THE IRAPÉ DAM AND HYDROELECTRIC PROJECT

This paper was written by Brasil Pinheiro Machado based on papers indicated in the references. Some figures and the technical features were based from documents furnished by Waldaisy S. Abreu Sifuentes from Leme.
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1. INTRODUCTION

The Irapé Project, is a 360 MW hydroelectric project, built in the upper reach of the Jequitinhonha River, in the State of Minas Gerais, Brazil. The main purpose of the project, besides local power generation, is to create a large reservoir to regulate the river flow for a series of downstream hydro-projects programmed to be built in the near future and including the already built 450-MW Itapebi Project. The reservoir, with a total volume of 6 billion cubic meters and flooding an area of 137 km², is created by an earth-core rockfill dam, 208-m high, the highest in Brazil.

The Project is owned and operated by CEMIG - Companhia Energética de Minas Gerais, the power utility for the State of Minas Gerais. The Project was built from May 2002 to August 2005, with a strict and very tight schedule of 40 months from start of site activities to initial power generation. It was designed by a consortium of two Brazilian engineering firms, INTERTECHNE Consultores and LEME Engenharia acting as engineers in an EPC joint-venture led by ANDRADE GUTIERREZ and including construction companies ODEBRECHT, HOCHTIEF and IVAI and with Voith-Siemens Hydro for equipment supply. CEMIG technical team acted as Owner's Engineer and managed as well the administrative and commercial issues of the realisation of the Project. A general view of the Project is depicted in Photo 1.

2. BACKGROUND

The Jequitinhonha River Basin is located mainly in the Northern part of Brazilian State of Minas Gerais with a minor part in the State of Bahia, as illustrated in Figure 1. The river basin was initially surveyed for its hydroelectric resources during the sixties as part of the major Brazilian Inventory of Hydroelectric Resources, covering the South-Central area of the country. These studies identified 20 feasible sites for power projects in the basin. A later study, carried out between 1984 and 1987, revised the conclusions of the previous survey and reduced the number of feasible sites to 14, in which the Irapé site was identified, and the project considered as a major upstream reservoir for regulating the river flows besides local electric power generation.

Comprehensive site topographical, geological and hydrological investigations were carried out by CEMIG...
during the nineties, associated with project feasibility studies. These studies considered the possibility of a RCC dam, but ended up by discarding this alternative because of the questionable quality of the economical locally available concrete aggregate. Therefore an embankment dam was the chosen alternative, and additional studies were concentrated between an earth-core and a concrete-face rockfill dam. After detailed studies of either type of dam and different project layouts, an earth-core embankment dam was selected.

In 1998 CEMIG obtained from the Federal Government the official concession for exploring hydroelectric power from the site, and in 1999 contracted the EPC Consortium which included the designers, INTERTECHNE and LEME, who carried out the basic design of the project. This basic design was submitted to ANEEL the Federal Government electric power regulating agency in January 2002 and approved for construction. Construction started in May 2002 and the first unit started power generation in the end of August 2005, a 40-month construction schedule.

3. SITE CHARACTERISTICS

The region of the project site presents a flat erosion surface deeply entrenched by river channel. The site properly is formed by a narrow canyon where the river runs between very steep abutments, about 80 to 100 m wide and 80 m deep, a narrow flat bottom and an additional river channel about 30 m deep and 20 m wide.

The rock mass at the site is a graphitic quartz-mica-schist with disseminated sulphide minerals, with good foundation and excavation conditions. The rock is usually fine grained, dark grey coloured, with a metamorphic milonitic foliation dipping around 11º to SE. Stratigraphically, it belongs to the pre-Cambrian Salinas Formation, of the Macaúbas Group. The soil cover is moderate to thin, but the zone of weathered rock is quite thick and irregular. Besides stress relief fractures and the ones related with the foliation, the rock mass has two main sub-vertical joints systems. Near the surface and the steep scarps, these structures were found open, weathered, or filled by soil-like or lateritic materials.

The occurrence sulphides in the rock mass - mainly pyrite and pyrrhotite - in percentages higher than normal were a key preoccupation of CEMIG and of the designers who carried out extensive studies to evaluate the impact of such a feature in the performance of the different project structures. This was considered to affect unprotected concrete as a result of oxidation of sulphites producing sulphuric acid, and could be deleterious to the rockfill. A detailed description of the problem and the solutions used for the Project is presented by Marques Filho et al. (2009).

The Jequitinhonha River, at the Irapé site, has a mean flow of 158.1 m³/s and well defined dry and rainy seasons. The average annual precipitation in the river basin is 1,300 mm, with a very clear distinction between the dry season (April through October) and the rainy season (November through March). The flood pattern at the Irapé site shows, correspondently, a clear seasonality: 5,950 m³/s during the rainy period versus 2,525 m³/s during the dry period, for a 1,000-year recurrence time. The PMF hydrograph has a peak of 11,446 m³/s and an average flow of 7,946 m³/s. As mentioned further down in this report, the spillway structure was sized with consideration of the flood routing in the reservoir, which of course, reduces substantially the peak of the outflow flood hydrographs.

4. PROJECT DESCRIPTION

The characteristics of the site and the schedule requirements set forth by the Owner, made it very difficult to develop a project layout capable of allowing the construction to overcome successfully these obstacles. In fact the topographic features made it difficult and expensive to provide access to the various construction fronts. On the other hand, for the Owner, the feasibility of the Project, based on economical and financial considerations, could only be achieved if the construction time for dam and other key structures could be reduced to allow initial power generation in an extremely tight period (for the size and type of structures) of 40 months. To accomplish this, a number of innovative design and construction features were adopted, some of which for the first time in Brazil. These allowed the achievement of this goal and an early reservoir impounding (previous to dam completion) permitting initial power generation, as mentioned, 40 months after start-up of contractor mobilisation. In relation to the original conventional planning there was an advance of seven months which was achieved by an engineering design in strict association with the construction planning and of course by providing construction equipment and manpower coherent with the production rates required. A detailed description of the challenges and solutions encountered is presented by Kamel et al. (2006).
The Project layout is depicted in Figure 2. As seen in this figure, the Project is developed across a rather tight river curve which has its inner side in the left margin, and ending in an almost 90° angle to the right of the incoming stretch of the river. The dam axis is placed upstream of the “vertex” of the river curve in a section which was considered, topographically and geologically best fitted for it. The power waterways and the spillway structure are routed in the left margin, bypassing the dam, but with features and locations especially designed to reduce construction quantities and to expedite construction activities.

4.1. River Diversion

River diversion for construction was done through two diversion tunnels designed with a cofferdam height compatible with a 50-year recurrence period during the first wet period and subsequently for higher floods and different crest dam elevations. The tunnels were built on the right bank, contrary to what seemed to be the logic design solution, because there existed a public road on the right margin of the river, from which access to the initial construction area would be easier and faster. This implied longer diversion tunnels as compared with the alternative location in the left abutment, but allowed achievement of river diversion 11 months after initial construction mobilisation, that is three months before originally scheduled.

One of the tunnels was designed and built with its entrance at a higher elevation than the other one, and without a flow control structure. This tunnel was 1060 m long. The other tunnel, was 1230 m long, had an entrance elevation lower than the other and had an underground gate controlled structure, accessible by a vertical shaft. This concept of one of the tunnels with a higher entrance without control structure is now a common practice in Brazil and is intended to save in concrete, equipment and of course in time because the logic is that it is closed with a soil embankment at the beginning of the dry season when the lower tunnel is capable to handle the river flows. This concept was first used at the Segredo Powerplant in the Iguaçu River, in 1992.

The diversion tunnels had a mushroom rectangular section, 13.0 m high by 13.2 m width, excavated in a crown and bench scheme. They had no lining except for localised shotcrete and rockbolting treatments.

4.2. Dam

The Irapé Dam is an earth-core rockfill dam, 208 m high, the highest in Brazil. Its crest elevation is 515.5 m and the crest length 551 m. The upstream slope is 1:1.5 up to El 484.0 and 1:1.3 in the remainder height. The downstream slope is 1:1.3 between the climbing road stretches, averaging 1:1.5. The maximum cross section of the dam is shown on Figure 3.
The material zones of the dam are indicated on Figure 3, and are described as follows:

1. Clay core
2. Filter (Natural sand)
3. Filter (Crushed sand)
3A. Fine transition
3B. Medium transition
5. Fine rockfill (Ø < 0.40 m)
5A. Medium to highly weathered rockfill
5B. Random. Highly weathered rock and saprolite
6. Slightly to medium weathered rockfill
6A. Slightly weathered to sound rockfill
7. Protection rockfill
9. Covering rockfill
10. Concrete block
11. Rockfill zone raised with dam crest at El. 475.00

A detailed description of the dam and of the studies carried out for its design, as well as the instrumentation used to monitor its construction performance is presented in Barhouch Aires (2006), Calcina et al. (2007) and Marques Filho et al. (2009).

The vertical dam core was designed with a mixture of clay and gravel up to about El. 407, that is inside the deep narrow channel, to ensure a more rigid material in this portion of the core. In addition, a layer of more plastic, self-healing soil was placed along the clay/rock contact on the valley channel walls to help the redistribution of stresses. Besides these measures, the lower part of the valley walls was trimmed by excavation to eliminate the stress-relief joints and minimise eventual arching problems. A concrete block, shown in Figure 3, was built to help expedite the construction work in this very difficult area. Above El. 407, the core was built, essentially, only with clay, to obtain a larger deformation modulus.

The rockfill shells used rock material obtained directly from the required excavation and from nearby deposits. The quality of these materials and the tightness of the construction schedule, and of course, the resulting economic impact of different alternative solutions, required an intense and minute monitoring of the design team, both from the engineers and from the Owner, to adapt the rockfill zoning to the performance and quality requirements. In this respect the use of sulphide rocks was deeply investigate both in Brazilian laboratories and in other projects where such problem existed (for example, Kangaroo Creek, in Australia). Non-sulphide rocks from a quarry located 5 km away from the dam, were used only for drains, filters, transitions and some external parts of the rockfill shells.

Foundation grouting was carried out using cement grout resistant to sulphides, with a low water-cement ratio, in which plastifiers, water reduction additives and micro-silica, were added. The use of anti-dispersion additives was limited to regions where the presence of water in rock fissures was confirmed.

The total volume of the dam is 10,300,000 m³. To comply with the schedule goals and overcome the difficult design and construction features, very high embankment production rates were necessary and were achieved. A production rate of about one million m³/month was maintained during many months. To allow start of reservoir impounding before reaching the final dam elevation, a feature required to achieve initial power generation on time, a downstream zone with a minimum width of 30 m was left to be raised in the downstream part of the dam section (see Figure 3) after the upstream part reached El. 475, which provided a safer margin to generate power at the minimum reservoir elevation.

4.3. Spillway and Intermediate Outlet

The original solution for the Irapé Project spillway was a conventional gate-controlled surface chute spillway, placed in the left bank and discharging in the river where...
it turns right in a almost 90º angle. However, because of
topographic configuration this solution required an
enormous amount of excavation, mainly in rock, which
would not only increase significantly the project cost,
but would prevent the achievement of the schedule goals
because it would be very difficult to synchronise
excavation and dam rockfill progress, and because
expensive intermediary rock stockpiles would be
necessary. The solution was to design a chute spillway
in which the chute was inside tunnels (actually two
tunnels, one for each gated passage), maintaining the
gate-control structure as an external feature, with
independent gravity blocks with a rock divider between
them and partially built against a downstream rock wall.
It allowed considerable savings in concrete quantities in
comparison with a conventional scheme. The chutes end
in flip-bucket structures, placed at an elevation of about
70 m above the normal water level of the river, and aligned
with the river stretch after the sharp curve mentioned
above. At the impact area of the issuing jet, a plunge
pool was excavated. Figure 4 depicts the longitudinal
profile of the surface spillway.

Besides the surface spillway, an intermediate outlet
was created, with a similar arrangement for the chute
and flip-bucket structure, but of course with a different
upstream control structure. The reason to provide the
intermediate outlet was related to the need of starting
reservoir impounding before completing the dam, that is,
creating a discharge device which provided safety and
which could be used if the impounding up to the dam
level - dependent of course on natural river flows -
occurred earlier than considered in the planning of the
construction work. The crest of the intermediate outlet
control floor was set at El. 449.40. The criterion for sizing
the intermediate outlet was to limit the maximum
probability of overtopping the partially completed dam to
0.1%. The closure of the diversion tunnels and start of
reservoir impounding occurred in November 2004 and the
intermediate outlet discharged safely the 2004/2005 wet
season floods with the partially built dam crest at El. 480.0.

The total spillway capacity of the combined surface
spillway plus intermediate outlet is 6,000 m³/s, which is
capable of damping the peak discharge of the
PMF hydrograph routed through the reservoir. The
maximum reservoir level reached during the occurrence
of the PMF hydrograph is El. 511, 50 m which is 4.0 m
below the elevation of the dam crest (El. 515.50).

Figure 5 depicts the plan and profile of the spillway
control structure and Figure 6 the control structure of the
intermediate outlet.

The control structure of the surface spillway contains
two independent water-passages controlled by radial
gates 11.0 m wide by 20.0 m high, with an ogee crest at
El. 491.0. The tunnel chutes are similar, and present a
slight convergence in plan, as shown in Figure 1.
The average length of the tunnels is 634.0 m with a slope
of 10.2%. The tunnel section is 12.0 m wide by 20.0 m
high at the portal and 11.0 m wide by 12.0 high in the
remaining length, with rectangular arch section. Photo 2
shows an upstream view of the entrance structures of
the surface spillway, the intermediate outlet and of the
power intake. The intermediate outlet has a 70-m high
tower structure and two radial gates, as seen in this figure.
The radial gates are 7.0 m wide by 9.62 m high and
close the passage 9.4 m high.

4.4. Power Waterway and Powerhouse

The generation circuit of the Irapé Project includes a
water intake, three power tunnels and an external
powerhouse for three generating units, each with a
133 MW rated power capacity.

The power intake is located at the right-hand side of
the intermediate outlet control structure, as shown in
Figure 2 and Photo 2. It is formed by a 59-m high tower
structure with three independent water passages,
equipped with emergency and service gates, with a water flow section 3.8 m wide by 5.1 m high.

The three independent power tunnels have a first portion as vertical shafts, 57 m long, followed by a 20 m curve and a 328 m long straight segment, with a slope of 12% descending into the external powerhouse. The initial 404 m of each penstock tunnel is lined with unreinforced concrete, 40 cm thick, with an internal diameter of 4.6 m. The remaining length of about 110 m is steel lined with an internal diameter of 3.8 m. The average total length of each penstock is 453 m. They daylight at a distance 8.5 m upstream of the centreline of the generating units. The criterion used for the hydraulic design of the waterway was based on the requirement of obtaining a net head for power generation of 158.50 m. At the initial section of the steel lined portion of the penstocks a grouting curtain and a drainage curtain were provided. In addition a drainage gallery with drainage holes connecting to the steel lined portion was provided as well.

The external powerhouse is a conventional concrete structure housing three vertical Francis turbine driven generating units. The service electrical and mechanical galleries are placed downstream of the units. These galleries house the main mechanical and electrical auxiliary and control equipment. The service bay is located at the right-hand side of the powerhouse in a structure which also contains the drainage pits and the auxiliary service transformers. Figure 7 shows a section of the powerhouse.

Photo 3 shows a general view of the powerhouse area.
Figure 7 - Irapé HPP - Powerhouse Section

Photo 3 - View of Irapé Powerhouse.
5. TECHNICAL FEATURES

Owner CEMIG (Companhia Energética de Minas Gerais)

Location
River Basin: Jequitinhonha
Distance from mouth: 566.9 km
Latitude: 16°44'17"S
Longitude: 42°34'29"W
County Right Bank: Berilo, Minas Gerais
County Left Bank: Grão Mogol, Minas Gerais

Hydrometeorological Data
Dam drainage area: 16,200 km²
Annual Aver. Rainfall (Basin): 700/1,300 mm
Annual Aver. Rainfall (Res. Zone): 900/1,100 mm
Annual Aver. Evap. (Reservoir): 1,315 mm
Mean Flow (1931 - 1988): 158.1 m³/s
Max. Flow (27/12/67): 3,930 m³/s
Min. Flow (30/09/89): 12.4 m³/s
Diversion Flows
Tr = 50 Years: 3,540 m³/s
Tr = 500 Years: 5,340 m³/s
Tr = 50 Years Dry Season: 1,240 m³/s
Tr = 500 Years Dry Season: 2,200 m³/s

Spillway
Design Flow: 6,000 m³/s
PMF (inflow): 11,446 m³/s

Reservoir
Min. Normal W.L.: 470.80 m
Max. Normal W.L.: 510.00 m
Max. Flood W.L.: 512.20 m

Floode Areas
Max. Flood W.L.: 144.03 km²
Max. Normal W.L.: 137.16 km²
Min. Normal W.L.: 59.99 km²

Storage Volumes
Maximum Normal W.L.: 5,963.92 x 10⁶ m³
Active: 3,695.98 x 10⁶ m³
Below Spillway sill: 3,790 x 10⁶ m³
Service Life of Reservoir: >300 years

Tailwater
Min. Normal W.L.: 330.20 m
Max. Flood W.L.: 340.60 m

Diversion
Tunnels: Bottom Tunnel with Horseshoe type section, 13 m in diameter in the vault and rectangular section of 10.8 m (W) X 5.2 m (H).
Top Tunnel with Horseshoe type section, 14 m in diameter in vault and rectangular section of 11.8 m (W) X 6.0 m (H).
Bottom Tunnel (with Control Structure): Inlet Sill: El. 327.00 m

Outlet Sills: El. 322.00 m
Length: 1,227.58 m
Top Tunnel (with no control structure): Inlet Sill: El. 350.00 m
Outlet Sills: El. 322.00 m
Length: 1067.50 m
Cofferdam Crest Elevations
Upstream Cofferdam: 376.00 m
Downstream Cofferdam: 342.00 m

Dam
Type: Rockfill with Clay Core
Crest Length: 540.00 m
Maximum Height: 205.00 m
Crest Elevation: 514.50 m

Spillway
Ogee
Type: Creager Profile with Radial Gate
Capacity: 4,000 m³/s
Sill elevation: 491.00 m
Number of gates: 2
Dimensions of gates: 11 m (W) X 20 m (H)
Chute
Type: Free Flow in Tunnel w/Ski Jump at El.395.00 m.
Section of Tunnel: Rectangular Arch 11 m (L) X 15 m (H) Transitioning to 12 m (W) X 11.4 m (H)
Aver. Length of Tunnels: 626.00 m
Dissipation
Type: Plunge Pool
Elevation of bottom: El.300.00 m

Intermediate Outlet
Control Structure
Type: Bottom controlled by radial gate
Capacity: 2,000 m³/s
Sill elevation: 450.00 m
Number of gates: 2
Dimensions of gates: 7.0 m (W) X 9.4 m (H)
Chute
Type: Free Flow in Tunnel with Ski Jump at El.395.00 m
Section of Tunnel: Rectangular Arch 17 m (W) X 15 m (H) Transitioning to 12 m (W) X 11.4 m (H)
Aver. Length of Tunnel: 622.00 m
Dissipation
Type: Plunge Pool
Elevation of bottom: El.300.00 m

Intake & Penstocks
Penstocks
Quantity: 3
Internal Diameter with Concrete Facing: 4.60 m
with Steel Facing: 3.80 m
Length

Diversion
Bottom Tunnel with Horseshoe type section, 13 m in diameter in the vault and rectangular section of 10.8m (W) X 5.2 m (H).
Top Tunnel with Horseshoe type section, 14m in diameter in vault and rectangular section of 11.8 m (W) X 6.0 m (H).
Bottom Tunnel (with Control Structure): Inlet Sill: El. 327.00 m

Outlet Sills: El. 322.00 m
Length: 1,227.58 m
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Cofferdam Crest Elevations
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Aver. Length of Tunnels: 626.00 m
Dissipation
Type: Plunge Pool
Elevation of bottom: El.300.00 m

Intermediate Outlet
Control Structure
Type: Bottom controlled by radial gate
Capacity: 2,000 m³/s
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Section of Tunnel: Rectangular Arch 17 m (W) X 15 m (H) Transitioning to 12 m (W) X 11.4 m (H)
Aver. Length of Tunnel: 622.00 m
Dissipation
Type: Plunge Pool
Elevation of bottom: El.300.00 m

Intake & Penstocks
Penstocks
Quantity: 3
Internal Diameter with Concrete Facing: 4.60 m
with Steel Facing: 3.80 m
Length
Stretch in Shaft 56.7 m  
Stretch in Vertical Curve 20.0 m  
Bottom Sub-horizontal Stretch 327.7 m  
In Concrete 110.0 m  
Shielded 20.0 m  
Intake 327.7 m  
Type Tower  
El. Crest 514.00 m  
El. Trashrack Sill 457.00 m  
Dimensions of Inlet 5.55 m (W) X 10.60 m (H)  
Fixed Wheel Gate  
Quantity 3  
Dimensions 3.8 m (W) X 5.1 m (H)  
Stoplog  
Dimensions 3.8 m (W) X 5.1 m (H)  

Powerhouse  
Generating Units 3  
Length of Assembly Area 25.00 m  
Width of Blocks of Units 1 & 2 13.50 m  
Width of Blocks of Unit 3 13.50 m  
Total Length 29.50 m  

Turbines  
Type Francis vertical axis  
Unit Power 125 MW  
Synchronous Rotation 300 Rpm  
Rated Unit Flow 85.22 m³/s  
Maximum Efficiency in power of 123.43 MW & Head of 172.20 m 95.48 %  

Generators  
Rated Unit Power/ Maximum 127/140 MVA  
Synchronous Rotation 300 Rpm  
Rated Voltage 13.8 kV  
Maximum Efficiency 98.5 %  
Rated power factor 0.95  

Time Schedule - Main Stages  
Start of Works April 2002  
River Diversion April 2003  
Closure of Diversion November 2004  
Commercial Generation 1st Unit Jul/2005  
Commercial Generation 2nd Unit Sep/2005  
Commercial Generation 3rd Unit Nov/2005  
Duration of Construction 40 months (to 1st Unit)  

Energy Studies  
Maximum Gross Head 179.10 m  
Maximum Net Head 174.50 m  
Reference Net Head 158.50 m  
Design Net Head 162.00 m  
Power of Plant 360.00 MW  

Firm Energy  
1st Unit 977,227 MWh Year  
2nd Unit 1,807,188 MWh Year  
3rd Unit 1,807,188 MWh Year  

6. ACKNOWLEDGMENTS  
The text and information presented in this paper were based on design documentation filed at INTERTECHNE and on published papers mentioned in the References, below, from which significant parts were freely transcribed without specific quotation. The Brazilian Committee on Dams acknowledges these sources and expresses its appreciation to the corresponding authors.  

7. REFERENCES  