FOZ DO AREIA DAM

1. Introduction

Companhia Paranaense de Energia - Copel, is a fully integrated utility located in the Paraná State, southern part of Brazil. Foz do Areia, officially known as Gov. Bento Muhoz da Rocha Netto Hydroelectric Power Plant, is COPEL`s main generation plant.

In May 1973, COPEL was awarded the concession for the construction of the Areia Project. In late 1973, COPEL signed a contract with the consortium Milder Kaiser Engenharia S.A. (MKE) and Kaiser Engineers International Corporation (KEIC) for the engineering studies required for the definition of the Foz do Areia Hydroelectric Development. At the same time COPEL set up a Board of independent consultants, whose members were respected authorities in planning and construction of hydroelectric projects, for the purpose of following up the development of the project and/or revising the conclusions and recommendations made by MKE/KEIC. Board members were Engineers J. Barry Cooke, James Libby, Victor F. B. de Mello and Nelson L. de Sousa Pinto.

By May 1974, the various analyses performed had led to the choice of a site dose to the confluence of the Areia River. An extensive geological exploration program of this site was then performed together with a detailed topographic survey. In August 1974, based on these additional studies, the final layout was chosen.

The main portion of the civil works was contracted to "Companhia Brasileira de Projetos e Obras — CBPO", as a result of a public bidding held in 1976. This company started the works in October 1976 and completed them in October 1980; it moved out from the worksite in November 1980, after all work had been completed, except for second stage concrete of the fourth unit, the construction of the drainage tunnel inverts and the completion of the upper portion of the dam parapet. These remaining services were carried out by "Construtora Gemar Ltda.". The overall finish work was done by "Encipar — Engenharia Civil do Paraná Ltda."

The installation of the penstocks was performed by "Barefame Instalações Industriais Ltda.", and the main installation job was performed by "Tenenge Nacional de Engenharia S.A.

2. Location

The Foz do Areia Power Plant, is the most important project on the Iguaçu River since it is the largest and the upper-most with regard to location on the river. It controls and regulates the river discharge to the other powerplants downstream.

It is located in south of Brazil, in Paraná State, 5 km downstream from the confluence of the Iguaçu and Areia Rivers, and 240 km from the city of Curitiba, as shown on **Figure 1**.

Figure 1. Foz do Areia Dam – Location

2.1. General Description

The Foz do Areia Project was developed taking into account the geological and topographical features of the site which are typical of the region as well as the rainfall pattern. Schedule and costs were important factors in the development of the project. Taking into consideration the high degree of natural humidity of the soil, and the difficulties of handling this material due to the frequency of rains, it was considered that a concrete face rockfill dam offered more economical and constructional advantages than a conventional rockfill dam with impervious core.

Foz do Areia is the first Brazilian dam of this type and also the world's largest, in height (160 m), volume (14,000,000 m^3) and in total area of concrete face (138,000 m^2) at the time of its construction.

The project includes a concrete face rockfill dam, two diversion tunnels, a power intake, six power tunnels and a powerhouse on the right abutment and a spillway on the left abutment, and an SF-6 substation adjacent to the powerhouse (**Figure 2)**.

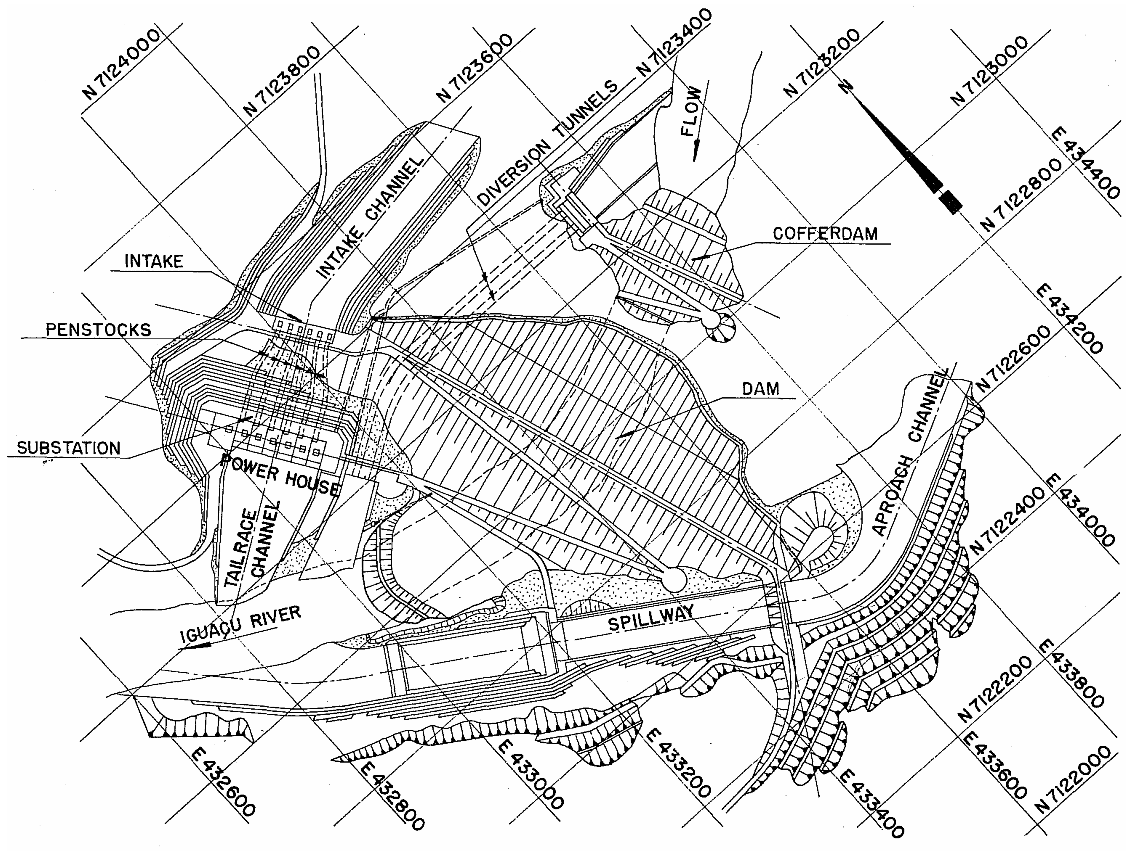


Figure 2. General Layout

3. Description of the Main Structures

3.1. Diversion Tunnels

The natural river channel, being quite narrow, dictated a single phase diversion scheme through two unlined tunnels, 12m diameter, with a horseshoe-shaped cross section, and each approximately 600 meter long.

The intake for the diversion tunnels is a concrete structure equipped with six steel slide gates, each measuring 4.1O m by 12.13 m which were closed by a mobile construction crane. A wheel type auxiliary gate was provided to be used in upstream slots in case of problems during slide gate closure.

3.2. Cofferdams

A saprolytic material, homogeneous cofferdam, about 45 meters high permitted the diversion of discharges up to 3,750 m^3/s through the tunnels, corresponding to 1:10 year flood.

The construction of the main dam was divided in two separate stages in such a way that the first stage constitutes a second cofferdam capable of diverting discharges of as much as 7,700 m^3/s through the tunnels, for a protection of 1:500 year frequency.

3.3. Dam

The dam is a compacted rockfill structure with an upstream concrete face. The crest is 828 m in lengh; maximum dam height over its foundation is 160 m and both face slopes are 1.4H:1V.

The dam is divided in different zones as shown on **Figure 3.**

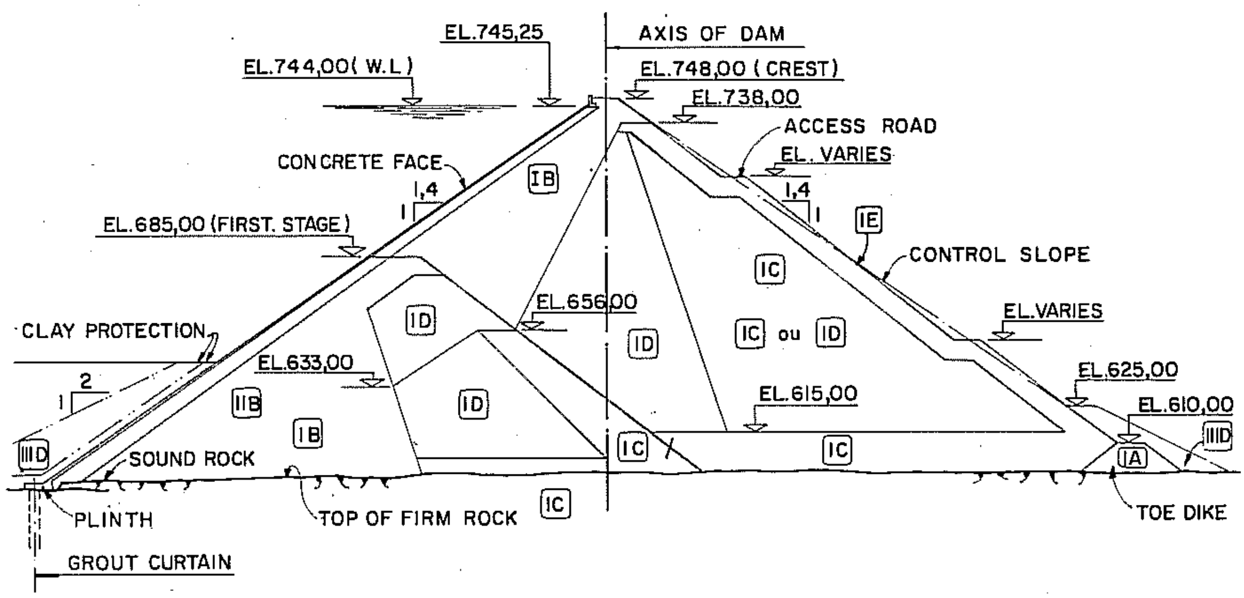


TABLE OF MATERIALS				
MATERIAL	CLASSIFICATION	ZONE	METHOD OF PLACEMENT	COMPACTION
ROCKFILL I	MASSIVE BASALT (upto 25% Basaltic breccia)	I A	DUMPED	———
		I B	COMPACTED IN 0,80m LAYERS.	FOUR PASSES OF VIBRATORY ROLLER (10ton.) 25% of water
		I C	COMPACTED IN 1,60m LAYERS	" "
	INTERCALATION OF MASSIVE BASALT AND BASALTIC BRECCIA	I D	COMPACTED IN 0,80m LAYERS	" "
	SELECTED ROCK - MIN. 0,80 m	I E	PLACED ROCK (Downstream face)	———
TRANSITION II	CRUSHED SOUND BASALT	II B	WELL GRADED - MAX-SIZE 6" COMPACTED IN LAYERS 0,40 m	LAYERS:- 4 PASSES OF VIB. ROLLER FACE :- 6 PASSES VIB. ROLLER(upslope)
EARTHFILL III	IMPERVIOUS SOIL	III D	MAXIMUM SIZE 3/4" COMPACTED IN 0,30m LAYERS	PNEUMATIC ROLLER OR CONSTRUCTION EQUIPMENT

Figure 3. Typical Dam Section

In the upstream zones placing of sound basalt with a 25% maximum percentage of breccia was required, compacted in 0.80 m layers. Sound basalt intermixed or interlayed with breccia, compacted in layers of 1.60 m, were used in the downstream portion of the dam.

In the entire dam construction water was applied, at a ratio of at least 25% of the volume of the rockfill material during the placing and compaction operations.

Compaction work was performed by vibratory rollers, each having a 10 ton static weight. In the upstream area, between the rockfill and the concrete face, there is a transition zone which supports the concrete slab. This zone is made up of special material, processed by crushing with a previously specified grading, and a maximum size of 6".

The transition material was placed in 0.40 m layers and it was compacted with the same vibratory rollers as the main rockfill. The upstream slope of the transition zone received a coating of asphalt emulsion and was covered with sprayed sand in order to prevent surface erosion and to facilitate the compaction of the face, which was done with six passes of the 10 ton vibratory roller, vibrating only in an upward direction.

The face slab is 0.80 m thick, at the base, and tapers linearly to 0.30 m at the top.

The critical point for this type of dam, as far as leakage is concerned, is the contact between the concrete face, and the plinth slab whose foundation rests on sound rock. Special care was taken in regards to this important detail of the project, in view of the fact that the plinth slab support is immovable, while the concrete face is subject to settlements which produce movements of the perimetric joint located between these two slabs. The solution adopted was a double waterstop system: a central one made of PVC and a bottom one made of copper.

A large volume of "IGAS" type mastic was also placed over the joint and protected by an elastic polyvinyl membrane. A large hollow neoprene cylinder was embedded in the IGAS along the perimetral joint.

3.4. Compensation Tunnel

In order to provide a minimum discharge of 100 m^3/s downstream from the dam, during the initial filling of the reservoir, or a possible need in the future, permanent compensation facilities were provided. Such facilities include: an unlined water intake tunnel with a rectangular cross-section 4.50 m high by 3.00 m wide and approximately 300 meter long an underground gate chamber with two sets of slide gates; an access tunnel and an open flow, concrete-lined discharge tunnel, approximately 250 m long ending in a flip-bucket.

3.5. Spillway

The spillway, located on the left abutment, was designed for a discharge of 11,000 m^3/s corresponding to the 1:10,000 year maximum project flood.

The spillway has a 70.60 m wide, 400 m long chute. The crest of the ogee section is at elevation 725.50 m and the flip-bucket type deflector is at elevation 625.50. The outflow is controlled by four tainter gates 14.50 m wide by 19.50 m high, hydraulically operated.

Ten stop-logs, each 2.00 m by 15.50 m, for maintenance use, are operated by a mono-rail system.

3.6. Power intake

The intake channel is 450 m long and 90 m wide, excavated in rock, with vertical slopes 10 m high and berms, 6 m wide.

The power intake structure is 70 m high by 108 m wide and permits a depletion of reservoir level as much as 47 m. The structure is provided with six wheel-type gates measuring 7.40 x 7.40 m. An additional gate, interchangeable in the slots of any intake, serves as an auxiliary maintenance gate. The service gates are powered by hydraulic hoists with a 175 ton capacity, and a 135 ton capacity gantry crane is used for moving the auxiliary gate and to perform maintenance work on the trashracks.

3.7. Power Tunnels

Six power tunnels convey the water down to the six turbines. These tunnels, lined with unreinforced concrete, have a 7.40 m diameter. The 70 meters of the lower horizontal section adjacent to the powerhouse is steel lined. Plate thickness varies from 1 3/4" to a maximum of 2". In this stretch, the internal diameter is 7 m The total length of the tunnel is approximately 220 m.

3.8. Powerhouse

The powerhouse is of .a semi-outdoor type, with four generating units each of 418.50 MW operating in its first stage. For the second stage, blockouts were left for the installation of two additional generating units, with the same characteristics, for a total installed capacity of 2,511 MW.

Erection loads were moved with an outdoor gantry crane, 800 ton capacity and a travelling bridge crane, inside the powerhouse, with a 50 ton capacity.

The turbinated water is returned to Iguaçu River through a tailrace channel approxirnately 250 m long.

About 150,000 m^3 of concrete was used in the construction of the powerhouse.

3.9. SF-6 Substation

The 500 kV switchyard is located immediately upstream of the powerhouse, at approximately elevation 623. The substation is a compact indoor SF_6 (Sulphur hexafluoride) type.

4. Geological and Geotechnical Features

The site has geology of basaltic rocks and is located in a deep valley, showing the particular step-type topography which reflects the superposition of basaltic flows. Both these factors considerably affected the design and construction activities.

Geological investigations started in 1973 and went on almost uninterruptedly during construction of the diversion works, in 1975, and the beginning of the main civil work, in 1976. During this period, nearly 7,000 m of rotary drilling were finished, (116 meters with "integral sampling"). Other work included test pits, geophysical prospection, as well as field and laboratory studies for the characterization of geomechanical features of local rocks and soil. An adit was excavated in the intake area to investigate a small fault found at this location and was subsequently incorporated in the drainage system.

The region is made up of medium to thick basaltic flows averaging 25 to 55 m. **Figure 4** shows a simplified geological section projected along the plinth, in the river bed and at the left abutment which gives an idea of the general sequence of basaltic flows, as well as location of permeable zones, flow contacts and stress relief fractures found in the river bed foundation. It also shows the overburden cover and the topography of the underlying hard rock surface.

The local sequence of rocks has a marked predominance of dense basalts, making up approximately 70% of the total volume. The remaining 30% are made up mostly of basaltic breccia, since amygdaloidal basalts are less common in this area. **Table 1** gives an idea of the average geomechanical properties of those rocks.

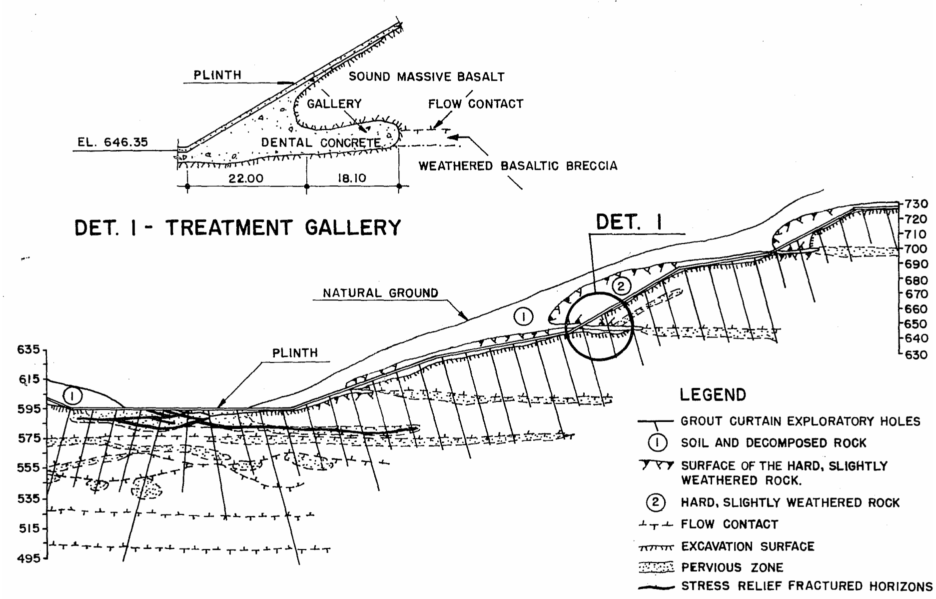


Figure 4. Geological Section along the plinth riverbed and left abutment

TABLE 1 - GEOMECHANICAL PROPERTIES OF LOCAL ROCKS			
PROPERTY (AVERAGE VALUES)		TYPE OF ROCK	
		DENSE BASALT	BASALTIC BRECCIA
DRY SPECIFIC WEIGHT (g/cm^3)		2.80	2.30
POROSITY (%)		1.30	11.80
COMPRESSIVE STRENGTH (Kg/cm^2)	DRY	2.380	380
	SATURATED	1.930	250
MODULUS OF ELASTICITY (Kg/cm^2)	DRY	680.000	260.000
	SATURATED	660.000	240.000
SOUNDNESS TEST - SODIUM SULPHATE (% OF LOSSES)	COARSE AGGREGATE	2	50
	FINE AGGREGATE	5	35
LOS ANGELES ABRASION TEST (TYPE E GRADING, 1.000 REVOLUTIONS) (%)		11	20

4.1. Soil and Weathered Rock Overburden

The soil and weathered rock mantle is unusually thick in this area and quite variable, ranging from nil, at the steeper slopes, to about 40 meters in flatter areas. Average thickness was found to be about 11.5 meters. This cover of residual and coluvial soils, plus thick layers of soft to hard saprolites raised considerable difficulties for the investigation program and for the design, excavation and treatment of the plinth foundation. A feature that raised further problems was the occurrence of deeply weathered zones and seams along flow contacts and sub-horizontal fractures.

The overburden was entirely removed and the resulting material employed in cofferdams, fills for leveling and protection of the perimetral joint at the lower portions of the dam. Most of it was dumped in spoil areas. The waste of this large volume of soil, instead of its use in another type of dam, was a decision based on a detailed evaluation of local climatic conditions, construction schedule and cost estimates.

Some soil cuts of considerable height were required in certain areas, particularly in the spillway approach channel. Slopes 1V:1.5H, with 6 meter wide berms at each 15 meters in height were used, with very good results. Two small slides occurred and were controlled by means of drainage and rockfill toes.

4.2. Influence of Geology on the Excavation Work

With more than 14,000,000 m^3 of rock excavation, the other activities of Foz do Areia Project were considerably affected by this excavation and by the dose relationship that exists between geology, blasting results and stability of high rock cuts.

Main geological structures of basaltic rocks are: sub-horizontal flow contacts, sub-vertical fractures of dense basalts, sub-horizontal discontinuities and stress relief fracture zones. Small intrusive dikes and two minor faults were also found at the site.

Concerning excavations, the main geological feature was the sub-vertical, columnar jointing of dense basalts, which raised considerable difficulties for the execution of vertical cuts.

The excavation for the powerhouse attained 200 meter maximum height, (10 meter high vertical cuts separated by 6.0 meter wide berms), and represents one of the highest rock cuts made in

Brazil. Local damage caused by blasting and chiefly related to the sub-vertical jointing of dense basalts was treated with anchor bars and shotcrete.

4.3. Foundation Treatment

The power intake, spillway and related wing walls, i.e., the high concrete structures, required deep excavations and, consequently, layed on foundations of sound rock, requiring no foundation treatment, except for normal cement grouting.

The plinth, however, being essentially a near surface structure, required more care and represented one of the most peculiar features of Foz do Areia Dam.

For concrete face rockfill dams built in steeply sloping valleys, the plinth has been frequently designed with varied geometries, sometimes as nearly vertical walls anchored against the rock, to reduce the amount of excavation. Nevertheless, this solution sometimes increases the thickness of rockfill immediately downstream of the perimetral joint, greatly enlarging the risk of excessive settlements at this critical zone. Also a vertically shaped plinth renders the grouting works quite difficult, specially because grout curtains in basalts have to be essentially sub-vertical to treat the highly predominant sub-horizontal pervious zones.

On this account and also to help the removal of sub- horizontal deeply weathered zones, the plinth, in Foz do Areia, was designed with the shape of a road, sometimes requiring considerably more excavation in the steeper abutments, having horizontal generatrix normal to the perimetric joint. Although this solution increased substantially the volume of rock excavation, it made possible a foundation located mainly on sound rock.

In some places, the occurrence of deeply weathered zones, especially along decomposed flow contacts, required other forms of foundation treatment, such as the unusually deep excavation followed by a 18 meter long adit shown in **Figure 4**. Concrete walls were also built in an upstream — downstream direction along the sub- horizontal decomposed seams, to protect the transition material. Filters were used between the weathered material and the rockfill from the end of the concrete walls down to the dam axis.

An extensive and possibly somewhat conservative grouting was carried out along the plinth and concrete structures. **Figure 4** shows the location of exploratory holes drilled along the plinth, giving an idea of maximum treatment depths, as well as the relation of grout curtain and pervious zones.

Grouting, in concrete-faced dams, is carried on along the plinth, outside the rockfill body, which is a particularly advantageous feature concerning construction schedule. At Foz do Areia, due to the height of the dam and, to a certain extent, to the pioneer character of the dam, added to the occurrence of some highly permeable and weathered layers found at the river bed foundation and portions of the abutments, conservative procedures were employed.

A single line grout curtain was used, with two rows of shallow consolidation holes immediately upstream and downstream of this line. Grout curtain depth was defined on geological basis, but usually kept around a third of the dam height. The more careful treatment was normally limited to the first 20 to 30 meters of depth, sometimes increased to attain sub-horizontal pervious zones within the abutments.

The grout works were to a certain extent complicated by the occurrence of quite pervious but extensively weathered stress-relief fracture zones found at the river bed. In these areas a grouting method based on the careful use of fairly high pressures (up to 5.0 kg/cm^2 immediately below the plinth foundation) was employed, with adequate results.

5. Characteristics of the Drainage Basin

The Iguaçu River, whose name in its original "T Guarani" language means "big water", is one of the major affluents of the Paraná River. It is originated in the vicinity of the City of Curitiba, in the State of Paraná, 875 m above sea-level and after a course of 1,100 km it flows into the Paraná River, at an altitude of 75 meters, where the breathtaking beautiful Iguaçu Falls are formed on the boundary line between Brazil and Argentina**.**

With a hydrographic basin of 67,000 km^2 the Iguaçu River has several natural falls. As the river shows an 800 m difference in level between its source and its mouth, it favors the implementation of dams with powerplants that are economical when compared with other Brazilian projects of a similar type.

The basin's relief is quite irregular. The river head is located in the foothills of the Serra do Mar and the river follows a northeast-southwest course as far as Porto Vitória, flowing along with a flat slope over crystalline rocks, with a small step at Porto Amazonas (Salto Caiacanga) down to Porto Vitória, where it cuts through the Serra Geral. Here, the river enters the basaltic region and takes an East-West course which determines the general direction of the basin, down to the river's fall.

The Iguaçu River enters the basaltic region at an altitude of 740 m and 630 km downstream it flows into the Paraná River at an altitude of 75 meters. Down to the confluence of the Jordão River, the Iguaçu slopes are rather abrupt. It is an uniform canyon type valley and the river-banks are relatively steep. From the confluence of the Jordão River down, it slopes less. The valley is not so steep and the banks are undulating and the river drops in a series of rapids such as Salto Santiago, Salto Osório, Caxias and Santa Maria (the Iguaçu Falls).

Climate conditions in Brazil's South Region, where the Iguaçu River basin is located, are usually determined by the relative movement of the air masses circulating over the area. Such movements are affected by the relative position of the anticyclones located over the continent. These centers vary both in position and intensity along the year, in an irregular manner; consequently, the region's climate characteristics, such as temperature, pressure, air humidity and precipitation are erratically distributed through-out the year.

The average annual temperature is approximately 20°C in Foz do Iguaçu and 16°C in União da Vitória. The absolute maximum and minimum recorded are + 40°C and -5°C respectively.

Rainfall is by far the basin's best known hydrological parameter, thanks to the existence of 350 stations, some of which have been keeping records for the last 90 years. Average annual rainfall over the basin is on the order of 1,400 mm/year. Rainfall distribution along the year is quite irregular and there is no well-defined dry season.

The Foz do Areia reservoir has a maximum water level at elevation 744 m, a total volume of 6,066 x 10^6 m and 167 km^2 of inundated area.

6. Main Features of the Foz do Areia Project

Foz do Areia dam was a benchmark in many aspects concerned with concrete-faced rockfill dams. It was the highest dam (160 m), the largest in rockfill volume (14,000,000 m^3), by the time of the construction, and it presents a total area of concrete face of 138,000 m^2.

Treatment of the downstream slope was executed in such a way as to ensure a face in accordance with the project's theoretical requirements, with minimal deviations. This gives an exceptional pleasing appearance.

It's also important to point out that was the first CFRD with a big reservoir, about 6 million cubic meters of total volume, without outlets provision for emptying the reservoir. The reservoir filling started in 1980 and the performance of the dam may be considered excellent according to the predicted design.

The spillway chute was provided with devices for aeration. This is a great step forward in the prevention of cavitation and, in this respect, Foz do Areia spillway is the first large size structure with air entrainment devices in Brazil and on the American continent.

The selection of a 500 kV, SF insulated substation in the powerhouse layout was of considerable help in solving space-related problems as required for a conventional type switchyard.

7. Construction

7.1. River Control

The river was diverted during a period of relatively high water, with flows of approximately 800 m^3/s. The final closure during diversion, was executed having a difference in water level of 6 meters. it was necessary to build simultaneously two rockfill dikes which were part of the upstream and downstream cofferdams.

The concept of the dam permitted to divert the river with relative lower cofferdams with a protection of 1:10 year flood. Then the first phase of the dam is built to act as a second cofferdam offering protection against 1:500 year floods. The difference in permeability of the transition material, located under the slab, with respect to the main rockfill, allowed to restrict the water flow through the first stage rockfill safely.

Once the dam and all the other hydraulic structures had been completed, the diversion tunnels were closed, by lowering the slide gates, by use of two mobile cranes. The wheel gate which had been designed to discharge flow downstream was not used in view that the Salto Santiago Reservoir, located downstream from the Foz do Areia Project, was able to maintain the flows for the Salto Osório Powerplant. The compensation tunnel which permits discharges up to 100 m^3/s was operated only for tests for the same reason. After the reservoir had surpassed the spilling elevation of the spillway, the reservoir level was controlled by means of this discharging facility.

The sequence of the various phases of diversion and control are shown in **Figure 5**.

Phases were as follows:

- Ist phase — Preparation for diversion of the river through tunnels — After completion of the works related with the construction of the tunnels, intake structure and the closure bridge, dumping of rockfill was initiated in dikes nº 2 and 3.
- 2nd phase — Diversion through the tunnels — Dikes 1 and 4 were blasted. Simultaneously dumping of rock in dikes 2 and 3 was accelerated to divert most of the river through the tunnels.
- 3rd phase — Construction of cofferdams — dumping of impervious material was executed on the outer slopes of dikes 2 and 3 and after pumping the water between dikes the upstream cofferdam was completed. The 45 m high cofferdam offered protection against floods. up to 3,750 m^3/s. Then the first stage of the dam was completed up to elevation 685, ensuring protection for floods up to 7,700 m^3/s (1:500 years).
- 4th stage — Completion of dam (rockfill and face slab), closing of the diversion tunnels and filling of the reservoir.

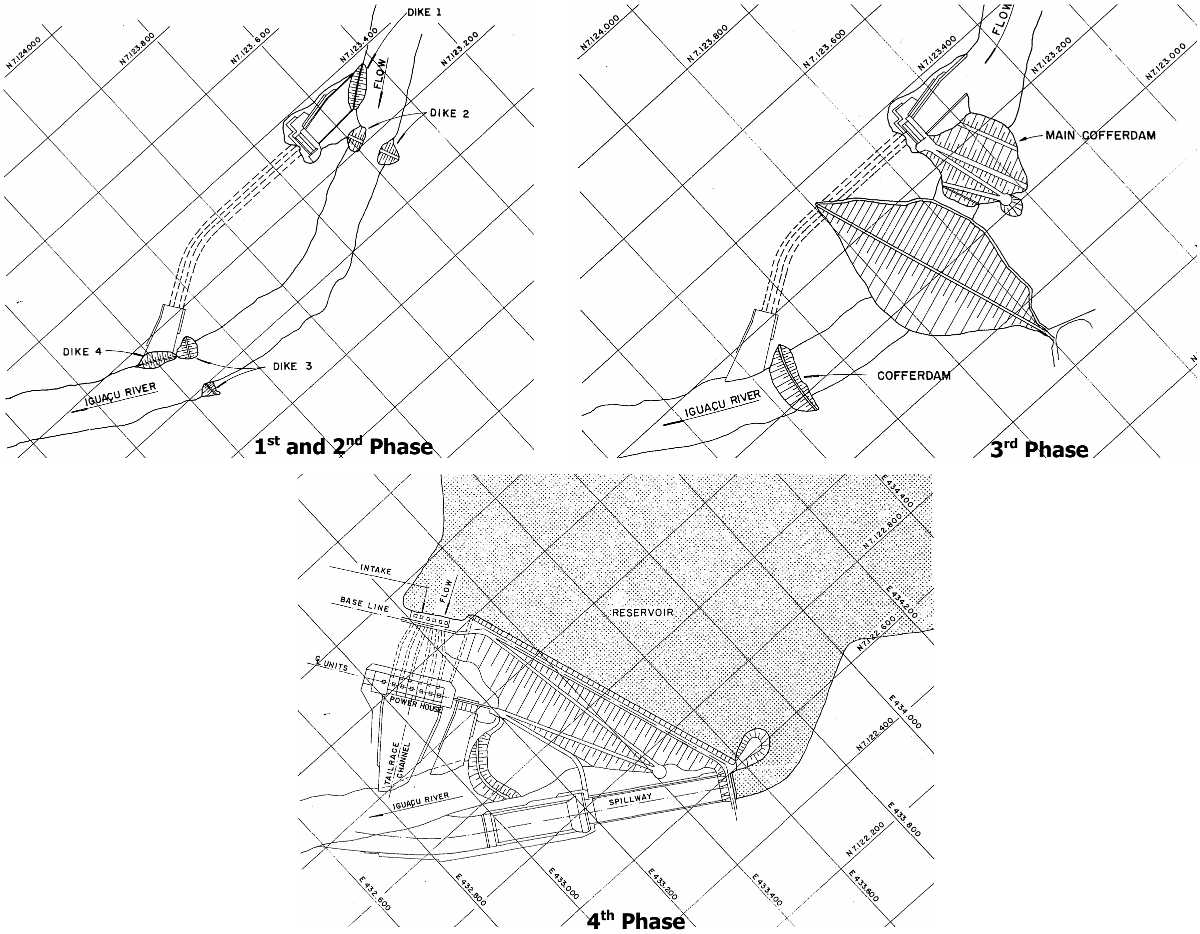


Figure 5. Diversion Phases

8. Execution of Civil Works

8.1. Dam

Foundations for the plinth, which is the connecting slab between the abutments and the dam slab, were laid over sound, groutable rock, similar to the foundations usually specified for a concrete structure support. The abutments were cleared of all overburden, down to the rock, especially over the first 30 meters, from the main slab.

Contacts between basalt and breccia were carefully cleaned and treated with dental concrete over the first 30 meters starting from the slab, and the remaining zones of weathered material, potentially erodible, were treated with filters.

Grading of the rockfill, mostly from structure excavations, proved to be much more uniform than those obtained for other dams. **Figure 6** shows the grading of the Cethana, Alto Anchicaya and Foz do Areia dams. The Foz do Areia void ratio of the rockfill is larger than those from the other two dams.

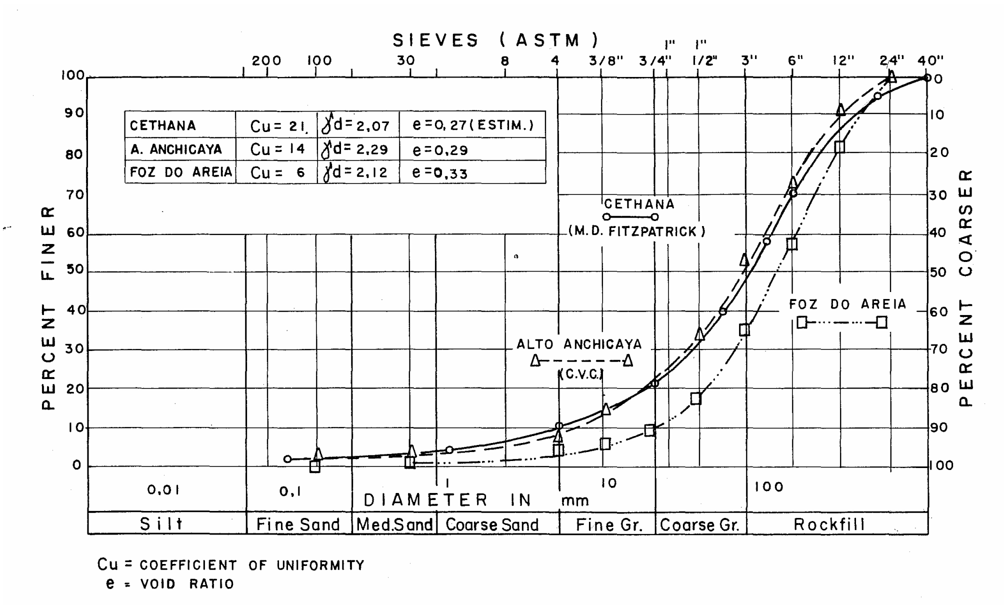
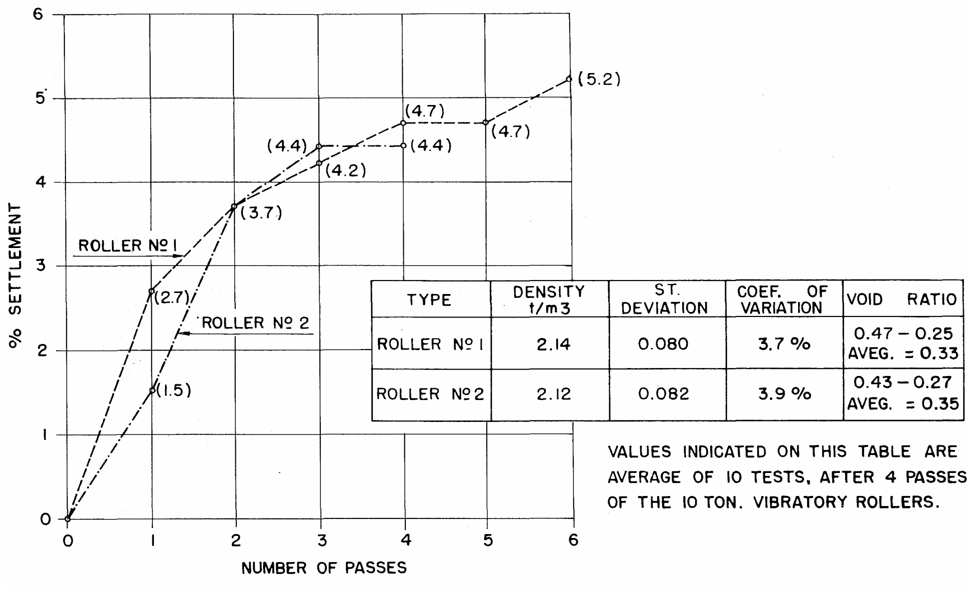


Figure 6. Main Rockfill - Grading Curves

To compensate this fact, the transition material zone under the slab (Zone IIB) was enlarged and the grading was improved by increasing the amount of fines, reducing also the maximum size. The objective was to obtain a material firm and dense for slab support. The rockfill consists of basalt and breccia, distributed in different zones, as shown on **Figure 3**.

Compaction was done by four passes of a 10 ton vibratory roller, with water being added; at a rate of 250 l/s. Experience with other types of rock shows that when water is added, the resistance of the sharp edges of the material is reduced, thus insuring a better density and stability of the fill with time. Number of passes *vs* settlement was investigated at the time the project was started, with results as shown on **Figure 7**.



TYPE	DENSITY t/m3	ST. DEVIATION	COEF. OF VARIATION	VOID RATIO
ROLLER Nº 1	2.14	0.080	3.7 %	0.47 – 0.25 AVEG. = 0.33
ROLLER Nº 2	2.12	0.082	3.9 %	0.43 – 0.27 AVEG. = 0.35

Figure 7. Compaction of Basalt Rockfill

The transition material located between the face slab and the main rockfill was compacted in two directions: horizontally, with four passes of a 10 ton vibratory roller and in the direction of the slope, moving the vibratory roller up and down the slope by a crane placed on the rockfill crest.

The compaction technique for the face had to be adjusted to the type of material available, depending of the content and type of fines of the rockfill. The methods used in other dams were adapted to Foz do Areia conditions.

A procedure which gave good results was as follows:

a. Level the face through manual work, by trimming the higher spots and filling in the lower ones.

b. Compact the face with two passes of a 10 t roller, without vibrating but adding water. It was noticed that when vibration was applied in the upward direction, the transition material got loose due to lack of cohesion.

c. Apply curing asphalt emulsion, at the rate of approximately 4l/m^2. Immediately following this, natural sand was applied by a shotcrete machine, in order to produce a more uniform face. It was noticed that the sand helps to break up the emulsion. Depending on the day, curing time varied from 4 to 12 hours.

d. Subsequently, the face was compacted with four passes of a 10 ton roller, with maximum vibration. When this was done at Foz do Areia, the result obtained was a smooth surface, relatively stable during the placing of reinforcement.

During rainy periods the asphalt ensures protection against erosion caused by the accumulation of water as experienced in other dams.

Placing of rockfill was carried out by using ramps inside the rockfill itself, except for the first 30 meters from the concrete slab. Ramps with a 15° to 18° slope were planned in order to speed up rockfill placing and reduce hauling distances between the excavation areas and the dam.

Rockfill production rates were on the average higher than 500,000 m^3/month, with a maximum of 670,000 m^3/month during August 1977. **Figure 8** shows recorded production rates, as compared with the contractual schedule.

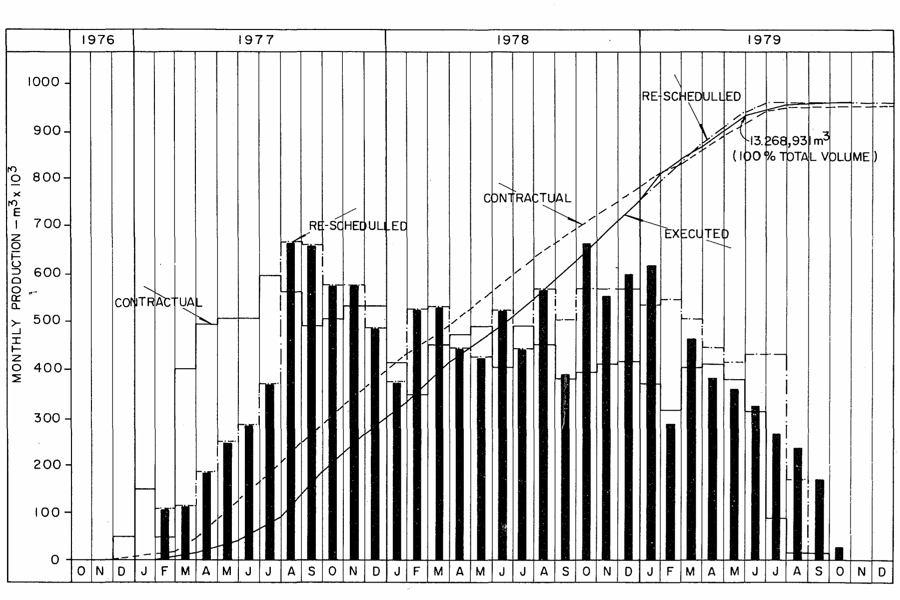


Figure 8. Dam Construction Program

The sequence of rockfill placing was planned to comply with the main project requirements: first stage construction, from elevation 595 to elevation 685, and optimization of transportation distances between the excavations of the main structures and the site of placing rockfill inside the dam. Work on the downstream face was carefully performed, with the following procedure being used: rocks with a diameter 0.80 m or more were separated from the rockfill and pushed by a dozer in such a way that the largest face of the rock would be in line with the 1.4H:1V downstream slope. Each rock was raised by the bulldozer blade and wedged in by smaller rocks in order to maintain the downstream face plane. The alignment was controlled by means of a triangular device to keep the alignment within tolerances 0.15 m from the theoretical line. General appearance of the face is exceptionally good.

Thickness of the dam concrete slab follows the formula:

$T= 0,30 + 0,00357 H$ (m),

where "H" equals the depth measured from the top of the dam and "T" represents the thickness of the slab in the section under consideration.

The slab acts as a membrane which follows the rockfill deformations, both at the construction stage and during the application of the hydrostatic load of the reservoir.

The slab reinforcement is 0,4% of its cross-section in both directions (horizontal and slope directions).

Experience with dams previously built has eliminated horizontal contraction joints which usually presented leakage problems. There is a minimum number of horizontal construction joints in modern dams and there are only eventual programmed construction joints as is the case of elevation 680, which separates the first and second stage phases at Foz do Areia Dam. Vertical joints located 16 meters apart are treated as shown on **Figure 9**. They are hard joints, without any filler, to reduce the total movement of the perimetral joint. The perimetral joints between the plinth, founded on rock and the slab which rests on the rockfill, are the most important since they open and off-set. At Foz do Areia, special care was given to the treatment of the perimetral joint, as shown on **Figure 9**.

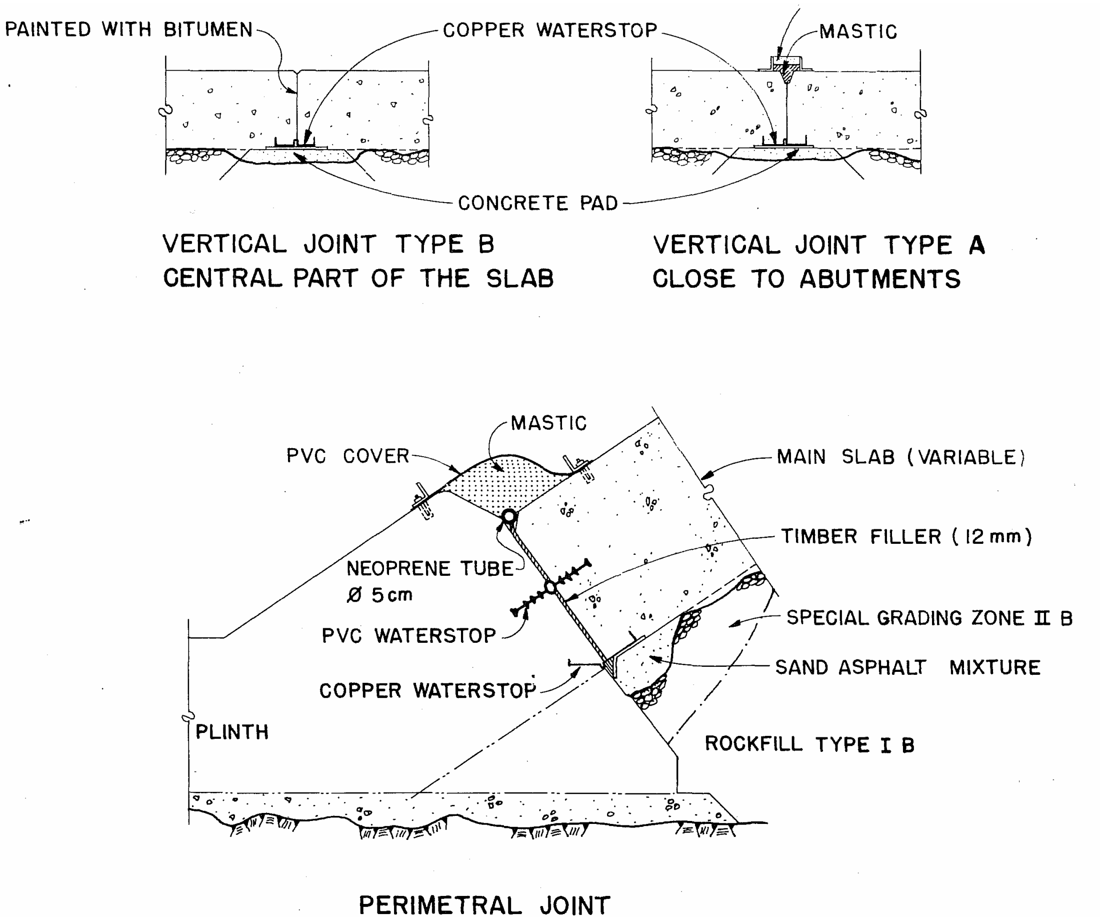


Figure 9. Joint Details

The main slab was built in two stages (**Figure 10**):

- First the filler slabs were built. Those are triangular slabs located next to the plinth, particularly in the immediate vicinity of the perimetral waterstops. Although in other projects those slabs had been built with slip forms, at Foz do Areia a system of fixed forms was adopted, with good results.
- Secondly, the main slab was built with a slip form which, prior to being approved was submitted to rigorous tests in order to ensure that deformations would be within specified tolerances.

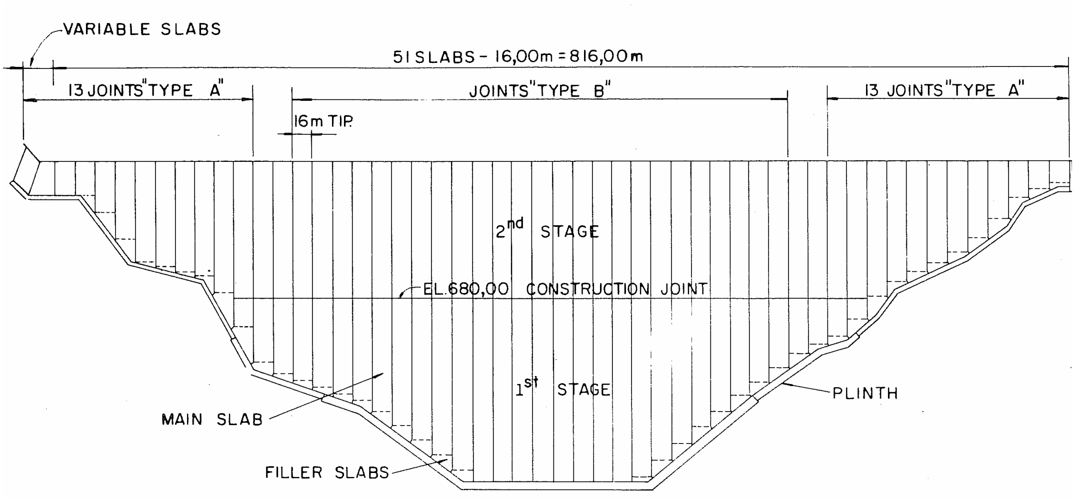


Figure 10. General view of the upstream face

The slabs are 16 m wide, between longitudinal joints, and the slip form was designed to cover this span; it had beams resting on the side rail tracks which served as guides for two double action hydraulic jacks, responsible for the upward movement.

Concrete was placed by truck-mixers and by cableway, to a hopper located on the crest of the dam and then concrete was poured by chutes down to the form. Slump required was between 8 and 10 cm inside the chute, to prevent segregation. Provisions to prevent excessive slump due to ram were taken. Also, sometimes it was required to cover the placing chutes to avoid losses of slump caused by sun rays. The concrete was compacted by vibrators which were always placed ahead of the form, thus avoiding any vibration under the sliding screed, as such operation brings undesirable floating of the form. The production rates attained were on the order of 62 m^3/h (maximum). The average slip rate was higher than 1.50 m/h and the maximum rate obtained was on the order of 6 m/h.

Instruments installed inside the dam are settlement cells, located at the center of the dam in two lines 100m apart, for the purpose of estimating settlements, deformation moduli, construction effects and future effects caused by the filling of the reservoir (**Figure 11**).

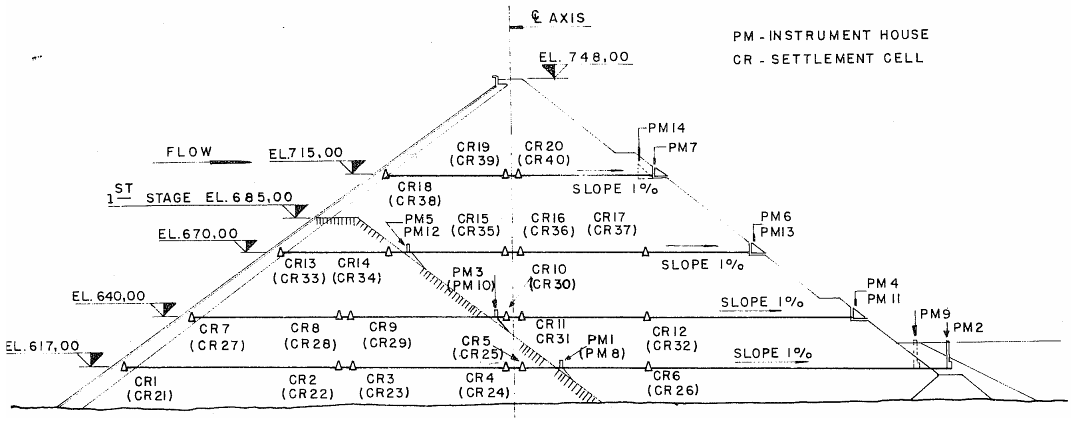


Figure 11. Dam Typical Section – Instrumentation within the rockfill maximum section

Additionally, settlement and displacement marks, of the conventional type were installed both on the upstream and downstream face, to observe the constructional and filling effects of the reservoir.

Seventy-four instruments were installed in the dam slab, located at different heights of the slab, as follows:

- 30 *deformation gauges*, type Kyowa CS-25F, located next to the abutments and on the central slab. Gauges permit to measure unit deformations (strain) in two right angle directions.
- 12 *reinforcement stress gauges*, Kyowa type, model RF-32C, located next to the abutments in potential tension areas. These instruments permit to measure stresses in the reinforcement.
- 18 *jointmeters*, Kyowa type, model CJ-60G, located on the abutment joints, to measure three types of movements: joint separation, settlement perpendicular to the slab and shear parallel to the perimetral joint.

Fourteen electric thermometers, of the LCEC type were also installed every 10 meters high in the central slab, for the purpose of measuring temperature variations.

A structure for measuring percolation through foundations and joints, was built at the downstream toe of the dam.

8.2 Concreting Works

The compact arrangement of Foz do Areia Project, permitted concrete placing work to be carried out simultaneously on several fronts using conventional equipment assigned to each area.

Distribution of concrete to the work-fronts, was accomplished by two cableway systems. The power intake structure and dam area were served by the upstream cableway. The powerhouse and spillway areas were covered by the downstream cableway.

Aggregates required for the preparation of the various types of concrete came from the excavations in sound basalt at the project site. The basalt was crushed by a crushing plant where four piles of coarse aggregate were produced (6″ - 3″; 3″- 11/2″; 11/2″ - 3/4″ and 3/4″ - 3/16″), as well as a pile of artificial sand. Additionally, natural sand was stock-piled and used for the purpose of improving workability of the concrete. The crushing plant had a 250 m^3/h capacity.

8.3 Quality Control

Two basic activities were emphasized in quality control of the project: control of rockfill and control of concrete. Special attention was given to those features due to their importance in the project. Since there were no permanent structures built of soil, only routine control was given to cofferdams and the impervious cover of the face slab.

8.4 Control of Rockfill

For the type of dam which was adopted at Foz do Areia – a oncrete face rockfill — it is extremely important to obtain a good quality rockfill next to the face slab to ensure a high compressibility modulus and consequently, a low rate of deformations, not affecting the performance of the slab.

Basic controls performed were as follows:

Field

- thickness of layers, controlled by wooden gauges, located at previously levelled points;
- number of passes of the 10 ton vibratory roller, in accordance with the type of rockfill. Also, the frequency of the vibratory compactors was determined by a tachometer;
- visual control of the percentage of breccia in zones where sound basalt was specified, considering that a breccia contamination of up to 25% of the total volume was acceptable;
- amount of water during the spreading of the rockfill and compaction with a vibratory roller (approximately 250 l/m^3);
- in addition, periodic density tests were carried out "in situ", with the help of steel rings with 1 and 2 m diameters, excavating to a depth equal to the specified layer.

Samples from those tests were taken to the laboratory for the purpose of determining grading, humidity content and typical features of the rock, such as: specific gravity, dry unit weight, absorption, void ratios, porosity, Los Angeles abrasion, curvature and uniformity coefficients.

Figure 12 and Tables 2 and 3 show average statistical results of checks made during the construction period.

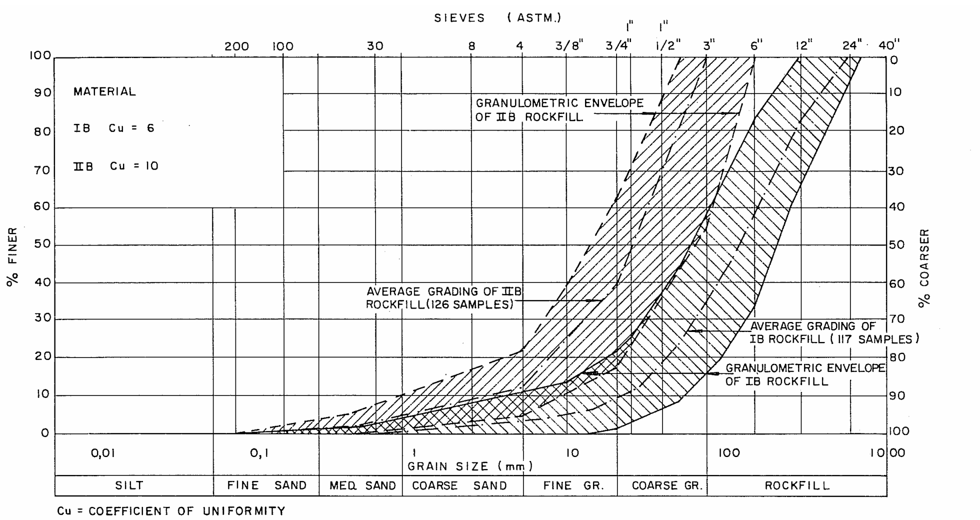


Figure 12. Main Rockfill (IB) and transition material (IIB) grading curves as recorded in the field

TABLE 2 - ROCKFILL TYPE I B - PHYSICAL PROPERTIES					
TEST	NUMBER OF SAMPLES	MAXIMUM VALUE	MINIMUM VALUE	AVERAGE VALUE	REMARKS
SPECIFIC GRAVITY	101	2878	2620	2801	UNIT kg/m3
MOISTURE	101	7,10	0,70	1,80	PERCENT
ABSORTION	101	3,36	0,16	0,98	PERCENT
DENSITY	101	2333	1806	2118	UNIT kg/m3
VOID RATIO	101	0,557	0,183	0,327	
POROSITY	101	36,0	15,0	25,0	PERCENT
LOS ANGELES ABRASION	49	15,9	6,0	10,0	PERCENT - 1000 REVOLUTIONS GRADING TYPE E
COEFFICIENT OF CURVATURE	107	3,50	0,8	1,3	NOTE: THIS TABLE SUMMARIZES PHYSICAL PROPERTIES OF BASALT COMBINED WITH BRECCIA, VARYING BETWEEN 0-25% PER VOLUME
COEFFICIENT OF UNIFORMITY	107	23,60	1,40	7,9	

TABLE 3 - ROCKFILL TYPE II B - PHYSICAL PROPERTIES					
TEST	NUMBER OF SAMPLES	MAXIMUM VALUE	MINIMUM VALUE	AVERAGE VALUE	REMARKS
SPECIFIC GRAVITY	128	2909	2528	2748	UNIT kg/m3
MOISTURE	128	5,95	1,42	3,21	PERCENT
ABSORTION	128	2,32	0,26	1,23	PERCENT
DENSITY	128	2294	1865	2100	UNIT kg/m3
VOID RATIO	128	0,441	0,154	0,312	
POROSITY	128	31,0	13,0	24,0	PERCENT
LOS ANGELES ABRASION	113	19,0	7,0	11,2	PERCENT - 1000 REVOLUTIONS GRADING TYPE E
COEFFICIENT OF CURVATURE	128	5,70	0,20	1,7	
COEFFICIENT OF UNIFORMITY	128	47,5	0,8	11,3	NOTE: PHYSICAL PROPERTIES OF THIS TABLE ARE FOR DIFFERENT KINDS OF BASALT.

8.5 Control of concrete

Concrete control at Foz do Areia was performed through the concrete and materials laboratory which supervised each one of the component elements of the concrete. Also it was determined some of the physical properties of concrete required for an adequate interpretation of the dam's instruments. This control can be summarized as follows:

- Cement: After testing various types of cement and in view of the difficulty of using a constant amount of pozzolanic material, it was decided that pozzolanic cement would be used and controlled through physical and chemical tests.
- Fine Aggregate: Two types of sand were used in the project's concrete: artificial sand obtained from basalt crushing, and natural sand taken from deposits in the vicinity of União da Vitória and Ponta Grossa. Fine aggregates were continually tested for grading control, percentage of organic material of natural sands, specific gravity, loose and dense unit weights and reaction to sodium sulphate, besides other routine tests as shown on Tables 4 and 5.
- Coarse Aggregates: The coarse aggregates were continually tested in the stockpiles, from 6" - 3"; 3" - 1 1/2"; 1 1/2" - 3/4"; 3/4" to nº 8, and results were statistically analysed.
- Concrete mixes: Two methods were applied for concrete mixes. One of them was the ACI method, using also the experience obtained from previous projects built by COPEL, such as Salto Osório. A second method was the "Fineness Modulus Method", which optimizes both the amounts of cement and its workability, after different tests using various combinations of sand percentages. Concrete was routinely tested for ages of 3, 7, 14, 28, 60 and 90 days, in order to establish the development of resistance particularly to observe the increment at 90 days in relation to the 28 days period, due to the existing pozzolanic content. Statistical resistance controls were drawn up for the purpose of determining the project's mean resistance as well as its variation coefficient. Also accelerated tests by boiling concrete cylinders during cycles of 28 1/2 hours were correlated with strengths at 28, 60 and 90 days.

Routine tests for determining slump and air entrainment were performed daily in the field, and tests for determining the thermal-expansion coefficient and elasticity modulus were carried out in the laboratory, in order to interpret the readings of the instruments installed in the dam slab, since the strain gauges are of the electrical resistance type.

TABLE 4 – ARTIFICIAL SAND

SIEVE		SPECIFICATION	AVERAGE VALUE	STATISTIC LIMITS — % PASSING				REMARKS
				68 %		95 %		
Nº	mm	% PASSING	% PASSING	$+\sigma$	$-\sigma$	$+2\sigma$	-2σ	
4	4,80	95 a 100	99	100	98	100	97	1) VALUES ARE AVERAGE OF 73 SAMPLES - FROM 01/06/78 TO 12/10/78 2) PHYSICAL PROPERTIES FINENESS MODULUS = 3.04 ORG. MATERIAL < 300 p p m. SPECIFIC GRAVITY = 2,869 UNIT WEIGHT = 1,479 kg / m^3 Na_2SO_4 LOSS = 8% 3) σ = STANDARD DEVIATION
8	2,40	80 a 100	88	95	81	100	75	
14	1,20	50 a 85	64	74	54	83	45	
28	0,60	25 a 60	32	41	23	50	15	
48	0,30	10 a 30	10	13	7	17	3	
100	0,15	2 a 10	3	4	2	5	1	
200	0,074	< 3 %	0	–	–	–	–	

TABLE 5 - NATURAL SAND								
SIEVE		SPECIFICATION % PASSING	AVERAGE VALUE % PASSING	STATISTIC LIMITS — % PASSING				REMARKS
				68 %		95 %		
Nº	mm			+ σ	− σ	+ 2 σ	− 2 σ	
4	4,80	95 a 100	97	99	95	100	93	1) VALUES ARE AVERAGE OF 111 SAMPLES FROM 01/06/78 TO 12/10/78
8	2,40	80 a 100	87	92	83	97	77	2) SAND IS COARSER ON SIEVES Nº 48 AND 100.
14	1,20	50 a 85	70	80	60	90	50	3) PHYSICAL PROPERTIES FINENESS MODULUS = 3,06 ORG. MATERIAL < 300 p.p.m. CLAY LUMPS 0,5 % SPECIFIC GRAVITY = 2,629 UNIT WEIGHT = 1,494 kg / m³
28	0,60	25 a 60	33	46	21	58	8	4) σ = STANDARD DEVIATION
48	0,30	10 a 30	6	10	2	14	0	
100	0,15	2 a 10	1	2	0	3	0	
200	0,074	< 3 %	—	—	—	—	—	

RECORDS

The maximum production rates attained at Foz do Areia were as follows:

	DAILY m^3	WEEKLY m^3	MONTHLY m^3
Common Excavation	34,407 (05/09/78)	176,308 (05-02/05-08/78)	572,608 (May 77)
Rock Excavation	26,444 (09/08/78)	135,843 (01-27/02-03/79)	564,920 (Jan 79)
Concrete	1,587 (10/28/78)	8,859 (10-23/10-29/78)	33,187 (Oct 78)

It should be stressed that three national records for rock-excavation were attained during the construction phase (excluding Itaipu). Initially, on September 77, 531,832 m^3 of rock excavation was reached. Later, in November 78, 527,571 m^3 was attained, and finally, in January 79, 564,920 m^3 were excavated.

In the Iguaçu river projects, a production of 33,187 m^3 of concrete was also a record as regards concrete pouring, per month.

Regarding tunnel excavations, the mark of 400.90 linear meters, in November 1976, was likely a national record.

Over the last two years of dam construction, an average monthly volume of over 500,000 m^3 of rockfill was dumped on the embankment. This means that at every minute, a 35 ton truck was loaded, unloaded, spread and compacted.

9. Performance

Filling of the reservoir was started on April 2, 1980, having reached elevation 740 on August 26, 1980. Since that date the reservoir has remained at about the same elevation with small variations, up to April 1981. During the period April-September the reservoir was lowered to approximately elevation 724 (**Figure 13**).

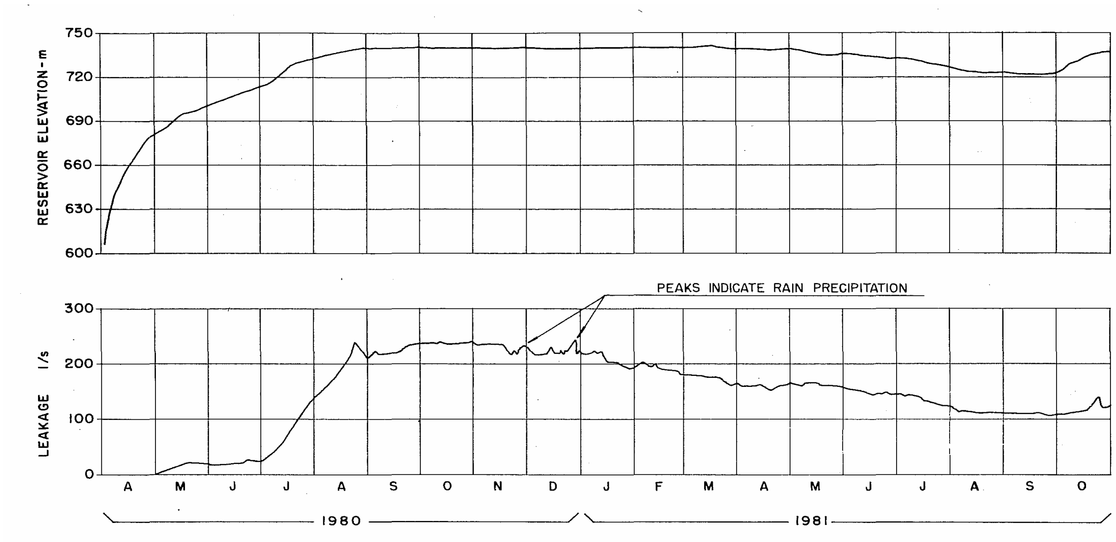


Figure 13. Filling of the reservoir – Evolution of leakage

9.1 Dam Settlement

Figure 14 shows the settlement observed in the hydrostatic cells, at the end of 1980, due to reservoir filling only.

Readings taken in September 1981 indicate stabilization of the cells. Maximum settlement at elevation 670 was 59 cm, which is the same reading recorded two months before.

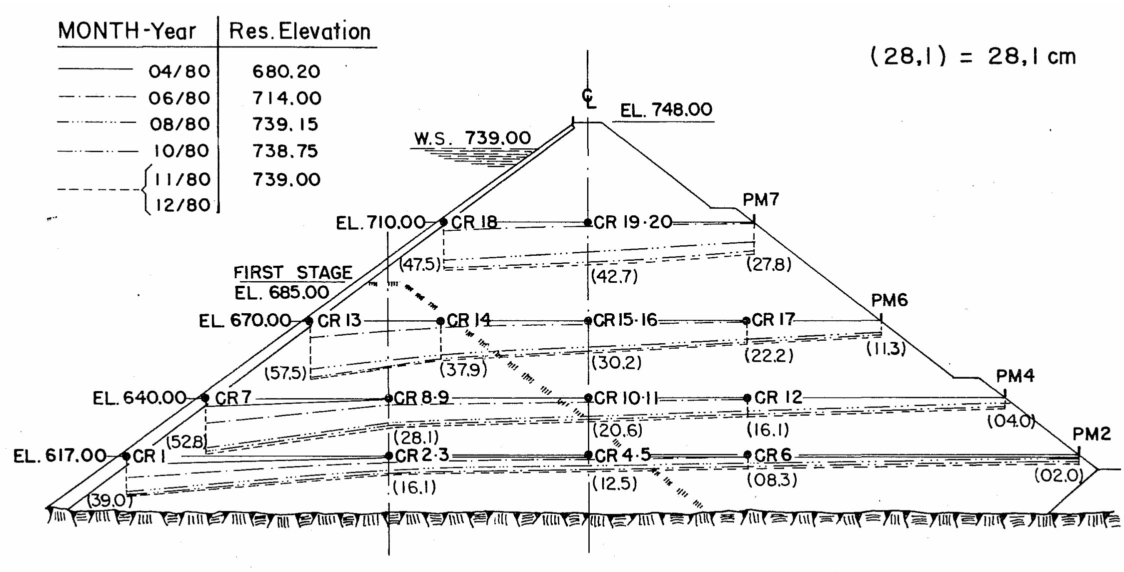


Figure 14. Vertical settlements after reservoir filling

Figure 15 shows an approximate interpretation of the equal settlement curves inside the dam, at the end of the filling of the reservoir.

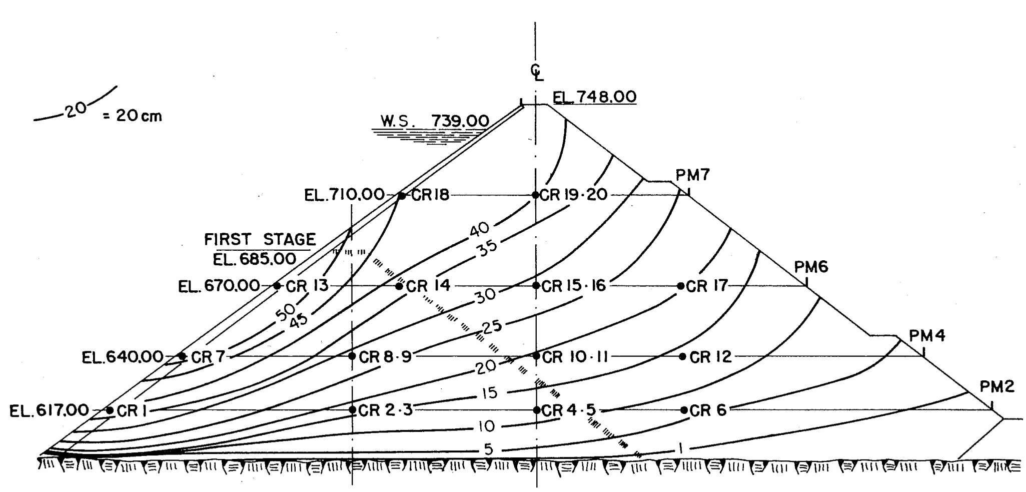


Figure 15. Equal settlement curves after reservoir filling – September 80

Figure 16 shows deformation of the slab deduced from reading of settlement cells located dose to the face slab, assuming deformation to be normal to the slab.

Maximum deflection is 7l cm, at elevation 713, which is 1 cm more than that observed six months before.

CR	30/04/80	31/05/80	30/06/80	31/07/80	29/08/80	30/09/80	31/10/80	30/11/80
1-21	10,33	21,77	27,74	44,03	47,35	49,13	49,70	50,13
7-27	8,79	22,76	30,93	55,84	61,44	63,89	64,52	64,83
13-33	1,90	12,55	20,66	50,40	61,19	66,23	68,52	69,19
18-36			8,73	35,61	46,99	52,39	56,40	56,58
ELEV.	680,00	702,50	714,08	735,80	739,15	739,05	738,75	739,00

CELLS	El.
CR 1-21	617,00
CR 7-27	640,00
CR13-33	670,00
CR18-38	710,00

CR= SETTLEMENT CELL

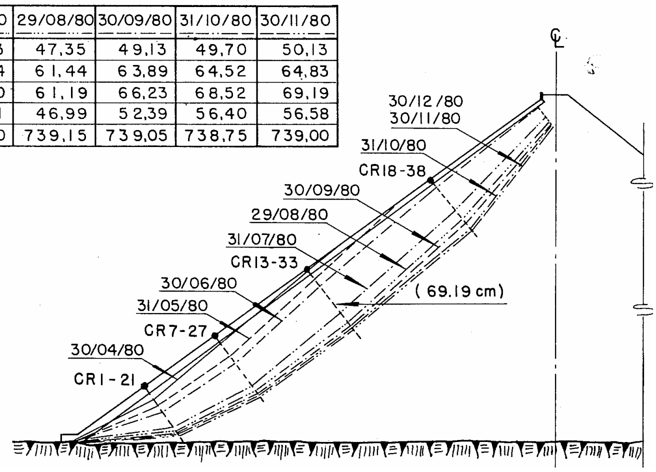


Figure 16. Slab deformations after reservoir filling

9.2 Joint Movements

Figure 17 shows the situation on Nov 27/80 when most of readings were almost stabilized. Readings after this date are relatively stable. Maximum opening of the joint was on the order of 24 mm, at elevation 685, at the right abutment.

Maximum shear movement occurred at the right abutment, elev. 685. This value was approximately 29 mm.

Movements perpendicular to the face exceeded 60 mm in two points: one at the right abutment, elev. 666, and the other at elevation 685, at the same abutment. See **Figure 17.**

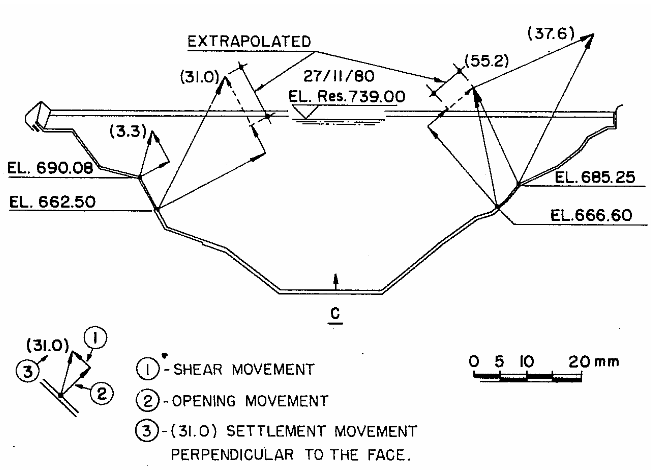


Figure 17. Perimetric joint movements

9.3 Slab behavior

Neither cracks nor spalling were present on the slab, during or after completion of the slab to elevation 680 (first stage), nor during the placing of the rockfill up to el.745.25.

During construction of the dam the first four longitudinal joints of the face next to the abutments opened as wide as 2cm. These joints closed gradually as the level of the rockfill rose during the construction of the second stage.

Before the filling of the reservoir, the unit strain values, indicated tension in the vicinity of the abutments. In the central portion of the dam, compression values were indicated by the strain meters.

The maximum recorded compression strain value was 120 x 10^{-6} in the middle section of the slab.

The filling of the reservoir caused compression in the major portion of the slab, except for areas dose to the abutments and the parapet, where tension was recorded.

As expected, some of the type A joints opened up to 4 cm, after filling of the reservoir.

9.4 Leakage

Leakage through the foundation and joints is measured in a triangular weir located at the downstream toe.

Figure 13 shows the evolution of percolation flows with time after reservoir filling. Maximum recorded discharge reached 236 l/s was for reservoir elevation of 740. The recorded discharge, for the same reservoir elevation, was 165 l/s, and 151 l/s in April 1981 and November 1981, respectively.

Variations in reservoir level have not produced major changes in percolation. In September 81, the recorded discharge was 129 l/s for a reservoir elevation of 734. The average historical leakage is 164 l/s.

Both the percolation values and their evolution confirm the excellent performance of the waterstop system, particularly if one considers the relatively important movements along the perimetral joint.

10. Dam views