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Lessons learned from dam construction in Patagonia Argentina

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Lecture

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Abstract

Three case histories of large dams built in North Patagonia which experienced unforeseen problems during construction, or after several years of operation are described. The necessary remedial and corrective works involved the development of important programs, being economic and programmatic impacts of great magnitude. The lessons learned from these experiences were very useful to the practice of design and construction of dams in the region. At the sites of three projects: Casa de Piedra, Alicura and El Chocon weak rock foundation are founded. In Casa de Piedra, located in a regional environments with clear evidence of limestone and gypsum formations, the use of microgravimetry was appropriate for the detection of cavities or discontinuities that traditional survey research may not detect. In Alicura where major structures were located on the left abutment, it was important to increase knowledge in the sector through early specific exploratory interventions, such as trenches, deep wells and exploration galleries. The importance of a good drainage system and percolation controls during operation through galleries and drains was fundamental. The case of El Chocon, where the situation becomes critical after ten years of normal operation, again shows the need for control and monitoring of the project throughout the useful life of the dam. The instrumentation system and the permanent control carried out by the Owner, Hidronor, made it possible to detect unfavorable conditions and plan an adequate corrective action in time.

1. Introduction

1.1 Victor de Mello. In Memoriam

It is really a privilege and an honor to have the opportunity to present this 7th Lecture in memory of Victor de Mello at the X Luso-Brazilian Congress.

Victor brilliant personal qualities has been described by the De Mello previous lecturers: “Friend, Engineer and Philosopher”, John Burland; “De Mello Foundation Engineering Legacy”, Harry Poulos; “My mentor and my role model”, M. Jamiolkowski; “Giant of Geotechnics”, Jim Mitchel; “A visionary”, Giroud; “Victor devoted his life to the betterment of people not only of Brazil, but also the world at large”, N. Morgenstern. I agree with all of them.

I had the honorable opportunity to write in the introduction to De Mello Volume, published in his tribute by his disciples and the unconditional support of his wife Maria Luiza, in 1989, my vision of Victor’s transcendent influence in the world of Geotechnical Engineering and especially in our region: “Victor de Mello in Latinoamerica” (Figure 1).

Some paragraphs included in that writing synthesize our relationship and I think it is appropriate to repeat:

“I made acquaintance with Victor de Mello during de 2nd Panamerican Conference on Soil Mechanics and Foundation Engineering, held in Brazil, in 1963”. I was 27 years old.

“I was deeply impressed by the clarity of his concepts and the acuity of his judgment.”

“His salient personality results in that in all areas in which he exercises activity, he achieves an outstandingly high level, as a consequence of the unusual compounding of natural gifts that are rarely encountered, developed to such a high degree in a single person: he has the indefatigable capacity of work of a Portuguese; the stoicism, and patience and interior peace of an Hindu; the preoccupation with perfectionism of a Swiss; the method and systematism of a Britisher; the pragmatism of an American; and the eloquence and enthusiasm of a Brazilian”.

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Figure 1. De Mello Volume - Victor signing my copy.

Figure 2. V Pan American Soil Mechanics and Foundation Engineering Congress, Buenos Aires, 1975.

The publication of the De Mello Volume was in fact coincident with the First International Society for Soil Mechanics and Foundation Engineering held in South America, in Rio de Janeiro in 1989. At this Conference I had the privilege to present a Special Lecture "Embankment Dams and Dam Foundations" written in collaboration with Victor De Mello, Peter Anagnosti and Norbert Morgenstern. It was really a great and a grateful experience.

In Argentina Victor participated in the most important events since our first conference on Soil Mechanics in 1968. His invaluable support for our country was continuous. In 1975 as Vice President of the ISMSFE for South America (Figure 2), he contributed as author and final evaluator of the Pan American Soil Mechanics and Foundation Engineering Congress, in Buenos Aires, and then in any meeting and technical event in which we ask for his participation.

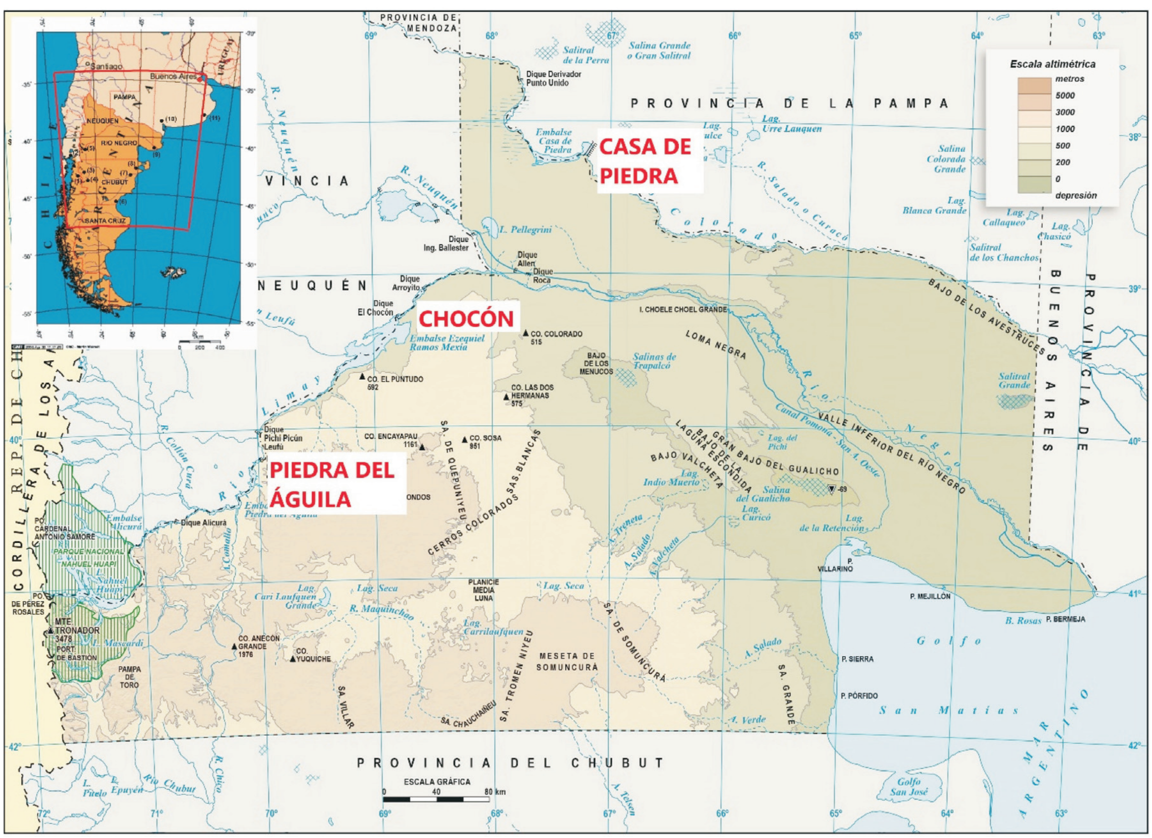


Figure 3. Location of dams: Casa de Piedra, El Chocón, Alicura.

In 1990 Victor made an unforgettable presentation of my Casagrande Conference in the Pan American Soil Mechanics and Foundation Engineering Congress held in Viña del Mar, Chile, in 1991 with its characteristic ingenuity and generosity that I keep on my mind as an unforgettable memory (Vardé, 1991).

His professional support was no less important. He participated as a Consultant and Expert in numerous large hydroelectrical projects in Argentina: Paraná Medio and Yacyretá as a Board Member; Potrerillos as a member of the Board with Giovanni Lombardi and myself; Casa de Piedra and Rio Hondo as an Independent Consultant, among others. In all of them he gave his experience and knowledge generously as was his characteristic.

I cannot fail to mention the role of Maria Luiza, who devoted her effort and life to Victor during long years of his brilliant career, and Maria, who gave peace and support to Victor in his last years.

Victor, an unrepeatable human being, had an enormous influence on my professional and personal life that was not only based on his abilities and his teachings, but on his very essence.

For me it was a before and after meeting him, becoming more than a mentor, colleague and friend but "brother" as he called a small number of people in the world.

For all that my eternal recognition and admiration to his memory.

1.2 This paper

Three case histories of large dams built in North Patagonia which experienced unforeseen problems during construction, or after several years of operation are described.

The necessary remedial and corrective works involved the development of importance programs, being economic and programmatic impacts of great magnitude.

The lessons learned from these experiences were very useful to the practice of design and construction of dams in the region.

2. Location

The three projects described in this paper: Casa de Piedra, Alicura and El Chocón were built, as mentioned before, in the region of Northern Patagonia Argentina (Figure 3).

The Patagonia, located in the southern end of South America, is bounded by the Colorado River on its north. On this river is located Casa de Piedra Dam.

El Chocón and Alicura Dams are across the Río Limay, in a subregion called Comahue, which is one of the richest in Argentina in natural resources. Particularly the third largest world reserve of natural shale gas, after China and U.S.A, is located in the Comahue region.

At the sites of the three dams weak rock formations are founded in which studies and specific investigations were carried by local and international geotechnical engineers in the last 50 years, related to the construction of large hydroelectrical projects (Deere & Vardé, 1986; Vardé, 1987; Vardé, 1988; Vardé et al., 1989; among others).

3. Case histories

3.1 Casa de Piedra dam

The Casa de Piedra dam, located on the Colorado River is owned by a multidistrict administration: "Ente Casa de Piedra", with representatives from the governments of the Provinces of La Pampa, Rio Negro, Buenos Aires, and from the National Interior Ministry of Argentina.

The dam was designed by a Consulting Group including Sir Alexander Gibbs and Partners from England, TAMS from U.S.A. and IATASA from Argentina. The Contractor was Impregilo, from Italy (Vardé, 1990).

The earth dam has a total length of 11 km, a maximum height of 54 m in the 200 m river Gorge, and an average height of 20 m founded on both banks on the river Terraces. Seven kilometers are on the left bank (partial view in Figure 4).

Foundations are mostly marine deposits of upper Cretaceous and Lower Tertiary, including marls, claystone, fossiliferous and coquina, and limestones (Figure 5).

Unfavorable geological features were detected during work dam foundations, consisting of caverns (Figure 6) and dissolution channels through massive gypsum below a zone of the left bank, with a length of 800 m, between stations 700-1500 m, (Vardé, 1986; Vardé et al., 1990; Etcheon & Speziale, 1989). The impact in the project construction and the total costs were very significant. The investigation of the problem was initiated because of a fortuitous event. The presence of a saline paleo layer prevented the setting of the concrete from a cut-off in that sector. The noticeable presence of gypsum, the loss of injection water in the boreholes and the fall of tools detected

Figure 4. Casa de Piedra - detail: spillway and powerhouse.

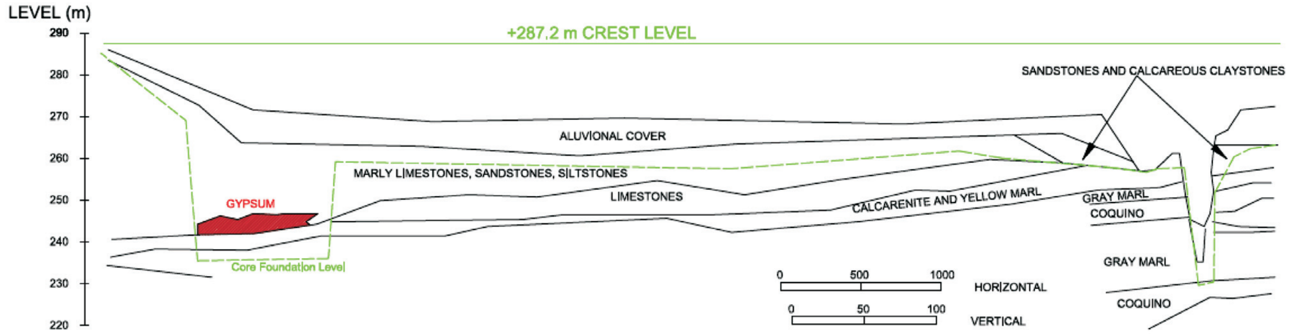


Figure 5. Longitudinal profile, showing location of the gypsum stratum.

Figure 6. Karst cavern.

the cavities. It was possible to define the extent of the anomaly and also rule out the existence of karst formations outside the delimited area.

It was necessary to carry out a very thorough investigation program specifically addressing these issues and introducing important changes in the design, which included geological, geomorphological and special geophysical methods using microgravimetric techniques.

A large open pit, 80.000 m^3 was excavated to check the effect of grouting test and pumping tests to determine the feasibility of dewatering a local ancient aquifer.

The program allowed to determine the extension of the karstic gypsum bed, 20 m deep 5 m thick, which lies over the pervious calcarenite and underneath the red claystones. The test excavation had a defining impact on the scheduling of subsequent tasks. Pumping tests were carried out and blasting test allowed the selection of the excavation method.

The microgravity survey has been a major help to assess the occurrence and location of karstic cavities. The technique was successfully used for cavities detection in important structures like the Great Pyramid of Cheops.

The investigation in Casa de Piedra was carried out by the Compagnie de Prospection Geophysique Francaise under de supervision of Geoconseil of France (Mariotti et al., 1990). A total of 619 gravimetric stations were installed, using a gravimeter of high precision, 0.5 cgal. The detection of negative anomalies, between -2 to -8 cgal in the zone of stations 700-1500 were in very good agreement with the location of cavities and gypsum dissolution phenomena, as was lately verified during the excavations for dam construction.

Several alternatives were considered for the foundation treatment of the affected zone, and a big excavation of about 2.000.000 m^3 was adopted to remove all the potential karstic materials in the core, filters and part of the shell foundation. The treatment through injections and the partial removal by sectors was evaluated. Finally, due to the associated uncertainties, the total excavation of the area affected by karsting was decided.

It caused one year delay in the dam construction program.

The central excavation was complemented by two symmetrical trenches, normal to dam axis, 86 m long, founded in the gray marls and filled with core material, to avoid potential short seepage paths through the more pervious materials (Figure 7). In situ permeability tests and numerical modeling were made to check critical hydraulic gradients and piping potential.

There have been many records of dams affected by the dissolution of salts, causing the formation of caverns, and increasing the permeability in foundations by enlarging rock discontinuities, dramatically increasing the flow rate. In the case of karstic foundations, like Casa de Piedra, where karsts were revealed during construction, although a very extensive conventional investigation (boreholes, geological mapping) was carried out without detecting the abnormality, adequate techniques and early works were required.

Proper and specific investigation techniques such as inspection adits and special geophysical methods, like gravimetry, are mandatory in order to achieve a successfully project of foundations on karstic formations.

3.2 Alicura dam

The Alicura Hydroelectrical Project has been constructed on Limay River, 100 km NE of San Carlos de Bariloche city, Argentina. The project includes a 130 m high

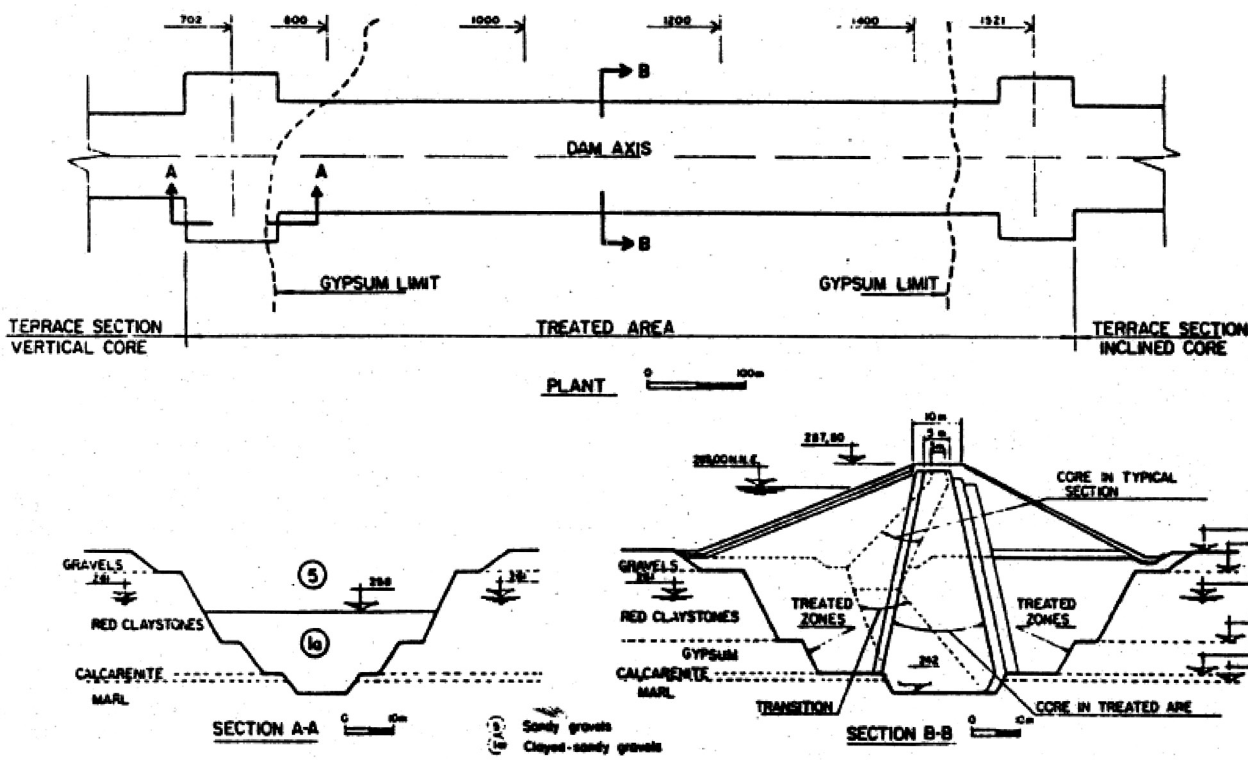


Figure 7. Casa de Piedra - Adopted Solution S700-S1500.

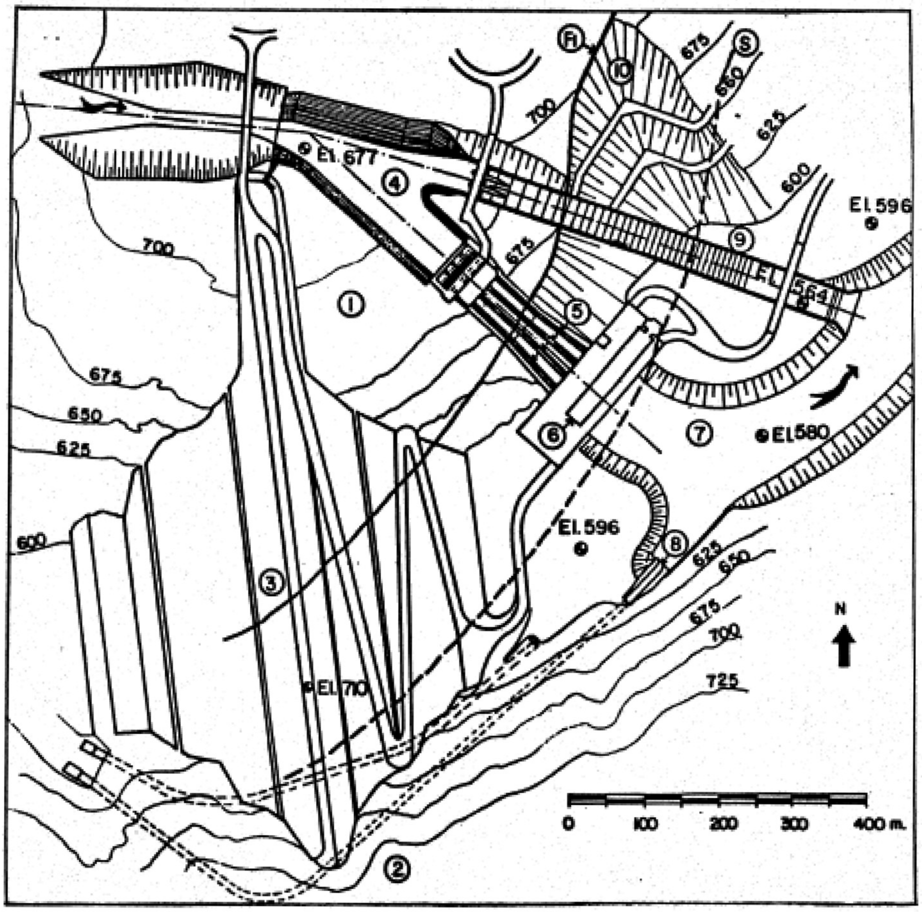
earth fill dam with the principal appurtenant structures located on the left bank, taking advantage of topographical features (Figure 8).

The total volume of the dam is 13 millions of cubic meters, with a central core of morainic material founded on rock, while the shells rest directly on 10-15 m of alluvium. The owner was Hidronor S.A. a state-owned public utility who also operated El Chocón Dam. The design and supervision of construction was made by Consorcio Consultores Alicura, a joint venture of local consulting firms of Argentina, Electrowatt from Switzerland, and Sweco from Sweden. The Contractor was Impregilo from Italy. The Hidronor Board of International Experts were Don U. Deere (USA), Giovanni Lombardi (Switzerland), Jack Hilf (USA), Flavio Lyra (Brazil), and Bolton Seed (USA), who reviewed the design, construction and performance.

Figure 8. Alicura. Aerial view.

The bedrock of the project area consists of a succession of psammitic (sandstone) and pelitic (mostly mudstone and siltstone, some claystone) rocks of Liassic (Lower Jurassic) age. Planar sedimentary structures do not persist over any considerable distance and individual sandstone or pelitic layers cannot be correlated between drillholes and outcrops, affected by neotechtonics movements (Vardé et al., 1986). The bedding interfaces are the dominant structural element in the bedrock, being in general horizontal with some very gentle folding. A major fault, denominated fault 1, running roughly NNE to SSW and dipping steeply SE intersects the penstock trench and spillway chute downslope of the corresponding intake structures (Figure 9 and Figure 10).

At the downhill side of the fault, in the penstock, as well as in the spillway area, the bedding of the rock abruptly changes to a dip of 20 to 38° SE to E, i.e. parallel to the slope (Figure 11). The existence of the zone of inclined bedding on the left bank was not known at the initial design stage, when the investigation was based mainly on geological mapping of outcrops and 8.000 meters of rotary drilling borings size NX. The inclined bedding was subsequently encountered in trench excavations in the area of penstocks



1. Left abutment
2. Right abutment
3. Dam
4. Approach channel
5. Penstocks
6. Powerhouse
7. Tailrace channel
8. Bottom outlet
9. Spillway
10. Stabilization excavation
F1: Fault 1 S: syncline

Figure 9. General layout - Fault alignments.

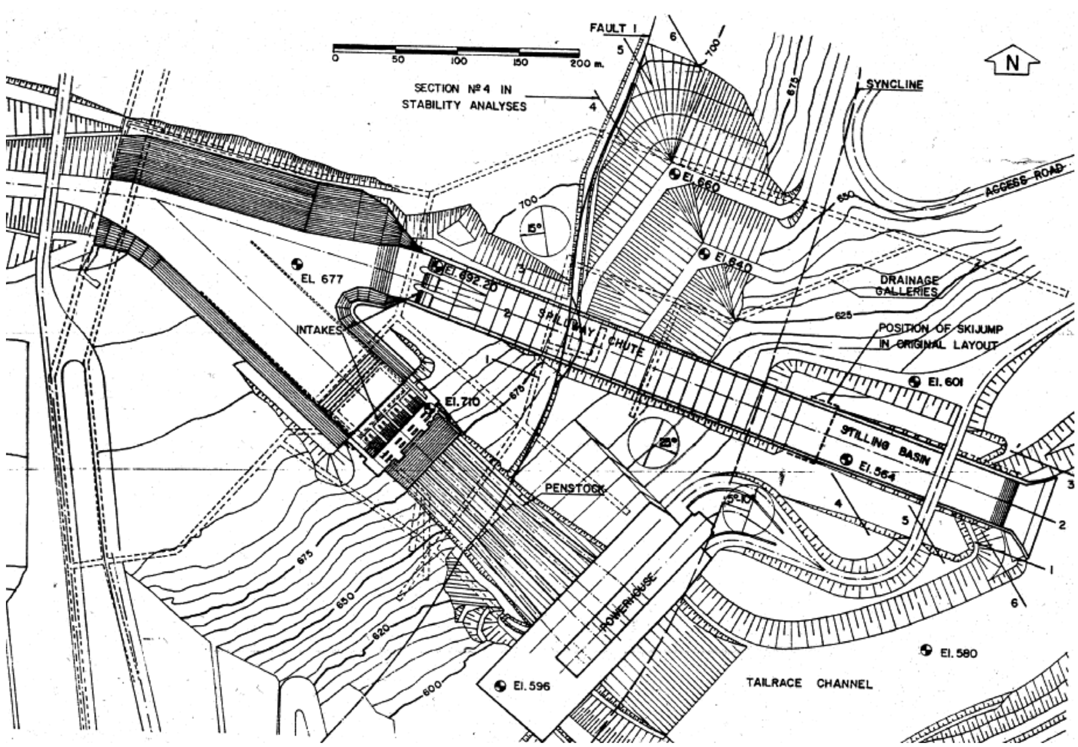


Figure 10. Detail of the left bank.

and considerable efforts were directed towards exploring this feature in detail.

An exploratory gallery at El 590 and a vertical shaft were also excavated. It should be noted that these investigations and the survey of the real conditions of the massif were defined during the excavations in the first stage of construction. At the design stage they were not detected despite having carried out more than 4000 m of exploratory boreholes and an exploration gallery with load plate tests.

The gallery crossed fault identified as 1, which proved to consist of a zone of plastic mylonitized rock, 2 m thick. Fault 1 appears quite impervious, forming a barrier that cut off seepage from the uphill side. The trenches carried out in the area of the penstocks and along the spillway provided useful information together with investigation borings and laboratory testing of samples. Drill holes, shafts and trenches in the valley floor indicated a return to flat dips probably due to the presence of additional faults.

A large number of pelitic interlayers are intensely sheared, predominantly along the upper contact with the sandstones. The shearing produced slickensides parallel to the bedding planes. There were also thin 5 cm thick clay and silt bands of totally crushed material. This clay/silt mylonite is quite frequently squeezed out. In the lower parts of the pelite beds randomly oriented "mirror" faces can be observed.

Moreover, horizontal layers of weak pelites were observed upstream of fault 1 in some investigation drillings. The most important one at El 655 to 660 below the penstock and spillway intakes became visible during the penstock trench excavation. It was deemed necessary to improve the safety by constructing shear keys under these structures, which were formed by excavating galleries and filling them with concrete.

The geotechnical properties of the sandstones are variable:

- Unconfined compressive strength from 30 to 40 MPa.
- Compression wave velocity of 2800 m/s.
- Friction angle (ϕ') from 35 to 55°; cohesion (c') from 200 to 250 kPa.

The competent pelites gave test values somewhat lower than the sandstones, but most of the efforts were concentrated on assessing the rock properties on the weaker pelites due to their crucial importance in the stability.

Weak pelites exists along the sliding planes. It can be classified as a clay to medium plasticity, with plasticity indices of 5 to 20 and liquid limit between 20 to 40. The most plastic samples contained about 40 % of clay (minor to 2 microns). The samples taken in the field had a natural water content at or below the plastic limit and were practically fully saturated. The dry density was between 1.8 to 1.9 g/cm^3 and the specific density ranged from 2.44 to 2.68 g/cm^3.

Since it was very difficult to extract good undisturbed samples, the drained strength parameters were determined by direct shear and triaxial tests mainly on remolded samples, which were reconsolidated to a density similar to the undisturbed ones. The results are summarized in Table 1.

The residual friction angles are plotted against the corresponding range of plasticity indices in Figure 12, where the empirical boundary curves proposed by Deere and Seycek, respectively are indicated.

It can be observed that the values obtained by direct shear tests are lower than those by triaxial tests and rather close to the lower boundary. The direct shear test in this case is the most appropriate procedure due to the possibility of orienting the samples and allows greater deformations of the specimens, including repeated and reverse cutting stages to reach the residual condition.

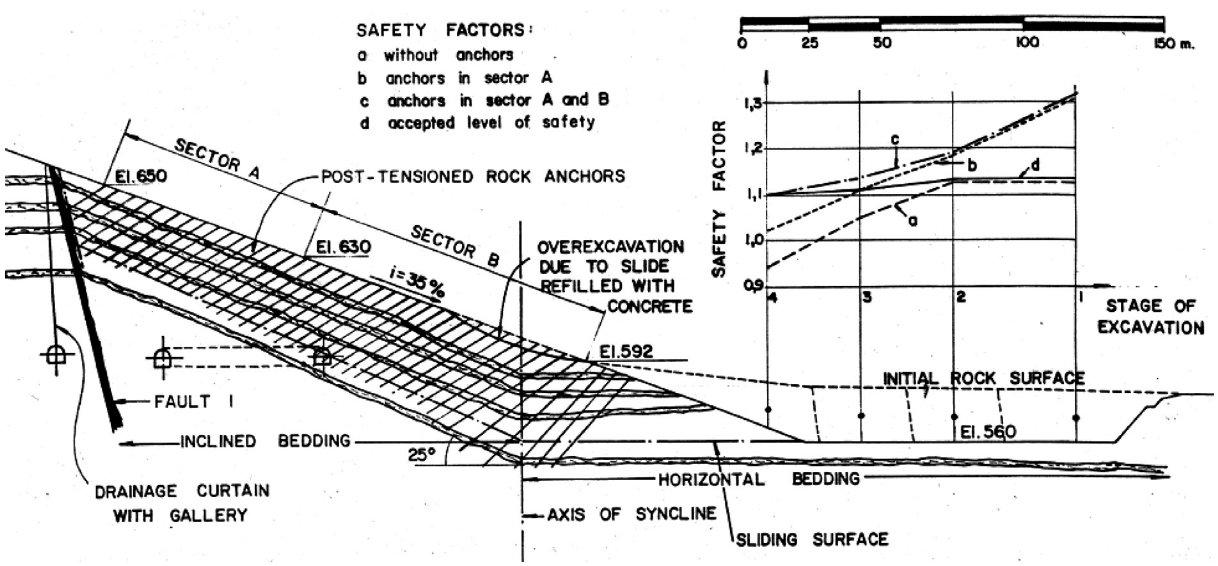


Figure 11. Cross section left bank, Stilling basin, spillway - stability analysis.

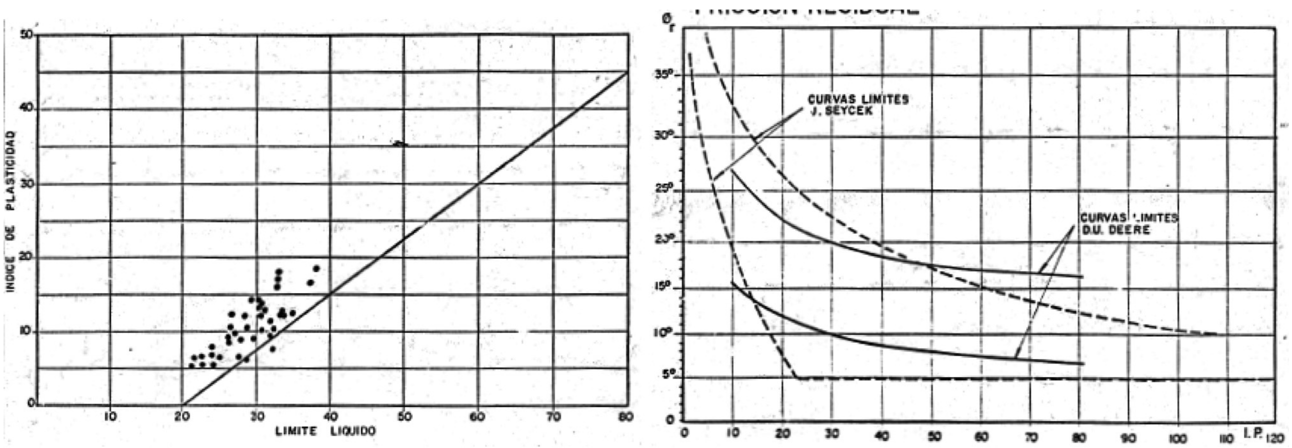


Figure 12. Plasticity chart showing results from sheared pelite; Deere and Seycek curves relating plasticity index and the residual friction angle.

Table 1. Summary of results from shear strength tests.

Type of test	Peak shear strength		Residual shear strength	
	c' (kPa)	ϕ' (°)	*c*' (kPa)	ϕ' (°)
Direct shear tests	0-30	22-31	0-10	17-21
Triaxial tests	0-30	22-30	0-10	21-24

Routine testing in the field laboratory of the Atterberg limits was then used to check that the plasticity indices fell in the known range. The geological and geotechnical characteristics of the left bank required a thorough design work in order to guarantee the stability of the slopes and its structures in the penstock area and the spillway chute and energy dissipator. The intake for the penstock and spillway structures required also an extensive stability analyses to guarantee that sliding would not occur on the weak horizontal pelite layers.

The stability of the left bank had to be improved by an extensive drainage system, shear keys and post-tensioned rock anchors (Andersson et al., 1985). These included 5.500 m of drainage galleries with 35.000 m of drain holes to form 150.000 m^2 of drainage curtain, a 1360 m long grouting gallery and 65.000 m^3 of grout curtain (Figure 13).

The purpose of the excavated galleries was to allow the execution, supervision and control of the screens of per-

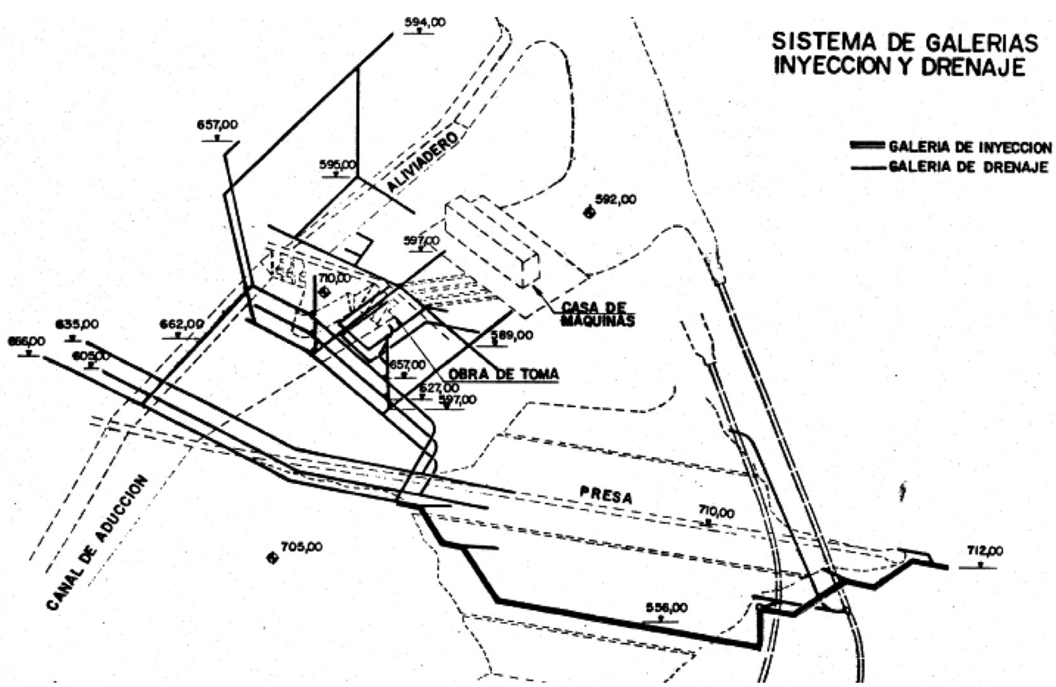


Figure 13. Drainage system on the left abutment.

forated vertical drains drilled by rotopercution equipment and spaced between 6 and 3 m.

On the left bank, three interceptors fronts of the water coming from the reservoir have been located by means of different galleries located at different levels, 605, 635 and 666. The first two are in the same vertical plane and are connected to each other by means of drains. The last one, at level 666 is 20 m upstream and continues towards the river valley to become an injection gallery below the earth dam, at level 556. Three parallel galleries conformed a second drainage curtain. They were connected to each other with vertical drains and to the previous ones. Those of the upper and lower levels extended parallel to the spillway to the surface of the hillside as a lateral drainage curtain. Two other superimposed galleries in the form of "U", also connected by vertical drains, were located at heights 635 and 655 in the area of foundation of the intake structure and in correspondence with the sheared pelite layers. The drainage galleries section was rectangular, 2 m wide and 3 m high; the walls and roof were protected with sprayed concrete and have a concrete floor slab with a draining gutter (Figure 14).

The drainage system is accessed from galleries on both sides of the Power House and also through two shafts 115 m deep, located near the intake and connecting the three levels of galleries in that sector. The effective drainage of the left bank was essential for the stability of the slope and the structures due to the inclined bedding and the fact that excavations would partly undercut the slope.

After a slide of around 120.000 m^3 occurred at a fairly early stage of construction in the area of the ski jump, revealing more unfavorable geotechnical conditions that had been considered in the original design, it was decided to substitute the spillway ski jump by a concrete stilling basin as the plunge pool could undercut the slope. Stabilizing measures were required including the removal of about one million of cubic meters of soil and rock to the left of the spillway chute to unload the slope, and the installation of about 600 anchors, 30-50 m long and post-tensioned to 1,000 kN. The Tensacciai system, similar to Freyssinet system was used. About 1 % of the anchors were provided with load cells in order to monitor the anchor forces. Additionally, horizontal extensometers were installed to detect any movement in the slope (Pujol & Andersson, 1985).

Figure 14. Drainage gallery.

The stability analyses were performed using a two-dimensional model composed of an active block and a passive one. The active block was delimited by the subvertical plane of fault 1 and by an inclined plane corresponding to a possible weak layer of pelite (Figure 11). The shear strength parameters assigned to the continuous plastic layers were $c' = 0$ and $\phi' = 17°$. Furthermore, a reduction of one third of the uplift water pressure and a minimum anchor pressure of 24 kPa with an active depth of 25 m were used.

The important conclusion obtained during the construction of the Alicura Project is that the adequate characterization of rock massifs affected by relatively small fault structures, but with shear planes between the strata, can only be achieved through a research plan that includes trench excavations, deep shafts and galleries. Conventional investigations through boreholes including special procedures do not adequately reveal the unfavorable features of thin sheared layers between more competent rocks, as in this case.

In the case of Alicura, where concrete structures are located on a terrace due to topographic advantages, guaranteeing the stability of the slopes is a critical factor for the execution of the works. Its economic impact can also be very important.

Consequently, the special work program must be carried out from the early stages of the studies.

In thin sheared strata, where in-situ tests have no application, it is important to define the continuity of weak planes and their shear strength properties through systematic sampling and characterization tests. It is of vital importance the implementation of an efficient drainage system. The installation of drains from different gallery levels allows control of the system operation.

3.3 El Chocón dam

El Chocón Hydroelectrical 1200 MW installed capacity, is located across the Limay river. A general view of the earth dam, the spillway and Power House is shown in Figure 15. Figure 16 presents the plan of El Chocón Dam.

The original design was made by a consortium formed by Italconsult, Sofrelec and Harza Engineering Co., between 1962 and 1965. The operation was assigned to Hidronor S.A. in 1968. The revision of the design and the construction supervision were carried out by Sir Alexander Gibb & Partners. The contractor was a joint venture formed by Impregilo, from Italy, and Sollazo, a local firm.

The first impounding of the reservoir took place in 1972. The earth dam is one of the largest in Argentina, 92 m maximum height, 13 millions m^3 volume and 2.245 m crest length. The plan layout was based on the topography. The

Figure 15. El Chocón dam - general view.

spillway is located on the right bank, about 100 m from the right abutment.

The dam cross section was designed using a thin sloping clay core between sandy gravel shells to reduce the differential settlements between the clay core and the gravel shells and also reducing the risks of horizontal cracking due to arching of the core between the shells. The top 14 m of the core were vertical (Figure 17).

The core material, obtained from a borrow area located in the reservoir area consisted of interbedded layers of clayed sands and silty sands, with a mean plasticity index of 23 %. Subsequent investigations on core material samples obtained between 1989 and 1990, from the right abutment by drilling into the core at different levels, have shown that the clay has dispersive characteristics. It is worth mentioning that at the time of design and construction stages of the works (1960 decade and early 1970 decade) the dispersive properties of the soils were not well known in civil engineering.

Various seepage control features were provided in both abutments and in the foundation of the dam. A cut off zone was excavated 5 m deep into rock in the river valley and a single line grout curtain was provided under the core centerline. The design grouting pattern consisted of holes on 10 m spacing, dipping upstream 35° along the dam axis to intersect the main joints. The depth of primary holes in the river valley was generally 25 m below the core-rock foundation rock contact. Grouting and drainage galleries were provided at the left abutment behind the Power Station to reduce seepage through the rock and to ensure that the phreatic surface exit point was kept below the toe of the cliff.

The geology at the site are predominantly horizontally bedded sandstone of late Cretaceous age. The Upper sandstone is formed by alternating layers of lithic sandstones with lenses of wacky siltstones and claystones generally 3 to 5 cm in thickness. Discontinuous thin gypsum levels of secondary origin were detected at the top and the base of the units. The sandstone forms the abutments as well as the left dam foundation.

Gypsum infillings in the foundation rock discontinuities were found in the river valley bottom and in both banks. These joint infillings are rather thin, ranging from several millimeters to 1 or 2 cm. Considerable evidence of

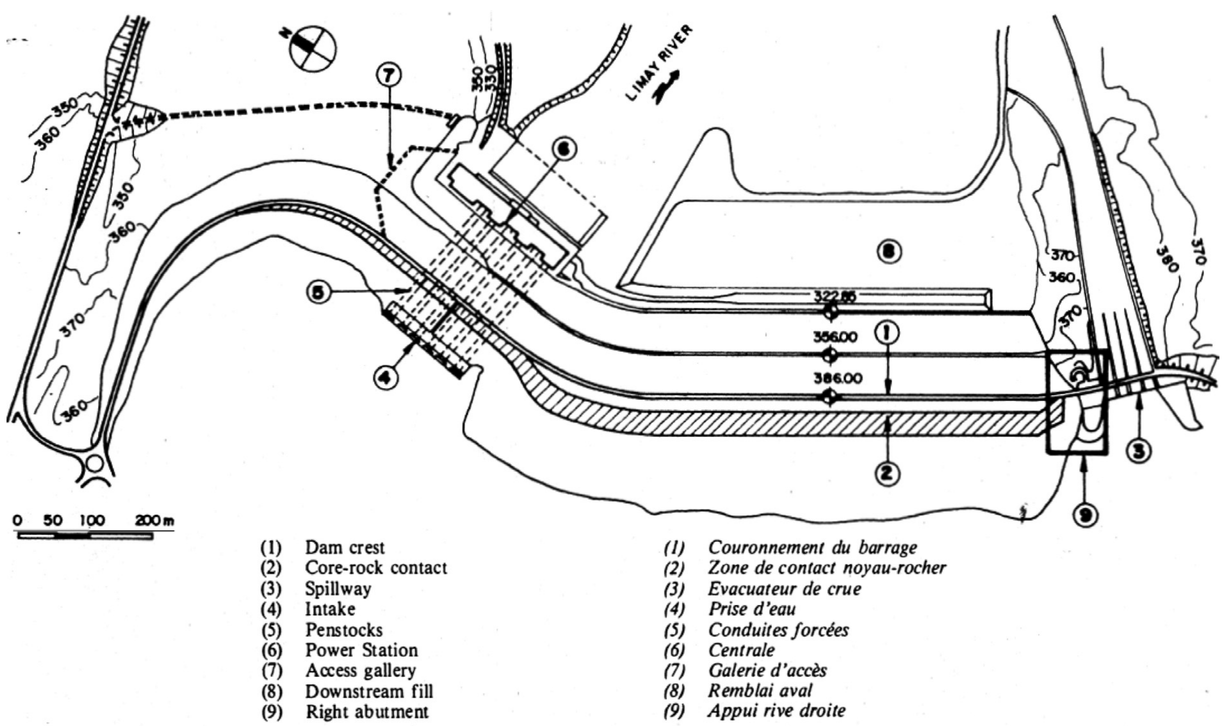


Figure 16. El Chocón dam - plan.

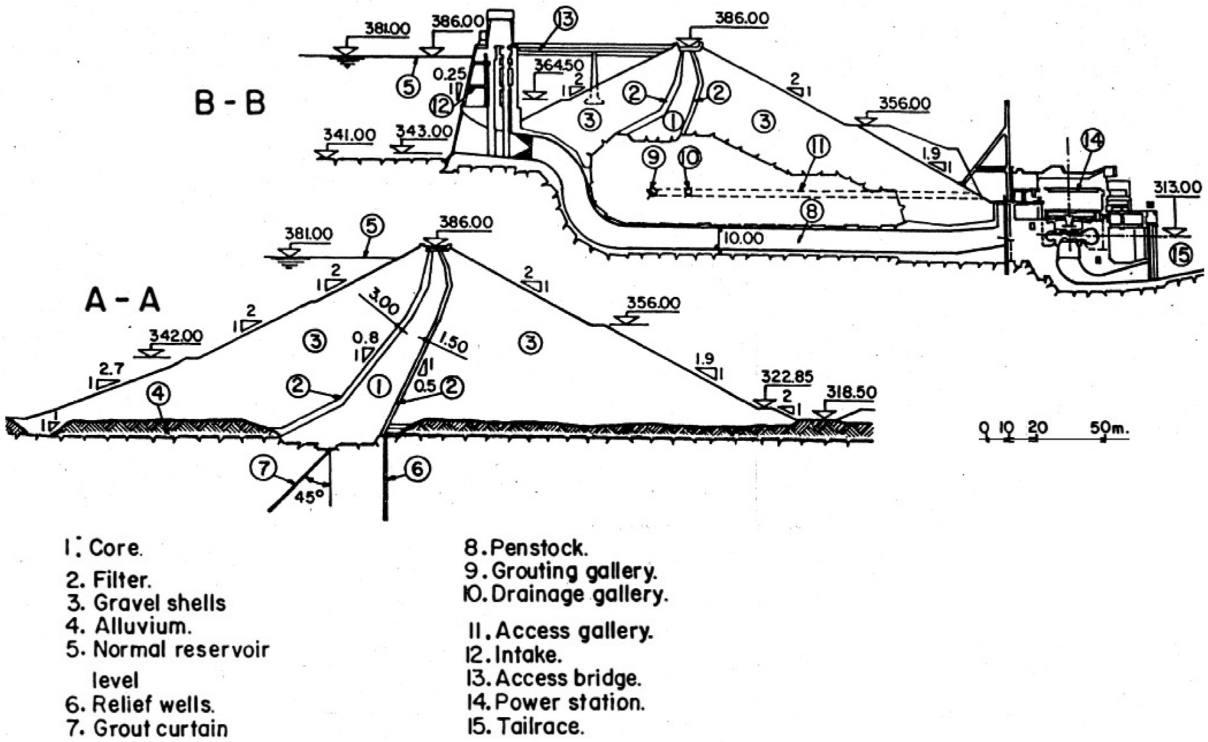


Figure 17. El Chocón dam - cross sections.

gypsum infillings was present on both banks, mainly as horizontal layers.

The instrumentation has generally performed satisfactorily. The piezometer levels in the core and the dam foundations had been considered reasonable during the first ten years after impounding. The performance of the dam and its foundation did not cause any particular concern in the period from 1972 to 1982.

In November 1982 the attention was focused on the development of high piezometric levels recorded in the core contact with the right abutment. An extensive program of studies and investigations was initiated by Hidronor to determine the causes of this behavior. A review of the pertinent data related to design, construction, geologic and geotechnical aspects were done and seepage and piezometric levels were monitored. Chemical analyses carried out on water samples from different drains showed that the measured seepage water of about 100 L/min contained an average of 2 g of soluble solids per liter. A boring program was performed to identify some of the existing fissures and joints and the amount of water seeping through them to know the percolation pattern through the dam foundation.

A first evaluation of the dam was made in August 1983. The conditions of the dam contact were of particular concern. The presence of high concentrations of soluble salts in the water effluents of drains and downstream of the dam, implied that an appreciable quantity of solids was removed during ten years of operation. It could be also indicative of progressive opening of joints in the rock foundations due of gypsum. Increasing flow could also result in some erosion of non-soluble fillings.

Field surveys had detected the presence of valley stress relief related joints, particularly on the right cliff between the dam and spillway area. A grouting program was considered necessary at both the right and left banks to reduce the potential of clay core piping through open rock joints (Aisiks et al., 1991a; Vardé, 1991).

A shaft in the rock, 107 m deep and three galleries at elevations of 346, 308 and 282 m were constructed to permit remedial grouting and drainage treatment of the rock abutment in the zone adjacent to the rock-core contact. The core is founded at the right bank in a cut off trench. A horizontal section of the abutment core-rock contact at El. 357 m is shown in Figure 18.

The core against the rock, face AB, and downstream, face BC, bears directly against the rock, without any protective filter. The layout of the shaft galleries in relation with the core is shown in Figure 19. Figure 20 shows the drillholes for right abutment treatment in detail.

The first few holes drilled towards the contact face AB revealed worse conditions that had been anticipated. A number of rack joints near the contact were found to be open and full of water at hydraulic pressures near reservoir level. The core when contacted was found in some cases to be either in a near fluid state or with very low consistency. The samples were recovered using special procedures.

All drilling and grouting done afterwards using a double gate system (SAS, Figures 21 and 22), and a pressure

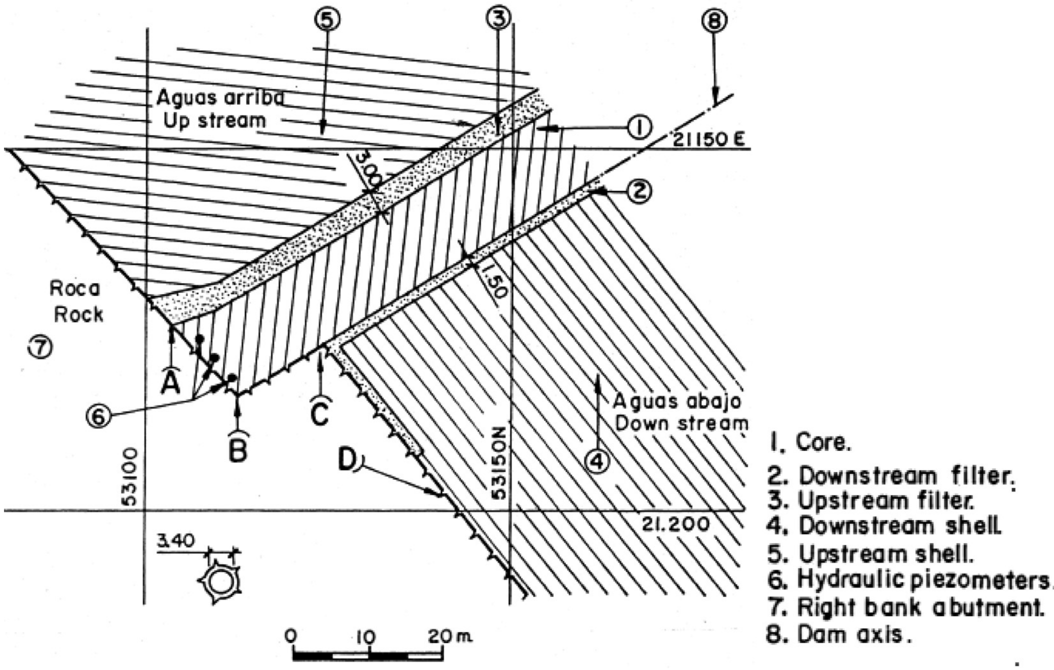


Figure 18. Horizontal cross section al elevation 357.

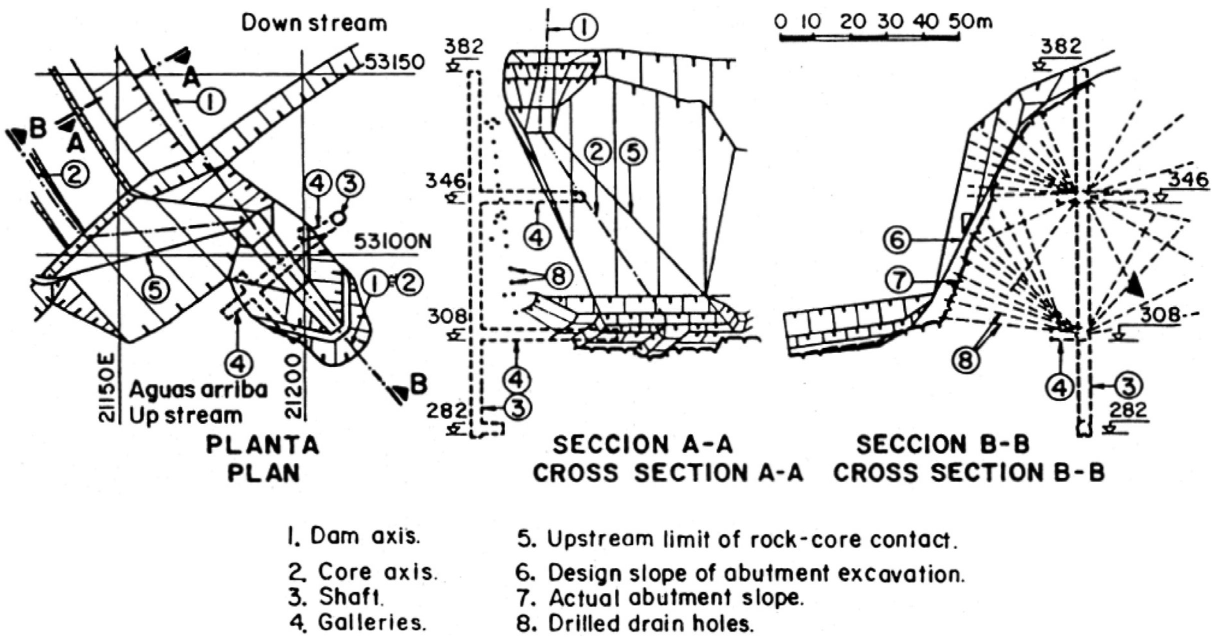


Figure 19. Layout of right abutment shaft and galleries.

regulation device (PRD), mounted at each borehole mouth. This system permits maintaining the pressure inside the hole equal or larger than the reservoir pressure to eliminate the danger of piping through the drillholes. More details of the special techniques used to treat the abutment of El Chocon can be found in Aisiks et al. (1991b).

It was concluded from the observation of the rock cores of drill holes that the rock near the contact was highly fractured due probable to stress relief and to blasting effects. Hydraulic piezometers located in the core adjacent to the steep abutment rock, 1 to 4 horizontal-vertical provided a way to assess the state of stress in the core. These tests show that with a reservoir level at El 369 m there was zero effective stress at the rock-contact above El 357 m. These data confirm that a crack existed within the core close to the steep abutment, cause by differential settlement and arch-

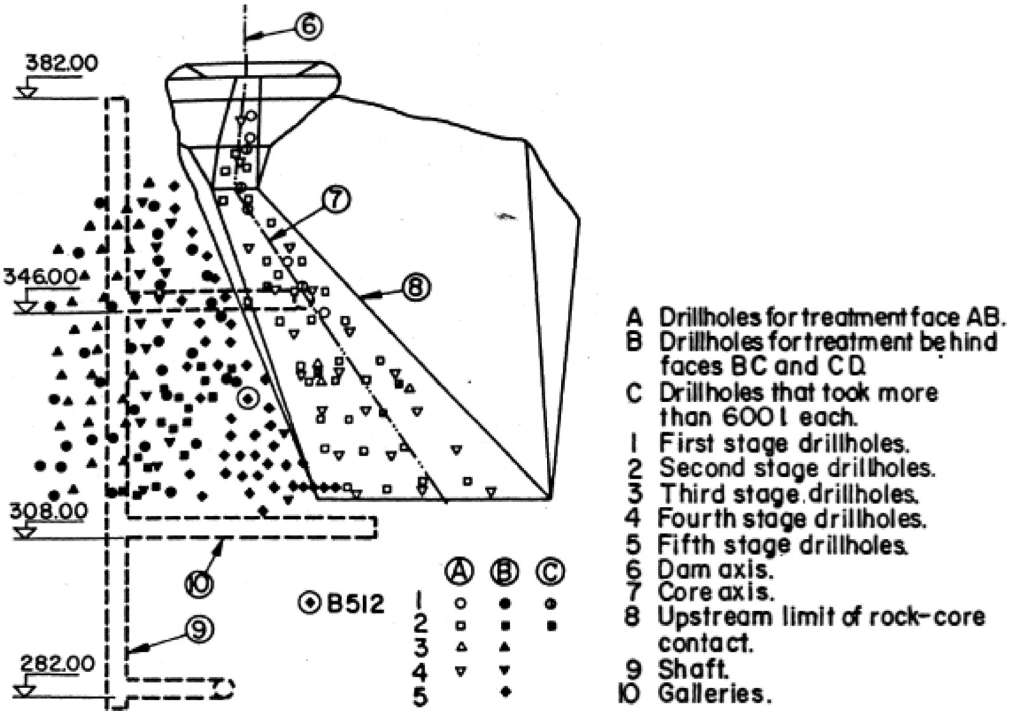


Figure 20. Drillholes for right abutment treatment.

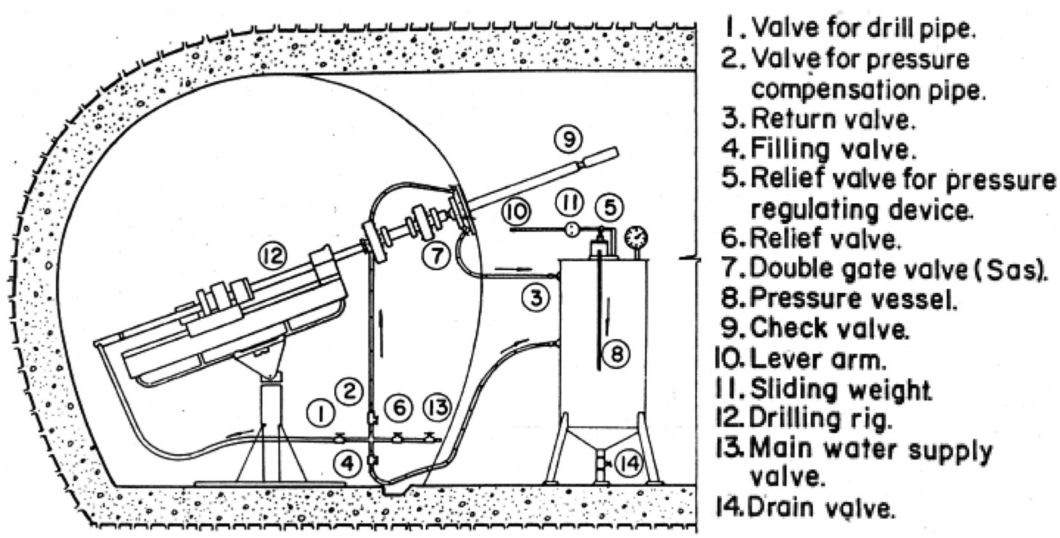


Figure 21. Gallery working station.

ing between the core and the rock face. An additional drainage system of the rock mass downstream of the BC rock-core contact face was installed. A second stage of grouting operations of the core-rock contact using a pressure controlled system. Grout takes up to 10,000 l per hole were recorded at El 340 m. Core samples containing hardened grout were recovered during the grouting program provided evidence of core cracking. Exploratory holes confirmed using impression packers showed open joints up to 2 cm wide.

The fissures, subvertical and parallel to the river valley, are attributed to stress relief in the recent geological past due to valley erosion and possibly widened by blasting during construction. This condition, considered critical to the dam soundness and its safety and remedial grouting and the drilling a new drainage system was programmed in several stages.

Stable mixes were used in conjunction with consistent volume-pressure relationships. A water-cement 0.67/1.00 ratio by weight was generally used (Deere, 1982)

Figure 22. Double gate valve (#7 in Figure 21).

with a relative cohesion 0.4 to 0.3 mm as defined by Lombardi (1969), to avoid the risk of hydraulic fracturing of the core. The volume of grouting resulted in about 53 m^3 at face AB, in 4 stages, and about 60,000 m^3 at face CD, in 5 stages. Total volume injected was more than 100 m^3, sealing the rock mass open joint system and the cracks existing in the core.

In the left bank the treatment behind the Power Station was performed from the original grouting gallery and extensions excavated at both extremes during 1984 and 1985. The amount of grout absorption was high in some areas, with more than 200 kg of grout per meter. The treatment was also extended to the left of Power Station. A total amount of 460 tons of cement was injected through a drilled length of 6700 m. Stable mixes were used with a 1:1 water cement ratio by weigh with 1 % of hydrated bentonite.

The potential problem of internal erosion of the core in contact with the rock in the foundation trench at the bottom of the valley led to the expansion of treatment work in successive stages. Between 1992 and 1994, the treated areas in both abutments extended to the valley.

On the right bank, a gallery of 100 m in length with an internal diameter of 3 m, lined with concrete, was excavated. Three rows of injection holes and a drainage curtain, located downstream, were made from the gallery to reinforce the existing system.

On the left bank a 622 m long gallery was built that descends from El 325 m to El 273, 25 m below the deepest foundation of the dam.

In February 1995, with the dam under the system of private concession it was decided to complete the treatment of foundations of the dam by building a section of central gallery joining both abutments. The gallery is 700 m long with a diameter of 3.4 m (Vardé, 1995).

The decision was based on having detected, through boreholes carried out from the galleries with double gate system and a pressure regulation device as mentioned hereinabove, areas of the core with a low degree of consistency, practically in the liquid limit. The excavation was conducted without the use of explosives by means of pilot advance drilling with a maximum unlined excavation length of 30 m. The injection curtain has three lines complemented by one line of drainage.

The serious conditions observed at El Chocón Dam are the result of the combination of various natural, design and construction features. The most important natural features are:

1) The flat-lying sedimentary weak sandstones and claystones beds having stress-relief joints in the bluffs and floor on the river valley;
2) The presence of subvertical and horizontal joints filled with salts, particularly gypsum;
3) The relative clean and neutral reservoir water with high dissolution capacity.

The significant design features comprise:

1) A layout leaving a rock nose between the spillway and the earth dam;
2) A steep right abutment face designed with a 2V:1H that was finally excavated to 4V:1H;
3) A core trench shape in the valley floor and the abutments that was difficult to blast in a weak rock;
4) A dispersive clay material used in the core;
5) Absence of a filter on the right abutment face B-C;
6) An inclined core with a relatively thin section and several changes in a slope, including a vertical upper section that tended to increase local arching effects.

The construction features worth mentioning are:

1) Reactivation and widening of right abutment joint and fissures due to blasting effects;
2) Grout mixes depending on takes but generally unstable and very lean;
3) The single-line grout curtain not guided throughout the foundation by geological evidence uncovered during excavation. The vertical primary grout holes could not seal the subvertical joints and were probable ineffective in sealing these potential water passages.

The experience and lessons acquired at El Chocón dam can be summarized as below:

- Drainage and grouting gallery: The construction of a drainage and grouting gallery under the foundation is generally very important for dams built on weak rocks with relatively low permeability and joints that, in some cases, are filled with soluble salts. Such a gallery allows monitoring the performance of grouting and drainage works during and after the first reservoir filling. In the case of El Chocón Dam, the availability of a bottom gallery under the entire dam, across the valley bottom and both abutments, would have provided a very efficient way to monitor the foundation behavior during operation. It would have also provided access to perform the remedial work needed;

- Grouting program and grout mixes: The remedial grouting program was guided by knowledge of the joint system gained through borehole investigations as well as gallery construction. It was very successful in sealing all the rock as evidenced by a small amount of grout takes in the core-rock contact zone in the final stage of grouting. Such grouting program has stopped an accelerated ageing process of El Chocón Dam and improved its safety;
- Instrumentation: The adequate instrumentation available and the efficient monitoring and analysis of foundation behavior performed at El Chocón Dam detected a condition of premature ageing and led to adequate action correcting a situation that had reduced the degree of safety of the structure and brought it back to acceptable levels.

4. Conclusions

The historical cases cited in this lecture clearly show that the project, construction and operation of large dams require high levels of competence in all the stages of the projects. The “unique work” character of a large dam is also evidenced in the sense that each project has individual characteristics that differs from other similar dams.

In the region of North Patagonia where there are weak rocks formations, the situations that could arise are even more demanding.

In Casa de Piedra, located in regional environments with clear evidence of limestone and gypsum formations, the investigation should pay attention to the evidence of water leaks or anomalies that imply the presence of soluble rocks and cavities. The use of microgravimetry is appropriate for the detection of cavities or discontinuities that traditional survey research may not detect.

In Alicura where important works are located on the left abutment, it was important to increase knowledge in the sector through early interventions in the work such as trenches, deep wells, exploration galleries. Due to their magnitude, these investigations require the presence of the Contractor due to the need for equipment and are intended to investigate geological features of little importance due to their size, but significant due to their influence on the safety of the works. The importance of a good drainage system and percolation controls during operation through galleries and drains is fundamental. The forecasts of investigations of this nature must be raised from the design for an adequate programming.

The case of El Chocón, where the situation becomes critical after ten years of normal operation, again shows the need for control and monitoring of the works throughout the useful life of the dam. The instrumentation system used and the permanent control carried out by the Owner, Hidronor, made it possible to detect unfavorable conditions and plan an adequate corrective action in time.

Proper management of large dams in all stages allows controlling contingencies and occurrences of unforeseen events, avoiding the risk of failure in some catastrophic cases.

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References

Aisiks, E., Giuliani, F., Tipping E., & Vardé, O. (1991a). Remedial works at El Chocón Dam. *Pro. 17th International Congress on Large Dams*, *Vienne*. ICOLD, 1211-1230.

Aisiks, E., Pallares J., Tipping, E., & Vardé, O. (1991b). Special techniques for corrective treament of right abutment at El Chocón Dam. *Proc. 17th International Congress on Large Dams*, *Vienne*. ICOLD, Q65 R 65.

Andersson, C., Jansson, S., & Vardé , O. (1985). Grouting and drainage system at the Alicura Hydroelectric project, Argentina. *Proc. 15th International Congress on Large Dams*, *Lausanne*. ICOLD, 701-719.

Deere, D. (1982). Cement-bentonite grouting for dams. *Proc. Conference on Grouting in Geotechnical Engineering, New Orleans*. ASCE, 279-300.

Deere, D., & Vardé, O. (1986). Engineering geological problems related to foundations and excavations in weak rocks. *Proc. 5th International Congress International Association of Engineering Geology*, *Buenos Aires*. IAEG/AAGI, 1-16.

Etcheon, A., & Speziale, J.F. (1989). Difficult foundations characterize the Casa de Piedra Dam. *International Water Power and Dam Construction*, 41(12).

Lombardi, G. (1985). The role of cohesion in cement grouting of rock. *Transactions 15th ICOLD Congress*, Lausanne, Switzerland.

Mariotti, G., Rondot, E., & Speziale, J.F. (1990). Le barrage de Casa de Piedra. Mise en evidence de condition de foundations dangereuses au cour de la construction. Adaptation du project. *Colloque International des Grands Ouvrages*, *Paris*. SGF, 77-85 (in French).

Pujol, A., & Andersson, C. (1985). Instrumentation of the Dam and left bank at the Alicura project. *Proc. 15th Congress on Large Dams*, *Laussane*. ICOLD, 387-416.

Vardé, O. (1986). Geotechnical investigation. Casa de Piedra dam. *Proc. 5th International Congress of Engineering Geology*, *Buenos Aires*. Balkema

Vardé, O. (1987). La mecánica de rocas débiles en la Argentina. *Proc. Conferencia en la Academia Nacional de Ciencias Exactas, Fisicas y Naturales*, *Buenos Aires*. National Academy of Sciences, 6-7 (In Spanish).

Vardé, O. (1988). La mecánica de Rocas Débiles. Tipo 2. Debilidad estructural. *Proc. I Simposio Argentino de Mecánica de Rocas*, *La Plata*. (in Spanish).

Vardé, O., Anagnosti P., de Mello, V.F., & Morgenstern, N. (1989). Embankment dams and dam foundations. *Proc. 7th Int. Conference on Soil Mechanics and Foundation Engineering*, *Rio de Janeiro*. Vol. 4, Balkema, 2177.

Vardé, O., Capdevila, E., & Reissian, E. (1990). Presa Casa de Piedra. Su comportamiento durante la primera etapa del llenado del embalse. *Proc. XI Congreso Argentino de Mecánica de Suelos e Ingeniería de Fundaciones*, *Mendoza*. SAIG (in Spanish).

Vardé, O. (1991). Presa El Chocón. Tareas de reparación. *Proc. IX Panamerican Conference on Soil Mechanics and Foundation Engineering*, *Viña del Mar*. Sochige (in Spanish).

Vardé, O. (1995). Presa El Chocón. Aspectos geológicos-geotécnicos vinculados al tratamiento de las fundaciones. *Actas de la Asociación Argentina de Geología Aplicada a la Ingeniería*, 9, 12-42 (in Spanish).