CAMPOS NOVOS HYDROPOWER PLANT ON CANOAS RIVER

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1. INTRODUCTION

Campos Novos HPP is located on the Canoas River around 21 km upstream from where it joins the Pelotas River between Campos Novos and Celso Ramos counties in the State of Santa Catarina, around 380 km from Florianopolis.

This plant and its related transmission system, with 880 MW capacity, belongs to Campos Novos Energia S.A. - ENERCAN, a private Brazilian company incorporated in 1999 by the shareholders: CPFL-G Geração de Energia S.A, CBA Companhia Brasileira de Alumínio, CNT Companhia Níquel Tocantins, CEEE Companhia Estadual de Energia Elétrica and CELEC Centrais Elétricas de Santa Catarina S.A.

Joint venture Consórcio Fornecedor de Campos Novos (CFCN) - consisting of Construções e Comércio Camargo Correa S.A, Engevix Engenharia S.A, CNEC Engenharia S.A. and GE-Hydro Inepar do Brasil S.A. was contracted for its turnkey project to provide all goods and services for implementing the plant and its related transmission system.

2. DESCRIPTION OF PROJECT

The project, designed solely to generate electricity, consists basically of a concrete-faced rockfill dam, 202 m maximum height, and river diversion through two tunnels provided with control structures for closing. The surface spillway is on the right bank with four gates 17.40 (W) by 20.00 m (H). The generation system also on the right bank consists of a concrete gravity intake, followed by three power tunnels with internal diameter of 6.20 m in the reinforced concrete stretch and 5.50 m in the steel lined stretch. The indoor powerhouse is equipped with three vertical axis Francis hydro-generating units, with total capacity of 880 MW. Figure 1 below shows the general layout of the project.

Figure 1 - General Layout of Project
The main quantities involved in building the project are listed below:

- Common Excavation: 3,941,000 m³
- Rock Excavation: 10,890,000 m³
- Underground Excavation (5,000 m of tunnels): 585,000 m³
- Rockfill: 12,500,000 m³
- Transitions: 280,000 m³
- Concrete: 334,000 m³

3. GEOLOGY, GEOTECHNICS AND FOUNDATIONS

The entire course of Canoas River runs over rock on the Southeast edge of the Paraná sedimentary basin. The river flows from its head to mid-course through sedimentary rocks represented by sandstones, siltites and shales from the Botucatu Formation and Passa Dois Group. It then flows into volcanic rocks of the Serra Geral Formation. These rocks occur as lava flows and have a sub-horizontal spatial distribution, with 20’ dip NW. The total thickness of this package is 600 m to 800 m and overlaps the Botucatu Formation sandstones.

In the reservoir area, Canoas River and its tributaries follow a course strongly influenced by local fracturing. These zones of weakness cause the river to carve out a channel, resulting in a deep straight valley with very steep slopes to approximately El. 700, in which the ratio between the length of the crest and height of the dam (C/H) is 2.9, reflecting the steep valley. The relief above it is characterized by a rolling plateau, from medium to slight rolling the farther away from the river valley.

Two different flow sequences are found on the site of the Campos Novos HPP. One of tholeitic basalts distributed in eight main overlapping sub-horizontal flows called from G to N, between elevations 420 and 590, perfectly correlatable in the investigated area, with the exception of the secondary flows called G’, H’/H”, and K’ occurring in restricted areas. Most of the basalt flows are characterised, as a rule, by a dense basalt zone in the lower and central part, with some vesicules at the base and a vesicular amygdaloidal basalt in the upper part, with a layer of basalt breccia consisting of basalt fragments enveloped and cemented by secondary minerals or by silty-sandy materials at the top. The vesicular basalt at the base and breccia at the top are missing in some flows.

A single flow was found in the other sequence above El. 590, called F, unlike the others because of its intermediary to acid composition, and has been classified as dacite and rhyolite of the hyperaluminous series. This flow is distinguished from the others because of its thickness of over 100 m and the presence of various vitreous levels. Another outstanding feature is the occurrence of a "magmatic banding" consisting of alternating darker and lighter centimetre portions, sometimes with signs of oxidation and micro-fissures, and they may, in conjunction with the primary fracturing in these horizons, strongly affect the alteration of the rock mass. In this flow, deep vertical "soil boxes" are common, to around 30 m from the top of the rock, and ten-metre blocks immersed in residual soil.

4. HYDROLOGY, HYDRAULICS AND ENERGY STUDIES

The drainage basin of Canoas River at the Campos Novos HPP site covers an area of 14,200 km². The long-term average flow is 298.5 m³/s. The maximum monthly average flow found in the historic period (1931-1997) was 3,243 m³/s in July 1983 and the minimum monthly flow 22.9 m³/s in January 1945.

Canoas River has a certain seasonality with regard to the probability of occurrence and magnitude of floods, and the hydrologically wettest period occurs between May and October. Figure 2 shows the 10 largest average daily flows during the historic period for each month of the year.

For the dimensioning of the diversion works, determining cofferdam elevations and risk assessment during the diversion works, a frequency study was undertaken of the annual maximum levels that would be reached, considering the river diversion through the tunnels.

The result of the studies was a design of two tunnels and a cofferdam for the dam construction on the river bed with a protection for the passage of the flood with a 20-year recurrence period in the wet season (approximately 5,500 m³/s), equivalent to protection against flooding of more than 200-year recurrence period in the dry season.

In addition to the flood frequency study, to determine the design flood of the spillway the Probable Maximum Precipitation (PMP) was calculated and its later transformation into Probable Maximum Flood (PMF).
The PMP calculation in the Uruguay River basin adopted the methodology recommended by the World Meteorological Organisation (WMO). The resulting Probable Maximum Flood hydrogram (PMF) for the project site showed a maximum peak of 18,870 m³/s, considerably higher than the 10,000-year flood peak (14,890 m³/s). As a criterion, the probable maximum flood was adopted for dimensioning the spillway.

The energy studies helped define the operating conditions, number of units and installed capacity, resulting in the adoption of three Francis 203.3 MW turbines under a head of 185 m for total installed capacity of 880 MW, with average generation of 3,310 GWh/year.

5. MAIN STRUCTURES

5.1. General Layout

The plant layout basically consists of:

- The river diversion through two tunnels excavated in rock on the right bank, 14.5 m in diameter and 16 m in height, in an arched-trapezoidal section with gate structure at the outlet, and a cofferdam on the riverbed for floods with a recurrence period of 1:20 years, corresponding to a peak flood of 5,500 m³/s.
- Concrete-faced rockfill dam with crest at El. 666 and a parapet wall at El. 667.00, 202 m in height, 576 m in length and a total volume of 12.5 million cubic metres; Spillway with four radial gates 17.40 m span by 20 m in height, on the right bank of the river, designed to discharge the probable maximum flow of 18,300 m³/s, with a 5 m surcharge above normal water level.
- The generation circuit consists of one intake, partly embedded into rock, followed by three power tunnels around 380 m long and a gross head of 180 m. The powerhouse, with access yard at El. 500 is indoors, consisting of three blocks for the Francis turbines and an installation area over a total length of 113 m and maximum height of 47.80 m. The construction of the conventional outdoor substation is planned on the left bank on a plateau at El. 735 m.
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- Single-bus switchyard and transmission line as far as the outlet and inlet regions of the tunnels to protect the tunnel excavation works and the concrete pouring of the diversion intake structure.

After concluding the excavation and intake concrete pouring, the diversion began through the tunnels by closing off the river when the pre-cofferdams in rockfill were laid. After fully closing the pre-cofferdams the cofferdams were heightened and at the same time the water was pumped into the coffer area, to permit the construction of the dam in the dry.

Each tunnel has a concrete intake structure with crest at El. 526.00 m, foundation at El. 474.50 m and a maximum height of 51.50 m. Each intake has three 4 m wide openings with base at El. 479.00 m and ceiling at El. 493.00 m.

All openings were fitted with two rows of slots for sliding gates, the one downstream designed for installing stoplogs, which will be responsible for the actual closure, and the one upstream for operating the auxiliary fixed wheel gate for inspection of the guides and helping in emergency manoeuvres of stoplogs.

The diversion tunnels were dimensioned so that the first stage of the dam designed for a 1,500-year flood event would be at a height of approximately half the final dam height.

In order to build the dam, two cofferdams were built, one upstream and the other downstream from the dam axis, both with a rockfill section with an impervious clay zone.

The main upstream cofferdam was built with its crest at El. 516.00 m, providing protection against floods of up to 20-year recurrence intervals. The slope of the rockfill dam has a gradient of 1V:1.2H and that of the compacted earthfill 1V:2.5 H.

The downstream cofferdam, with crest at El. 489.00 m, is incorporated to the main dam. Its section consists of rockfill embankment and an impervious zone downstream. The impervious earthfill above the water level was compacted with the traffic of the actual transport equipment. The external slope of the rockfill dam has a gradient of 1V:1.3H and the compacted fill above El. 479.00 m, 1V:2.50H.

5.3. Dam

The main dam structure of Campos Novos Hydropower Plant is a concrete-faced rockfill dam (CFRD) 590 m in length (L) and maximum height (H) 202 m. The L/H ratio is 2.92 and an indication of the steepness of the valley, with the abutments having average gradients of close to 45°.

The total rockfill volume from excavating basalt rock was 12.5 million cubic metres, dumped and compacted over a sound rock foundation. The upstream shell has a slope of 1(V):1.3(H), while the downstream shell has an average slope of 1(V):1.4(H), while the effective gradient of the slopes is 1(V):1.2(H) between the access berms. Figure 3 shows the cross-section of the dam with distribution of the materials and grain size increasing from up to downstream.

The rockfill embankment was zoned so that the third upstream and the central region of the dam - materials called 3B and 3D - were compacted in 1.0 m thick layers watered at a ratio of 200 l/m². The void ratio of these zones was an average of 0.22 and density = 2.14, the unconfined compression tests showed that
the strength of the material forming the rockfill was higher than 75 MPa. The third downstream, consisting of 3C/3D materials was compacted in unwatered layers of 1.6 m, the average density of this zone was 2.02. Transitions under the slab - materials 2B and 3A - were compacted in 0.50 m layers and reached void ratios of 0.20.

For protecting the processed transition and bedding for concreting the upstream face an extruded concrete facing was done concomitantly with the transitions, at an average cement rate of 75 kg/m³.

The upstream face slabs each are 16 m wide, covering a total area of around 105,000 m², with thickness varying from 0.30 to 1.00 m, adopting the following equations used worldwide:

- \( e (\text{cm}) = 0.30 + 0.0020 \, H (\text{m}) \quad H < 100\text{m} \)
- \( e (\text{cm}) = 0.0050 \, H (\text{m}) \quad H > 100\text{m} \)

The reinforcement rates adopted for the concrete area of the slab were distinguished as shown in Figure 4 below.

Reinforcement Rate and Distribution:
- Area A: Double reinforcement (in the two faces - top and bottom) at a 0.5% Ac rate in both directions (horizontal and vertical);
- Area B: Reinforcement in the central region of the concrete at rates of 0.3% Ac in the horizontal and 0.4% Ac in the vertical direction.

All vertical joints are protected by copper waterstops and are placed at the slab base, and in the region of tension (abutments), the joints are coated with a PVC cover with filler, as shown in Figure 5. The perimetral joints of the plinth are also in copper and a PVC cover with filler.

Analyses in a 3-D mathematical model in finite elements were prepared before filling the reservoir in order
to foresee deformations in the rockfill embankment and actual dam face.

5.4. Spillway

The outflow system is located on the right bank, and the water is returned downstream from the tailrace canal, consisting of an approach channel, spill structure and chute ending in a flip bucket (see Figure 6).

The spillway was designed for a discharge of 18,300 m$^3$/s to permit evacuation of the probable maximum flood with the water level surcharge in the reservoir to EL 665.00 m, with a specific flow of 263 m$^3$/s/m.

The approach channel is around 730 m in total length, the first 635 m with the bottom at EL 635.00 m and the last 65.00 m with floor at EL 630.00 m, to reduce the velocity of the flow next to the control structure.

The control structure consists of four adjacent blocks separated by joints to form a set of four 17.40 m wide bays, where tainter gates are installed, between pillars 4.50 m in width. The ogee crest was set at EL 640.00 m and the top of the structure at EL 666.00 m. A gallery was planned at EL 629.60 m next to the foundation close to the upstream face to undertake a grout and drainage curtain.

The chute downstream of the control structure is fully concrete-lined, 83.10 m in width and 92.85 m in length.

The stilling pool was excavated with the bottom at EL 455.00 m, 70.00 m in length towards the flow, 92.00 m in width.

5.5. Headrace and Intake

The intake consists of a hollow gravity structure, with maximum height of 32.00 m and a sloping upstream face with 1V:0.18 H. The intakes are separate for the three generating units and each of the blocks has two openings due to the existence of central pillars for support of trashracks. Each headrace is provided with two rows of slots for gate installation, the one downstream corresponding to the emergency gate and the upstream to the stoplog.

The role of the fixed wheel gate is to protect its generating unit by closing the intake under any flow conditions of the turbine.
5.6. Power Tunnels

The power tunnels were excavated in rock and concrete lined. In their final stretch the tunnels are steel lined to resist the forces from the hydraulic transients from the operation of the units.

The excavation of the vertical stretch has a circular section with a 7.20 m radius and around 155 m long. The initial excavation was done using raising boring, with the later widening by excavation in benches, with the material being removed from below. The horizontal stretch has a horse-shoe section with diameter of 7.20 m.

The final stretch of the power tunnels is steel lined, including a conical reduction to a 5.50 m diameter over a length of 101.50 m. The armouring was designed to resist a maximum external pressure of 0.35 MPa, when the conduits are empty, i.e. without internal hydrostatic pressure. Figure 7 shows a longitudinal section of the generation circuit.

5.7. Powerhouse

The powerhouse is indoors, with three blocks for the generating units, each 18.00 m wide lengthwise and 32.75 m cross-wise, and houses three Francis vertical axis hydraulic turbines, with rated power of 300 MW and rated velocity of 200 rpm, adapted to direct coupling to three-phase synchronous generators of 311 MVA.

The maximum height of the structure is 47.80 m, measured from the foundation of the draft tube elbow at El. 452.20 m, to the transformer yard at El. 500.00 m. The erection area, with floor at El. 479.10 m, is an extension of the generator hall, consisting of two blocks situated on the left of the generating units, each with the same dimensions defined for them.

Loads inside the structures are shifted with the help of two overhead travelling cranes with separate capacities of 2,800 kN, a rail track at El. 491.00 m and span of 17.25 m.

The unloading area is situated to the left of the installation area, but at El. 500.00 m, coinciding with that of the transformer platform. Unloading operations are carried out using an overhead travelling crane, whose rail track at El. 500.00 m extends all along the powerhouse roof and installation area. Lowering the equipment to the installation area is done using an opening in its roof, provided with a mobile hatch, and using the unloading gantry crane.

The same crane shifts the stoplogs in the draft tubes and lowers equipment to the electromechanical galleries situated on the upstream side, through two opening at the ends of the transformer platform using downstream and upstream cantilevers.

The electromechanical equipment is set out on four floors inside the powerhouse, situated at El. 470.15 m (mechanical gallery), 479.10 m and 485.85 m (bottom and top electrical galleries) and 492.80 m (electromechanical gallery), in addition to the transformer platform at El. 500.00 m. Vertical access to the galleries is by means of steps, one at each end or a lift only on the left side. A control building was also built upstream from the powerhouse above the level of the access yard.
The dewatering and drainage sumps are on the right of the generating units in a stretch between the crane rail tracks. The pump house for these sumps is over them based at El. 467.5 m.

Access to inside the powerhouse is through a reception hall in the unit 1 block, over the galleries at El. 500.00 m. Next to the reception is the dispensary. From there access to the upper gallery is by lift and staircase, as in the control building.

The tailrace canal is fully excavated in rock, 54.00 m in width.

6. CONSTRUCTION

A major challenge in building the Campos Novos HPP was the creation of access routes to carry out the diversion works, since the canyon where the plant was built was completely inaccessible with almost perpendicular slopes, and it was only possible to reach the river bank after five months work.

Another basic point in planning the works was to be able to divert the river having previously built the plinth on the two abutments. This condition was necessary, bearing in mind that the diversion would be at the end of the wet season (October 2002) and it was also necessary to reach the elevation of the first stage of the dam (around 100 metres high) before the end of the dry season (April 2003), over a six-month period or so.

The rockfill of the dam was carried out in three stages and the face slabs were concreted in two stages. First the rockfill was raised upstream to El. 570 m and the concrete slabs to El. 568 m. This heightening corresponds to approximately 52% of the final height of the dam and to the recurrence interval against floods of 1:500 years. Photo 1 shows a view of the dam’s construction during the slab construction in this 1st stage.

Next, the downstream rockfill was raised to El. 570 and later complemented to El. 660.

After completion of the rockfill, the second stage of slab concreting began and was concluded in February 2005, around eight months before starting to fill the reservoir. Photo 2 shows a view of the heightened dam and the start of the concreting the slabs in the second stage.
The rockfill of the dam was compacted using 12-ton vibrating rollers. The average production of compacted rockfill was 700,000 m³/month, while the slab strips 16 m wide were carried out at an average speed of 2.9 m/hour.

Photo 3 shows the complementation of the dam crest after concluding the face slabs, by placing precast parapet walls upstream and downstream, and later filling the spaces between them with in situ concrete in the case of the upstream wall.

At the end of the construction period, the obtained average values of the deformability modules were as follows:
- Upstream zone rockfill \( E = 60 \text{ MPa} \)
- Central zone rockfill \( E = 50 \text{ MPa} \)
- Downstream zone rockfill \( E = 30 \text{ MPa} \)

7. OCCURRENCES IN SLABS OF DAM

Filling the plant lake began on 10 October 2005. When the reservoir was filled to El. 642 m, around 80% of its maximum depth, it was observed that the compression joint between slabs 16 and 17 of the face had broken. An underwater investigation was performed with divers and a robot, by which it was discovered that the failure extended to a depth of more than 100 m.

The concrete and joint above the water level were repaired and below the water level clay-silt material was dumped to seal the joint and the damaged concrete of the slabs; this initial dumping reduced the seepage flows from 14 m³/s to around 0.8 m³/s.

Lowering the lake as a result of the accident in the diversion tunnel 2, helped view the configuration of the cracks, which were mapped as shown in Photo 4. The description of the accident in the diversion tunnel 2 and recovery works are presented in the book on river diversion published by CBDB Brazilian Committee on Dams.

The cracks appearing in the compression zone of the slabs showed spalling caused by excess compression in the cross and lengthwise directions to the slab faces. This phenomenon also occurred in Mohale and TS-1 dams in South Africa and China, respectively.

Prior to re-filling the reservoir, the damaged slab strips and joints were fully recovered. Although the embankment had undergone almost all reservoir head during the first fill, and therefore had mobilized most of its deformation, four vertical joints were opened in the central compression region in order to permit horizontal displacements between slabs in order to prevent high stress levels.

8. ECONOMIC, ENVIRONMENTAL AND SOCIAL ASPECTS

The Basic Environmental Project (BEP) was prepared in accordance with the precepts and objective in the EIA/RIMA environmental reports, and as recommended
by the Santa Catarina State environmental agency FATMA. The BEP defined a series of programs and projects to be implemented during the construction and operation of the project.

The programs comprising the BEP are being implemented by forming partnerships with the population and relevant public agencies. Some of the programmes developed worth mentioning are: environmental education, preservation and rescue of plant and animal species - by creating nurseries and planting more than 240,000 native seedlings around the reservoir, relocation and compensation for families, historic and cultural heritage of the towns that have areas affected by the project, implementing a Conservation Unit and developing a Plan for Environmental Conservation and Use of the Reservoir Surroundings. One of the programmes in the Basic Environmental Project (BEP) of Campos Novos Hydropower Plant is to implement a Conservation Unit. The result of this programme was the implementing of the River Canoas State Park. The area for forming the park - 1200 hectares, equal to 18,000 football fields, was donated by Enercan to the Santa Catarina state government. The company was seeking to implement the Conservation Unit in the actual region where the Campos Novos HPP was built, on the banks of the River Canoas, close to Barra do Rio Ibicuí in the county of Campos Novos. Before buying it, Enercan undertook an ecological assessment that points to characteristics that made the area ideal for installing a park, among which is the presence of endangered plant species, such as the araucaria, Brazilian walnut and soft tree fern (Dicksonia sellowiana), and more than 350 species of fauna.

9. PERFORMANCE OF PROJECT

The instrumentation plan for the dam embankment planned to install 28 settlement cells and four magnetic settlement meters to help determine the deformability modules of the different zones in the rockfill.

The upstream face was instrumented with electro-level sections in the central region and on the right bank, with triorthogonal joint meters between the plinth and slab and with joint meters between slabs, plus surface benchmarks placed at the top of the dam. Figure 8 shows the instrumentation installed in the face and the downstream surface benchmarks.

The initial plan for monitoring the Campos Novos dam was the installation of two main instrumentation sections, which were placed at stake 13+10m (riverbed) and st. 21+10m (left bank). In addition to these two instrumented sections, measuring instruments were installed in the concrete face, as shown in Figure 8.

Overall 133 instruments were installed to monitor the performance of the rockfill and concrete face of the dam, such as settlement cells, multiple extensometers, triorthogonal joint meters, electro-levels, surface
Due to lowering of the reservoir as a result of the accident in the diversion tunnel, cracks were exposed resulting from excess axial compression the central slabs. After recovering the affected slabs of the dam face, new instruments were installed in the concrete face to monitor the performance of the repaired stretches during filling.

Overall more than thirty instruments were installed, with special mention to 13 joint meters to monitor the closure of the central joints between the slabs 16 to 20 created to absorb the deformations when the reservoir is being refilled, seven concrete stress meters to assess the stress levels which they would undergo again and an extra row with ten electro-levels placed in the right abutment of the dam to assess the deflections of the face in this abutment. The following graph (see Figure 9) shows the settlements over time for the instrumented section on the riverbed at st. 13 + 10.00.

The settlements undergone by the dam in the second impounding of the reservoir varied between 4% and 10% in relation to the preceding period, considering the readings of the settlement cells until April 2008 and the rate is gradually dropping with a tendency to reach stabilization, as is typical of similar rockfills. Slow deformation (creep) in the central part of the dam was 6 mm/month, average between April 2007 and April 2008. Fifteen months after its second filling, the dam behaviour is fully satisfactory.

The powerhouse, drainage gallery of the power tunnels and spillway were also instrumented using rod extensometers, triorthogonal joint meters between blocks, piezometers, thermometers and stress meters, in addition to the direct pendulum in the powerhouse, which showed normal behaviour during the construction, fill and operating period.

Photo 5 shows an overview of the Campos Novos Hydropower Plant fully operating.
Figure 9 - Settlements measured by Settlement Cells

Photo 5 - Campos Novos HPP, in operation
10. TECHNICAL CHARACTERISTICS

General
Site Canoas River between the Santa Catarina towns of Celso Ramos and Campos Novos
Starting year August 2001
Finishing year January 2006
Owner Campos Novos Energia S.A. - ENERCAN
Designer Engevix Engenharia S.A., CNEC Engenharia S.A.
Civil contractor Construções e Comércio Camargo Correa S.A.
Manufacturers and assembly firms GE-Hydro, Inepar do Brasil S.A.

Basic data
Drainage basin area 14,200 km²
Annual average precipitation 1,500 mm
Annual average temperature 16º C

Reservoir
Area at maximum normal level 32.9 km²
Total storage volume 1,472 hm³
Active storage volume 129 hm³
Length 58 km
Maximum width 600 m
Maximum Normal water level 660.00 m
Maximum Flood water level 666.00 m
Minimum water level 655.00 m

Tailrace canal
Maximum Normal water level (3 units at full load) 480.10 m
Maximum Flood water level 496.70 m
Minimum water level (1 unit in operation and Machadinho reservoir depleted) 475.00 m

Flows
Average inflow 298.50 m³/s
Maximum flow recorded (July 1983) 8,303 m³/s
Minimum daily flow recorded (September 1974) 27 m³/s
Maximum diversion flow and return time (TR~50 years) 6,461 m³/s
Probable Maximum Flood (peak inflow) 18,870 m³/s

Dam
Type concrete-faced rockfill
Length 590 m
Height 202 m
Crest elevation 660.00 m
Crest width 10.00 m

Spillway
Type surface with gate control
Length: 94.00 m
Width 83.10 m
Capacity 18,300 m³/s
Maximum specific discharge 263 m³/s/m

Spillway gates
Type tainter
Quantity 4
Dimensions 17.40 (W) x 20.00 (H)
Width 17.40 m
Height 20.00 m
Manufacturer GE Hydro-Inepar

Intake
Type hollow gravity
Length 51.50 m
Maximum height 32.00 m

Intake gates
Type fixed wheel
Quantity 3
Dimensions
Width 6.20 m
Height 6.50 m
Manufacturer GE Hydro-Inepar

Diversion tunnels
Quantity 2 tunnels
Type and dimensions of section horseshoe section with 14.50 m (W) x 16.00 m (H)
Length of Tunnel 1 860.90 m
Length of Tunnel 2 915.80 m

Power tunnels
Type excavated in rock
Quantity 3
Internal diameter 6.20 m
Vertical stretch - height 173 m
Horizontal stretch - unit length 211 m

Steel lined stretch
Diameter 5.50 m
Length 85 m
Manufacturer GE Hydro-Inepar

Powerhouse
Type indoor
Height 47.80 m
Length (including service area) 113.00 m
Installed capacity 880 MW
### Turbine
- **Type**: Vertical axis Francis
- **Quantity of units**: 3 units
- **Rated power**: 300 MW
- **Rated head**: 175.60 m
- **Maximum discharge of unit**: 186 m³/s
- **Rated velocity**: 200 rpm
- **Manufacturer**: GE Hydro-Inepar

### Generator
- **Type**: synchronous
- **Rated power**: 311 MVA
- **Voltage**: 13.8 kV
- **Frequency**: 60 Hz
- **Rotation**: 200 rpm
- **Manufacturer**: GE Hydro-Inepar

### Step-up transformers
- **Quantity**: 3 units
- **Type**: three-phase force
- **Rated power**: 330 MVA
- **Rated voltage - upper winding**: 230 2 x 2.5% kV
- **Rated voltage - lower winding**: 13.8 kV
- **Manufacturer**: WEG

### Bibliography
