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Foundation Investigation and Treatment for the Main Dam, Itaipu Project

John G. Cabrera
Consulting Engineering Geologist, Las Cruces, New Mexico

SYNOPSIS: After selection of the site for this major hydroelectric project on the Upper Paraná River, initial borings in the river bed revealed an extremely fractured zone near the base of the dense basalt flow on which the main dam would be founded. This zone was later seen to correspond to intersecting faults beneath the river bed, therefore upon dewatering the river channel several exploratory tunnels were driven parallel and perpendicular to the dam axis, foreseeing their incorporation into an elaborate "shear key" system to prevent renewed movements of the shear zones. The tunnel grid was backfilled with concrete through holes drilled from the foundation surface and grouted at low pressures.

INTRODUCTION

Itaipu, the world's largest hydroelectric project, is situated on the Paraná River, between Brazil and Paraguay, approximately 14 km upstream from the bridge joining the two countries (Figs. 1 and 2) and consists of a concrete dam and spillway 2,610 m long flanked by embankments totaling 5,150 m in length (Fig. 3). The maximum height of the hollow gravity main dam across the river gorge is 196 m, and over 56 million cubic meters of earth and rock were excavated for the project. The total cost is presently estimated at close to 18.6 billion U.S. dollars.

The Itaipu project is a binational undertaking established by Brazil and Paraguay in accordance with a treaty signed by the two governments in 1973 and the statutes of an entity created a year later, known as "Itaipu Binacional," which determine equal sharing of generated power (Cotrim et al, 1977).

The engineering consortium selected to carry out the feasibility studies, a joint venture of International Engineering Co., U.S.A., and Electroconsult of Italy, began the study of the stretch of the Paraná River between Guaira Falls and the mouth of the Iguazu River with a regional geologic appraisal to characterize the basalt flows and select viable dam sites. This work was aided by the interpretation of satellite imagery and aerial photos to determine the principal structural lineaments and to differentiate soil and rock types.

A general study of various alternatives for locating the dam was made by means of borings and geophysical profiling; this resulted in the selection of a reach of the Paraná in which the project could be situated. More detailed investigations followed to determine the best axis within the reach, considering river channel configuration, proximity of abutments and the lithologic and structural conditions of the basalt sequence at the various sites. Thereafter an axis was defined by closer-spaced borings and short adits in which rock mechanics tests were made, followed by the sinking of a deep shaft and more detailed in situ tests of the principal discontinuities.

During the construction phase, a program of exploratory tunneling was followed to investigate shear zones encountered beneath the river bed.
From this program an elaborate treatment plan of shear key tunnels was developed to prevent renewed movement along the shear zones.

Starting at first with eight alternative layout axes, the preliminary borings and geophysical profiles narrowed down the choice to four, namely Axis A-1, just downstream from Itaipu Island, Axis A-2, 300 m farther downstream, Axis C-1a, 800 m downstream from A-1 and Axis E, 3,100 m downstream from A-1 (Fig. 2). The information obtained from pre-feasibility investigations made at or near the four axes indicated that Axis E had the best geological conditions beneath the river bed. However, that axis had the disadvantage of requiring that the spillway chutes be located close to an existing hydroelectric plant in Paraguay, which could be affected by high discharges. It would also have adversely affected the location for the future river navigation facility.

Of the three upstream axes, Axis A-2 was shown to be preferable, chiefly because of a much better river cross section and spillway location.

SITE INVESTIGATIONS

Preliminary Investigations - Phase 1

Investigations of the foundation area for the high dam within the deep channel carved by the Paraná River began with some preliminary conventional borings in the banks, abutments and on the shallow Itaipu Island itself. The first stage of the studies called for only a few borings and seismic refraction traverses to define rock types at several alternative dam axes. Five basalt flows with interflows of sedimentary breccia were identified during this stage and named, from the bottom up, Flows A through E (Fig. 4).

Initially borings were made in 1973 using only a double tube core barrel, consequently, recovery of core in zones of closely-jointed or highly fractured rock was low. It soon became apparent that more accurate drilling methods had to be employed, specially within the river channel itself. Most of the initial river borings encountered difficulties such as:

- Flow velocities of 2 to 3 meters/second, which tended to shift the drilling barge and bend or break casing.
- An average depth of water of 50 m (164 ft) and 10 m to 15 m of relatively loose alluvium at the bottom.
- Sudden variations of water level, which snapped anchoring and guy cables.
- Dense morning mist during the winter months (June-September) which would last until 10 A.M.
- Extreme fracturing of the top of rock in the river bed.
- Strong winds funneled along the narrow valley.

All of the above contributed to slowing down and decreasing the efficiency of borings made in the section for the main dam.
Investigations - Phase II

Once the axis was selected, a more intense phase of investigations for the feasibility report proceeded. Small adits were driven into interflow sedimentary breccia B, overlying flow B, and determinations of elastic moduli and direct shear tests were made to characterize this horizon of relatively weak rock, having an average thickness of 10 m. The new phase of core borings consisted of wireline N diameter holes drilled using double tube core barrels with core lifter sleeves (M series). This improved core recovery somewhat, but not enough to recover soft, altered rock.

Drilling within the river took on a more thorough approach, benefiting from the earlier attempts. The drilling barge had more anchoring points and an 8 inch (20.3 cm) outer casing, well-guyed to the barge's hull, protected a 5 inch (12.7 cm) casing that penetrated the alluvium and in which H and N casings were used during wireline core drilling.

Due to the high degree of fracturing of the uppermost dense basalt in the river bed (flow B) pressure tests gave inordinately high water loss readings, but this was primarily due to difficulties in properly seating the packers as the drill string rocked to and fro with the barge.

The river borings carried out to below flow B revealed a continuous zone of highly fractured rock near or at the base of that flow. It was speculated that this signified the presence of a zone of lamellar jointing. Although core recovery techniques were still not adequate to retrieve all of the material in that zone, it became possible to drill some borings in excess of 200 m depth and determine the conditions of five additional flows below flow A.

Investigations - Phase III

It was then decided to drill several integral core borings at both banks to define the fractured zone at the base of flow B and the condition of the open contact with flow A. These were among the deepest borings (100 m) made up to that date (1975) using that technique of core reinforcing. Nevertheless, at the considerable depths where the zone of fragmented rock occurred, binding of the fragments could not be efficiently achieved with cement grout, even using quick setting additives. It was only by using a fast-setting epoxy resin that some core of the weak seams was recovered.

The in situ or residual stresses within the basalt sequence were determined near the location of the main dam by several borings in which overcoring strain measurements were made using a stress tensor gauge (Rocha et al., 1974). It was surprising to find that horizontal stresses had been relieved in proximity to the river channel, whereas normal stresses remained moderately high (to 4.41 MPa).

Investigations - Phase IV

A fourth, more intensive phase of investigations was decided upon. This included a deep shaft in the right bank, with short tunnels radiating from it to investigate and test the principal adverse features in situ. The shaft, 4 meters in diameter, was excavated from El. 120 m to El. 7 m and tunnels driven at El. 70 m, 59 m and 12 m. From the latter drift, an upward-sloping tunnel reached the contact between flows A and B, approximately at El. 20 m.

The tunnel at El. 70 m was made to study breccia B and carry out plate jacking and grouting tests. The results indicated an average modulus of elasticity of 6,800 MPa. A part of this...
Breccia had been grouted from holes made before initiating the shaft and the results could be verified. There was an extremely low grout acceptance in the compact breccia, classified as type I.

The drilling program had revealed the existence of a zone of closely-spaced, parallel fractures in basalt flow B, between elevations 55 and 62 m. The gallery at EL. 59 m was raised somewhat and a test niche made at EL. 62 m to carry out a series of direct shear tests on blocks measuring 1 m on the side. The discontinuity (named "B") consisted of parallel joints, well-cemented in some areas by calcite or silica, and the average angle of shearing resistance was found to be 35°, with a cohesion of 0.5 MPa.

The most important feature revealed by the deep shaft and the tunnel at EL. 20 m was the zone of crushed rock at the base of flow B. This was determined to be a shear zone filled with a mylonite gouge. The zone could only be observed within the short length of the tunnel, which was not continued fearing it could intercept one of the borings made in the river channel, that was not plugged with cement. Therefore, it was decided that a crash program of subsurface investigation would be carried out by tunneling after unwatering the river channel in the foundation area for the main dam.

Three direct shear tests were made at the base of flow B where the sheared basalt could be cut into blocks with a diamond saw without disintegrating it. The angles of shearing resistance obtained were $\theta = 25.1^\circ$, $26.1^\circ$, and $29.2^\circ$. It should be emphasized that this was a highly fractured, blocky, dense basalt with some clay filling the fractures, not the basalt entirely reduced to a mylonite, (Cabrera and Barbi, 1981).

Final Investigations - Phase V

After unwatering and excavating the thick alluvium in the foundation area for the main blocks of the dam, five shafts were sunk to elevations 8 m to 10 m above sea level, that is, 1 m to 9 m below the contact between basalt flows A and B. A sixth shaft was made in the downstream part of the area between the stems of double block F15/16 to investigate and treat a local concentration of highly sheared zones.

In all of the shafts one or two zones of closely-spaced open fractures filled with clay were found. In shaft 13, situated just downstream from the head of double buttress F-15/16, a very thick zone of weathered and highly fractured dense basalt, with oxidized, open fractures filled with clay, was found between elevations 38 m and 31 m.

The river thalweg was being excavated in the powerhouse foundation area at the same time and a zone of lamellar, weathered basalt was found dipping approximately 20° E-SE. This zone was followed upstream along its nearly N-S strike, by ripping and light blasting, and the suspicion that it coincided with the zone encountered in shaft 13 was confirmed. Its extent was further verified by two short tunnels that followed the ascending shear zone westward from shafts 9 and 12. It soon became apparent that several intersecting shear zones traversed the remnant of flow B in the river channel. While various solutions were being sought, an accelerated program of borings was initiated to determine the distribution and extent of the shear zones. In addition, it was decided to follow those zones, to permit direct observations and more accurate appraisal of the geomechanical parameters by tunneling parallel and perpendicular to the dam axis whenever possible (Fig. 5).

The borings made from the foundation level and the exploratory tunnels eventually permitted a three dimensional visualization of the relationship among the shear zones within the foundation area for the high concrete dam. When the investigations were completed it was possible to differentiate the zones as follows (Fig. 6):
1. Eastward-dipping shear zone. This is the uppermost shear zone and the first to be exposed along the deepest portion of the river, following the stem of half block F-17. It intercepts the originally proposed foundation level of El. 40 m at the head of half-block F-15, dips 15° - 20° E-SE and varies from very weathered, with a filling consisting mostly of clay at higher elevations, to almost a rock to rock contact of lamellar slabs at the lower levels, close to the contact between flows A and B. Slickensides on the surface of this zone indicated that the rock mass above it was thrust westward, towards Paraguay.

2. Westward-dipping zone. This zone is tangential to contact A/B on the west side of the river channel, as seen in the deep, right bank shaft and the tunnel at El. 20 m. It rises at a low angle until it is intercepted by the eastward-dipping zone. Shear displacements could not be observed on the west side, probably because of a high degree of weathering and considerable clay. Eastward updip displacements of as much as 30 cm were measured.

3. Upstream southward-dipping zone. This zone has the main disadvantage that it dips downstream. It was initially located in shaft 14 at El. 29.5 m. In a downstream direction it tends to become tangential to contact A/B. It consists of crushed rock, with some clay filling between fragments. Proper investigation of this shear zone required an additional shaft (No. 17) from which a short drift followed it eastward to a N-S tunnel that ramped down to the A/B contact.

4. Downstream southward-dipping zone. This zone, with an approximate area of only 4 m by 50 m, was found during the final phase of investigations of the foundation area. Core drilling in an area nearly 46 m south of shaft 17 had revealed another zone of lamellar fragments with a low clay content. As it progresses downstream from El. 35 m and El. 30 m to lower levels, it tends to narrow down and disappear, forming a fault slice imbricated upon the upstream southdipping zone described above.

5. A fifth, less important shear zone was found at the downstream portion of the high dam foundation area.

GEOMECHANICAL PROPERTIES OF THE SHEAR ZONES

The various forms in which the shear zones beneath the river bed occur were differentiated into five classes, based on detailed observations, measurements on the tunnel walls and the results of laboratory direct shear tests on 20 cubes. The principal factors considered in the following classification are the amount and type of clay filling.

1. Fracture zone with rock-to-rock contact; the angle of shearing resistance $\phi = 35^\circ$. This form of occurrence is generally restricted to greater depths or close to where the fracture pinches out.

2. Zone of subhorizontal rock fragments, with clay coating or partial clay filling ($\phi$ between 25° and 30°).

3. Zone of crushed rock (mylonite) with small, angular rock fragments and a small percentage of clay ($\phi < 30^\circ$).

4. Open zone with well-defined borders, filled principally with clay gouge and angular fragments of sound or slightly weathered basalt ($\phi > 25^\circ$).

5. Zone filled principally with highly plastic clay ($\phi < 25^\circ$). As the shear zones approach the original river bottom level the clay filling is mostly an orange-tan, very plastic, colloidal clay that appears to have been introduced by infiltrating river water.

PROPOSED TREATMENT

The 1974 cost estimate in the feasibility report included a sum of $2.8 million for special treatment of breccia zones, thought at the time to be the weakest features in the foundation. This treatment was described as "removal of weaker breccia from certain strata of the foundation and replacement by concrete to improve the shear strength of the foundation for the dam".

Once the program of exploratory tunnels following the shear zones at various levels was well underway, it became apparent that treatment or elimination of the fractured seams was necessary in order to have the required factor of safety. Initially, several alternatives were considered, including: excavating part of the foundation area to below contact A/B; hydraulic excavation of the material filling the shear zones using high pressure jets; a series of trenches filled with concrete; concreted tunnels acting as shear keys. Of these, lowering of the downstream third of the foundation area and concrete shear keys along the fault zones and contact A/B were analyzed in detail. Although both of these solutions would yield equivalent safety factors, an open excavation would have required nearly 7 months to remove the rock by controlled blasting, then backfill with concrete from El. 20 m to El. 40 m, which would have seriously delayed the construction schedule. The second alternative would permit underground treatment while the high blocks of the main dam were poured to a safe limiting height (El. 90 m) and would be 20% less costly, in view of the smaller amounts of excavation and concrete required, (Caric et al., 1982).

Based on these considerations, the alternative of underground treatment by shear keys was selected. Already in 1978, before the river channel foundation area was unwatered, a comparative study of subsurface treatment methods carried out at other projects (e.g. Green Peter Dam, Oregon, New Bullards Dam, California) had been made. Conventional limiting equilibrium stability analyses against shear failure, utilizing very conservative assumptions, showed that
to obtain an adequate factor of safety the minimum area to be treated by concrete keys had to be approximately 25% of the general foundation area under the four main double blocks of the dam (Itaipu Binacional, 1981).

Finite element analyses were used to determine the proper orientation of the concreted tunnels to serve as shear keys. It was found that with keys only parallel to the dam axis (transverse to the applied load) there was a tendency to concentrate the stresses in the upstream rows of keys in a manner that could cause progressive failure. On the other hand, keys aligned in an upstream-downstream direction and located directly under each stem of the blocks would provide a continuous, effective shearing resistance over the whole foundation width, even bridging over unexcavated areas of the foundation.

In order to include the best elements of both orientations and increase the area of treatment, it was decided to have a grid of 8 tunnels parallel to and 12 perpendicular to the dam axis, surrounded by peripheral drainage tunnels. Such a grid would have the advantage of a more uniform stress distribution, reducing the concentration in the shear keys across the river (Fig. 7).

The grid beneath the high central blocks in the river channel consisted of tunnels 3.5 m wide and 2.5 m high, totaling 2,600 m in extent, which were filled with approximately 30,000 m$^3$ of concrete and then grouted.

The shear keys are not at a uniform elevation. Practically all the keys parallel to the dam’s axis rise from either extreme towards the area beneath the stems of double block F-17/18, where two of the shear zones intersect. The upstream-downstream key tunnels are somewhat more regular in elevation. A separate grid was designed around shaft 17 in order to cope with the south-dipping shear zones at approximately El. 30 m. In addition, short stub tunnels were driven upstream under the heads of half blocks F-16, F-17 and F-18 and westward in the area beneath the stems of F-17 and F-18, where the existence of very soft mylonitic gouge was a source of concern with respect to settlement.

**Fig. 7 Itaipu Dam. Grids of Tunnels under River Bed for Concrete Shear Keys and Drainage.**
SHEAR KEY TUNNEL GRID CONSTRUCTION

Excavation of the tunnels for the shear keys was carried out with extreme care to prevent damage to the sound foundation rock, always controlling the blasts so that the concrete rising above the area would not be affected. Blasting vibrations were very carefully monitored. The maximum advance for each round of blasting was 1 to 6 m, after which scaling was done with special lances and the muck removed with a mini-loader. Where two or more shear zones occurred close above one another it was necessary to heighten the tunnel, excavating it by benches to a vault 8 to 10 m high. These chambers were concreted as soon as possible to reduce stand up time, in spite of the use of tensioned rock bolts and wire mesh. The use of gunite together with rock bolts and wire mesh was very effective in some areas where shear zones overlapped and a larger excavation was required.

Prior to concrete placement, the tunnel surfaces were thoroughly washed with air/water jets to remove all loose material and all surface clay was removed from the shear key to guarantee good concrete adhesion. Detailed geologic mapping of the tunnel walls and roof was carried out, indicating all the important lithologic and structural features. This was done with reference to station and elevation marks placed by the surveyors.

The concrete used to backfill the tunnels was generally of a specified compressive strength of 27.4 MPa (3,973 psi) after one year, with a maximum aggregate size of 3.8 cm (1.5 inch). However, for the stub tunnels beneath the heads of the blocks, 34.3 MPa (4,973 psi) strength was specified. In order to distribute the concrete thoroughly within the tunnels it was fed through 6 inch holes bored from the foundation surface with "down the hole" percussion drills on 4 m to 6 m centers.

Up to a certain height within the tunnels, concrete was vibrated by a crew positioned upon a platform made of rebars. This was followed by some vibration from the ends of the sections being concreted and by lowering vibrators down the placement hole. The last 50 cm of tunnel height were filled with concrete containing an expansion agent.

Grouting of the concreted shear key tunnels was carried out from the surface through holes drilled at 2 m centers to fill any voids left due to concrete shrinkage, insufficient filling of the irregularities of the rock surface or to opening of joints by blasting at nearby continuations of the grid system. Grout pressures were limited to 0.6 times the height of the overlying rock and the slurry generally consisted of a 1:1 water/cement ratio. When grouting was completed, core bores were made to verify grout penetration and pressure tests carried out to determine if the fractures had been filled.

FOUNDATION INSTRUMENTATION

To monitor the behavior of the rock mass in the foundation area of the high blocks, instruments were installed even before backfilling the shear key tunnels. The most successful type of instrument has been the multiple-position borehole extensometer. The behavior of these instruments showed that the beginning of excavations was characterized by continuous deformations, specially at the heads of double blocks F-15/16 and F-17/18, which reached 1 mm/month during the opening of the stub tunnels under those blocks. After the tunnels were backfilled with concrete and grouted, the rate of settlement decreased to 0.7 - 0.4 mm/month, then stabilized. Tangential movements were determined by triorhogonal gages installed along contact A/B in the peripheral drainage gallery and by lowering vibrators down the tunnel. During reservoir filling measured by a series of Casagrande type piezometers and open wells that permitted piezometric measurements at any point along the hole. Permission to continue pouring concrete above El. 90 m on the high blocks depended on the settlement rates measured by the extensometers. As tunneling progressed while the main blocks rose from the foundation, the behavior of the rock mass subjected to increased loading became critical, specially in areas where chambers had to be excavated and the overlying rock burden was decreased. The instruments monitoring these areas indicated higher deformations and thus they were the first tunnel sections to be concreted.

REMEDIAL MEASURES IN OTHER AREAS WITH FOUNDATION STRUCTURAL DEFECTS

Following is a brief account of the principal problems encountered in basalt flows at progressively higher elevations due to the presence of discontinuities, and the measures taken to remedy them within the area of the main dam.

Discontinuity B

This is the imbricate series of fractures that was tested at El. 61.5 m in a niche excavated beside the gallery leading from the deep shaft at El. 59 m, within the upper portion of the dense basalt of flow B.

Shortly after the river was diverted in October 1978, this discontinuity was investigated by tunnels into both abutments. It soon became evident that on both sides its shear strength is high, for it consists principally of a zone of closely-spaced, subhorizontal joints cemented by calcite and/or silica. It was, however, found that some seepage along it could cause uplift pressure once the reservoir was filled, therefore the exploratory adits were converted into drainage tunnels extending 165 m at El. 60 m in the right abutment and 210 m at El. 55 m within the left flank.

Contact C/D

A shear key tunnel grid similar to that beneath the river bed, but of much smaller dimensions, was required to treat the foundation of blocks F-1, E-6, E-5 and E-4, the E blocks being simple buttresses on top of the right abutment. In that area the contact between flows C and D, approximately at El. 112, was found to be filled...
with a highly plastic orange-tan clay derived from the river when it flowed at much higher levels in the past. In the left abutment contact C/D is tightly cemented, with no clay filling. In comparison to the main grid, this smaller one consists only of four upstream-downstream tunnels and three tunnels parallel to the dam axis, having the same cross section as the river tunnels. The upstream-downstream tunnel coincident with block E-6 turns sharply east under the head of block E-6 and passes under the heads of half blocks F-1 and F-2. Access to the grid was through a shaft downstream from the stem of block E-6. The three tunnels parallel to the dam's axis were extended westward below the stem of buttress E-2 in order to make certain that the contact had progressively greater rock to rock contact in that direction. The total length of tunnels is nearly 600 m and filling them required approximately 6,000 m³ of concrete.

Discontinuity D

This is one of the most extensive discontinuities present at the Itaipu site. It became apparent not only in borings made on the left side of the river but in road cuts and the sides of the Diversion Canal excavation. In that area it developed within a band of expansive clay minerals concentrated in the upper third of flow D and is filled with colloidal clay derived from the river and even some small pockets of fine, uniform sand.

The concrete dam foundation on the east (left) side of the Diversion Canal was 20 m to 34 m above Discontinuity D, with sound, dense basalt in the dam's axis. The first two concrete blocks adjacent to the Diversion Structure on the east side of the canal were changed from single buttresses to massive gravity blocks for greater stability. With the dam founded at higher elevations to the east, the load imposed upon the rock would be distributed over a considerable area at the depth of the fracture.

Originally it was intended to found the uppermost double buttress blocks of the main dam at El. 125 m on the left abutment, but in that area Discontinuity D, which had evidence of shear displacement, bifurcated into two branches approximately at elevations 117 m and 115.5 m. In addition to being one and a half the size of the first grid it was affected by blasting vibrations during canal excavation, so that the rock mass above it underwent stress relief and some movement. Consequently, the foundation level had to be lowered below the fracture to El. 115 m.

In the right abutment, Discontinuity D was followed for over 120 m by a tunnel excavated at El. 125 m and was found to have enough areas with rock to rock contact to yield an estimated friction angle $\phi$ of 35°. Only the foundation of half-block F-1 had to be lowered from El. 125 m to El. 123 m due to the occurrence of a branch of the discontinuity observed exposed on the cut slope between the foundation berms of buttresses F-1 and F-2. This exploratory tunnel was left as a drainage tunnel after adequate roof supporting treatment.

FOUNDATION GROUTING AND DRAINAGE OF THE MAIN DAM

Prior to curtain grouting, consolidation grouting was carried out under the heads of the double buttress blocks in vertical holes 3 m deep and on 6 m centers, just after the first lift of concrete was poured. The main purpose of this grouting was to seal any fractures that could open near the foundation surface due to excavation blasting.

Contact grouting was carried out from the upstream berm of the blocks in two rows of inclined holes extending about 6 m below the block foundation level and with a spacing of 3 m.

The main grout curtain is situated upstream of the dam blocks and is connected by means of lateral curtains in the abutments with the downstream grout curtain of the powerhouse. Thus the entire foundation area of the main dam in the river channel is surrounded by a grout curtain to reduce seepage to a minimum and the shear key grid within this area is encircled by a drainage tunnel system with drains and weirs to collect and measure what little infiltration passes through the encircling curtain.

The upstream grout curtain consists of 3 rows of vertical holes drilled from the upstream platform of the blocks. It would have been preferable to initiate the curtain directly under the buttress heads but, by doing so, concreting would have been delayed. Moreover, calculations showed that a grouting gallery across the blocks would have weakened them.

The grouting sequence consisted of primary holes on 6 m centers, secondary ones at half-spacing and tertiary and quaternary holes at split-spacing, depending on grout absorption. The first line to be grouted was the one nearest the drainage curtain. Grout absorptions greater than 12.5 kg/meter would dictate the need to grout tertiary or higher orders of grout holes. The depths of grout holes reached 40 m below the foundation rock surface in the river channel area.

All the grout holes were drilled with percussion equipment. It was found that the "down the hole" type of air drill produced faster, straighter holes than those drilled with a rotary rig and cuttings could be readily flushed out. Rotary borings drilled with diamond drill bits tended to produce much fine-grained "rock flour" slurry that could be forced into the very fractures that had to be sealed with cement grout. Furthermore, the cost of percussion drilling was considerably less than diamond drilling.

Pressure increase during grouting observed the following specifications:

<table>
<thead>
<tr>
<th>Depth</th>
<th>Maximum Effective Pressure (pressure at level being grouted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 5 m</td>
<td>not greater than 1 kgf/cm² (98 KPa)</td>
</tr>
<tr>
<td>6 m-20 m</td>
<td>0.3 x depth kgf/cm² (98 KPa)</td>
</tr>
<tr>
<td>20 m 6+0.75(depth-20)</td>
<td>50 kgf/cm² (98 KPa)</td>
</tr>
</tbody>
</table>


to a maximum surface gauge pressure of 50 kgf/cm² (4.9 MPa)
CONCLUSIONS

The foregoing account of the selection and investigation of the Itaipu dam site can illustrate the fact that the input from the engineering geologist is often not sufficient to prevail over other factors that influence the location of a dam's axis. At site E, farther downstream, the river eroded down to flow A, which investigations showed did not have zones of fragmented and comminuted rock corresponding to shear zones.

In the area of the axis finally selected for the dam (Axis A-2) it appears likely that as basalt flow B was eroded by rapid river downcutting to nearly 20 m from its base, the decrease of normal load in the valley allowed the high residual horizontal stresses to cause buckling and shearing of the remainder of the flow.

No evidence of shear displacement was found on the surfaces of contact A/B, only filling by colloidal clay, probably transported by water infiltrating from the river. Directly under the river thalweg the contact is more open, probably separated by an arching effect.

ACKNOWLEDGMENTS

The investigations for site selection, and the feasibility and design stages of the Itaipu Project were directed by IECO-ELC, an engineering consortium consisting of International Engineering Co. (IECO) of San Francisco, USA and Electroconsult (ELC) of Milan, Italy. IECO-ELC was also made responsible for the design of the canal and all structures required for river diversion and the rockfill embankment of the left (East) side of the river. Eventually it became the coordinator of all engineering design carried out by consortia of Brazilian and Paraguayan firms. The writer was principal Engineering Geologist for IECO from 1975 to 1981.

Most of the final investigations for the treatment of the shear zones under the river channel and other major discontinuities affecting the foundation of the main dam were carried out by Itaipu Binacional's Division of Geology, Rock Mechanics and Instrumentation, headed by Engineering Geologist Adilson L. Barbi.

REFERENCES


SALIENT FEATURES OF THE INVESTIGATION AND SELECTION OF THE ITAIPU DAM SITE

1. Core drilling in the river channel had to be done in nearly 50 m of water.

2. The dam axis was chosen by giving point ratings to essential features, for example, width of site, geological conditions in the river bed, necessary height of embankments, etc.

3. Although downstream site E was found to have a relatively unfractured thick basalt (flow A) as a foundation for the main dam, it was not selected primarily because spillway discharges from it, and probably the powerhouse discharge as well, would affect the small Acaray project which develops a stream of the same name a short distance downstream from that site, on the right side of the river (Fig. 2). It also received a low rating for the location of future navigation facilities.

4. After the five basalt flows that influence the selected site were investigated by core borings and defined as flows A through E from the bottom up, deeper borings were made and five additional underlying flows were characterized.

5. A sum of 20 million dollars was included in the construction estimate to cover the cost of any radical foundation treatment program.

6. The core borings made before the foundation area of the main dam was unwatered did not reveal the true character of the shear zones in the lower part of flow B. The first confirmation of the nature of these zones was obtained in a tunnel advanced from the bottom of the deep shaft on the right bank.

7. All the basalt flows in the area have a subhorizontal discontinuity, generally in the upper third of the flow's thickness. Some of these discontinuities developed in horizons of expansive clay mineral concentration. The clays in turn appear to have formed due to deuteric alteration of volcanic glass along a horizon controlled by the cooling rate of the flow (Cabrera, 1985).

8. The results of the detailed investigations of the shear zones below the river bed near the base of flow B, indicate that they occurred due to unloading by rapid downcutting of the river and consequent high horizontal stresses, which buckled and overthrust the remaining thickness of flow B (Cabrera, 1986).
