The Agua Prieta Powerhouse Slope Instability

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Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 2.23

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SYNOPSIS

The slope instability that endangered seriously the powerhouse of the Agua Prieta Hydroelectric Project and involved about one million cubic meters of soils and breccias is described. The evolution of the geotechnical instrumentation measurements done from October 1987 to date, the geomechanic analysis performed and the stabilization works are discussed.

INTRODUCTION

The Agua Prieta Hydroelectric Project is located 4 km North from Guadalajara City in Mexico. This project uses residual water from the city sewerage. The water is collected at two points in the Northeastern part of the city and carried to a regulation reservoir through gravity tunnels, with a total length of 13.6 km (Fig 1). From the reservoir about 26 m^3/sec of water will flow to each unit in the power plant through a penstock 3.8 m in diameter and 1.6 km long. The generation units are two Pelton turbines of 120 MW each to produce peak load electricity. The installed capacity will be doubled in the future.

The powerhouse is located at elevation 953.8 on the left bank of the Santiago River, at the bottom of a 520 m deep canyon carved in volcanic rocks, to take advantage of the elevation difference from the upper plateau where the regulation reservoir has been built (Fig 2).

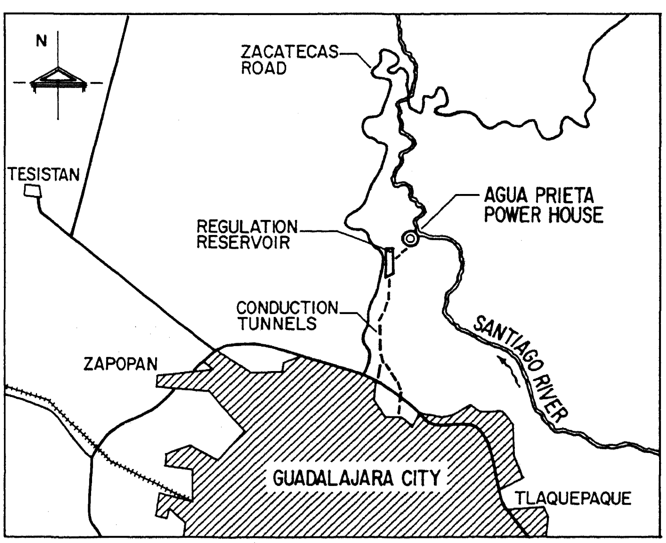

Fig. 1 Project location

The project construction began in 1987 and some excavations were made to trim and increase the stability of the natural slope behind the powerhouse by removing the loose materials forming six berms. At that time, seven inclinometers were installed to monitor the potential local movements between the upper four berms. The frequent surveying and inclinometric measurements showed that during the rainy season of each year (June-September) the slope moved horizontally outwards at a rate of about 2 mm per month and then almost stoped. In September 1990 the rate of displacements increased to 9.8 mm per month and the slide did not stops after the end of the rains. Moreover, in July 1991 the velocity of the displacement increased dramatically to 118 mm per month on the average, putting in serious jeopardy the powerhouse at the toe of the slope.

TOPOGRAPHIC AND GEOLOGICAL FACTORS

Differential erosion by the Santiago River on the diverse geological units described below produced the topographic characteristics of the site shown in Fig 3, including steep slopes and scarps. The sliding mass is located between elevations 1055 and 1150 m and is laterally limited by two diverging crevices which are 150 m appart at the top and 250 m at the base. At elevation 1055, the approximately 35° slope turns down to an almost vertical scarp 100 m deep.

Regional geology

The site is located within the physiographic province of the East-West volcanic alignment across Mexico called "Eje Neovolcánico" between parallels 19° and 20° North. Tertiary and Quaternary volcanoes are typical and their lava flows of basalt, andesite and rhyolite interbeded with pyroclastics cover the region.

This province relates directly to the subducting zone of the Cocos Plate along the Pacific Ocean coast, and thus is seismically active according to both tectonic features and historic records (Ref 1).

Site geology

The lithologic column at the site is a Tertiary sequence of volcanic flows and pyroclastics. Interbeded horizontal pseudostrata of andesite, basalt, tuff and ignimbrite underlie lacustrine tuff and rhyolite. Over this sequence there are pyroclastics ejected by "El Tempisque" volcano and either lahar flows or breccias from the same collapsed volcano (Ref 2).

The geology of the slope was studied in more detail from 1988, after the slope instability was detected. Until the first months of 1991 the limits of the unstable mass were really known after more drillings were made and is described as follows.

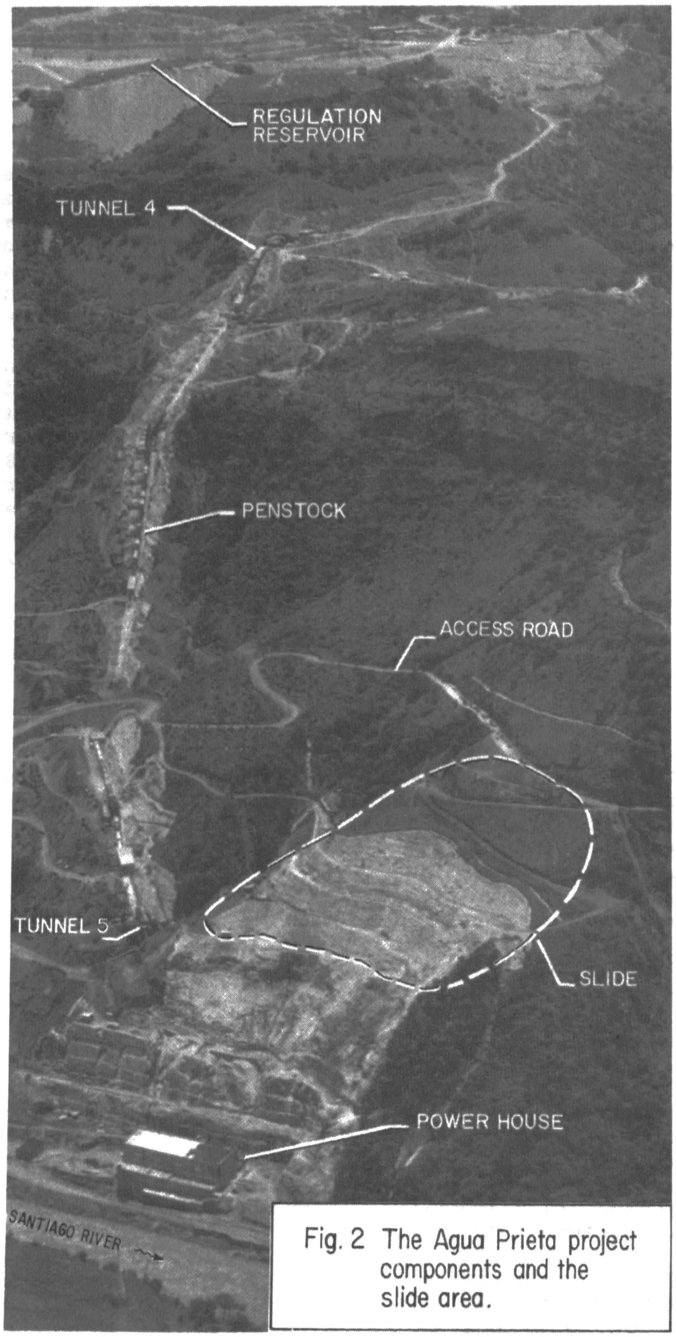

Fig. 2 The Agua Prieta project components and the slide area.

In the site there are basically three lithologic units shown in Fig 4:

Tom-ata Unit. Is made up by vesicular andesite and basalt flows with two interbedded layers, one of ignimbrite 25 m thick and another of tuff with thickness varying from 10 to 20 m. There are also some layers of petrified clay between lava flows present in this unit.

Bv Unit. Is a volcanic breccia characterized by fragments of basalt between 40 and 100 cm in size, in a clayey sand matrix. Lenticular layers of pumitic sand and an horizon of tuff are also present. The unit weight varies from 2.3 to 2.5 ton/m^3 and the internal friction angle was estimated between 40^o and 45^o. The sliding mass is constituted by this material which exhibits the properties of a well compacted rockfill.

Ar Unit. Located on the base of the Bv Unit, this material is a highly plastic clay derived from weathering of the lacustrine tuff. The x-ray difraction test reveals that the predominant constituent mineral is montmorillonite. This layer shows striations and slickensides.

The geotechnical properties of this materials are presented in more detail by Herrera and Reséndiz in Ref 3.

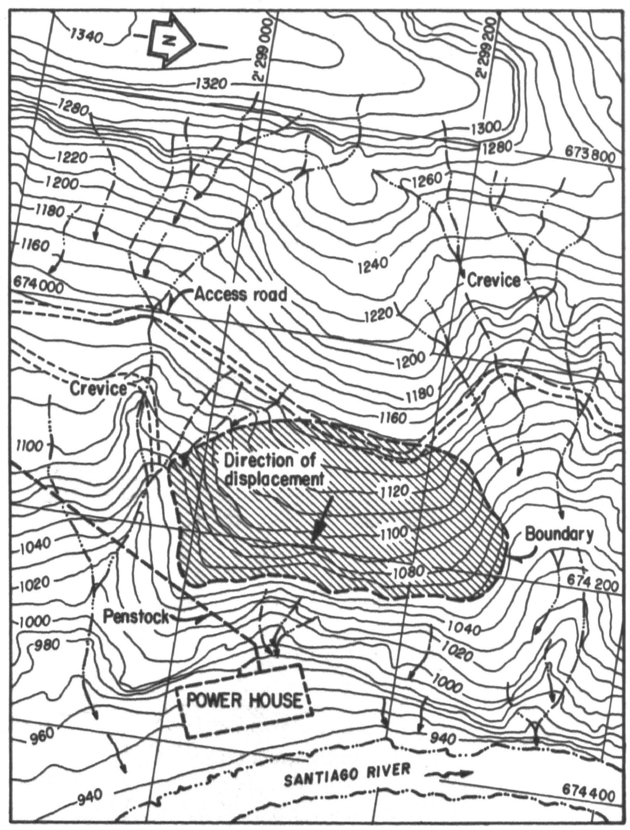

Fig. 3 Site topography and boundary of the sliding mass

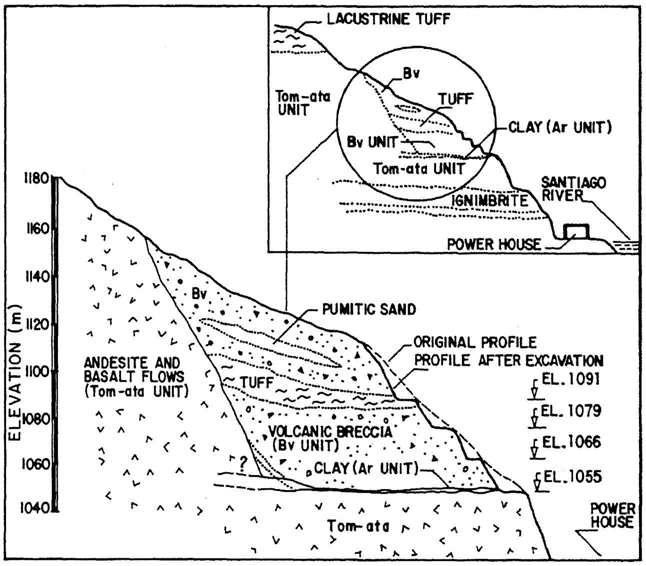

Fig. 4 Geological cross section of the uphill slope

INSTRUMENTATION

During the construction of the berms at the powerhouse slope in 1987, seven short inclinometers (I-3 to I-10) were installed in a telescopic array. Only four of them intersected the clay layer Ar. Also several surveying references were installed, both climbing the natural slope and forming a network outside the slope as a basis for precision triangulated measurements.

After the first year of monitoring and when the displacements were big enough to disregard the instrument's incertitude, the slide became apparent and it was necessary to install piezometers and more inclinometers, this time long enough to intersect the sliding plane. The boreholes were also used to determine the geological profile of the slope. This action was complemented with the installation of more topographic references along the berms.

When the movements accelerated in 1990, there was necessary to increase the geological exploration to determine the back extention of the mass in movement and the boreholes were used to install more inclinometers and piezometers. The array of the instrumentation in the slope is shown in Fig 5.

Topographic measurements

The displacements of the tip of inclinometers obtained by surveying are shown in Fig 6, along with the monthly rainfall records.

As said before, the slope moved at a rate of 1.8 to 2 mm per month during the dry season and accelerated to about the double during the rains. But in September 1990, the slide continued to move at the same rate even in the absence of rainfall and accelerated again in the next raining season up to 118 mm per month, showing clearly the beginning of a failure.

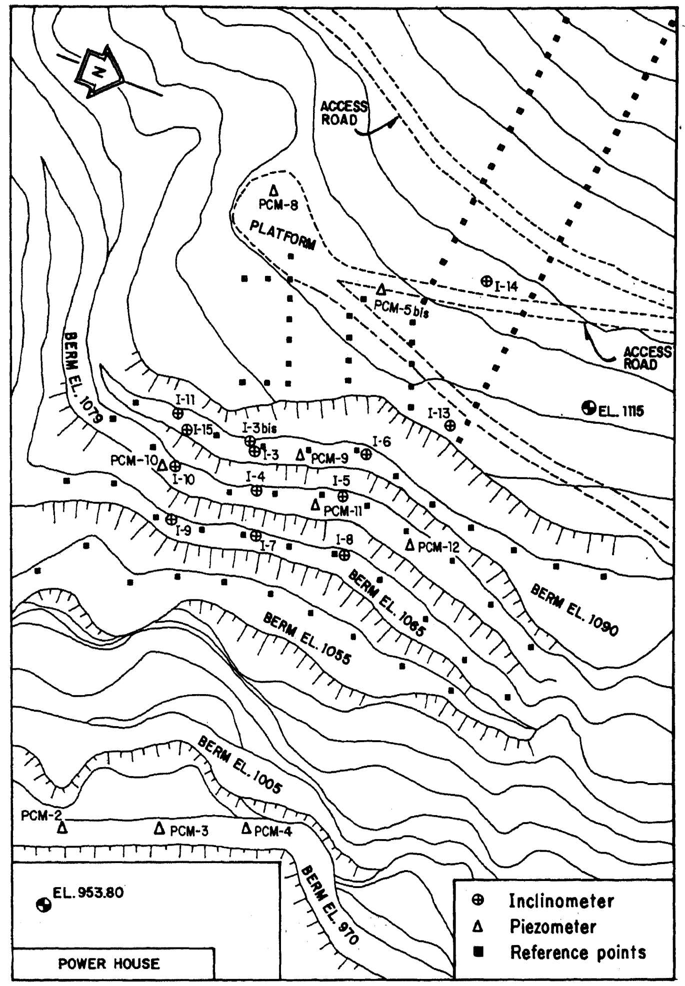

Fig. 5 Geotechnical Instrumentation and trimming berms

In August 23, 1991, after reaching a total displacement of 358 mm, the movement stopped as result of the stabilization actions described later. Little oscillating movements were observed from July 1992 to date as a consequence of the new instrument's incertitude and methodology used to determine faster the coordinates of the reference points.

Another valuable data obtained from the topographic reference points was the location of the up and lateral slide limits before it was revealed by the occurrence of cracking uphill the slope.

Inclinometric measurements

Because the telescopic initial array of inclinometers, three instruments (I-4, I-5 and I-6) did not show significant displacements while the others did. This obscured the first data analysis until when with the longer inclinometers an almost horizontal sliding plane was detected. Then the data was reprocessed considering the topographic coordinates of the mouthpiece of the casings as the fix point and then by integrating downwards the inclinometric geometry. The presence of a rigid block moving over the clay layer became then clear. Figure 7 shows the simplified deformation profiles for

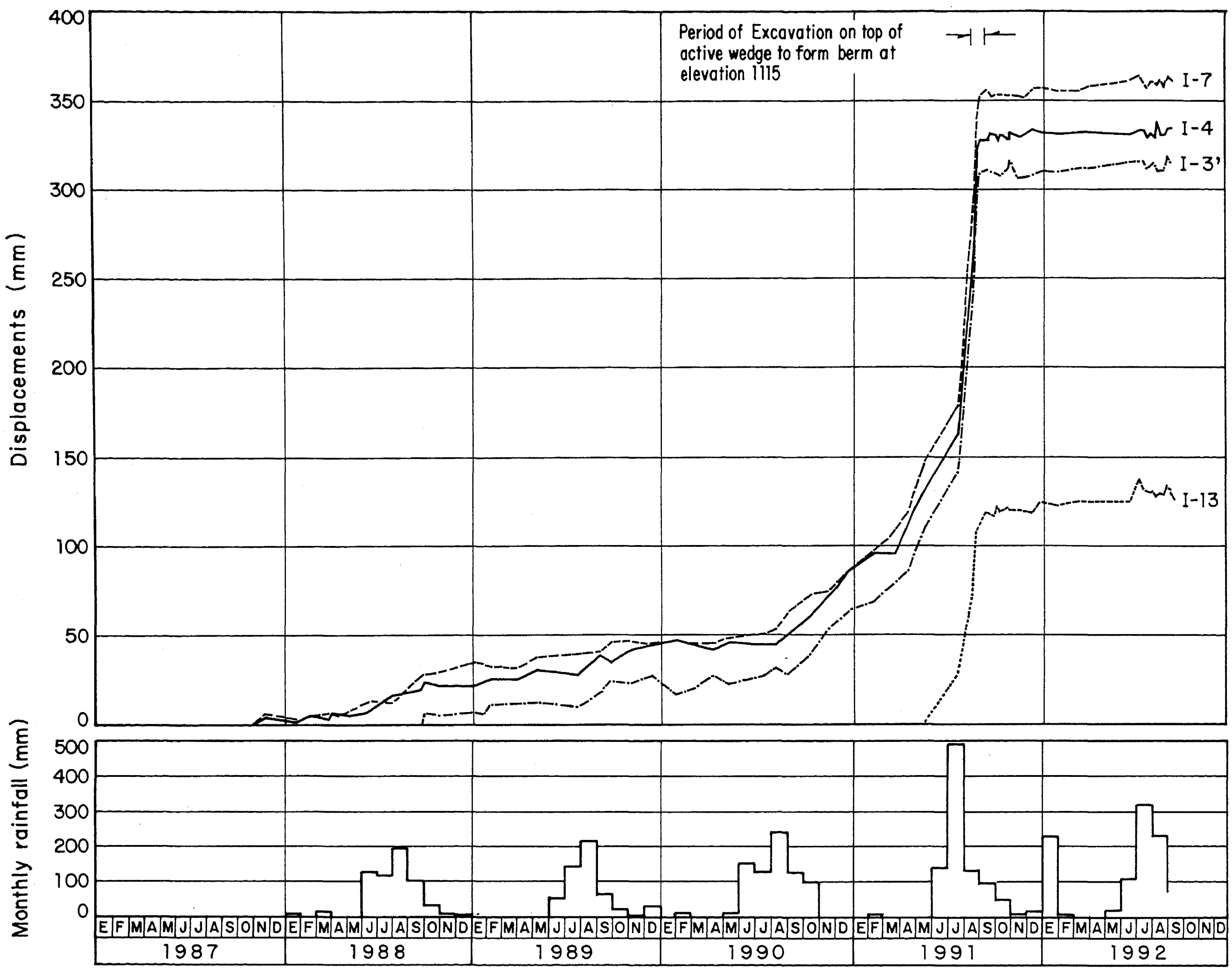

Fig. 6. Horizontal displacements of the mouth piece of four inclinometers

three inclinometers along the central section of the slope. It can be observed the great magnitude of the slide between May and December 1991.

Piezometric measurements

Because the presence and flow of water inside the mass is a relevant factor in slope instability, it was judged important to measure the piezometric level. This goal was reached at the beginning by using the inclinometer casings as observation wells and afterwards by installing open pipe piezometers.

The piezometric levels inside the sliding mass registered by several instruments from 1991 to date are shown in Fig 8. It is clear the raising of these levels during the raining seasons. Although their values vary according to the location and type of each instrument. The data are consistent and reliable and permit the evaluation of water pressures inside the slope.

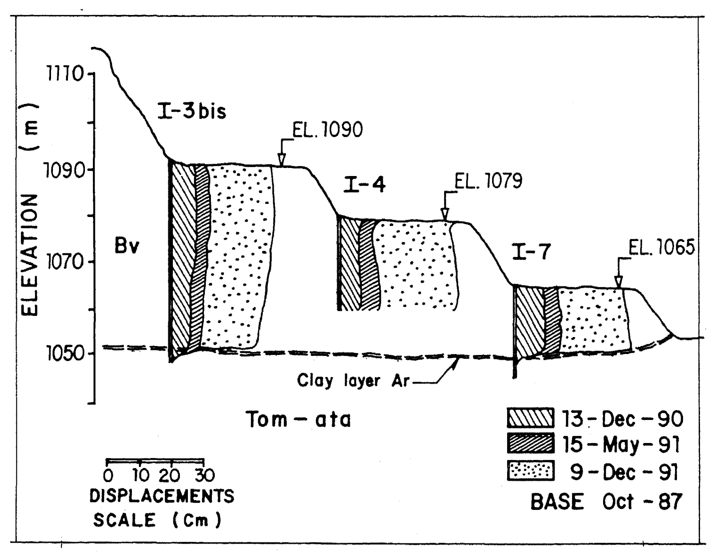

Fig. 7 Deformations along three Inclinometer casings

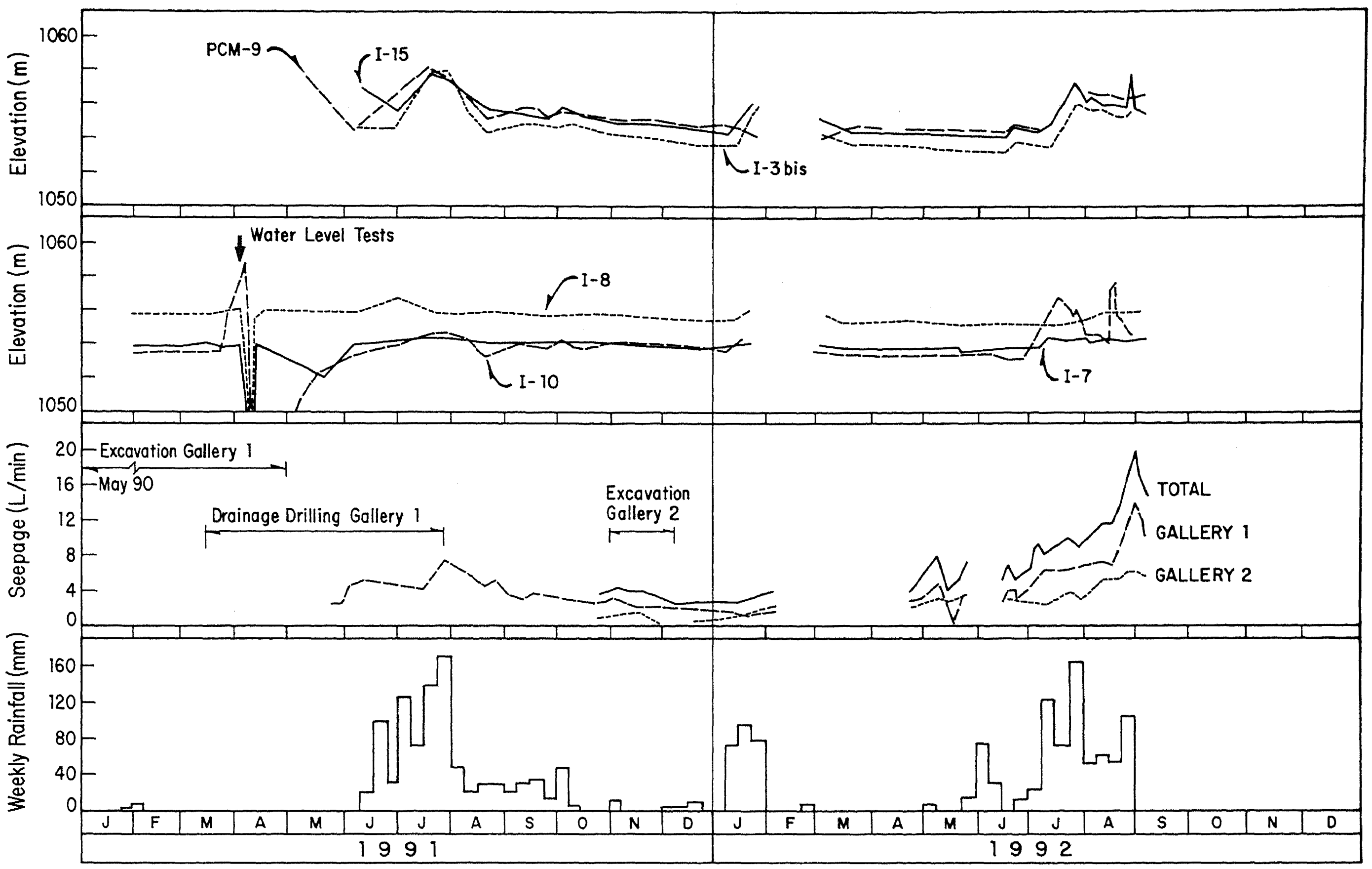

Fig. 8 Piezometric levels, seepage through drainage galleries and rainfall

The piezometric behavior inside the slope are governed both by the permeability diferences between the Tom-ata and Bv Units, and by the impermeable layer formed by the Clay Ar Unit. The first keeps high the piezometric level at the back of the sliding block and the second allows the water table raising inside this block.

To down the piezometric level two drainage galleries were excavated in 1991 (Fig 11). The profile along the main instrumented section for the readings made in September 7, 1992 is shown in Fig 9, where it can be observed both the permeability effect and the water table lowering at the block as result of the drainage gallery construction.

The seepage water collected by the new drainage galleries is also shown in Fig 8. It can be seen the increase of the drained water volume during the rainy season, as another evidence that the galleries are working as planned.

STABILITY ANALYSIS AND REMEDIAL WORKS

The geotechnical instrumentation data together with the geological explorations and site inspections permitted the definition of the sliding block boundaries. It is constituted by nearly one million cubic meters of the breccia material Bv and limited by the clay layer (Ar Unit) at the base and the contact with the andesite basalt material (Tom-ata Unit) at the back (Figs. 3 and 4).

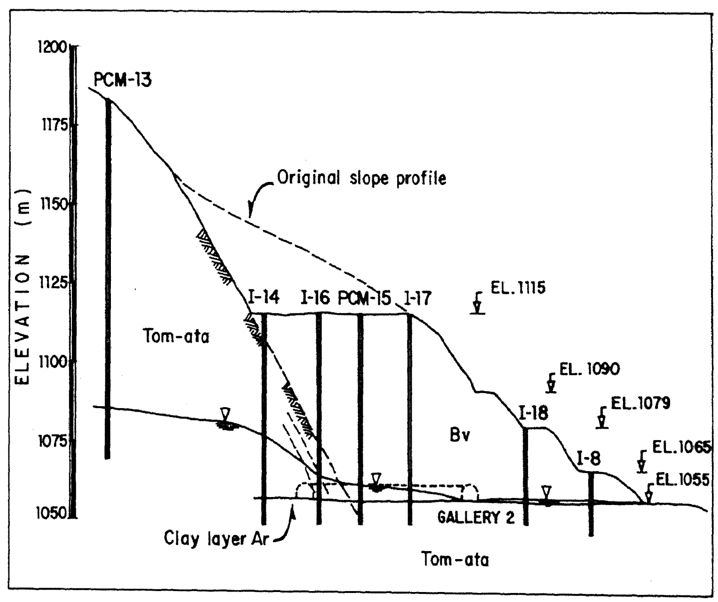

Fig. 9. Piezometric profile along the main instrumented section

The failure mechanism conceived is presented in Fig. 10. The active wedge of the block (ABC zone) pushes outward the passive block (BCD) with a force Ea which slides horizontally against the shearing resistance Fr of the clay layer. The other acting forces are the weight of the block W, the percollation water force P and the upward reaction U from the underlying rock, including effective stress plus pore pressure. When the trimming cuts were performed (Excavation I), the movement began because the decrease in Fr due both a reduction of the block weight and the decrease of the angle of internal friction of the clay after the occurrence of some displacements. The slide accelerated during the rainy seasons due to the increase in percolation force by accumulation of water above the clay layer. Moreover, from June to December 1990 five boreholes were drilled to install more inclinometers and piezometers using bentonitic mud as perforation fluid. Due to the high permeability of the breccia sytematic loss of perforation fluid happened, thus introducing about 40 ton of bentonite. It is presumed that this mud created low permeability zones within the breccia block therefore increasing the percollation water force P during the rainy season of 1991, resulting in the great block displacements measured.

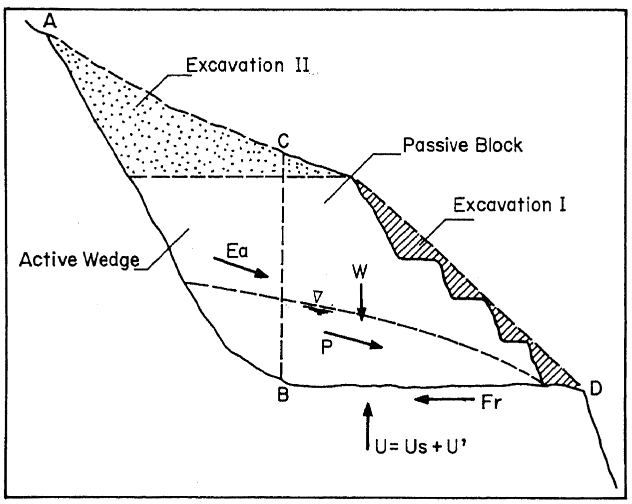

Fig. 10 Failure mechanism adopted for stability analysis

As soon as the slide was detected (1988) several actions were undertaken to regain the slope stability including the prevention of water infiltration from the ground surface by applying shotcrete in the trimmed berms, by constructing several gutters, and then by the excavation of two draining galleries with a large number of boreholes from the inside.

As the slide continued and accelerated, the information acquisition was also accelerated in order to define a geomechanical model and to carry out the stability analysis. A detailed description of this process can be found in reference 3.

The analysis results leads to recomend the excavation of 100,000 m^3 at the top of the active wedge (Excavation II in Fig. 10) forming a platform at elevation 1115. Because of the weakness of the rock, this was performed immediately without the use of explosives and achieved from August 11 to September 9, 1991.

The slide stoped in August 23 (Fig. 6) when the excavation volume was 37,200 m^3 approximately, equivalent to about 89,300 ton. This was the force needed to stop the slide and that means that the slide could have not been stopped economically and in time by other means. As the final excavation volume was 99,708 m^3 it is estimated that the static safety factor increased 25%.

To minimize the rainfall percollation force, the platform at elevation 1115 was covered with a layer of compacted clay and then paved with asphalt, complemented with surface drains along the berm and uphill the slope.

Finally, it was necessary to install three new inclinometers and two open piezometers to monitor the slope behaviour afterwards, because the former inclinometers were disabled by shear at the clay layer elevation due to the big displacements of the slope, and some piezometers were destroyed during the plaform excavation. The new instrumentation and the drainage galleries location as well as the elevation 1115 platform are shown in Fig 11.

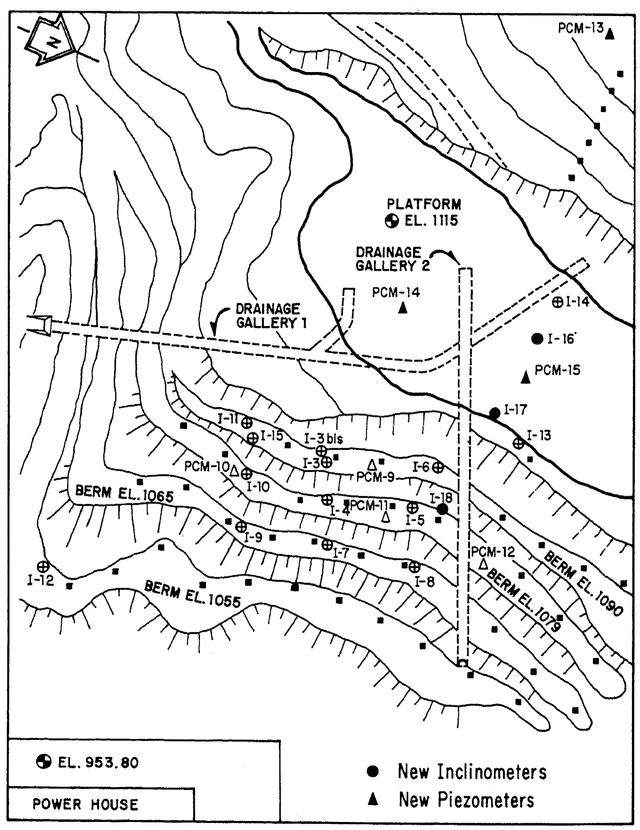

Fig. 11 Drainage Galleries, Excavated Platform and location of new instruments

CONCLUSIONS

The stability of the natural slope uphill the Agua Prieta powerhouse was at the limit and the trimming excavations induced its movilization, thus obtaining the opposite effect.

Nevertheless, appropriate remedial works based on drainage and excavation on the top of the slide allowed to stop the movements.

The stabilization works have performed adequatelly so far and the new instrumentation will allow to monitor the slope behaviour afterwards.

This article shows the relevant role of the geotechnical instrumentation in order to detect the existence of the instability problem and to monitor the changes in the acceleration of the slope displacements. Based on these readings the remedial works were undertaken just in time to prevent the slope failure.

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