousand less, and the benefit-cost ratio (B/C) would be also better.

The layout of the surface type for El Siete No.2 Power Station is shown in Dwg.-38 and the underground type shown in Dwg.-49.

In the case of the surface type, it would be a simple combination of penstock and powerhouse.

In the case of the underground type, it would be composed of an underground penstock, underground powerhouse, tailrace tunnel, outlet, access tunnel, cable tunnel, and work adit.

The difference between the two would be an effective head of 357.5 m for the surface type, in contrast to a 358 m for the underground type, with a maximum output of 85 MW and 85.2 MW, respectively.

However, with respect to total construction costs, it would be US$114,771 thousand for the surface type, and US$128,958 thousand for the underground type, for the benefit-cost ratios (B/C) of 1.825 and 1.624, respectively.

The surface type would cost US$14,187 thousand less, and, the benefit-cost ratio (B/C) would be also better.

Consequently, for the El Siete No.2 Power Station powerhouse, a surface type was applied, similarly to the case of the El Siete No.1 Power Station.

The study results are shown in Tables-12.2 and 12.3.

Table-12.2 Powerhouse Type for El Siete No.1 Project

Item	Unit	Ground Surface type(1)	Under ground type (2)	Difference (2) - (1)
Max. discharge	m^3/s	25	25	-
Effective head	m	353	357.5	+4.5
Max. Output	MW	75	76.2	+1.2
Annual Energy	Gwh	508	514.5	+6.5
Total Const. Cost	10^3US$	134,740*	148,925*	+14,185
Penstock	"	3,000	3,150	
Power house	"	7,480	10,000	
Tailrace	"	690	2,870	
Access tunnel	"	-		
Cable tunnel	"	-	9,860	
Adit	"	-		
Hydraulic Eq.	"	16,112	12,092	
(Sub-total)		(27,282)	(37,972)	(+10,690)
Others	"	107,458	110,953	
Annual benefit (B)	10^3US$	22,275	22,556	
Annual cost (C)	"	16,169	17,871	
Benefit-cost ratio (B/C)		1.378	1.262	

* Transmission line costs not included

Table-12.3 Powerhouse Type for El Siete No.2 Project

Item	Unit	Ground Surface type(A)	Under ground type (B)	Difference (B) - (A)
Max. discharge	m^3/s	28	28	-
Effective head	m	357.5	358.0	+0.5
Max. Output	MW	85	85.2	+0.2
Annual Energy	Gwh	588.3	589.1	+0.8
Total Const. Cost	10^3US$	114,771*	128,958*	+14,187
Penstock	"	2,210	3,240	
Power house	"	4,940	7,630	
Tailrace	"	-	2,930	
Access tunnel	"	-	8,140	
Cable tunnel	"	-		
Adit	"	-		
Hydraulic Eq.	"	12,271	8,444	
(Sub-total)		(19,421)	(30,384)	(+10,963)
Others	"	95,350	98,574	"
Annual benefit (B)	10^3US$	25,100	25,135	
Annual cost (C)	"	13,772	15,475	
Benefit-cost ratio (B/C)		1,825	1,624	

* Transmission line costs not included.

12.2 Design of Individual Structures

12.2.1 Design of El Siete No.1 Power Station Civil Structures

The El Siete No.1 Power Station civil structures are as follows:

- El Siete No.1 Dam

Type	:	Concrete gravity
Height	:	55 m
Crest	:	207
Volume	:	143,000 m^3
Design flood	:	1,160 m^3/s
High water level	:	1,450 m
Low water level	:	1,440 m
Avilable drawdown	:	10 m

- El Siete No.1 Auxiliary Dam

Type	:	Concrete gravity
Height	:	21.5 m
Crest length	:	148.0 m
Volume	:	36,000 m^3
Design flood	:	1,110 m^3/s
High water level	:	1,460 m
Avilable drawdown	:	0 m
Maximum intake wate	:	25 m^3/s

- El Siete No.1 Auxiliary Sedimentaton Basin

Type	:	Double tank
Tank length	:	67 m
Tank width	:	40 m
Tank depth	:	6.16 m
Tank capacity	:	16,500 m^3

- El Siete No.1 Axiliary Connecting Headrace Tunnel

Type	:	Non pressure
Length	:	858.2 m
Inside diameter	:	3.4 m
Cross section	:	Standard horseshoe shaped
Gradient	:	1/600
Capacity	:	25 m^3/sec

- El Siete No.1 Intake

Type	:	Inclined
Maximum intake wate	:	25 m^3/sec

- El Siete No.1 Headrace Tunnel

Type	:	Pressure
Length	:	3,144.53 m
Inside diameter	:	3.4 m
Cross sectio	:	
Gradient	:	1/1000
Capacity	:	25 m^3/sec

- El Siete No.1 Surge Tank

Type	:	Restricted oriffice, vertical shaft
Height	:	43.05 m
Inside diameter	:	7.8 m
Cross section	:	Circular
Port diameter	:	1.4 m

- El Siete No.1 Penstock

Type	:	Surface, rocker bearing
Number of lines	:	1 - 2 (after branched)
Capacity	:	25 m^3/sec
Inside diameter	:	3.40 m - 1.25 m
Steel thickness	:	9 mm - 29 mm
Length for No. 1 unit	:	1,300.80 m
Length for No. 2 unit	:	1,287.80 m
Branch type	:	T-branch

- El Siete No.1 Powerhouse

Type	:	Surface
Width	:	20.00 m
Length	:	55.50 m
Height	:	20.80 m
Turbine	:	Vertical-Shaft Pelton x 2 units
Generator	:	Vertical-Shaft synchronous generator x 2 units
Turbine center elevation	:	1,074.00 m
Tailwater level	:	1,071.00 m

- El Siete No.1 Tailrace tunnel

Type	:	Non-pressure
Length	:	184.72 m
Inside diameter	:	3.60 m
Cross section	:	Standard horseshoe shaped
Gradient	:	1/1,000
Capacity	:	25 m^3/s

- El Siete No.1 Outdoor Switchyard

Lot	:	8,370 m^2
Width	:	94 m
Length	:	89 m

(1) El Siete No.1 Dam

El Siete No.1 Dam is to secure an effective storage capacity of 540,000 m^3 for daily regulation of water used at the El Siete No.1 and No.2 Power Stations. The basic planning and designing details for No.1 dam are as follows:

- Design flood discharge, handling of 1,160 m^3/s (1,000-year return period flood)
- Sediment discharge disposal measures for 6,075 m^3/km^2
- 10 m rapid water level fluctuation for daily regulation
- Meteorological conditions during construction

- Conditions of local materials for dam

- Cross section and geological conditions of damsite

These basic details were considered and studies made of various dam types. As a result, as a gravity type concrete dam would be optimum for coping with the above conditions and would also be inexpensive from an economic standpoint, it was decided to adopt a concrete gravity type as described in 12.1.

Regarding the regulating pond, high water level comparison studies were made of the economics of various water levels and EL. 1,450 m being the most economical was selected.

Regarding available drawdown, 10 m, at which a daily 540,000 m^3 regulating capacity could be secured was adopted. (See Area-Capacity Curve, Dwg.-16)

As a result of these studies, the dam height was made 55 m from the foundation rock.

In selection of the dam axis, the topographical and geological conditions at the prospective damsites were considered and two alternatives, upstream and downstream, were selected, comparison studies made, and the downstream axis providing better economics selected. (See 11.2.4)

The matters considered in the design of El Siete No.1 Dam are given below.

1) Spillway

The design flood discharge at El Siete No.1 Dam is taken to be 1,160 m^3/s as stated in 8.4.

Spillways to handle this flood are to be provided at the middle of the dam, and these were made a combination of normal-use and emergency spillway types.

A normal-use spillway would discharge floods up to 200 m^3/s-class which corresponds to past maximum flod by free overflow of the dam crest at surcharge water level without operating the gates.

On the other hand, in the event of a flood exceeding 200 m^3/s, the structure is to be such that this flood will be discharged by an emergency spillway having two 9.00 m high, 10.00 m wide sluice gates together with the normal-use spillway.

Gate operation errors can be prevented through the use of the two spillways, while the normal-use spillway will be effective against unforeseen floods induced by slope failures which may occur in the reservoir area or farther upstream. These spillways will occupy a 88 m section of the 207 m dam crest length, corresponding to 42 percent of the entire dam crest length.

2) Gate for El Siete No.1 Regulating Pond Sediment Flushing

No.1 dam will have two sediment flushing gates to discharge sediment deposited in the regulating pond to the area downstream of the dam.

To handle sediment deposited in the No.1 regulating pond and to maintain the daily regulating capacity, it is necessary, when the river runoff at the regulating pond exceeds a given level, to fully open the previously described gates to return the interior of the regulating pond to its original stream condition and cause inflow to flow down naturally, and through the force of the flow to turn the deposited layer into slurry for discharge downstream.

Sediment flushing gates are necessary for the previouslymentioned operation, and therefore highly important are facilities.

The sediment flushing gate ducts are to have sluice gates at the front and tainter gates at the back giving consideration to ease of opening and closing operations. The duct cross sections are to be 6 m high and 6 m wide, with consideration given to facilitate supplementary sediment disposal work by construction equipment.

3) El Siete No.1 Dam River Diversion

The El Siete No.1 dam site is comparatively small width, while the stream is swift, and the results of hydrological analyses show that maximum floods at 200 m^3/s based on the probability in 25 years return period are large compared with the river channel, and the rise of the water level is great. These conditions were considered and it was decided to adopt a tunnel diversion system.

Regarding the diversion tunnel location, on considering the geologies of the two banks at the El Siete No.1 dam site, the right bank where deposited layers and a weathered zone extend deep inside was avoided, and the left bank where the original ground is comparatively stable was selected. The length is to be 356.4 m, the gradient 1/18.72, and the cross section a semi-circular top, rectangular bottom shape of inside diameter 4.50 m. The diversion tunnel capacity was designed at the past maximum flood of 200 m^3/s.

The upstream and downstream cofferdams were made rockfill types, the heights being 11.0 m and 6.5 m, respectively.

Regarding abnormal floods during construction, considerations were given so that the sediment flushing ducts in the dam body can be used as diversion.

4) El Siete No.1 Dam Foundation Rock Treatment

The rock comprising the foundation of El Siete No.1 Dam is basalt as described in 10.4.2, and it was decided that foundation rock treatment for the dam would be carried out giving consideration to the fact that there are many seams.

Foundation rock treatment was designed to consist of consolidation grout, and from the inspection gallery in the dam body, one row of curtain grout at 1.5 m intervals and 20 m depths at the middle of the dam.

5) El Siete NMo.1 Dam Right Wing Cut-off Concrete Treatment

The right abutment of El Siete No.1 Dam has distributions of deposited layers and a deep layer of a weathered zone as shown in Dwg.-09 and Dwg.-17. As the mountainside is comparatively steep at 1/1.3, if the dam foundation were to be excavated by surface work, the cut slope would be high and the quantity of excavation large. It was therefore decided that the comparatively stable weathered zone would be excavated in tunnel form and filled with concrete to construct a cut-off wall.

(2) El Siete No.1 Auxiliary Dam and Auxiliary Intake

El Siete No.1 Auxiliary Dam is a facility to be used as an alternate to the No.1 inake facility during flushing operations at the No.1 regulating pond.

For this purpose, El Siete No.1 Auxiliary Dam will serve to take in the 25 m^3/s of water required for power generation before it enters the No.1 regulating pond, with the remaining excess water discharged to flush out the No.1 regulating pond.

Intake from the No.1 auxiliary dam would be commenced at the time the inflow at the auxiliary dam site exceeds the 25 m^3/s maximum available discharge of El Siete No.1 Power Station.

The location, selected from the standpoints of purpose and function, is to be at the end of the No.1 regulating pond backwater, approximately 1 km upstream of El Siete No.1 Dam, and approximately 350 m downstream of the confluence of Sanchez River and the Atrato River. The location, from a topographical point of view, is where the river width is comparatively large and a low, flat open space of river terrace also exists so that the auxiliary dam, auxiliary intake, and a sedimentation basin can be laid out in compact form.

The No.1 auxiliary intake dam is to be a concrete gravity type, and spillways to discharge the design flood of 1,110 m^3/s having similar structures as the No.1 dam are to be provided. The dam height is to be 21.50 m, the dam crest length 148 m, of which the spillway portion is to be 52 m so that 39 percent of the entire crest length is to be the spillway portion.

The method to handle sediment at the No.1 auxiliary intake dam when that dam is in use, is to partially open the two spillway gates and allow discharge while operating in a manner that the intake water level will not be lowered. When the No.1 regulating pond is being used, the two spillway gates are to be left fully open, causing all inflow to be discharged into the No.1 regulating pond so that sediment will be floused out by the force of the water.

The auxiliary intake is to be provided adjacent to the No.1 auxiliary dam.

Two control gates are to be installed at the auxiliary intake to control inflow.

(3) *No.1 Auxiliary Sedimentation Basin*

Intake from the No.1 auxiliary intake dam is to be during sediment disposal at the No.1 regulating pond. The intake period is to be an average of approximately 3 months annually, and during the high-water period of the Atrato River when the sediment content of inflowing water is high. For this, an auxiliary sedimentation basin is to be provided between the auxiliary intake and the auxiliary connecting waterway.

The structure is to be such that there will be a capacity for sediment of 0.1 mm and over contained in a maximum available discharge of 25 m^3/s to be settled. Two sedimentation tanks 67 m long, 20 m wide and 6.16 m average depth are to be laid out in parallel for a structure that sediment settled in the tanks can be removed without stopping power generation.

In effect, the sediment flushing system is for water to be conducted into one of the two tanks, partially or fully opening a partitioning gate installed at the upper part of the middle partitioning wall to cause the water conducted to overflow from the top to the other empty tank, and utilizing the force of this falling water to agitate the settled sediment in the tank and make it into slurry form, flushing this out of the Atrato River main stream by drain gallery holes provided at the two sides of the tanks. Such a system is frequently applied to this type of sedimentation basin and the effectiveness of the system has been varified.

The front part of the tank is provided with a transition portion to lower the flow velocity of the water taken in at the auxiliary intake in a limpid condition without vortex, and the downstream part for water storage with apurtenant control gates to temporarily hold water after settling and to control the flow of water into the connecting tunnel and spillway for excess water discharge, thereby providing all the required functions of a settling basin.

(4) No.1 Auxiliary Connecting Headrace Tunnel

The No.1 auxiliary connecting headrace tunnel is to conduct water to the No.1 headrace tunnel drawn at the No.1 auxiliary dam during flushing operations of sediment deposited in El Siete No.1 Regulating Pond, to be used for power generation. The maximum available discharge is to be 25 m^3/s, the length 858.2 m, inside diameter 3.4 m, gradient 1/600, and is to be a non-pressure tunnel.

The connecting headrace tunnel route would go mainly through shale and chert bedrock. Since it would pass directly under Santa Lucia Ravine, thorough attention was paid to the matter of cover. In view of the geological conditions, the entire length is to be concrete line.

For the junction with the No.1 headrace tunnel, which is a pressure tunnel, the connecting headrace tunnel is to be joined to the No.1 headrace tunnel by a inclined-shaft tunnel to prevent the intermixture of air.

The head of the inclind-shaft tunnel is also to be utilized in an arrangement by which the water in the headrace tunnel will be kept from showing reverse flow to the sedimentation basin while the No.1 regulating pond is being used.

(5) No.1 Intake

The No.1 intake is a structure to take in daily-regulated water at El Siete No.1 Regulating Pond in accordance with the power station load. The topography, geology, available drawdown, and the estimated sedimentation condition in the regulating pond were considered for the No.1 intake location, and it was decided to provide it apurtenant to the dam, parallel to the dam axis at the left bank of the damsite where functional requirements would be satisfied and where most economical.

The intake was made a completely inclined type with gate shaft in consideration of the topography (particularly, the slope of the left bank mountainside). The maximum intake is 25 m^3/s.

An inclined screen is to be provided in front of the intake to facilitate removal of inflowing floating trash. A bucket is also to be provided at the front of the intake to temporarily store sediment flowing into the tunnel, the bucket being to connected to the sediment flushing gatesd and ducts in the dam body for removal by the force of flowing water.

The control gate for intake control or for use during repair of the headrace tunnel is to be installed inside the intake shaft. This gate was designed as a double-sided watertight gate to be watertight against water pressure from inside the tunnel during intake at the No.1 auxiliary intake dam (No.1 regulating pond empty) and against water pressure from the No.1 regulating pond side.

(6) No.1 Headrace Tunnel

No.1 headrace tunnel is designed for a maximum discharge of 25 m^3/s, with a 3.4 m inside diameter, 3,145 m length, and a circular cross section. The tunnel is to be a pressure type. (See 12.1)

The No.1 headrace tunnel route is to be laid out in a form to short-cut a meandering portion of the Atrato River where it changes course from west to south, and was selected for the shortest tunnel length to the El Siete No.1 powerhouse giving consideration to topography, geology, earth covering, and work adits.

In consideration of experience with pressure tunnel construction in Colombia, stability of the original ground, and distributions of faults and joints, it was decided that the entire length would be concrete lined. Consolidation grouting and high-pressure grouting are also to be performed as reinforcing work for the stabilization of the original ground.

Work adits for construction are to be Work Adit No.1 (L = 320 m) at the No.1 intake side and Work Adit No.2 (L = 120 m) at the No.1 surge tank site, with the plan to excavate of the headrace tunnel from both upper and lower portals.

The headrace tunnel gradient is to be 1/1,000 in consideration of drainage during excavation.

(7) No.1 Surge Tank

No.1 surge tank is to be provided at the end of No.1 headrace tunnel (L = 3,145 m). The surge tank is to be a restricted-orifice, vertical-shaft type in consideration of the topography, geology, and work execution conditions.

The inside diameter of the vertical shaft was made 7.8 m considering conditions for stability against surging vibrations (Thoma's conditions) and with a cross secton design to ensure minimum construction costs.

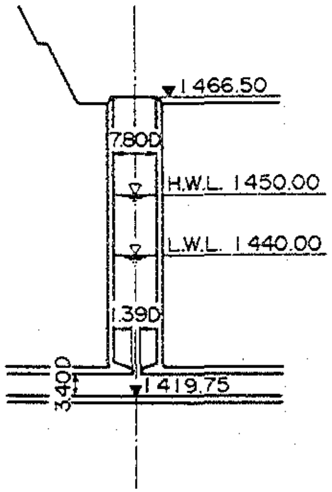
The inside diameter of the restricted orifice was made 1.4 m as a result of studying the critical flow of the restricted orifice and the optimum inside diameter at the time of load breaking and at sudden load increases.

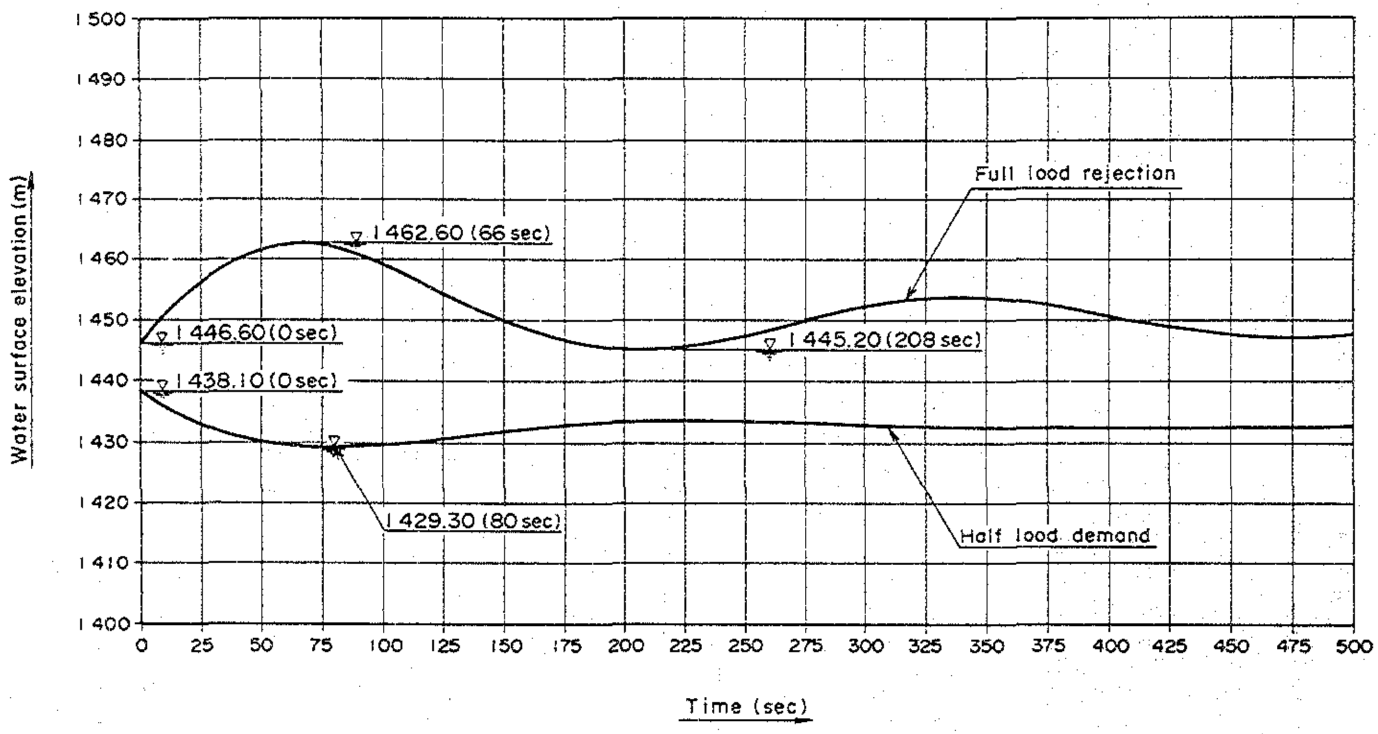
The surge tank was designed for a height of 43.05 m analyzing the upsurge water level at total load breaking (full closure from 25 m^3/s discharge) at high water level of No.1 regulating pond, and downsurge water level at half-load surging (sudden increase from 12.5 m^3/s discharge to 25 m^3/s discharge), and considering an upper allowance on top of the water level fluctuation range and an allowance at the bottom to prevent the intermixture of air.

The entire vertical shaft and bottom tunnel portions of the surge tank are to be lined with reinforced concrete. Further, in consideration of work execution and prevention of seepage into the ground, the surfaces are to be reinforced with steel liner up to a height of 4 m at the bottom of the vertical shaft and the bottom tunnel (L = 40 m).

The Runge-Kutta Formula was applied to surging design calculations, and calculations of damping waves were made employing an IBM S370-M 155 computer.

Fig. 12-5 Surging Curve of No.1 Surge Tank





In the case of full load rejection

Q = 25.0 m³/s → 0.0 m³/s
n = 0.0110
ϵup = 0.4788
H.W.L. = 1450.00

In the case of half load demand

Q = 12.5 m³/s → 25.0 m³/s
n = 0.0150
ϵdown = 0.8904
L.W.L. = 1440.00

(8) No.1 Penstock

Regarding the No.1 penstock, a study was made of the two types of surface type and underground embedment type. From these studies, the economic surface type and with which work execution, would be easier, and for which it would be possible to shorten the construction period, was adopted. (See 12.1 (4))

No.1 penstock route connecting the surge tank and No.1 powerhouse was laid out on a ridge of the mountainside to provide the shortest distance with lengths for No.1 unit of 1,300.8 m and for No.2 unit of 1,287.8 m. Of these, the horizontal 148 m section, corresponding to the connection with the surge tank, was made an embedded tunnel type.

These would normally be one penstock line when the maximum discharge of 25 m^3/s and penstock length are considered, and it was designed to connect to two turbines on bifurcation at the end of the penstock. The bifurcation pipe was made a T-type since the penstock and powerhouse axis parallel each other.

From the economic comparisons of various inside diameters for the penstock, a starting point inside diameter of 3.40 m and an ending point (inlet valve) inside diameter of 1.25 m were selected, and the variation in inside diameter between the two is shown in Dwgs.-27 and 28. (See 12.1(3))

The surface of the penstock is to be fixed by twelve anchor blocks with intervals provided support by rocker saddles 12 m apart. Connection of the individual lengths of pipe are to be by field welding. The pipe shell material is to be SM50 (JIS specifications) or ASTM A 440 in consideration of pipe thickness and field welding. The Individual pipe length thicknesses were decided calculating water hammer pressures. The results are shown in Dwg.-29. The water hammer pressure rise at the turbine inlet will be 11 percent of hydrostatic pressure.

(9) El Siete No.1 Powerhouse

Two vertical-shaft Pelton turbines were adopted for the El Siete No.1 power station in consideration of available discharge, head, number of units, utility factor, operation and maintenance. A comparison study was made of the two cases of surface type and underground type for the powerhouse, and the surface type providing better economics was adopted. (See 12.2.1(4))

Regarding the location of the El Siete No.1 powerhouse, since it would be necessary for the water after power generation at El Siete No.1 Power Station to be conducted to the El Siete No.2 powerhouse on passing through the dam body of the El Siete No.2 intake dam, it was decided to build a lot on a table near Aguila Ravine at the left-bank side of the Atrato river and which would be as close as possible to the El Siete No.2 intake dam where the foundation rock is stable, and floods of the Atrato River can be avoided.

The powerhouse building is to be a surface type of reinforced concrete construction, the required dimensions being 20 m wide, 55.5 m long, and 20.8 m high.

Two Pelton turbines and generators, an erection bay, control room, cable handling room, storeroom, office room, etc. are to be accomodated in this building.

The powerhouse foundation structure is to be provided on bedrock by open excavation below the lot at El. 1,081 m.

Two vertical-shaft Pelton turbines and appurtenant equipment are to be accomodated in the foundation, and open draft waterways are to be provided to conduct discharge water to the outlet. Further, appurtenant to the drafts, an open concrete afterbay is to be provided.

Since the water of the Aguila Ravine will come down on the mountain side of the powerhouse, a small intake weir is to be provided at Aguila Ravine to prevent inflow to the powerhouse lot, and the water is to be discharged directly to the Atrato River by an open Channel 150 m in length (width 1.0 m, height 1.5 m).

An access road is to be newly constructed from the national road highway (Medellin City - Quibdo City) at the right bank side for access to the powerhouse. Crossing of the Atrato River to the left bank side is to be by bridge over which heavy equipment such as generators, turbines, and transformers are to be transported. Accordingly, a steel bridge, 95 m long and 5 m wide is to be constructed across the main stream of the Atrato River.

(10) No.1 Tailrace Tunnel

No.1 tailrace tunnel is to conduct a maximum available discharge of 25 m^3/s used at El Siete No.1 Power Station to the downstream El Siete No.2 Power Station and to connect conduit provided inside the body of the No.2 Intake Dam. The length is to be 185 m, and to avoid water level variation (affecting operating efficiency of Pelton turbines) at the El Siete No.1 Power Station afterbay, it is to be a non-pressure type of standard horseshoe shape and 3.60 m inside diameter.

To ensure there will be no hindrance to the operation of El Siete No.1 Power Station at maximum output during stoppage of operation and repairs of El Siete No.2 Power Station, a spillway is to be provided appurtenant to El Siete No.2 Intake Dam at the outlet section of the No.1 tailrace tunnel.

The tailrace tunnel route is to go upstream along the Atrato River from the El Siete No.1 Power Station afterbay, giving consideration to earth cover and curve radius. The entire length of the tailrace tunnel is to be reinforced by concrete lining.

(11) No.1 Outdoor Switchyard

No.1 outdoor switchyard, when stepping up the voltage at El Siete No.1 Power Station to 230 kV, taking out to Ancon Sur Substation (ISA), and taking in from El Siete No.2 Power Station are considered, will require a lot 94 m wide and 89 m long (area 8,370 m^2).

To build a lot for this switchyard adjacent to the El Siete No.1 powerhouse will require excavation of a broad area behind the powerhouse, and a large volume of excavation, demanding higher construction costs. Consequently, it was decided that a lot for an outdoor switchyard would be built at the same EL. 1,081 m at a comparatively flat tableland on a hill adjacent to the national road at the right bank side of the Atrato River. This lot would be developed by partial open cut and the remaining part banked with the excavated soil and compacted.