The Tablachaca Dam slide N° 5 problem

Le problème du glissement N° 5 du Barrage Tablachaca

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SYNOPSIS

Slide N° 5 is an ancient slide of about 13 million m3 located in Perú upstream of the Tablachaca dam, the main structure of the Mantaro Hydroelectric Plant; whose installed capacity comprises 50% of the country's total capacity. In February 1982 the slide movements accelerated alarmingly, prompting ELECTROPERU to decide to undertake emergency stabilization works simultaneously with the complementary studies and investigations. The emergency works consist of: a fill buttress at the slide toe; treatment of reservoir sediments that form the buttress foundation; prestressed anchors in a stretch adjacent to the dam where buttress size had to be reduced; underground and surface drainage; and excavation of a rock protrusion in the river curve upstream of the slide. At present slide movements have decreased significantly, which indicates the effectiveness of the stabilization works; consequently this can be considered the largest slide in the world whose fall has been avoided by human action.

INTRODUCTION

The objective of this lecture is to describe the emergency works intended to avoid the fall of Slide N° 5 as well as the difficult process of decision making related to their design and construction. These works had to be designed and their construction started with preliminary information only regarding the site geology, the subsoil characteristics and the nature of slide movements. Due to space limitations, details about site investigations and instrumental measurements carried out, site geology and methods of analysis used are not included in this document.

Slide N° 5 is an earth and rock slide of about 13 million m3, located in the Peruvian Andes eastern range, 300 km from Lima. It is located on the right abutment and immediately upstream of the Tablachaca dam. This dam was constructed between 1968 and 1972 on the Mantaro river, in an area where a deep canyon has been eroded. The dam crest is at an elevation of 2697 m above sea level.

The Tablachaca dam is a 72 m high concrete arch-gravity dam. Its objective is to divert water from the Mantaro river through a 20 km long pressure tunnel to the fall in the Santiago Antunez de Mayolo hydroelectric plant, which has an installed capacity of 798 MW. From there turbinated water is conducted through another tunnel to a second fall where the Restitución 216 MW capacity hydroelectric plant is located. These two power plants are owned by the government company ELECTROPERU and represent, at present, 50% of the total hydroelectric power installed in Perú and approximately 55% of that of the interconnected Central-Northern region of the country where Lima is located.

The Tablachaca reservoir provides a usable storage volume for weekly regulation of 2 to 3 million m3; in order to maintain this usable volume and since the Mantaro river carries a very high volume of solids (more than 4 million m3 per year), the reservoir was provided with a system to eliminate the solids deposited. Sediment elimination is done by lowering the reservoir level when the flow is appropiate. This operation is carried out by means of four sluice gates located in the body of the dam. One of these gates (A-1) significantly influenced the design of the stabilization works of Slide N° 5, because it is located on the right side of the dam, very close to the affected slope, and it is very important for the elimination of reservoir sediments. For this reason, it was necessary to take into account for the design of the stabilization works that the operation of this gate should not be affected.

The maximum and minimum reservoir operation levels are elevations 2695 and 2676 m respectively. Since the river bottom is at elevation 2630 m approximately, a considerable volume of sediments which reached more than 40 m of depth in front of Slide N° 5 was deposited in the reservoir during the first years of operation. Subsequently, the sediments level remained constant because of the permanent operation of the sluice gates. In the vicinity of the dam, the sediments level descends towards the sluice gates, forming the "approaching cones". The existence of these sediments also required special considerations in the design of the stabilization works.

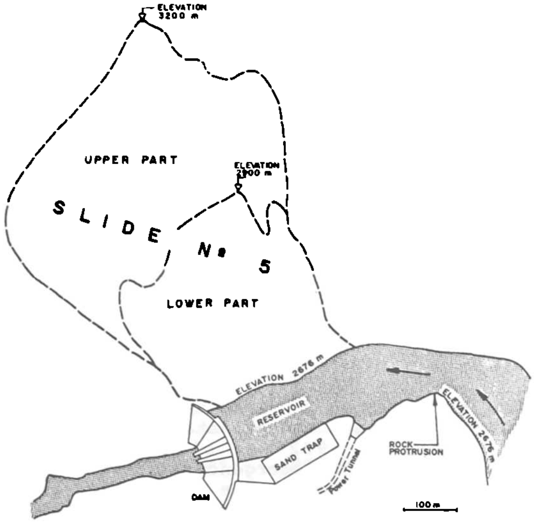

Fig. 1 Tablachaca Facilities and Slide N° 5

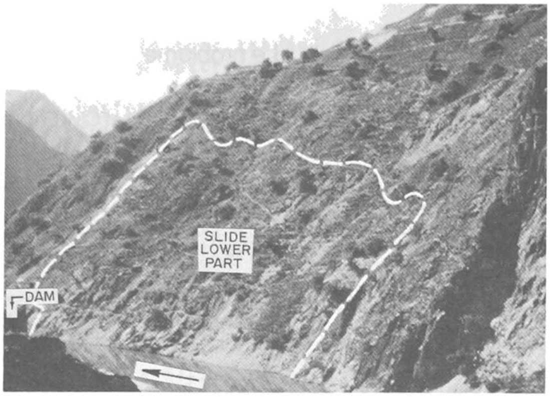

Fig. 2 Slide N° 5

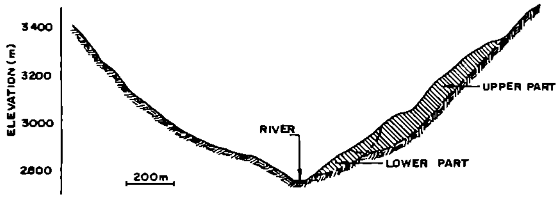

Fig. 3 Mantaro Valley and Slide N° 5

Slide N° 5 was detected for the first time during the geological studies made before the location of the dam was decided; in those studies eight ancient slides that would be affected by the reservoir were identified (Electroconsult, 1962, 1964 and 1971).

Slide N° 5 has a length of about 300 m along the reservoir. It begins immediately upstream of the dam and it extends from elevation 2660 m to 3200 m, with an average inclination of 37° to 40°. Figure 1 shows the slide shape and dimensions in plant, together with the intake structures of the hydroelectric plant, and Figure 2 shows a panoramic view.

Slide N° 5 consists of two parts, called lower and upper part. The lower part is the most active, with a volume of about 3 million m3. It extends from elevation 2660 to 2850 m approximately. The upper part has a volume of about 10 million m3, and reaches up to elevation 3200 m. It initially was inferred to be inactive. The slide was possibly originated by seismic action, when in its erosion stage the valley was at about elevation 2800 m. At a later stage, while the erosion process that deepened the valley to its present level continued, the material of the upper part of the slide displaced downwards, covering the rock slope of the lower part of the valley and the alluvial terraces of the river bed; the present lower part was formed in this manner. Figure 3 shows a cross section of the valley, in which the upper and lower part of the slide are indicated.

After the dam was constructed and shortly before reservoir filling started, cracks indicative of activation of the slide were observed. Later, when the reservoir filling started in September 1972, the slide underwent displacements of several meters and about 65,000 m3 of material fell on the reservoir. During this period the movements reached velocities of up to 50 cm/day and continued during several months (Electroconsult, 1972 and 1973).

PREVIOUS INVESTIGATIONS

Between 1973 and 1979 no systematic measurements of the slide movements were made; however, since observations indicated that movements continued, in 1978 ELECTROPERU contracted the Polytechna firm from Czechoslovakia to carry out a geophysical prospecting of the lower part of the slide. These measurements made it possible to estimate that the slide reached a depth of about 50 meters (Polytechna, 1979). It was recommended that additional investigations be conducted and instrumentation installed, in order to confirm the slide depth and establish its other characteristics and degree of danger.

In the period between January 1980 and April 1982, investigations were made and instrumentation installed in the lower part of the slide, with participation of the Czechoslovakian advisory group. Nineteen exploratory drillholes were made, which were also used as open pipe piezometers. In 5 drillholes, slotted inclinometric casings were installed; in 11 others, smooth casings were installed in which measurements were made with a "poor boy inclinometer" (this technique consists of introducing steel rods of different lengths hanging from a wire line, in order to detect the depth at which movements have deformed the casing, blocking the passing of the rods). A system of 34 monuments for topographic control of surface displacements was also installed and, in the perimeter of the lower part of the slide, points for tape extensometer measurements were installed. In addition, the excavation of a tunnel was started through bedrock behind the slide, which confirmed its deep nature and the presence of ground water. The construction of drainage ditches and the sealing of cracks on the slope surface were also started.

During the rainy seasons (January to March) of 1980, 1981 and 1982, movements increased progressively, confirming that the degree of activity of the slide was directly related to the regional precipitation and to the variations in reservoir level. As a result, since 1981 reservoir operation was restricted and maintained at minimum level. Based on the different velocities of surface movements, a preliminary zonification of the slide area was established, and the existence of a zone where movements were much greater than the rest was defined. This zone comprises the downstream third of the slide (adjacent to the dam) and was labeled the "critical zone" because of the high risk that it represented (ELECTROPERU, 1983).

In February 1982 movements accelerated, reaching surface velocities of up to 5 cm/day, perfectly correlated with deep movements recorded at depths of up to 50 m; this indicated that the slide was deep and that it represented a serious danger for the hydroelectric plant facilities. Under these circumstances, in April 1982 ELECTROPERU decided to carry out emergency stabilization works and simultaneously to carry on the studies and investigations required to determine the slide geometry and characteristics; this would permit making adjustments that could be necessary in the design of the emergency works and subsequently to design the definitive stabilization works.

CONCEPTION OF EMERGENCY WORKS

Basic Considerations

Due to the urgency of the situation, ELECTROPERU, with the collaboration of advisors and specialists, proceeded immediately to contract by tender specialized firms to design the stabilization works. A program that would permit the immediate start of the works in the shortest possible time was required; its first priority was to immediately improve the slide stability. In preparing that plan, it was considered necessary that the emergency works meet the following basic requirements: (1) that they could deform together with the slide without being damaged, since the slide would continue moving during their construction; (2) that they could be started immediately; (3) that they form part of a total program of stabilization works, so that temporary works that had to be replaced later on would not be constructed; (4) that their dimensions would permit modifications during construction, since their design would be based on preliminary information; and (5) that they would not impair reservoir operation.

For the selection of the stabilization works program, several alternatives and combinations of stabilizing elements were considered, including: (a) fill buttress at the slide toe; (b) rigid retaining structures at the slide toe, consisting of piles, walls or sheet piling; (c) prestressed anchors fixed to bedrock behind the slide mass; (d) underground and surface drainage; (e) stabilization by means of chemical or cement grouting; (f) unloading excavation of the upper end of the lower part of the slide, to reduce driving forces; and (g) removal of the slide mass.

Use of rigid retaining elements at the slide toe was discarded, because as a consequence of the slide displacements they could be damaged as they were being constructed due to their stiffness. Any type of unloading excavation was also discarded, since the available information about the slide depth and characteristics was insufficient to assure that the upper part would not be destabilized (subsequent studies have shown that because of the considerable slide depth, it would have been economically impossible to replace the retaining effect of the lower part with respect to the upper part of the slide, which would have been destabilized). The use of chemical or cement grouting was also discarded, due to the great slide size and the little information available about its constitution.

Finally, after evaluating the effectiveness of the different types of works from the geotechnical point of view and considering their constructability, cost and schedule, preparation of the emergency works program was finished in June 1982 (WCC/MR, 1982a), including, as the basic stabilization elements, a non-rigid passive force at the slide toe and drainage.

Works Program

As stabilizing passive forces, a comparison was made of prestressed anchors fixed to bedrock and a fill buttress along the slide toe founded on the reservoir sediments previously compacted. The fact that both the anchors and the buttress are sufficiently flexible structures and would not be damaged by slide movements was taken into account. A parametric study was made to compare the cost-effectiveness ratio of both types of stabilizing elements, and led to the conclusion that in order to obtain the same increment in factor of safety, the cost required using a fill buttress was approximately one-sixth of that required using anchors. For this reason, after evaluation of the hydraulic implications of

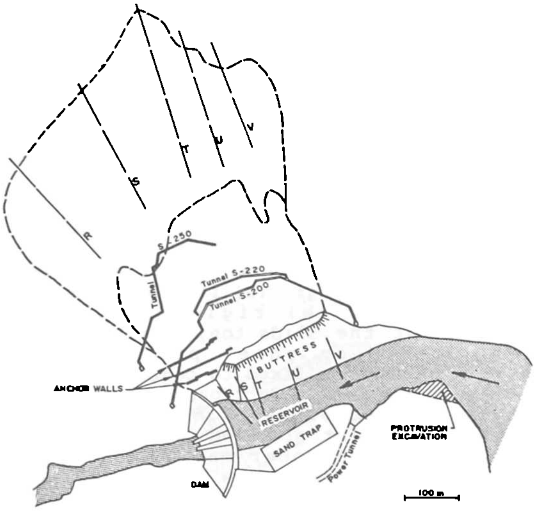

Fig. 4 Emergency Stabilization Works - Plant

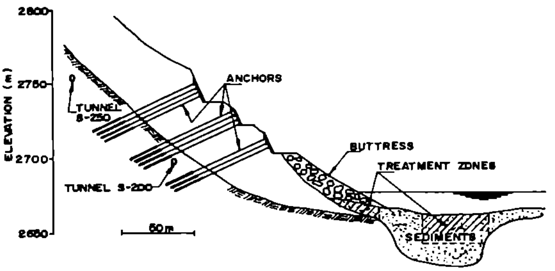

Fig.5 Emergency Stabilization Works-Section R

its construction in the reservoir, a buttress was selected as the main passive stabilization element. It provides a counterweight against rotational slides and a passive resistance against translational slides.

Since the buttress would be founded on reservoir sediments - believed to be sandy and loose, and hence susceptible to liquefaction during earthquakes - it was necessary to include their densification in the works program. In accordance with the estimated sediment characteristics, vibroflotation from floating barges was considered the most adequate system. In addition, since the part of the buttress comprised between its base (elevation 2672 m) and maximum reservoir level (elevation 2695 m) would not be compacted and would be subjected to immersion, it was to be constructed with screened gravel larger than 1 1/2 inch, so that it would drain freely and thus avoid susceptibility to liquefaction. Moreover, in order to protect the buttress from the erosion that could be caused by the river flow and variations in reservoir level,

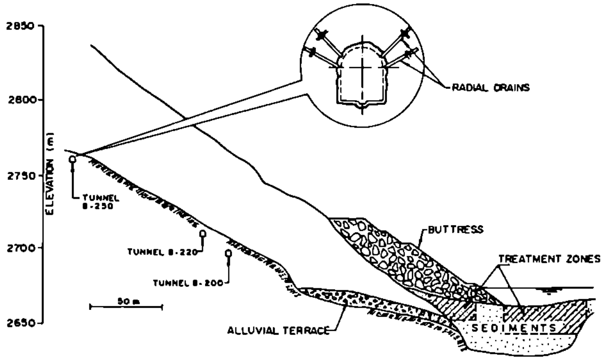

Fig. 6 Emergency Stabilization Works-Section T

a layer of rock rip-rap was to be placed on its outer face.

In order to avoid the buttress material obstructing sluice gate A-1, which would have seriously affected hydraulic operation of the reservoir, it was decided not to extend the buttress with constant cross section to the dam, but rather to progressively reduce the cross section near it. To complement the lower stabilizing effect of the buttress, in that stretch coinciding with the slide's "critical zone", the works program included prestressed anchors which would provide an additional passive force in the lower part of the slide.

The drainage works consisted of tunnels excavated in bedrock behind the slide and drainage galleries transversal to them, radial drains drilled from inside the tunnels and drainage galleries, and horizontal drains drilled from the slide surface; in addition, it was decided to increase the number of drainage ditches to evacuate surface run-off water and to carry on sealing of cracks to reduce the infiltration of rain within the slide mass.

The described program was complemented with excavation of the rock protrusion located in the left bank of the river, in the curve located immediately upstream of the slide. The objective of this excavation is to achieve a good alignment of the river flow as it approaches the dam. In addition, the construction of a hydraulic model was programmed, in order to verify the effect of protrusion excavation and to analyze the adequate reservoir operation with the stabilization works, establishing norms for the future operation of the sluice gates.

Figure 4 shows a plan of the emergency works program and Figures 5 and 6 show two cross sections; section R near the dam (in the stretch with anchors and reduced buttress section) and section T farther upstream (in the stretch with entire buttress section). These figures show the buttress, the anchor walls, the drainage tunnels, the zones of sediment treatment and the protrusion excavation.

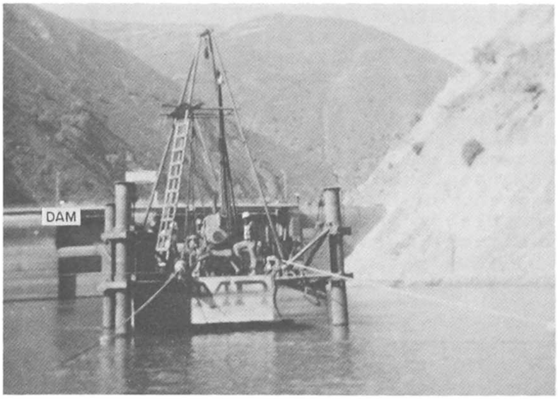

Fig. 7 Sediments Investigation

The emergency works program also included site investigations and instrumental measurements to improve knowledge of the slide characteristics, record its movements and the variations of the different external agents that affect it (ground water, rain, reservoir level), and to evaluate the efficacy of the stabilization works.

DESIGN AND CONSTRUCTION OF EMERGENCY WORKS

Immediately after the emergency works program was approved and during contracting, construction of access and studies and investigations were started.

Investigation of Reservoir Sediments

An investigation of reservoir sediments was carried out between June and August 1982, to determine physical and mechanical characteristics (MR, 1982). That investigation was conducted from a platform with jack up legs (See Figure 7), and included boreholes with standard penetration tests and soundings with dynamic cone. The results indicated that the sediments were predominantly sandy and loose, as had been estimated, with N values in the upper 15 m generally between 2 and 15; however, zones with sands having a high fines content were also found, as well as intercalated layers of soft clay; the static undrained shear strength of the clays was considered insufficient to provide an appropiate factor of safety during rapid buttress construction.

Due to the presence of sands with high fines content and clay layers in the sediments, the vibroflotation method initially considered for sediment treatment was discarded, and instead, compacted gravel columns intruded by the Franki technique (compaction gravel Franki piles) was selected. The gravel columns were to be installed from a fill platform constructed up to 2 m above minimum reservoir level which would constitute the lower part of buttress.

Buttress Materials

Simultaneously geological investigations were carried out to locate the sources of materials required for buttress construction (WCC/MR, 1982b). To obtain the gravel that forms most of the buttress, a terrace of the Mantaro river located immediately downstream of the dam was chosen and a screen was installed there. As rock quarry, a sandstone outcrop located 16 km from the slide was chosen.

Stability Analysis

Between June and November 1982 static stability analyses were made for dimensioning the emergency works (WCC/MR, 1983e). Those analyses were made by backanalysis, using two cross sections of the slide: one proposed by Hutchinson (1982) that includes the entire buttress section and the other downstream, in the stretch with anchors and reduced buttress section, that coincides with the critical zone. As initial condition for the backanalysis (factor of safety 1.0), the reservoir near minimum level and ground water conditions corresponding to dry season were considered. In these analyses the variation of the slide factor of safety was evaluated for several volumes of stabilization works (buttress, anchors and unloading), improvement in sediments shear strength by the installation of gravel columns, several hypothesis about ground water level drawdown that could be achieved with the drainage, several levels and rapid drawdown of reservoir, and partial excavation of reservoir sediments.

Static stability analysis of the buttress foundation were also made (WCC/MR, 1983e), which permitted preparation of different alternatives for its staged construction. This construction method would be used in case the instrumentation indicated that excessive deformations or excess pore pressures were ocurring in buttress foundation.

Parallely to static stability analysis, seismic analyses were made; in order to obtain the basic information for these analyses, a seismicity study was made (WCC/MR, 1983a), and the characteristics of ground vibrations were determined (WCC/MR, 1983d). Based on these studies, a peak ground acceleration of 0.15g was adopted for design. The liquefaction potencial of reservoir sediments was evaluated (WCC/MR, 1982c), verifying that they were susceptible to liquefaction under the design acceleration. Likewise, the seismic stability of the slide and buttress were evaluated (WCC/MR, 1983f) by means of the estimation of seismic induced displacements. It was verified that the slide displacements would be within reasonable limits for the works planned; whereas in order to maintain the buttress displacements within acceptable limits, it was confirmed that it would be necessary to densify reservoir sediments under buttress as well as in front of it (See Figures 5 and 6).

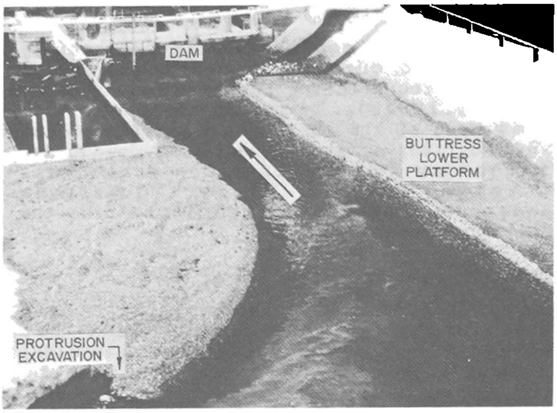

Fig. 8 Hydraulic Model

In addition, it was established that the design earthquake would produce liquefaction of reservoir sediments in their natural state and, consequently, their flow towards the approaching cones of the sluice gates. This would cause the accumulation of sediments against the dam and the obstruction of the sluice gates. This foreseen situation, of sediment pressure acting against the upstream face of the dam, was used to calculate solicitations that would occur in this estructure (WCC/MR, 1983g); these analyses indicated that those solicitations would exceed the allowable design values, and hence sediment treatment is necessary independently of buttress stability requirements.

Hydraulic Model

Between July 1982 and February 1983 studies were made in the reservoir hydraulic model; the model was constructed at scale 1:60 with a movable bottom. The model represented, in addition to the involved structures, 2.5 km of the river bed. The objectives of the tests were to study the adequate reservoir operation with the buttress, the optimum alignment of protrusion excavation, expected sediment scour at the buttress toe, size of the protection rock rip-rap, and buttress behavior in case an extraordinary discharge occured during its initial construction stage; based on these studies norms for the operation of the sluice gates were prepared (WCC/MR, 1983j). Figure 8 shows the hydraulic model during one of the tests made to establish the alignment of protusion excavation.

Slide Investigations

The complementary investigations of the slide were started in October 1982, making it possible to gain knowledge progressively about the slide characteristics and to make adjustments in the works design as necessary. These investigations included geological surveys, exploratory drillholes, in-situ and laboratory tests, instrumentation installation, geophysical prospecting (Arce, 1983), and discharge and pressure measurements of drains (WCC, 1983).

During the course of the investigations it was determined that bedrock consists of metamorphic rocks, predominantly black graphitic phyllite, which exists in the upstream three fourths of the slide; this rock presents a very thin foliation with a dip between 45° and subvertical. In the downstream fourth section, bedrock consists of quartzitic phyllite. The slide mass is very heterogeneous and is formed by rock fragments of the parent rock embedded within a matrix of detritus originated from the same rock, formed by a mix of gravel, sand and silt.

The slide mass is crossed by multiple sliding surfaces, associated with clayey shear zones, some of which are located at the slide base while others are shallow or of medium depth. Some of these surfaces start at the upper end of the upper part of the slide and others at intermediate heights, forming cracks or scarps. Sliding surfaces outcrop at their lower end at intermediate heights or at the slide toe. One of the main objectives of the slide study is to determine the geometry and activity of the different sliding surfaces.

Since site investigations could not be finished at the expected date and due to the need of verifying the design of the stabilization works, a geological model based on the information available up to June 1983 was prepared. That model was represented by five cross sections of the slide (WCC/MR, 1983h), called R, S, T, U and V (See Figure 4), on which bedrock, ground water level and the infered sliding surfaces were plotted.

Drilling of the drillholes and instrumentation installation were completed in September 1984. Thirty nine drillholes were carried out in the slide area with a total length of 3,230 m and depths up to 145.5 m; slotted inclinometric casings were installed in 28 of them and, in the other 11, 24 electric piezometers were installed. Likewise, in the floor of a drainage tunnel 3 open pipe piezometers were installed. In addition the system of topographic monuments and points for extensometric measurements was enlarged. Extensometric measurement points were also installed within the tunnels in places were they cross the basal sliding surface.

As a result of the investigations, it was determined that the slide is deeper than initially estimated; it reaches depths up to 80 m in the lower part and up to 120 m in the upper part.

Design and Construction

The buttress design was completed in August 1982; its dimensions are: 48 m high, 300 m long, 60 m wide at its base and 22 m wide at crest (467,000 m3); these dimensions would permit obtaining, for maximum reservoir level, a factor of safety of 1.12 to 1.20, depending on the ground water level drawdown that would be reached with the drainage and that could not be estimated with certainty.

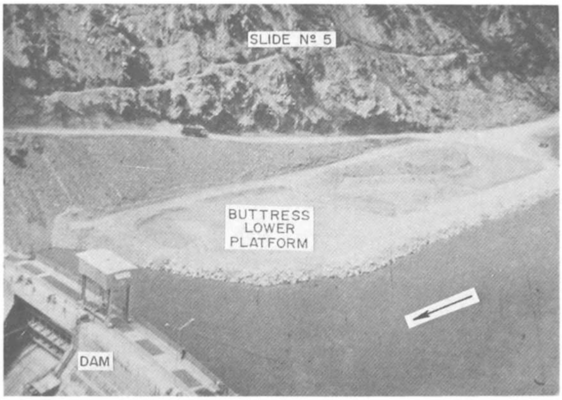

Fig. 9 Buttress Lower Platform and Access Road

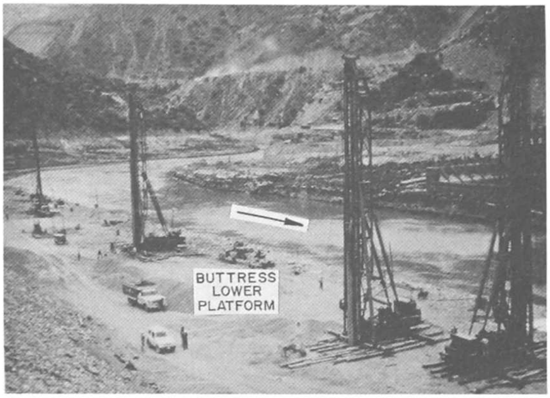

Fig.10 Sediment Treatment - Franki and Vibroreplacement Rigs

Buttress construction was started immediately, with a road leading to the reservoir constructed by dumping fill. First, the buttress lower platform was constructed; this platform reached 2 m above minimum reservoir level and from it sediment treatment was made; Figure 9 shows the platform and the road leading to the reservoir.

In order to determine the sediments characteristics with greater precision and to control their behavior both during their treatment as well as during the subsequent buttress construction, investigation drillholes were carried out in which standard penetration tests, static cone tests and Menard pressuremeter tests were conducted; undisturbed samples were also obtained in which the sediments microstratigraphy was studied. These investigations confirmed the sediments' characteristics described above (WCC/MR, 1983i). Inclinometers and piezometers were then installed in those drillholes.

Sediment treatment had to be extended during the 1983 rainy season, in spite of lack of certainty that the reservoir level could be maintained at the minimum during that season. If high discharges had occured that year, exceeding the capacity of the sluice gates, the reservoir level would have raised several meters above the platform. In order to prevent that situation an evacuation plan involving personnel and equipment was prepared and the effects that the flood might produce were studied in the hydraulic model. Fortunately 1983 was a dry year and it was possible to maintain the minimum reservoir level.

In addition, it should be mentioned that upstream of Tablachaca there are cities that dump garbage on the Mantaro river banks, and the river washes away many plastic bags and other materials at the beginning of each rainy season. In order to avoid obstruction of the tunnel entrance by rubbish, each year the sluice gates are closed and the reservoir level raised, so that all floating solids pass above the dam spillways. In 1983 it was not possible to raise the reservoir level while the works were under construction, so that year more than 100 Tons of garbage were eliminated along several hundred kilometers upstream of Tablachaca before rains started.

In order to speed up sediment treatment, the Franki technique was complemented with vibroreplacement in the areas where sediments are mainly sandy. Figure 10 shows three Franki rigs and one vibroreplacement rig operating on the buttress lower platform. The Franki technique used consisted of driving a 52 cm diameter and 22 m long steel casing through the fill platform and sediments, with a 4 Ton and 6 m drop hammer pounding, inside the casing, a gravel plug formed at the casing bottom. Once the casing was driven to rejection or to its full length, the plug was pounded out and casing lifting started, adding and compacting gravel so that a column of this material with a volume twice that of the casing was formed. The spacing between gravel columns was 2.4 m, center to center. The vibroreplacement technique used a 175 H.P. vibroflot with 2 Ton self weight and 35 cm outside diameter, which penetrated through the fill and sediments by effect of vibration and water jets. Then while the vibroflot was lifted, gravel was introduced and compacted continuously; the spacing between gravel columns was 2.3 m, center to center.

At the beginning of the sediments treatment, tests were conducted to verify the efficacy of the methods used (WCC/MR, 1983b and 1983c). Likewise, treatment results were continuously verified by means of drillholes, confirming that the Franki and vibroreplacement techniques provided satisfactory results in densifying sediments consisting of clean sands or sands with few fines. It was also determined that the Franki technique additionally provided adequate densification of the sandy sediments with high fines content and increased in short term the shear strength

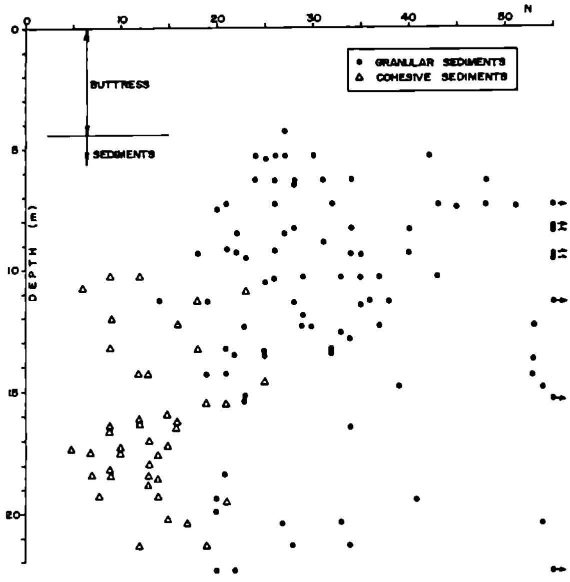

Fig.11 Results of Sediment Treatment by the Franki Technique

of clay layers. Figure 11 shows the results of standard penetration tests made in the verification drillholes carried out in the area treated by the Franki technique.

Sediment treatment in the buttress area was completed in March 1983; a total of 1,583 22 m deep compacted gravel columns were installed, 882 by the Franki technique and 701 by vibroreplacement, in a total area of 7,611 m2; twenty five drillholes were made to verify the results. Sediment treatment in the reservoir area in front of the buttress was postponed due to financial and schedule reasons; it is required for seismic considerations.

The installed instrumentation permitted measuring a rapid dissipation of excess pore pressure and deformations of buttress foundation that did not represent any danger to the structure, both during sediment treatment and during the subsequent construction of the rest of the buttress. As a result it was possible to construct the entire buttress without any restriction in the velocity of material placement; its construction was completed in September 1983. Figure 12 shows the finished buttress; in the lower part at the height of reservoir fluctuation, the rock rip-rap protection is seen; several access roads to the instrumentation points are also observed.

In the stretch comprising the reduced buttress section, the decision was made to install 405 anchors with 120 Ton working load each (549 Ton/m). With this number of anchors a lesser factor of safety than in the rest of the slide would be reached in that stretch; however, due to schedule reasons it was not

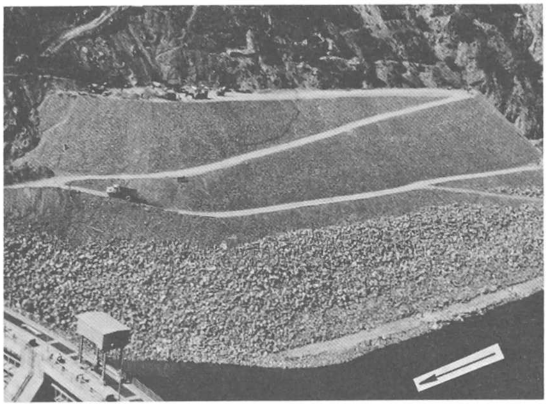

Fig. 12 Buttress

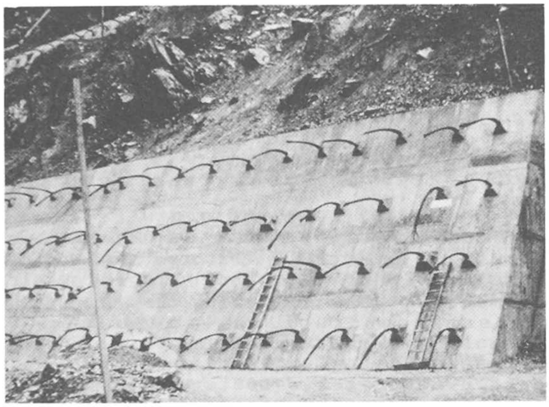

Fig. 13 Anchor Wall

considered possible to install at that stage a greater number of anchors.

Each anchor is formed by 12 high-strength steel cables, 1/2 inch in diameter, placed around a 1 1/4 inch grouting plastic pipe. The anchors were drilled through the slide mass and fixed to rock behind it by a fixed length grouted at high pressure; the fixed length varies from 20 m in the graphitic phyllite zone to 15 m in the quartzitic phyllite zone. Anchor reaction is transferred to the slope surface by three continuous reinforced concrete walls, 60 cm thick; anchors are located in these walls forming 3 to 5 rows spaced vertically 2.5 m apart. Figure 13 shows one of these walls, and Figure 14 shows the set of three walls located in the stretch with reduced buttress section.

Anchor tensioning was completed in April 1984. As part of the tensioning procedure, 10% of the anchors were tested to 180 Ton and all of the rest to 150 Ton; additionally, hydraulic load cells were installed in 20

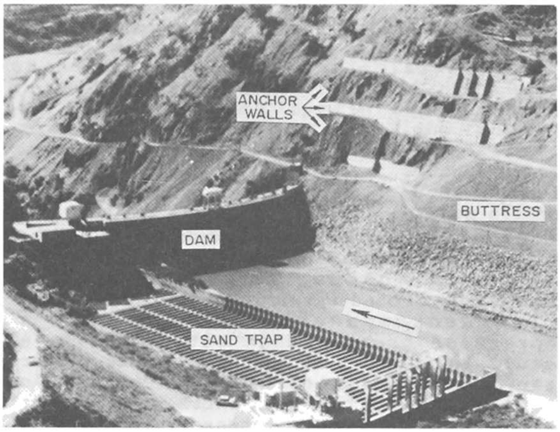

Fig. 14 Buttress and Anchor Walls

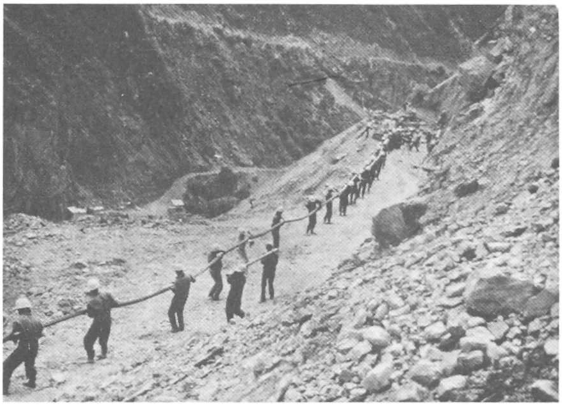

Fig. 15 Anchors Cables Transportation

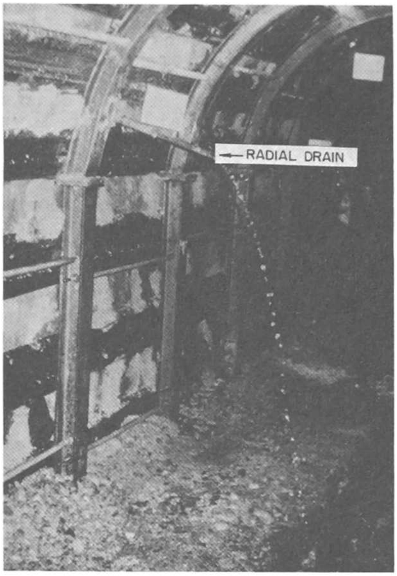

Fig. 16 Tunnel and Radial Drain

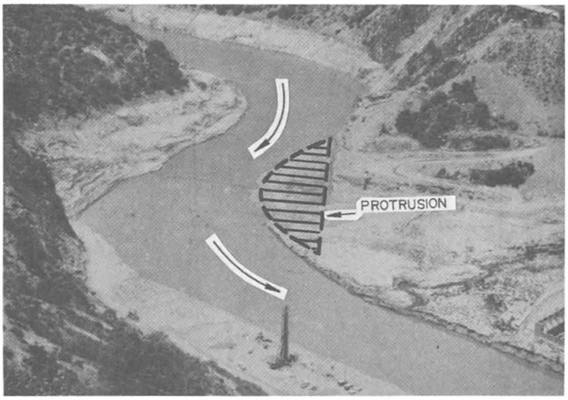

Fig. 17 Protrusion Excavation

anchors to control their subsequent behavior. It should be mentioned that the length of the anchors ended up substantially greater than expected, since the slide is deeper than initially estimated. Anchor lengths reached an average of 80 m and a maximum of more than 100 m, which is exceptional worldwide and constituted a serious construction challenge in the steep Tablachaca slope. Figure 15 shows transportation of one of the anchors cables on the slope.

Drainage works consist of two main tunnels (S-200 and S-250) located at about elevations 2700 and 2750 m; the tunnels were excavated from portals located in rock outcrops outside the slide area. Since this is a permanent drainage system, it was required that the tunnels be located in firm bedrock behind the slide. However, bedrock location was not known yet when construction was started, so it was necessary to try the alignment, changing direction each time the slide base was intersected. In order to increase drainage, transversal drainage galleries were excavated at different locations along the tunnels, both within bedrock and into the slide mass. A total of 1,527 m of tunnels and transversal drainage galleries were excavated, in which 190 radial drains were drilled with a total length of 3,290 m. Figure 16 shows a tunnel and a radial drain.

Simultaneously the other works included in the program were executed, consisting of 21 horizontal drains drilled from the slope surface, with a total length of 1,282 m; ditches for surface drainage, with a total length of 3,924 m; and excavation of 68,487 m3 of the rock protrusion located in the river curve upstream of the slide (See Figure 17).

Results Obtained

Instrumental measurement records indicate that the deep slide movements have decreased considerably in most cases, and in a few others they remain more or less constant; in no case has acceleration of movements occured. Similarly, surface movements also show considerable reductions; Figures 18 and 19 show the change in the movements for some deep sliding surfaces and for some surface monuments. However, during the last three

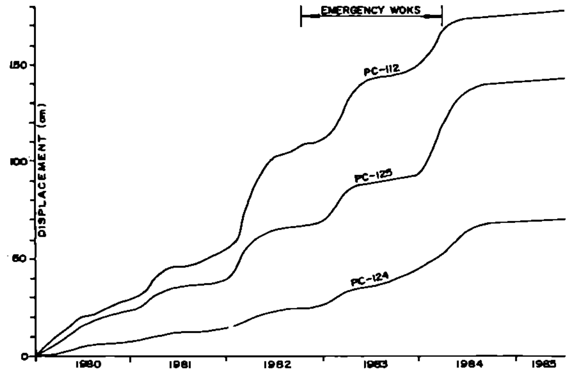

Fig. 18 Surface Displacements

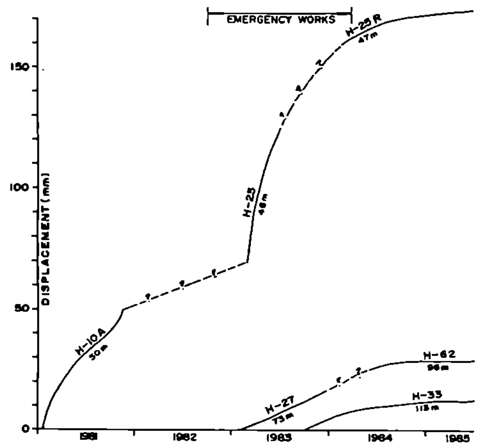

Fig. 19 Displacements in Deep Sliding Surfaces

rainy seasons (1983, 1984 and 1985) precipitations were less than in previous years, and measurements covering a longer period and including more intense rains are considered necessary, in order to confirm this tendency.

The most recent stability analyses were carried out with the geological model prepared with information available through June 1983. They indicate the following factors of safety for the emergency works constructed (for maximum reservoir level):

Section	FS
R	1.12
S	1.18
T	1.21
U	1.26
V	1.11

It should be mentioned that these analyses were made before the emergency works were completed, hence the position of the depressed ground water level was estimated. Subsequent measurements indicate a depression greater than estimated, as a result the factors of safety would be higher than those indicated above. However, it is also necessary to wait until measurements obtained during more intense rainy periods are available, in order to confirm the ground water level depres-ion reached.

These initial results are indicative of the efficacy of the emergency works constructed and have permitted reducing the restrictions regarding reservoir operation.

DESIGN OF DEFINITIVE WORKS

The static and seismic stability analyses required for design of the definitive works were carried out using the geological model prepared with information available through June 1983. Although it was clear that such a design could require some modifications when the investigations were completed, this procedure was adopted so that ELECTROPERU could be informed of the cost of those works. Between July and November 1983 detailed static and seismic stability analyses were made, and in November 1983 the definitive stabilization project was completed (WCC/MR, 1983k).

Static stability analyses were made by backanalysis, considering as the initial condition the minimum reservoir level and ground water level corresponding to dry season. Analyses were made using the Lowe and Karafiath method (1960) and considered several conditions for the ground water and reservoir levels. The effects of rapid reservoir drawdown, saturation of slide mass, the emergency works under construction and several additional works alternatives were analyzed.

Seismic stability analyses were made by the Newmark method (1965), using yield accelerations determined by Makdisi and Seed (1977), and also similar curves derived specifically for a Peruvian earthquake representative of motions in rigid materials similar to those of the slide. On this basis, the possible seismic induced displacements of the slide and buttress that could occur for several conditions of reservoir and ground water level, saturation of the slide mass and several volumes of stabilization works were obtained.

The definitive stabilization project was prepared considering as a goal increasing the factor of safety to 1.2 in all sections of the slide, as well as limiting seismic induced displacements to 0.4 m in the zones with anchors and 1 m in those without anchors. The design includes the following main works in addition to those already constructed: (1) additional anchors in the zone adjacent to the dam; (2) anchors in other zones of the slide to stabilize sliding surfaces outcropping above the buttress; and (3) treatment of reservoir sediments in front of buttress.

Subsequent to the stability analyses and design of the definitive works mentioned above, in September 1984 site investigations and instrumentation installation were completed. These have provided some additional information, on the basis of which an up-dated geological model was prepared (ELECTROPERU-WCC/MR, 1984). This model is also represented by five cross sections of the slide. Figure 20 shows section R, representative of the "critical zone", in which the anchors and the reduced buttress section are seen. Figure 21 shows section T, representative of the central part of the slide, in which the entire buttress section is seen.

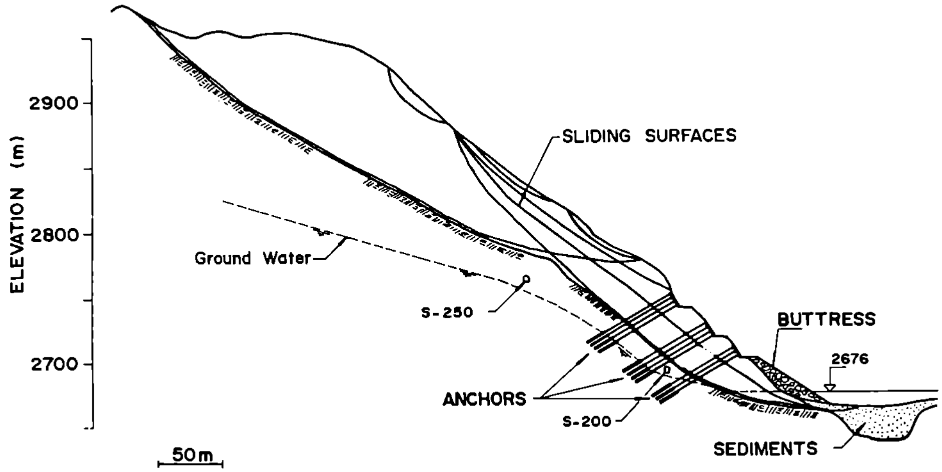

Fig. 20 Section R

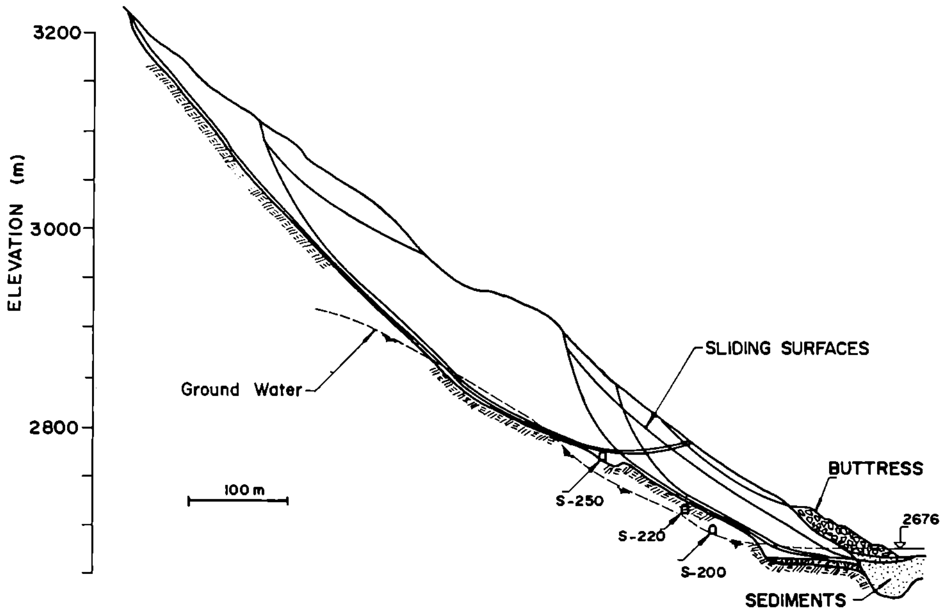

Fig. 21 Section T

At present new stability analyses are being carried out, using the up-dated geological model. The results of those analyses will reveal with greater certainty the degree of safety achieved with the emergency works and to review the design of the definitive stabilization works.

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REFERENCES

Arce, J. (1983). "Seismic refraction profiles - Slide N° 5 - Upper part"

Electroconsult (1962). "Appendixes to preliminary report - Volume 1: Geology"

Electroconsult (1964). "Geology - General report"

Electroconsult (1971). "Geology - Final report"

Electroconsult (1972) (Vinci, A. and F. Ramos). "Tablachaca reservoir slopes situation to October 17, 1972"

Electroconsult (1972) (Vinci, A. and A.B. Cleaves). "Tablachaca reservoir. Slope stability. Consulting geologists mission"

ELECTROPERU (1983) (Bustamante, J., S. Novosad and J. Kopic). "Slide N° 5 stabilization study - Stability conditions"

ELECTROPERU - WCC/MR (1984). "Geological interpretation of Slide N° 5 (up to September 30, 1984)"

Hutchinson, J.N. (1982). "Tablachaca reservoir - Report on Slide N° 5".

Lowe, J. III and L. Karafiath (1960). "Stability of earth dams upon drawdown", Proceedings of the First Panamerican Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 2.

Makdisi, F.I. and H.B. Seed (1977). "A simplified procedure for estimating earthquake-induced deformations in dams and embankments", Report N° UCB/EER-77/19, Earthquake Engineering Research Center, University of California, Berkeley

Michelena, Repetto y Asociados (MR) (1982). "Sub-aquatic geotechnical investigations of Tablachaca reservoir sediments"

Newmark, N.M. (1965). "Effects of earthquakes on dams and embankments", Geotechnique 15, N° 2

Polytechna, Stavební Geologie, Geotest (1979) (Novosad, S., R. Votoček, J. Kopic and J. Knejzlik). "Geophysical, geoacustic and geotechnical investigacions"

Woodward-Clyde Consultants (WCC) (1983). "Results of field hydrogeological measurements"

WCC/MR (1982a). "Program of static stabilization works"

WCC/MR (1982b). "Materials search for buttress construction"

WCC/MR (1982c). "Evaluation of liquefaction potencial of sandy reservoir sediments and of seismic induced deformations of fill buttress"

WCC/MR (1983a). "Seismicity study"

WCC/MR (1983b). "Gravel columns installation test by the Franki compaction piles method"

WCC/MR (1983c). "Gravel columns installation test by the vibroreplacement method"

WCC/MR (1983d). "Selection of ground seismic vibrations characteristics"

WCC/MR (1983e). "Static stability analysis of the active slide and buttress"

WCC/MR (1983f). "Seismic stability evaluation of the slide and buttress"

WCC/MR (1983g). "Estimation of the effect on the dam of a soil mass"

WCC/MR (1983h). "Geological interpretation of Slide N° 5"

WCC/MR (1983i). "Results of site investigations"

WCC/MR (1983j). "Hydraulic model of Tablachaca reservoir"

WCC/MR (1983k). (Reviewed March 1984). "Stability analysis and conception of definitive stabilization works"