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AAR MANAGEMENT AT PAULO AFONSO IV POWER PLANT - BRAZIL

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ABSTRACT

The Paulo Afonso IV Power Plant, which is part of the Paulo Afonso Hydroelectric Complex, was built between 1975 and 1979 on the São Francisco River, near the Paulo Afonso falls. The underground powerhouse is equipped with six 410 MW Francis turbines and began operation in December of 1979.

The concrete structure of the Paulo Afonso IV Power Plant is undergoing a process of expansive reaction which is seriously affecting the generating units. An investigation program was established, in January 1995, to study remedial measures to mitigate the expansion effects.

Several laboratory tests were made with concrete samples extracted from the structure and with the aggregates used in the construction as well. A reduction of about 34% in the elastic modulus of the concrete affected by the AAR was detected. In situ stresses were measured in the concrete structure using the overcoring process. The maximum compressive stress found was near 1 MPa. A monitoring system of the concrete structures was designed and installed. The first year of observation points to a mean vertical expansion rate of 35 $\mu\varepsilon$/year.

A 3D mathematical model of the underground powerhouse, taking into account a stress-dependent expansion law, was developed and calibrated using the electro-mechanical equipment monitoring data. The cutting of expansion slots in the powerhouse was studied through this model.

Keywords: Alkali-aggregate reaction, mathematical model, monitoring, concrete laboratory tests, expansion slots, stress measuring, power plant

INTRODUCTION

The Paulo Afonso IV hydroelectric power plant belongs to the CHESF power generation complex, located on the São Francisco River, near the Paulo Afonso falls, at the border of Bahia and Alagoas states. It consists of an underground Powerhouse, measuring 210 m long, 24 m wide and 54 m height, equipped with 6 generating units having Francis type turbines with rated horsepower of 410 MW per unit and total head of 112.5 m. This hydroelectric development was built during the period 1975-1979. The reservoir was filled-up in August 1979 and the start-up of the six machines occurred between December 1979 and May 1983. This was the last power plant built in the Paulo Afonso complex, which also comprises the Paulo Afonso I, II, III and Apolônio Sales (Moxotó) plants.

The coarse aggregate used in PA-IV was obtained from the necessary excavations. The cements were the Portland type, with equivalent alkali content between 0.6 % and 1 %. Because pumped concrete was used in the construction of the Powerhouse, the average cement content was of 360 Kg/m^3, that is, about 3 Kg of alkali per cubic meter of concrete. The aggregate maximum size was of 19 mm, with a water/cement ratio of 0.53. Quartzose natural sands were used as fine aggregates. (Cavalcanti et al. 1997).

The first evidences of the alkali - aggregate reaction in the PA-IV Power House date from 1985, resulting from a situation of cracks in the concrete structure, mainly in the walls of the generator hall and in the main floor slab.

The control measurements performed on the electromechanical equipment, usually at each 9,000 hour period of operation (~1 year), show the following typical problems of power plants affected by the AAR, which have presented a certain evolution through time and have been more intense in the GR1 unit and less severe in the GR6: Inclination of the turbine-generator shaft, turbine's cover out of level, variation of the vertical gaps and inclination of the wicket gates, ovalling of the turbine and generator pits and reduction of clearances in the guide and support bearings.

The verification of the alkali-aggregate reaction in the de PA-IV concrete was carried out by the CHESF's consultant Mr. Mielenz, in September 1988, detecting evidences of AAR in two samples of concrete taken from the drainage gallery. The results of the petrographic examinations showed deformed quartz as the reactive mineral, although with an incipient stage of reaction.

INVESTIGATION PROGRAM

Laboratory Tests

The AAR research at laboratory level was carried out on concrete samples removed from the structures through rotating boring. A few samples were extracted with diameter of 150 mm, mainly to establish the concrete strength and strain parameters, while most of the samples have a 75 mm diameter, using the boring holes provisioned for installation of the multiple extensometers.

Petrographic tests - In the first stage, petrographic tests were performed, as per ASTM C 856-83 (Furnas: DCT.T.04.002.98. 1998), in order to accurately define the concrete aggregates, evaluate porosity or voids in the mortar, check existence of gel resulting from reaction, check concrete internal microcracks and dark rims around the aggregate particle. Six 75 mm diameter samples from the holes of vertical extensometers in the GR1 to GR6 units, located at the left/downstream region near the spiral case, were examined. (Table 1). No dark rims or microcracks were found in the aggregate.

TABLE 1: Results of the Petrographic Tests of Concrete Samples

Sample	Coarse aggregate	Filling of the voids	Reactive Mineralogy	U E A *
GR1 el. 113m	Granite biotite and microgranite biotite	Totally or partially filled	Strained quartz and weathered feldspar	$< 35°$
GR2 el. 130m	Microgranite biotite and amphibolite	Empty or lined with vitreous material	Strained quartz and weathered feldspar	$26°$ to $31°$
GR3 el. 130m	Granite	Empty or lined with white material	Strained quartz and weathered feldspar	$< 45°$
GR4 el. 130m	Granite	Empty or lined with white material	Strained quartz and weathered feldspar	$< 50°$
GR5 el. 113m	Granite and biotite gneiss	Empty or lined with white material	Strained quartz and weathered feldspar	$< 45°$
GR6 el. 130m	Granite and amphibolite biotite gneiss	Empty	Strained quartz and weathered feldspar	$< 45°$

* Undulatory extinction angle of the quartz crystals

Alkalis Content- Tests to determine the alkali content were carried out both for total and for soluble alkali content on the samples used for the petrographic tests. The results (Table 2) showed soluble alkali content between 0.06 % and 0.12 %, indicating a heterogeneous reaction intensity.

The methods used were the ASTM C. 114 for the total alkali content and the procedimento 1.02.36 adopted by the Furnas Laboratory for the soluble alkali content. The samples preparation was made according to the same method described by Grattan-Bellew (Grattan-Bellew et al. 1992).

TABLE 2: Total and Soluble Alkali Content (alkaline equivalent in Na2O)

Sample	GR1 el. 113m	GR2 El. 130m	GR3 el. 130m	GR4 el. 130m	GR5 el. 113m	GR6 el. 130m
Total alkali (%)	2.51	2.54	3.09	2.89	3.00	2.15
Soluble alkali (%)	0.10	0.10	0.12	0.06	0.09	0.09

Aggregate Potential Reactivity - The determination of the aggregate potential reactivity was performed through accelerated reaction tests, in mortar bars and test specimens prepared from the concrete cores extracted from the powerhouse, from the boreholes of vertical extensometers and from the drainage gallery. Tests with mortar bars were carried out according to the ASTM C-1260/94 method. The cement used had an alkali content of 0.92% mix of 1:2.25 and water/cement ratio of 0.53. The results, (Table 3), showed expansions lower than 0.1% even after 30 days, which may indicate that samples have less available silica.

TABLE 3 - Expansion of Mortar Bars (%)

Sample	12 days	15 days	18 days	21 days	24 days	27 days	30 days
GR1 el. 113m	0.02	0.02	0.03	0.04	0.05	0.05	0.05
GR2 el. 130m	0.02	0.03	0.04	0.04	0.05	0.06	0.06
GR3 el. 130m	0.03	0.03	0.04	0.05	0.05	0.06	0.07
GR1 el. 117m	0.03	0.04	0.05	0.05	0.05	0.06	0.06
GR5 el. 117m	0.03	0.03	0.04	0.04	0.05	0.06	0.06

The test specimens prepared with 75 mm ∅ concrete cores, were stored at a temperature of 38 °C under two different conditions: 100% of relative air humidity or in a 1N solution of NaOH. Test specimens length variations were measured up to the age of 1 year (Furnas, DCT.T.1.094.97. 1997).

TABLE 4 – Expansion of the Concrete Test Cores (%)

Sample	Condition	30 days	60 days	90 days	120 days	150 days	180 days	360 days
GR2	100% RH	0.07	0.07	0.07	0.04	0.04	0.04	0.04
el. 130m	NaOH	0.07	0.07	0.07	0.07	0.07	0.07	0.09
GR3	100% RH	0.02	0.02	0.02	0.02	0.02	0.02	0.02
el. 130m	NaOH	0.02	0.02	0.02	0.02	0.02	0.02	0.03
GR4	100% RH	0.05	0.04	0.04	0.04	0.03	0.04	0.03
el. 130m	NaOH	0.06	0.06	0.05	0.05	0.05	0.07	0.08
GR5	100% RH	0.06	0.06	0.05	0.05	0.05	0.05	0.04
el. 112m	NaOH	0.05	0.06	0.05	0.05	0.05	0.06	0.09
GR6	100% RH	0.05	0.05	0.05	0.04	0.04	0.04	0.04
el. 130m	NaOH	0.05	0.04	0.04	0.04	0.04	0.04	0.05

By the results shown (Table 4) and considering the expansion limit of 0.04% at 360 days for tests with 100% relative humidity and at 180 days for tests in NaOH, the GR3 sample is considered innocuous in both tests. The content of soluble alkali of this test sample (Table 2) was the higher one, confirming a less intense reaction. All the other samples had moderate reaction.

Damage rate – In order to evaluate the stage of reaction and the resulting damages, the procedure proposed by Grattan-Bellew was used. Determination of the damage rate was performed utilizing a concrete polished surface, prepared by cutting samples taken from the structure and examined using a stereo-binocular microscope, with 16 magnifying power (Grattan-Bellew et al. 1992).

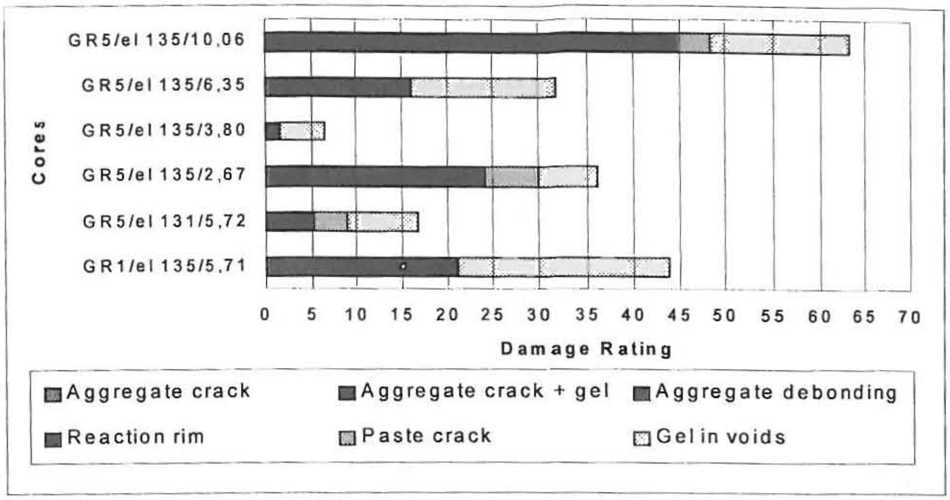

Fig. 1: Damage rate of concrete cores

The 75 mm diameter samples were taken from the boreholes for installation of the horizontal extensometers in units GR1 and GR5. Results are shown in Figure 1 (Furnas, DCT.T.4.014.98. 1998). Compared to the Canadian dams Saunders (Di = 35 ~ 148), Beauharnois (Di = 100) and Lady Evelyn Lake (Di = 213), where high damage ratings indices were observed, which are able to classify the PA-IV Powerhouse structure (Di = 7 ~ 63) , as not much affected by AAR, in terms of damage, up to date. The damage rating index scale ranges from Di = 17, in a core taken from a 40 years old wall made with a granitic gravel aggregate, with no visible cracks, up to Di = 213 in a core taken from old Lady Evelyn Lake dam, containing a very reactive argillite aggregate (Grattan-Bellew et al. 1992). This dam was completed in 1927, removed and replaced in 1973 due to the deterioration extent (Thomas et al. 1992).

Mechanical Properties – In order to establish the Young's Modulus and compressive strength, concrete test cores were used.. The following average values were obtained (Furnas, DCT.T.1.094.97.1997):

Test	Average	Standard Deviation	Variation Ratio	Number of tests
Young's Modulus (GPa)	19.86	2.38	11.96 %	12
Compressive Modulus (MPa)	35.90	3.60	9.97 %	7

Tests showing the original Young's Modulus value for this concrete are not available. However, a correlation between compressive strength values and Young's Modulus for other concrete mixes used for PA IV (Table 5), allows estimating the expected Young's Modulus value if concrete were not affected by AAR (Fig. 2).

Chart of Figure 2 shows value close to E= 30 GPa for f'c=36 MPa. The value found for the Young's Modulus aged approximately 20 years, shows a 34 % decrease in relation to the expected value. Compressive strength apparently was not affected.

TABLE 5 – Properties of Some of the Concrete Types Used for PA IV

Mix	Cement (Kg/m^3)	W/C Factor	D max (mm)	Age (days)	E (GPa)	fc (MPa)
175B76III	185	0.65	76	90	21	20.0
210B76IIIa	220	0.58	76	90	23	21.4
150B38IVa	281	0.50	38	28	27	30.0
185B19VIIIa	357	0.53	19	28		25.9

Note: Mix 185B19VIIIa was used for the PA IV Power House .

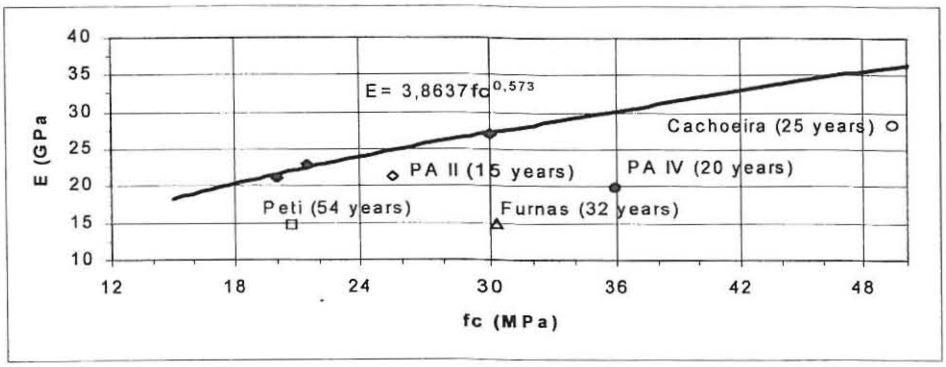

Fig. 2: Correlation between compressive strength and Young's modulus of PA IV concrete

The chart shows also the values for the PA II concrete, Furnas (Castro et al. 1997), Peti (Magalhães et al. 1997) and Cachoeira (Kuperman et al. 1997), all of which affected by AAR.

Stress Measurement

Systematic stress measurements were made for the concrete structure at points located close to the joint between units GR1 and GR2 and between unit GR2 and the assembly area. Boring hole SR1 is located in unit GR1, 3 m downstream the power room axis and 2.6 m from the joint between units GR1 and GR2. The SR2 hole is located in unit GR2, 3 m downstream the power house axis and 2.45 m from the joint with the assembly area. The SR3 hole is located in unit GR2, 10.32 m downstream the power house axis and 2.5 m from the joint between units GR1 and GR2. Three tests were carried out for each hole (STT1, STT2 and STT3) at elevations 132 m, 131.5 m (horizontal axis of the spiral case) and 131 m.

Tests were carried out on boring holes EX (37.4 mm) with concentric overcoring SW (143.5 mm) using the STT - Stress Tensor Tube, developed by the Laboratório Nacional de Engenharia Civil from Lisbon. This technique used for determining stresses is based on the measurement of the deformation caused by stress relief through 9 extensometers distributed in three 3 rosettes along the circumference of the tensor tube. The elasticity parameters of the material were determined through tests carried out on test samples taken, using biaxial chamber and uniaxial compression tests in a rigid servo-controlled press.

TABLE 6 - Intensity and Orientation of Principal Stresses

Hole	E (GPa) ν (Poisson)	Test	Principal stresses (MPa, + compression) and orientation (degrees)					
			σ1	α1/β1	σ2	α2/β2	σ3	α3/β3
SR1	21.00	STT1	0.841	325/3	0.500	232/3	-0.164	265/83
		STT2	0.585	336/1	0.423	228/5	0.148	1/86
	0.17	STT3	0.715	113/5	0.474	198/8	-0.028	339/80
SR2	18.60	STT1	1.032	286/4	0.513	191/3	-0.09	71/80
		STT2	0.796	124/0	0.714	211/11	0.235	327/78
	0.17	STT3	1.061	169/52	0.727	62/14	0.381	323/33
SR3	20.00	STT1	1.210	107/27	0.672	11/37	0.244	138/44
		STT2	0.774	322/7	0.539	233/9	-0.127	280/87
	0.17	STT3	0.759	345/11	0.570	243/0	0.124	101/64

Note: α is the angle with the power house axis in degrees, clockwise, from the left bank
β is the angle with the vertical axis.

Test samples taken from SR2/STT3, SR3/STT1 and SR3/STT3, presented sometime after being taken, longitudinal cracks, and this may explain the discrepancy in values referring to orientation of the principal stresses in these points. (Furnas. DCT.T.4.030.97. 1997).

We may conclude that the smaller principal stress (σ3), sometimes tensile but always with low intensity, is noticeably vertical; the other two are compression stresses and practically horizontal with the largest one (σ1) near 1MPa.

Monitoring

The instrumentation plan of the concrete structures was designed to find out the concrete expansion rates and the abnormal changes in structure dimensions around the generating equipment. The attention was concentrated in the observation of the concrete expansion rates, in the hydroelectric power plant longitudinal, vertical and horizontal directions, mainly using simple and multiple extensometers, totaling 15 instruments, being 6 vertical, 6 horizontal and 3 longitudinal extensometers.

The joints between blocks were instrumented with 12 triorthogonal joint meters located at el 140 m and el 144 m. The benchmarks (12 units) were installed from the generators floor (el. 144 m) and will be used for measurement of expansion vertical displacements through accurate leveling from topographic stations located at the rock mass wall. (Cavalcanti et al. 1997)

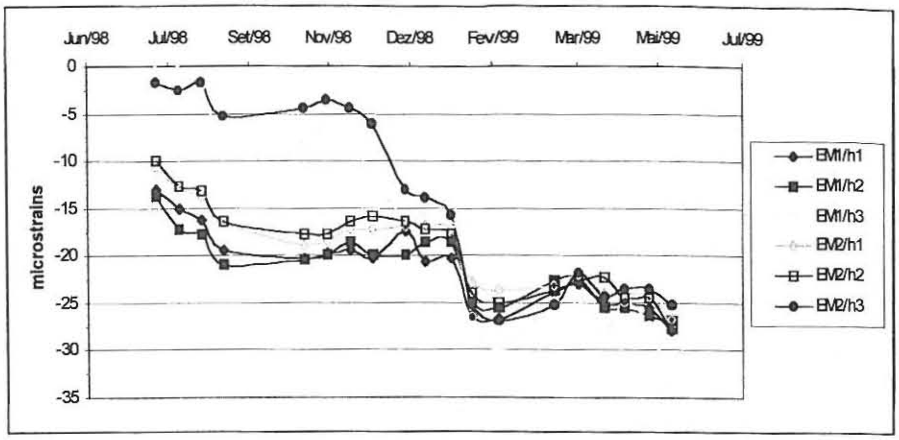

Fig. 3: Concrete expansion measured by vertical multiple extensometers EM1 and EM2

Figure 3 shows concrete expansion measured by vertical extensometers at the GR1 (EM1) and GR2 (EM2) units. Average annual expansion rates were 12 $\mu\varepsilon$ for EM1 and 15 $\mu\varepsilon$ for EM2, considering the average between rods 1 and 2. Rods 1 and 2 are anchored immediately below and above the concrete/rock contact and suffer the effects of the restriction caused by the rock pillar between the units along almost half of its length. Rod 3, which is not subject to such restriction has annual rates of 36 $\mu\varepsilon$ at EM1 and 34 $\mu\varepsilon$ at EM2. A longer observation period would be required for better evaluation.

MATHEMATICAL MODEL

For research into diagnostic and corrective solutions, tridimensional mathematical models were prepared and processed through the ANSYS 5.2 program, comprising the GR1 and GR2 units and a part of the assembly area, penstocks and the encircling rock massif. The discretization adopted resulted in a total of 8552 nodes, 5661 plane elements SHELL type, 22462 solid elements SOLID type and 1499 linear elements BEAM type, totaling about 40.000 degrees of freedom. (Cavalcanti et al. 1997)

In order to simulate a concrete expansion rate depending on the stress condition, a stage by stage analysis was made, for which it was assumed as valid the various effect overlapping principle and considering as one year the duration of each stage. The analysis were carried following the iterative method, in the elastic regime, stage by stage, obtaining a concrete expansion rate as a function of the value of the calculated stresses up to the previous stage, in every structure element and considering the three Cartesian directions.

A 30% reduction in the concrete Young's Modulus after 30 years of expansion was adopted, that is, a rate of 1% per year. This is done to take into consideration the reduction of the Young's Modulus, observed in structures subject to the AAR action. During this time the value of the Poisson coefficient was kept constant.

It was also simulated the concrete visco-elastic behavior, through the variation of the Young's Modulus as a function of time and the relaxation of the stresses in the concrete. Relaxation curves families were obtained, calculated from creep curves and the fields of relaxed stresses were calculated, at the ages when loads occurred, adding them to the ages under study. The function used by the Bureau of Reclamation: was adopted in this study for concrete creep:

$f(k,t\text{-}k) = 1/E(k) + F(k).Ln(1+t\text{-}k)$ [1]

As calibration factors we used the Young's Modulus for concrete at 1 year of age (E), expansion rate without confinement ($\in o$) and limit stress (Su) obtaining a good adjustment with the following values: E =30 GPa, $\in o$ =100 $\mu\varepsilon$/year and Su = 4 MPa.

Because at the beginning instruments measures were not available , a calibration between theoretical and "in situ" measured results was adopted, using for such purpose the displacements supplied by electromechanical measurements, obtained during the equipment maintenance operations. The main data used for such calibration was the unevenness, observed in the turbine's cover, the inclination of turbine shafts and variations in vertical gap of the pre-distributor vanes.

CORRECTIVE ACTIONS

The opening of expansion joints in the structures affected by AAR has been used frequently, in the last years for mitigating the problems caused by the concrete expansion. The main purpose of opening these joints is the relief of stress accumulated in the direction perpendicular to the joint plane or to allow the expansion of the concrete in a more symmetric manner, reducing in this way the undesirable effects. However, the relief of these stresses will cause an intensification of the concrete expansion, in the region affected by the reduction of stresses, that has to be taken into consideration when choosing this type of solution. (Charlwood et al.1995).

Four alternatives of isolated or combined expansion slots were studied: Transverse expansion joints between generating units and longitudinal expansion joint along the interface between the concrete of the generating units and the rock downstream the cavern. The most promising alternative to reduce problems at the generation equipment is the opening of the longitudinal expansion joint downstream between the generating units and the rock mass. More detailed studies, using a mathematical model based on the instrumentation data and on the concrete sample tests must be made for a final evaluation of the effects of opening the expansion joints. (Cavalcanti et al. 1997)

CONCLUSION

The investigations made for the concrete of the UHE PA IV have shown that the Alkali-Aggregate reaction process caused a reduction of 34% in the Young's modulus of the concrete, after 20 years. Maximum compressive stress values measured were 1 Mpa. The average annual expansion rate in the vertical direction during the first year of operation of the extensometers, was 35 $\mu\varepsilon$.

The study of the corrective actions shall be based on the analysis of stress conditions and displacements of the structure, from tridimensional mathematical models, simulating in the most realistic way, the rheology of the materials involved, as well as the AAR process. Within this approach it is of paramount importance the simulation of the concrete creep and the degradation of its elastic properties, as well as the dependency between the concrete expansion and the restraining stresses.

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