Chapter 7 Geology and Construction Materials

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7.1 Geology

7.1.1 Introduction

In this section, geotechnic conditions of the Los Llanos Project area and its main civil structure sites are described based on all of topographic and geologic data collected, in cooperation with the ICE and JICA study team, from literatures published in Costa Rica, the field investigations and ICE's laboratory tests between August, 1994 and June, 1995.

It is, also, explained in this section that the said geotechnic conditions have satisfied assessing works for the feasibility study of this project.

A summary of the field investigations conducted by the JICA study team at the project site and of the geological/geotechnical investigations conducted by ICE are given in 7.1.2. A summary of the topography and geology of the regional area, which includes the project site, is explained in 7.1.3. Topographic and geologic conditions, as well as a geoengineering evaluation of the construction sites for the major civil structures (including the reservoir, dam, powerhouse, and the waterway route) are discussed in 7.1.4 through 7.1.7. The results of the in-situ rock testing conducted by ICE, as well as the results of the geophysical prospecting and drill-core laboratory test, are described in 7.1.8 through 7.1.10.

The main data discussed in this section are shown in Figures 7-1 through 7-14 attached with this report.

7.1.2 Summary of Geo-Investigation Works

ICE had conducted a considerable number of geological/geotechnical investigations prior to the feasibility study for this project. Several additional investigations were recommended by the JICA team and conducted by ICE during the study. A summary of the investigations is given in paragraph (2).

A summary of the on-site field investigation, conducted by the study JICA in conjunction with ICE, and the results of the geological/geotechnical investigations are given in paragraph (1).

Regarding the topographic maps of the project site, topographic maps prepared by ICE (scale for the greater area: 1/10,000, scale for the dam and powerhouse site: 1/2,000) were used in the early stage of the study, and aerial topographic maps prepared for the project (scale for the greater area: 1/5,000, scale for the dam and powerhouse site: 1/1,000) were used in the later stage of the study. Hereinafter, the former maps are referred to as the old topographic maps, and the latter maps as the new topographic maps in this report.

(1) Field Investigations by JICA Team

The on-site field investigations were conducted by the JICA study team between September 1994 and July 1995 a total of five times, for which the total number of survey days was 34 days.

The first field investigations included observation and photographing of the drill core, and examination of the exploratory adits. In the second field investigations, geological mapping of the major civil structure sites was conducted using the old topographic maps. In the third field surveys, an additional geological mapping of the major civil structure sites (especially the sites for the structures which require extra geotechnological attention), was conducted using the drafts of the new topographic maps. Additional observing and photographing of drill cores were also conducted in the third field surveys. In the fourth field surveys, a further geological mapping of the major civil structure sites, observation and photographing of drill cores were conducted using the new topographic maps. In the fifth field surveys, a further geological mapping were conducted.

The topographic interpretation of the project site was reviewed by the JICA study team using the aerial photographs taken for the aforementioned topographic maps. The results of the interpretation of the aerial photos, as well as the results of the field surveys, were reflected in the geological maps of the feasibility study.

(2) Geologic/Geotechnic Investigations by ICE

As mentioned previously, ICE had conducted a considerable number of geologic/geotechnic investigations prior to the feasibility study. Tables 7-1 through 7-8 show the items and numbers of geologic/geotechnic investigations. As for the subsurface explorations by core drilling 19 holes (739.55m in total) have been drilled in this project area as shown in Table 7-1. Of these holes, Drillholes PHLL18TP on the penstock route and PHLL19CM which were additionally recommended by the JICA team and conducted by ICE are also included in Table-1.

Two exploratory adits (36.15 m in total and a 5.5 m long open-cut section) and two pits (19.05 m in total) were excavated at the damsite as shown in Tables 7-2 and 7-3. Plate jack tests were carried out in the Adit No.1 and pit No.2. These test results are shown in Table 7-8.

ICE conducted the following geophysical prospectings: seismic prospecting and at the damsite Vp/Vs measurement in Adit No.1, on the right bank along the down-stream dam axis; seismic prospecting and resistivity survey along the headrace tunnel route and penstock routes; and seismic prospecting at the powerhouse site; as shown in Tables 7-4 to 7-7. The JICA team re-analyzed several profiles of the seismic prospecting. Details are given in 7.1.7.

Investigation and tests conducted by ICE related to examination of construction materials are discussed in 7.2.

7.1.3 Regional Geology

(1) General Topography

The project site is located intermediate area between a mountainous area (1,000 to 1,500 m above sea level) and low lands faced the Pacific Ocean. The damsite is approximately 500 m above sea level, and the site for the powerhouse is approximately 100 m above sea level. The rivers are generally narrow throughout the area, forming a V-shaped gorge in certain places. The cliffs along the Rio Naranjo near the damsite are very prominently hanging over the river. The powerhouse site is located on the left bank of the Rio Paquita, which runs through the boundary of the mountainous area and the alluvial lowlands along the Pacific Ocean, thus forming a complex topography with an irregular and rugged shoreline.

No prominent landforms caused by landslides were observed in the project area.

(2) General Geology

The geology of the project area, located at the middle-stream portions of the Rio Naranjo and the Rio Paquita, is composed of sedimentary rocks of the Jurassic to the Eocene and is partially overlain by nonconsolidated sediments of the Quaternary.

Sedimentary rocks which make up the bedrock of the area are composed of ophiolite (volcanic rocks from undersea eruptions) and other oceanic sedimentary rocks called the "Nicoya Complex". The project area is underlain by characteristic large-size and circular conglomerate, fine to coarse grained sandstone, siltstone and marlstone, in other words, relatively coarse grained sedimentary rocks. These rocks indicate shallow sea environment when deposition. In addition, limestone is outcropped among marlstone as allochthonous block (olistolith) in the lower reaches of the river downstream from the site for the powerhouse.

Sediments of the Quaternary period contains of residual soil, colluvial soils, talus deposits, terrace deposits and riverbed deposits. A relatively large amount of residual soil and colluvial soil are widely distributed in gentle slopes, whereas the rest of the sediments are distributed only at limited areas.

Table 7-9 shows the geological sequence and lithological characters of the project areas.

The above-mentioned sedimentary rocks have bedding planes striking mostly in the northwest-southeast direction with northeast dipping. A summary of the geologic and geoengineering viewpoints of the project is given in the following section.

7.1.4 Site Geology

(1) Damsite

As mentioned previously, three potential locations for the damsite were compared and contrasted in this feasibility study, namely the up-stream damsite as proposed in the master plan, the mid-stream damsite, and the down-stream damsite, the latter two of which were proposed independently by ICE after the master plan was presented. The on-site geological investigations were conducted at the three locations by ICE, and JICA team analysed and examined of the results of the previous geological investigations conducted by ICE.

The geological conditions, geotechnical characteristics identified in the study of each potential damsite, plus a geoengineering evaluation for each site, are discussed in this section.

In addition, the rock classifications for the drilling core and the standard of rock classification for the adits which were adopted to geotechnically evaluate the rocks at the drill core and the exploratory adit wall of each damsite, as well as the rock mass classification, which were introduced to comprehensively evaluate rocks, are discussed in 7.1.5.

(a) Topography

Each site is topographically located in a deep V-shaped gorge, with a cliff of 100 to 150 m in height (measured from the riverbed), a portion of which hangs over the river at the left bank, exposing hard, massive conglomerate. In contrast, the pitch of the slope at the right bank (between the up-stream damsite and the lower part of the mid-stream damsite) is rather gentle, with a height of between 70 to 80 m and 100 to 120 m (measured from the riverbed). The conglomerate bedrock at the portion with the gentle slope has somewhat thick residual soil, yet is still sufficiently stable. As to the riverbed, at all three locations, the width of the river water was 10 m or less during the dry season, and exposure of the hard bedrock was outcropped at both the right and left banks. Both of the bank are extremely steep and covered with a thick tropical forest, thus its finer details are not depicted even in the new topographic maps. Therefore, at the post feasibility study stage, it is highly critical to observe the topographic condition and distribution of residual soil.

<u>Up-stream Damsite</u>

This site is located at the point where the Rio Naranjo changes its course from the south to the southwest. Topographically, the shape of the valley is extremely asymmetrical. There is a steep cliff at the left bank, rising 100 to 150 m (measured from the riverbed), a portion of which hangs over the river. In contrast, the right bank rises between 70 to 80 m and 100 to 120 m (measured from the riverbed), forming gentle slope.

<u>Mid-stream Damsite</u>

This site is located at the start point where the Rio Naranjo changes its course from the southwest to the south. The topography of the valley at this site is asymmetrical. There is a steep cliff at the left bank rising 100 to 150 m (measured from the riverbed), a portion of which hangs over the river. In contrast, the right bank rises with a relatively gentle slope. However, it is not extremely asymmetrical, comparing to that of the up-stream damsite. The overhanging portion on the left bank is most prominent in the mid-stream damsite. However, there are no visible cracks in that hang, and, therefore, it is considered sufficiently stable.

Down-stream Damsite

This site is located at the point where the Rio Naranjo completes the change of its course to the south (the river first starts changing its course toward the south at the mid-stream damsite). The both banks at this site are no longer nonsymmetrical, with a partial overhanging portion along the left bank (the distinctive characteristics at the up-stream and mid-stream damsites). Instead, the area has a virtually symmetrical V-shaped topography. There is a steep cliff at the left bank. However, no potion of it hangs over the river as it does at the up-stream and mid-stream damsite. The right bank also has a steep slope with approximately the same inclination as the left bank. No gentle slopes are observed on either side.

(b) Geology

Up-stream Damsite

At the up-stream damsite, conglomerate is outcropped at both banks. Exposure of hard, massive conglomerate is especially evident on the left bank in its overhanging cliffs. Conglomerate in this location includes extremely large rounded boulder with a diameter over 1 m. Sedimentary layers are generally not clearly observed. The main gravels of the conglomerate are of conglomerate, sandstone and shale. The fresh conglomerate is extremely firm and hard, making it virtually impossible to crush, even when hard hitting with a hammer. In contrast, the conglomerate at the foundation of the gentle slope on the right bank is rather heavily weathered, with clearly defined residual soil. On the top of the weathered conglomerate, secondary nonconcrete talus deposits are distributed. No fault with a prominent fractured zone is observed.

One drillhole (PHLL1SP, depth: 70.70 m) and one exploratory pit (Pit No. 1, depth: 9.55 m) were conducted on the right bank of the site. According to the data observing the PHLL1SP cores, weathering is progressed to approximately 20 m in depth. However, fresh rocks are observed at deeper ground levels. The results of the Lugeon tests conducted sporadically at a depth of between 12 and 70 m indicated that injected water was not able to be discharged. However, considering the drill-core observation results mentioned above, it is hard to conclude that such a portion is composed of rocks with high water permeability. Leakage of the injected water at the contact of the packer and the hole wall is considered one of the reasons for such a phenomenon, since the double packer method was employed for the Lugeon tests only after drilling was completed for the whole depth. The water table was observed at approximately 40 m in depth.

The progression of hard weathering is observable all the way down to the bottom of Pit No.1, 9.55 m in depth.

As to the riverbed, the width of the river water was 10 m or less during the dry season, according to the field survey, and exposed hard bedrock was observed on the both banks.

Considering all of the above conditions, the site is strong enough to support the foundation of a dam whose the height will be approximately 60 m with a concrete gravity type, as long as the strongly weathered portion on the surface is removed. As to the gentle slope on the right bank, which is mostly higher than the height of the dam, it is considered sufficiently stable at this site.

Mid-stream Damsite

Conglomerate is distributed on the both banks of the mid-stream site. In particular, exposure of hard, massive conglomerate is observed on the left bank in the overhanging portion. Conglomerate in this area includes extremely large rounded boulders with a diameter over 1 m. The sedimentary layers are generally not clearly observable. The main components of the conglomerate are of conglomerate, sandstone and shale. The fresh conglomerate is extremely firm and hard, making it virtually impossible to crush, even when hard blows with a hammer. In contrast, secondary nonconcrete talus deposits is distributed. No fault with a prominent isolately fractured zone is observed.

One drillhole (PHLL2SP, depth: 83.30 m) and one exploratory pit (Pit No. 2, depth: 9.50 m) were conducted on the right bank of the site. According to observation of the PHLL2SP cores, weathering is recognized to approximately 5 m in depth. However, fresh rocks are observed at deeper ground level. The results of the Lugeon tests, conducted sporadically at a depth between 10 and 80 m, indicated that injected water was not able to be discharged. However, considering the drill-core observation results mentioned above, it is hard to conclude that that portion is composed of rocks of high water permeability. Leakage of the injected water at the contact of the packer and the hole wall is considered one of the reasons for such a phenomenon, since the double packer method was employed for the Lugeon tests only after drilling was completed for the whole depth. Therefore, it is safe to consider the value 0 Lu, obtained at the 80 m in depth, as the water permeability of the area. The water table was observed at approximately 5 m in depth.

A progression of strong weathering is observed all the way down to the bottom of Pit No .2, which is 9.50 m in depth. Owing to the effects of weathering, extremely low values were obtained as the results of the plate-bearing tests conducted near the bottom of the pit.

Therefore, it is not recommended to use these figures in estimating the strength of the fresh bedrock.

As to the riverbed (although we were not able to conduct a field survey), the width of the river water is estimated at 10 m or less during the dry season, and exposure of hard bedrock on both banks is expected, based on the results of the field surveys conducted at the up and down-stream damsites, as well as the new topographic maps.

Considering all of the above conditions, the site is considered sufficiently strong to support the foundation of a dam approximately 60 m in height with a concrete gravity type, as long as the strongly weathered portion on the surface is removed.

Down-stream Damsite

As to the geology of the down-stream site, conglomerate is outropped on the both banks. The outcrops of hard, massive conglomerate are especially prominent on the both banks. Conglomerate in this area includes extremely large rounded boulders with diameters of over 1 m. Layers are generally not clearly observed. However, layers of coarse sandstone, which trends east-west and dips north, are observable in certain places. The main components of the conglomerate are of conglomerate, sandstone and shale. The fresh conglomerate is extremely firm and hard, making it virtually impossible to crush even when hard hits with a hammer.

One drillhole (PHLL3SP, depth: 83.00 m) and one exploratory adit (Adit No. 1, length: 30.15 m) were conducted on the right bank of the site. One drillhole (PHLL4SP, depth: 60.00 m [inclined]) and one exploratory adit (Adit No. 2, length: 6.0 m) were conducted on the left bank of the site. According to core examination of PHLL3SP, the bedrocks are weathered to approximately 5 m in depth. However, fresh rocks are observable at a deeper ground level. Fresh, hard rocks are also observed in Adit No. 1 at a depth of 10 m or greater.

The results of the Lugeon tests, conducted sporadically at a depth of between 10 and 70 m in PHLL3SP, indicated fairly low water permeability (1 - 9 Lu) except at one section approximately 10 m in depth. In addition, the water table was observed at approximately 10 m in depth. Considering the above, there are no problems in the area from the standpoint of geology.

As mentioned above, fresh, hard rocks are observable at approximately 10 m in depth in Adit No. 1. In addition, the results of the plate-bearing tests conducted at four locations in the adit confirmed that the area is rockmechanically strong enough for the proposed civil structures (further details are given in 7.1.6).

In seemed that weathering has slightly deeper on the left bank than on the right bank. Weathering is observable as far as approximately 16 m in depth in PHLL4SP, and all the way of Adit No. 2, which is 6.0 m in length (11.5 m minus 5.5 m for the entrance area). Since the left bank of the area is located within a small ridge not even shown on the topographic map, it is safe to say that there are no significant differences in the weathering conditions of the bedrocks between the both banks.

Considering all of the above conditions, the site is considered sufficiently strong to support the foundation of a concrete gravity type dam approximately 60 m in height, as long as the strongly weathered portion on the surface is removed.

As to the riverbed, the width of the river water was 10 m or less during the dry season, according to the field survey, and outcrops of hard bedrocks were observed on the both banks.

<u>Intake Site</u>

The power intake site in this project will eventually be located on the right bank of the Rio Naranjo, whichever of potential upstream, mid-stream or downstream sites may be chosen as the damsite.

In this section, I geotechnical comments are given on the assumption that the dam will be constructed on the downstream site.

The power intake structure is planned to be installed between EL460m and EL480m on a slope on the right bank, some 30m up from the down-stream damsite.

The landform of this intake site shows a steep slope at an angle of about 50 degrees from the level of land with relatively a few uneven portions, and currently its surface is covered with plants grown fairly thick.

Topographically this slope appears stable.

This site has not been subjected to subsurface exploration yet, but referring from the results of past surveys conducted in the neighborhood, the geology of this site is as shown below.

- It may safely be said that the bedrock of this site is composed of conglomerates as is the aforesaid damsite.

- Judging from conditions of the drillhole PHLL3SP bored at a spot on the right bank side of the downstream damsite, there is strong probability that the bedrock of this site is covered with unconsolidated surface deposits (estimated maximum thickness: 2m) and that the surface layer of the bedrock itself has turned a highly or moderately weathered rocks (estimated total depth: 8-10m).

(c) Engineering Geology

The engineering geological assessment of potential damsites in this project can be briefed as follows:

- Unweathered portion of the conglomerate constituting the bedrock of potential damsites in this project has very compact and hard lithologic characters and is massive as a whole.

- The bedrock of these potential damsites is overlain by unsolidified surface deposits (the maximum depth: approximately 3m). Of the potential three damsites, the downstream site has the least distribution of surface deposits, both horizontally and vertically.

- Including highly weathered portion (CL) and moderately weathered portion (CM) altogether, the maximum thickness of weathered conglomerate of the bedrock of these potential damsites (to the right angle direction against the slope) reaches about 15m near the upstream and mid-stream sites on the right bank, and around 10m at the downstream site on slopes on the both banks.

- Exclusively from the viewpoint of topographic and geologic conditions, the downstream site is considered the most favorable.

- At the downstream damsite, construction of an about 60m high concrete gravity dam is planned. In this case, it will be reasonable to set the excavation line as the foundation rock in CM-weathered rocks aforesaid from the viewpoints of lithological and rock mechanical properties.

- Consequently, the foundation excavation depth at the downstream damsite will become greater at the upper half on the both banks and will be 5-8m from the ground surface to the right angle direction against the slope.

- The drillhole PHLL3SP has proved that the groundwater level at the downstream site lies at a depth of about 11m from the opening. No detailed data are available concerning the groundwater level on the left bank, but judging from the topographic and geologic conditions there, it appears almost the same as on the right bank. This suggests that the groundwater level probably lies somewhere slightly below CH-weathered rocks or around the uppermost portion of unweathered rocks.

- On the other hand, the seepage of the bedrock at the downstream site is 10 Lu to 20 Lu in CM-weathered rocks, 1 Lu to 10 Lu in CH and 0 Lu to 3 Lu in non-weathered rocks according to past Lugeon tests.

- Judging exclusively from Lugeon values, the range of water barrier treatment for the dam foundation at the down-stream site should be sit near the lowest portion of CH-weathered rocks. However, based on a more conservative side estimate, it is desirably anticipated that the range should extend up to just below the groundwater level at abutments on the both banks, and at the riverbed it be at a depth of half the dam height provided if there is no fault.

- For preparing a definite design concerning the downstream site in the future, core drilling reaching up to unweathered rocks as well as measurements of the groundwater level and Lugeon tests covering rock portions should desirably be implemented on the left bank side near a spot where the dam crest will abutd. At the riverbed, core drilling to a depth of half the dam height or more and Lugeon tests covering the total length shall desirably be carried out.

The engineering geological assessment concerning the power intake site in this project can be summarized as follows:

- It seems that the planned power intake site on the right bank of the upstream side of downstream damsite has no fatal defect under topographic, geologic and engineering geologic conditions.

- For preparing a definite design of power intake structures, at least one core drilling reaching up to unweathered rocks shall desirably be implemented near the center of the intake site. At the same time, measurements of groundwater level and drillhole deformation tests in part shall desirably be carried out.

(2) Reservoir Area

As mentioned above, the down-stream site is of the most optimum for the dam of this project according to the feasibility study, the reservoir area is described assuming the damsite at the down-stream site in this Chapter.

(a) Topography

The dam of this project is designed to have a high water level of 485 m above sea level. In such a case, the backwater of the reservoir will reach approximately 1 km upstream along the Rio Naranjo. There is a confluence of the Rio Naranjo and its tributary, the Rio Naranjillo, approximately 500 m upstream of the damsite. The reservoir will form a Y-shape according to the confluence. Generally, the reservoir will stretch from the south to the north with light irregularities.

The reservoir will be widest at the confluence of the Rio Naranjo and the Rio Naranjillo, approximately 50 m in width.

The reservoir area is surrounded by 1,000 m class mountains (above sea level). Therefore, most of the reservoir's shoreline will be facing steep slopes.

The greater part of the surrounding slopes of the reservoir is undeveloped and covered with a tropical forest, except for the portion near the mountain top which is utilized for pasturage and coffee plantations.

No trace of landslides or collapse scars were observed at the reservoir area.

(2) Geology

Conglomerate observed at and around the damsite is distributed at the lower to middle portion of the reservoir area. Sandstone (consisting of coarse to fine grains) is distributed at the middle to upper portion of the reservoir area. Sandstone overlies conglomerate.

The rock formations in the reservoir area are composed of the same sedimentary rocks found on the damsite, namely sedimentary rocks from the Jurassic to the Eocene, which include ophiolite (volcanic rocks from undersea eruptions) called the "Nicoya Complex" and other oceanic sedimentary rocks. Fresh, hard bedrock is observed on the steep cliffs especially near the riverbed.

The same conglomerate forming alternated layers on the damsite is distributed in the area between the damsite and the confluence of the Rio Naranjo and the Rio Naranjillo which is about 500 m up-stream from the damsite. The size of sandstone grains gradually becomes larger from medium to coarse as they are observed in the upstream direction. The somewhat rhythmical alternation of sandstone is consisted with coarse grained sandstone including granule to middle sandstone. The thickness of each layer is generally 5 to 6 cm. The conglomerate layers at the damsite trends east-west and dips north, and the coarse grained sandstone layers in the upper reaches of the river trends northwest-southeast and dips northeast. These layers are considered to have a slight unconformity or conformity boundary, instead of a fault boundary, since rock formations grade at the same time, and no prominent faults are found in the boundary area. No prominent faults with fractured zones are observed in the reservoir area.

In addition, no karstic carbonate rocks (which largely effect on problems of reservoir water leakage conditions), and no new volcanic rocks (which are porous and nonconcrete), are observed in the reservoir area.

The reservoir area is mostly located in a steep V-shaped gorge with no loose sediments except for partial drift of the riverbed deposits.

(c) Engineering Geology

The outlines of the topography and geology of the reservoir area have been made clear from data offered by ICE and various surveys conducted by the JICA in conjunction with ICE. Engineering geological assessment of the reservoir area is summarized as follows;

- According to the examination of data collected and the results of the geological mapping up to now, the reservoir area does not have possible large-scale water leakage to another river basens. The reasons are as follows: The reservoir area is surrounded by topographically thick mountains, the underground water levels on both banks of the river near the damsite increased towards the mountains, and no carbonate rocks nor new volcanic rocks with high water permeability (which could cause major leakage from the reservoir) are observed in the area.

- No landslides nor major collapse scars are observed except for a partial and limited collapse scars in the reservoir area. In addition, no major unstable slopes are, also, observed in the area.

- Considering all of the above, it is safe to conclude that there are no unstable slopes which threaten the stability of the reservoir rims and the dam abatements.

(3) Headrace-Tunnel Route

(a) Topography

The total length of the headrace tunnel (which starts at the right bank of the Rio Naranjo, over the dividing ridge, then onto the left bank of the Rio Paquita) is approximately 5.5 km.

The headrace tunnel will be mainly routed through mountains between 600 and 700 m above sea level. At one point, in the middle of the course, it will pass underneath fairly large swamp (Quebrada Jilguero). The topography of the route has minor ups and downs, and the tunnel cover at most parts of the route will be between 100 and 200 m. Approximately 95% of the total length has more than a 100-m thick tunnel cover, except for the area near the intake and the surge tank sites. The maximum cover will be approximately 300 m near San Isidro. The tunnel cover underneath the Quebrada Jilguero will be only 20 m or so.

Surface water streams were observed during the field survey in the Quebrada Jilguero area.

Table 7-20 and 7-21 show general figures of the headrace tunnel and locations with thin cover.

(b) Geology

The geology of the route for the headrace tunnel is discussed below, according to an examination of data offered by ICE, and aerial photographs interpretation, as well as the results of the geologic survey conducted by the JICA in cooperation with ICE.

Boulder conglomerate, sandstone/shale alternating layers, conglomerate (pebble size), fine grained sandstone, and conglomerate (volcanic) are distributed in the headrace tunnel route, respectively, from the upper to the lower stream.

The rock formations at the headrace-tunnel route are consisted with the same sedimentary rocks found at the damsite, namely sedimentary rocks from the Jurassic to the Eocene, which include ophiolite (volcanic rocks from undersea eruptions) called the "Nicoya Complex" and other oceanic sedimentary rocks. Fresh, hard bedrock expose, especially near the riverbed, except for the shallow area near the surface.

Conglomerate in the uppermost reaches of the headrace tunnel route includes extremely large rounded boulder with a diameter over 1 m, which can be found in the same layer in the damsite. The layering structures are not clearly observed as a whole. However, a partial layer of coarse sandstone, which trends east-west and dips north, is observed. The main gravels of the conglomerate are of conglomerate, sandstone and shale. The fresh conglomerate is extremely firm and hard, making it virtually impossible to crush, even when hitting with a hammer.

Sandstone and shale (medium to fine, partially coarse) form rhythmic alternating layers in the lower reaches of the boulder conglomerate along headrace tunnel route. The thickness of each layer is generally 5 to 6 cm. No prominent laminations with thickness of 1 mm or less are observed.

The conglomerate distributed in the lower reaches also contains the rounded gravels. However, unlike with conglomerate at the damsite, their diameter are mainly 50 to 60 cm at largest. Coarse sandstone layers are observed in some portions.

The grain size of fine grained sandstone distributed at the lower reaches are relatively small with poor variation of grain size. Silt-grade shale layers are observed in some portions.

In addition, the volcanic conglomerate is distributed in the lower most reaches of headrace tunnel route. It is considered to belong to the same layer of the conglomerate distributed at the powerhouse site. Various kinds of lithology, including breccia, are clustered together. However, all of them are basically considered to belong to the same layer, with a slight variation in lithology.

Existence of minor folding or fault structures is expected, since the strikes and dips of the layers varies slightly according to the position along the headrace tunnel route. In general, they trend northwest-southeast, and dip northeast, showing conformity or slightly unconformity boundaries. For example, the data offered by ICE indicates that the sandstone/shale alternating layers, which disappear approximately 1,000 m east of the headrace-tunnel route, have a slightly different strikes and dips compared with the conglomerate distributed at the upper and lower reaches of tunnel. Such dispositions are conformable with the strikes and dips of the alternating layer observed on surface. Therefore, the sandstone/shale alternating layers are assumed to disappear at the depth of the tunnel route.

According to the results of the field survey at the Quebrada Jilguero, where the tunnel cover will be the thinnest (approximately 20 m), exposure of the rather fresh, hard bedrock consisting of conglomerate and sandstone, as well as the surface stream, were observed.

Hard and firm rocks are observed at the fresh part of each layer. In addition, no prominent tectonic topography or collapse of land are observed. Therefore, it is safe to conclude that there are no disadvantageous conditions for the tunnel plan for this project.

According to the geologic plan and section along headrace tunnel route offered by ICE, the existence of 2 or 3 faults can be expected. However, none of these are expected to have a large displacement or fractured zone. Therefore, it is safe to conclude that there are no critical conditions for the tunnel plan for this project.

(c) Engineering Geology

As mentioned above, lots of geo-investigation works involving topographic, geologic, geophysic and geotechnic methods have made clear the outlines of geo-conditions along the headrace tunnel route area.

From a view point of engineering geology, the said outlines are summarized as follows;

- As shown in Figs. 7-8 and 7-9, the headrace tunnel is expected to pass through the three types of conglomerates, such as, boulder-size gravels (Cgb), pebble-size gravels (Cgp) and volcanic gravels (Cgv) and fine-grained sandstone.

- Lithological characters of the unweathered portion of all of the aforesaid rocks are compact, hard and sound and are massive as a whole.

- The surface layer of the aforesaid rocks found at the ground surface is weathered in greater or lesser degree. Normally the maximum depth from the surface of rocks to unweathered rocks is 20m to 30m.

- Joint sets developed moderately in places are found in the aforesaid rocks, but the conditions of such joint sets over the entire span of the headrace tunnel route have not been made clear yet. Geological mapping realized so far has discovered no joint set whose level should require particular precaution.

- No remarkable fault has been found out crossing the headrace tunnel route.

- On the other hand, the aero photograph interpretation for this project area has picked out several lineaments, three of which are crossing this tunnel route.

- These three lineaments, taking the peripheral landform and geology into consideration, could not function as serious hindrance factor to construction of this headrace tunnel even though they may be faults. However, in any event, whether these lineaments are faults or not should be made clear by surveys to be conducted in the future.

- Of the total span of the headrace tunnel (about 5,500 m long), there are three sections which have tunnel rock cover less than 50 m, namely near the intake site (about 50 m long), near Quebrada Jilguero located nearly at halfway point of the headrace tunnel (approximately 100 m long) and near the surge tank (some 250 m long). For the second and third sections stated above, core drilling accompanied with water level measurements should desirably be executed in the future.

- At present no detailed data are available concerning groundwater in the headrace tunnel route area. However, judging from the topographic and geologic characters of mountains along this route, it is estimated that the groundwater level over the entire span lies at a depth of not greater than 20 m.

- Judging from all available data up to present, the topographic and geotechnic conditions will not strike a fatal blow against construction of this headrace tunnel. However, it is needless to say that excavation and supporting methods should be decided carefully for the aforesaid sections which have tunnel rock cover less than 50 m.

(4) Penstock Route and Powerstation Site

General geo-conditions about topography, geology and geotechnology of surgetank site, penstock route and powerstation site are described in this section.

There were two alternatives for the penstock route-powerstation site; one was called as "No.1 route" proposed in the master plan of this project, and the other "No.2 route", which was located on the up-stream side of the "No.1 route", proposed by ICE.

Prior to the feasibility study done by the JICA team and ICE, the said two routes were investigated by ICE.

The final "penstock route-powerstation site" proposed by this feasibility study is located between No.1 and No.2 routes, as shown in Fig. 7-8.

As for the field investigation works in the area including the surgetank site, penstocks routes and powerstation sites are summarized as follows:

- The area around the surge tank site was surveyed with geophysical prospecting and spot-trenching.

- The penstock routes were investigated with geophysical prospecting, spot trenching and core drilling.

- For the powerstation sites, geophysical prospecting and core drilling were carried out.

- During the feasibility study, geological mapping for the area including surgetank site, penstock routes and powerstation sites and additional core drilling with in-situ tests and logging in the holes on the lower section of No.1 penstock route (Drillhole PHLL 18 TP) and the No.1 powerstation site (Drillhole PHLL 19 CM) were, also, done by ICE, according to the JICA teams recommendation.

- As for the final powerstation site, a drillhole PHLL 13 CM done prior to the feasibility study has penetrated to the sound rocks and given useful information to assess its foundation rocks.

Geo-conditions of the final penstock route-powerstation site are, hereinafter, explained.

(a) Topography

The left bank of the Rio Paquita, where the surgetank site, penstock route and powerstation site are located, is situated at the boundary of the mountainous area and the alluvial lowlands along the Pacific Ocean, thus forming a complex topography with an irregular and rugged small ridges and shoreline.

The surge tank will eventually be placed near the end of a slightly lean ridge extending to the SW direction, no matter which penstock route may be chosen.
At the beginning, studies were made to install the penstock on a slope situated at the west side of the surge tank - a slope with relatively gentle inclination as a whole, accompanied with slight unevenness, but finally decision has been made to place it underground as Figs. 7-9 and 7-11 illustrate.

The final power station site will be situated near a location where a long and narrow formed river terrace abuts against the mountain slope on which the penstock runs, on the left bank of the Rio Paquita, upstream from the "No.1 site" proposed in the master plan. The spot behind the power station site forms a slope with relatively gentle inclination as shown in Figs. 7-10 and 7-11.

(b) Geology

The main component of the bedrock distributed in this area is conglomerate Cgv, the same volcanic conglomerate distributed in the lowermost portion of the headrace-tunnel route. It's surface portions have been rather strongly weathered (maximum depth of weathering: approximately 15 m). Fresh bedrocks are exposed at the riverbed. However, it is hardly observed on top of ridges. All most of slope surface in this area are covered with a rather thick topsoil or weathered residual soil, and outcrops of the bedrocks exposing on slope cuts along passways or local roads are generally weathered.

Surge Tank Site

The bedrock of the surge tank site is constituted of conglomerate (Cgv).

This conglomerate Cgv is characterized by its containment of gravels originated from volcanic rocks and the lithologic characters of its unweathered portion is very compact, hard and sound and is massive.

The surface of the bedrock of this site is capped by top soil and residual soils. It is difficult to determine the thickness of these overburdens and the level of weathering of the bedrock judging from the data of past surveys.

No data are available at present concerning the groundwater level of this site.

Considering that this site is located at a slightly lean ridge, the weathering degree of the bedrock of this site is estimated fairly high. An integrated analysis of topographical, geological and geophysical data and information on this spot obtained until now leads to a supposition that the depth from land surface to unweathered rocks at this point may be 30m or more.

Penstock Route

This feasibility study has concluded that it is the best policy to install the penstock underground taking the aspects of civil engineering technology and cost into account. Therefore, the geology of the underground portion where the penstock runs is explained in this section. (See Figs. 7-8 through to 7-12).

Geo-investigations so far carried out for regions where the penstock may probably run have been focused on the aforesaid routes No.1 and No.2. The outcome of geo-investigations made on route No.1 is briefed as shown in Fig. 7-11.

According to the cores of drillhole (PHLL18TP) conducted at the halfway point of the penstock route No.1 (elevation 297.2 m), strongly weathered bedrock was observed from the surface to approximately 9 m in depth, followed underneath by fresh, hard and massive conglomerate Cgv.

The following shows the geological conditions of the underground portion where the penstock runs, summarized by consulting the past geo-investigation results.

- There is high probability that the penstock will pass through Cgv of the same type as conglomerate distributed throughout the surge tank site.

- The aero photograph interpretation has picked out one lineament which is crossing this route. However, whether it is a fault or not is yet to be made clear.

- The data of the seismic prospecting conducted along the aforesaid penstock routes No.1 and No.2 indicate that seismic primary wave velocity Vp of conglomerate Cvg of the bedrock is 2.3km/sec or less from land surface to a depth of 30m to 40m. The data also show that Vp of the portion which is supposed to be fresh rocks is 3.0-4.6km/sec, and that even at fresh rock portions the low velocity zone (Vp:2.1-2.8km/sec) exists depending on locations.

- Fresh bedrock was observed directly underneath the strongly weathered conglomerate, even at the portion where weathering is very strong. The maximum distance from the surface of strongly weathered bedrocks to the fresh rocks along the penstock route is estimated to be approximately 20 m.

- The water-level measurement, standard penetration test (conducted at the strongly weathered portion), deformation test (conducted both at the strongly weathered and fresh portions), and borehole velocity log, were conducted in the drillhole PHLL18TP by ICE. The results of these tests are analyzed in 7.1.8. According to the observation results of the drillhole cores, the geologic and geotechnical conditions of the area have proven to be more or less as expected.

Power Station Site

The bedrock on the left bank of the Rio Paquita where the power station site is situated is composed of conglomerate Cgv and marlstone. From the viewpoint of geologic sequence, the former conformably overlies the latter. However, the ICE survey provides data proving that the boundary of the both is partly sheared.

As already stated, the lithological characters of the unweathered portion of conglomerate Cgv are very compact, hard, and massive, while marlstone shows properties that it is well-compacted under loaded conditions but is likely to soften easily under unloaded conditions, for example, in a form of drilled core. Marlstone outcrops at a part of the riverbed and a part of the small ravine bottom near the No.1 power station site, and its outcrops are found in softened and friable conditions.

At the final power station site, conglomerate Cgv and marlstone are distributed as bedrocks covered with top soils (thickness: 2-2.5m) and terrace deposits (thickness: 4.5-5m). The drillhole PHLL13CM bored within this site has ascertained that conglomerate Cgv is ranging from a depth of 7.05m to 44.9m and marlstone from a depth of 44.9m to the bottom of the drillhole of 50m.

Appendix "Geologic Log of Drillhole PHLL13CM" names rocks from a depth of 7.05m to 25.5m as "Breccia", but they should be construed as being included in conglomerate Cgv in terms of geologic sequence.

(c) Engineering Geology

Engineering geological assessments concerning the surge tank site, penstock route and power station site are briefed as follows:

Surge Tank Site

- It seems that judging from the outcome of past geo-investigations, the surge tank site proposed in this feasibility study will not pose any particular problems. However, because the slopes of the southeast and south sides of this lean ridge where the surge tank will be located are rather steeper than that of the southwest side, attention should be paid to these topographical conditions upon making a definite study of the surge tank in the future.

- At the surge tank site, one core drilling reaching up to the bottom of the structures should be penetrated to grasp the real geologic and geotectonic conditions for their detailed design works.

Penstock Route

- Conclusion has been reached as a result of this feasibility study that installing the underground penstock is the most favorable viewing the aspects of civil engineering technology and cost.

- No subsurface exploration has been carried out yet along the penstock routes shown in Figs. 7-8 and 7-10. However, the data obtained in geo-investigations already carried out at its periphery make engineering geological assessments of the said route possible to some extent.

- As illustrated in Fig. 7-12, the penstock tunnel is supposed to pass through conglomerate Cgr throughout its total span. On the other hand, generally, the lithological characters of the bedrock (Cgr) in areas where the penstock route runs have possibly been deteriorated in greater or lesser degree from its surface to a depth of 30m-40m.

- In consequence, very careful attention is necessary for the definite of the tunnel excavation and supporting in the sections where penstock tunnel rock cover is 40m or less. On the other hand, it appears that the geotechnical conditions in sections having rock cover of 40m or more will not pose any problem for tunneling works, excepting for a limited part whose lithological characters have been deteriorated.

- For a definite study of penstock, subsurface explorations consisting of at least two core drillings with measurements of groundwater level at the same time and drillhole deformation tests as occasion demands are required to sections with relatively thin tunnel cover out of the total penstock tunnel span.

Power Station Site

- Past geo-investigations have made clear that at the power station site, about 7m thick overburden (top soils and terrace deposits) overlies conglomerate Cgv of the bedrock and that marlstone in its turn underlies this conglomerate Cgv.

- The foundation of power station should be the conglomerate underlying the aforesaid overburden which shall be dug and removed. As far as checking the drilled cores, this conglomerate is vested with satisfactory aptitude as the foundation rocks of a power station.

- Marlstone lying beneath conglomerate at this spot will not appear directly in the power station foundation.

As mentioned previously, this marlstone is very dense in a completely loaded condition, but exhibits the property of readily softening in a condition of load removed.

In the design of the powerhouse foundation is to be to the depth at which conglomerate is distributed, and it is not necessary to consider this marlstone in particular. Even in the event of a change made to a plan for excavation to the depth of marlstone distribution at a stage after detailed design, it is thought there will not be any problem in particular as foundation rock for the powerhouse. However, in executing work, measures should be provided to avoid repetitions of wetting and drying such as to pneumatically apply mortar immediately after excavation. The same may be said for a case of excavation done so that this marlstone appears at the surface of a slope.

- The boundary between conglomerate and marlstone has outcrops in part which present indications of sheared zones, but as previously mentioned, the relationship between the two is thought to be that of conformity or very slight unconformity. Also, a large-scale sheared zone is not found at the boundary between the two at Drillhole PHLL13CM at the powerhouse site. Consequently, there is no necessity to consider a sheared zone in particular at this boundary which is a place deeper than the depth of excavation given in the Final Report.

7.1.5 Rock Classification

A rock classification and evaluation system designed to examine the bedrock of the drill core and adit was adopted in the feasibility study for this project in order to enable an objective evaluation of the geotechnical characteristics of the bedrock at the dam foundation.

With regard to the drill core, the rock core was classified on a scale of 1 to 5 under 3 categories--weathering (W), hardness (H), and intervals of cracks (C)--which were recorded in the geologic log of drillhole shown in Table 7-16. Table 7-17 shows the evaluation of the core by grouping the results of the classification. Figures 7-3 through 7-5 show geological cross sections which were obtained based on the results of the evaluation of the core.

With regard to the adit, the bedrock was classified on a scale of 1 to 5 under 3 categories--weathering, hardness and intervals of cracks--according to the "Rock Classification for Adits" (see Table 7-18). Figures 7-3 through 7-5 show the results of the classification based on the grouping according to the "Rock Mass Classification."

The relationship between the above-mentioned "Rock Classification for Adits" and "Rock Mass Classification" is shown in Table 7-19.

7.1.6 In-situ Rock Foundation Tests

(1) Introduction

The plate-jack tests were conducted at four locations each in exploratory Adit No. 1 and exploratory Pit No. 2, both located on the right bank of the damsite.

In addition, the borehole deformation test were conducted at five locations in the drillhole PHLL2SP, six locations in the drillhole PHLL3SP, seven locations in the drillhole PHLL4SP, eight locations in the drillhole PHLL18TP and at six locations in the drillhole PHLL19CM, located at the penstock route and powerhouse site. The results of the tests are discussed below.

(2) Plate-Jack Tests

The plate-jack tests were conducted at four locations each in exploratory Adit No. 1 and exploratory Pit No. 2, both located at the right bank of the damsite of the project.

Table 7-8 shows the results of the tests.

(3) Borehole Deformation Tests

In addition, borehole deformation tests were conducted at five locations in the drillhole PHLL2SP, six locations in the drillhole PHLL3SP, seven locations in the drillhole PHLL4SP, eight locations in the drillhole PHLL18TP and at six locations in the drillhole

PHLL19CM, located at the penstock route and powerhouse site. The results of the tests are discussed below.

Table 7-22 and 7-23 shows the results of the tests.

(4) Results and Evaluation of the Tests

The results of plate bearing test and borehole deformation test indicating the deformation characteristics of the rock masses at the damsite are shown in Table 7-8 and 7-22. The results of borehole deformation test indicating the deformation characters of the rock masses at the penstock route and powerhouse site are shown in Table 7-23.

As analysis of plate bearing test, moduli of deformation (D) of the rock masses were determined in the incremental loads 10, 20, 30, 40, 50 (and 60 at some point) kgf/cm^2, and secant moduli of elasticity (Es) and tangential moduli of elasticity (Et) were obtained in the straight-line section of 20 to 50 or 60 kgf/cm^2 in the final loop of the maximum load level (50 or 60 kgf/cm^2) by ICE.

As analysis of borehole deformation test, moduli of deformation (D) of the rock masses were determined in the incremental loads 15, 30, 45 and 60 kgf/cm^2, and moduli of elasticity (Eb) were obtained in the straight-line section of 20 to 60 kgf/cm^2 in the final loop of the maximum load level (65 kgf/cm^2) by ICE for drillhole at damsite and penstock route, and by JICA study team for drillhole at powerhouse site based on the data supplied by ICE.

A value of 0.2 was used for the Poisson's Ratio of the rock mass in calculating the various coefficients at the damsite, since the testing sites were of hard rock (as a measure, unconfined compressive strength of rock more than 600 kgf/cm^2). Generally in case of hard rock, Poisson's Ratio of 0.2 is frequently hypothesized. Assuming a case of increasing the value of Poisson's Ratio by 0.05, the variations in the various coefficients will be reduction of roughly 1,000 kgf/cm^2. On the other hand a value of 0.25 was used for the Poisson's Ratio of the rock mass in calculating the various coefficients at the penstock route and powerhouse site. Rock mass condition at the test sites are not so hard compared with those of damsite (as a measure, unconfined compressive strength of rock is 100 to 300 kgf/cm^2).

On looking at the rock mass characteristics in the plate bearing test results, the proportion of plastic deformation in total deformation is approximately 77% for CH class and 55%

for B class. Value of CH class is 1.4 times greater compared with those of B class. This indicates that CH class is more subject to the effects of fissures or joints, and the plastic deformation is great.

Moduli of deformation (D) are from 60,500 to 75,100 kgf/cm^2 with CH class and 12,800 to 16,560 kgf/cm^2 with B class. Values of CH class are representative values according to overall judgement, on the other hand those of B class are lower compared with those of CH class. It is suggested that the values of B class were possibly influenced of small and local cracks because of blasting.

Tangential moduli of elasticity (Et) are 86,200 to 256,000 kgf/cm^2 with CH class and 19,050 to 25,150 kgf/cm^2 with B class. The value 256,000 kgf/cm^2 with CH is unusually large value, it is suggested that this is caused something unfavorable occurred through test or hardness during analysis (for example hardness for determine straight line). On the other hand values of B class are lower compared with those of CH class. It is suggested that the values of B class were influenced of small and local cracks because of blasting as same as moduli of deformation. To summarize the above results, only 86,200 kgf/cm^2 for CH class value is the only reasonable value as tangential moduli of elasticity.

Secant moduli of elasticity (Es) are 97,000 to 144,700 kgf/cm^2 with CH class and 18,100 to 23,240 kgf/cm^2 with B class. It is too hard to determine reasonable value and interpret those values because of too much variations of value. Only one thing to be able to suggest is that the value 1,441,700 kgf/cm^2 with CH is unusually large value and caused something unfavorable occurred through test or hardness during analysis (for example hardness for determine straight line).

All of test points in exploratory Pit No. 2 were located in strongly weathered bedrock. So these values are representative of hard weathered bedrock not fresh nor sound rock. As mentioned in 7.1.4, it is suggested to remove the strongly weathered rocks for the foundation of a dam.

The following may be said when judged only from the results of plate bearing test.

Regarding classification of the rock masses, the characteristics are equal the standard values for CH class which are general rock mass classifications and so the classifications are appropriate. On the other hand, test points in B class were influenced of small and local cracks because of blasting, so those values are not representative of B class rock mass.

Strength and deformation characteristics of rock masses showed slight effects of fissures or joints, but comprehensively these are hard rock masses. Therefore it is judged that rock mass characteristics are adequate for the dam foundation.

Table 7-22 shows the results of deformation tests in drillholes PHLL2SP, PHLL3SP and PHLL4SP at the damsite.

Moduli of deformation (Db) are 123,000 to 51,200 kgf/cm^2 with B class and 51,000 to 26,600 kgf/cm^2 with C class. These values are representative values as result of borehole deformation tests according to overall judgement.

Moduli of elasticity (Eb) are 139,000 to 55,500 kgf/cm^2 with B class and 72,000 to 27,400 kgf/cm^2 with C class. These values are representative values as result of borehole deformation tests according to overall judgement.

Regarding classification of the rock masses, the values of each class are representative values and so the classifications are appropriate. Therefore it is judged that rock mass characteristics are adequate for the dam foundation. This result is concordant with results of plate bearing tests at the damsite.

Table 7-23 shows the results of deformation test in the drillhole PHLL18TP at penstock route.

Moduli of deformation (Db) were 26,000 to 126,000 kgf/cm^2 at the drillhole PHLL18TP located on the penstock route. The values are 101,000 to 126,000 for Cg-a class, and 54,000 to 108,000 for Cg-b class.

The values are 55,000 to 105,000 kgf/cm^2 for Cg-b class and 36,000 to 43,000 kgf/cm^2 for Cg-c class. These values are representative values as result of borehole deformation tests according to overall judgement. Therefore it is judged that the bedrock mass characteristics are adequate for the penstock tunnel except topsoil or residual soil.

Table 7-23 also shows the results of deformation tests in the drillhole PHLL19CM located powerhouse site. Moduli of deformation (Db) were 4,000 to 9,000 kgf/cm^2 for Ma-b class. Value at a point in terrace deposits was 3,000 kgf/cm^2. These values are flash bulletin values calculated by the JICA study team based on investigation data of ICE. These agree approximately with values calculated by ICE and do not affect the conclusion of this report. As a result the bedrock conditions of the area were considered to be sufficiently stable for the foundation of the powerhouse.

7.1.7 Geophysical Prospecting

(1) Introduction

The seismic prospecting were examined as a part of the geophysical prospecting at the damsite, headrace-tunnel and penstock routes, and powerhouse site. The Vp/Vs measurement was conducted in the exploratory Adit No. 1 located on the right bank of the damsite. In addition, resistivity survey was also conducted along the penstock route and at the powerhouse site. All these investigations were conducted by ICE prior to the feasibility study.

In addition, the JICA study team re-examined and re-evaluated some of the prospecting data offered by ICE about seismic prospecting. The results are discussed below.

(2) Damsite

The seismic prospecting at eight measuring traverses at the damsite was conducted by ICE. Table 7-10 summarizes the prospectings, and Figure 7-2 shows the locations of the measuring traverses.

All measuring traverses for the seismic prospecting were located in the gentle slope on the right bank of the up-stream damsite.

The results of the prospecting are consistent with the results of the drill-core observations, indicating the strongly weathered layer near the surface as the low-speed layer.

(3) Headrace-Tunnel Route

The seismic prospecting and resistivity survey along the headrace-tunnel route for this project were conducted by ICE. Table 7-11 summarized the results.

According to those investigations, the elastic wave velocities and specific resistances in vicinities of the headrace tunnel excavation level are grasped as follows:

	Vp	ρ
Conglomerate (pebble) C_{gp}	3.0 - 4.0 km/sec	30-120 ohm-m
Fine-grained sandstone f_{Ss}	2.2 - 2.5 km/sec	30-50 ohm-m

Compared with elastic wave velocities, the values of specific resistances measured are small, and it is possible that this is the effect of weathered zones near the ground surface. In the area of distribution of f_{Ss}, the overburden of the tunnel is planned to be more than 200m, and it may be that the depth of exploration possible for seismic prospecting is exceeded.

(4) **Penstock Route**

The seismic prospecting and partial resistivity survey at two measuring traverses along the penstock route were conducted by ICE. The routes are located at the No. 1 and No. 2 routes in the powerhouse site selected by ICE.

The results of the prospecting, especially at the No. 1 route, are consistent with the results of the drill-core observations, indicating the strongly weathered layer near the surface as the low-speed layer. Table 7-12 summarized the results.

At the surge tank site, the elastic wave velocities and specific resistances of the conglomerate parts distributed there have been measured as follows:

	Vp	ρ
Strongly weathered part	0.6 - 0.8 km/sec	
Slightly weathered part	2.5 - 3.0 km/sec	40-150 ohm-m
Fresh part	> 5.0 km/sec	

(5) **Powerstation Site**

The seismic prospecting at six measuring traverses and resistivity survey at one traverse at the powerstation site were conducted by ICE.

The results of the prospecting are consistent with the results of the drill-core observations and geological investigation of the surface, indicating the terrace deposits and the strongly weathered layer near the surface as the low-speed layer.

The elastic wave velocities of the conglomerate portions distributed at this site are indicated as 2.3 to 2.5 km/sec, while at fresh parts, velocities of 2.8 to 3.5 km/sec are indicated. The specific resistances are from 15 to 40 ohm-m, and compared with values of elastic wave velocity, the values are low, and there is a possibility that they have been affected by terrace deposits and strongly weathered layers in the vicinity of the ground surface.

(6) **Re-Analysis of the Seismic Prospecting**

The JICA study team re-examined and re-evaluated the data offered by ICE at several measuring traverses in seismic prospecting. The following are the measuring traverses for which the analysis was conducted:

Damsite: PS-3
Headrace-Tunnel Route Measuring Traverse
Penstock Route: PP-1 and PP-2
Powerhouse Site: PC-1 and PC-2
Vp/Vs Measurement in Exploratory Adit: Adit No. 1 at the damsite

In Table 7-10, results of both of ICE's analysis ("analysis A") and JICA study team's re-analysis (only for PS-3) ("analysis B") are shown as the distribution of seismic velocity layers at the damsite.

According to "analysis A", the geological structure of these prospecting traverses are divided into four (4) layers based on the velocity values. On the other hand according to analysis B the geological structure of these prospecting traverses are divided into three (3) layers based on the velocity values. According to the table there is a difference about the number of velocity layer. But the forth layer of "analysis A" was limited only in part. For example, in the geologic section of up-stream damsite (Figure 7-3), there are four (4) traverses PS-4, PS-5, PS-8 and PS-3 are crossed with the section. The geological structure of traverses PS-4, PS-5, PS-8 based by "analysis A" are divided into three (3) layers based on velocity values, and same as PS-3 based by "analysis B". Seismic velocity or depth to the layer's lower bottom are similar with "analysis A" and "B". So it is possible to say that both of velocity layers of two analysis are concordant according to overall judgement.

7.1.8 Drilling Core Tests

Drilling core tests were carried out on the conglomerate at the damsite, penstock route and powerhouse site.

The test results are as given in Table 7-24 and 7-25 and the evaluation are as follows:

Conglomerate at the damsite (PHLL1SP, PHLL2SP, PHLL3SP and PHLL4SP)

- Specific gravities are 2.74 ± 0.05(*), and those of mid-stream damsite are slightly lower compared with those of up-stream and down-stream damsites.
  There are no serious variations between those of up-stream and down-stream damsites.
  They are heavy in weight as a conglomerate and satisfactory in quality.

- Absorptions are 1.26 ± 0.55%, and those of up-stream damsite are slightly lower compared with those of mid-stream and down-stream damsites. There no serious variations between those of mid-stream and down-stream damsites.
  They are comparatively low in absorption, but indicating good quality.

- Unconfined compressive strength are 675 ± 293 kg/cm^2, and there are no serious variation between those of up-stream, mid-stream and down-stream damsites.
  The strength of rock as the dam foundation is adequate.

Conglomerate at the penstock route and powerhouse site (Drillholes PHLL10TO, PHLL11TP, PHLL12CM and PHLL13CM and PHLL14CM)

1) Specific gravities are 2.51 ± 0.09, and there are no serious variations between those of penstock route and powerhouse site. They are slightly low compared with those of the damsite, but heavy as a conglomerate and satisfactory for the foundations of the penstock and powerhouse.

2) Absorptions are 5.67 ± 2.79%, and there are no serious variations between those of penstock route and powerhouse sites. They are slightly high compared with those of the damsite, but indicating good quality.

3) Unconfined compressive strength are 218 ± 113 kg/cm^2, and there are no serious variation between those of up-stream, mid-stream and down-stream damsites.
   The strength of rock as a penstock or powerhouse foundations is adequate.

To summarize the above results, the conglomerate at the damsite, penstock route and powerhouse site possess satisfying rock strength for a dam foundation, penstock and powerhouse.

*) Values after ± are SD (Standard deviation).

7.2 Construction Materials

7.2.1 Introduction

In this project, a gravity concrete dam with about 60m high class is planned to be constructed. In this section, results of investigations and tests for the concrete aggregates conducted so far in the project area are explained and given some comments for the future potential sites of concrete aggregate required for construction of the concrete dam.

7.2.2 Investigation Works

Spots where ICE has already conducted some survey or test in an attempt to select potential sites of concrete aggregate include three riverbed deposits sites and three rock quarry sites.

Of the said sites, the only site where field investigation has been conducted for the selection of potential quarry of aggregate is Quebrada Azul Site. Particulars of the survey are shown in Tables 7-26 and 7-27, of which outline is as shown below.

Core Drill Hole: 9 holes total 85.2m

Seismic prospecting: Six (6) lines total 794m (data of two (2) lines, total 341m in length were only available to the JICA team)

Laboratory tests were executed at all six points, with results as shown in Table 7-26. Measurements of soundness and abrasion were carried out as laboratory tests. These tests were conducted conforming to ASTM C-131, C-33 and C-88. The number of samples of which soundness and abrasion were measured at each point are as follows: