dam world 2018 Third International Dam World Conference
BRAZIL · Foz do Iguaçu · September 17-21, 2018

ANALYSIS AND INTERPRETATION OF THE STRUCTURAL BEHAVIOUR OF BOUÇOAIS-SONIM DAM DURING THE FIRST FILLING OF THE RESERVOIR AND THE FOLLOWING 10 YEARS OF OPERATION

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Keywords: Bouçoais-Sonim dam, First filling of the reservoir, First period of operation, Monitoring, Thermal and structural modelling, Finite element method

Abstract. *This paper presents the relevant monitoring data and the main results of the analysis and interpretation of the structural behaviour of Bouçoais-Sonim dam, a medium-size gravity concrete dam of 42.5 m high, during the first filling of the reservoir, in March 2005, and the subsequent 10 years of operation.*

As usual, the dam safety control is performed by considering the data provided by the monitoring system, which is analysed and interpreted with the support of numerical models. These activities are particularly relevant during the first filling of the reservoir, which is a real load testing, and during the first years of operation, when the dam's body and its foundation adapt to the local serviceability conditions.

A three-dimensional finite element model of the dam and of the rock mass foundation, considering the concrete viscoelastic behaviour and the variation of the reservoir water level and temperatures, allows simulating numerically the thermal and structural dam behaviour. The dam's concrete delayed behaviour was represented by the Bažant and Panula's basic creep function, evaluated from ordinary concrete laboratory tests.

The monitoring results, namely the displacements measured through plumb-lines, exhibit a good agreement with those obtained by the structural analysis, showing that the observed response is coherent with the evolution of the main actions.

1 INTRODUCTION

1.1 Dam's characteristics

The Bouçoais-Sonim dam integrates the Bouçoais-Sonim hydroelectric scheme, which is located just after the Rebordelo scheme, in the north of Portugal, on the Rabaçal river, a tributary of the Tua river that is, in its turn, tributary of the Douro river (Figure 1).

The scheme mainly consists of the dam, the hydraulic circuit and the powerhouse. The headrace tunnel has a length of 1350 m and a slope of 0.7%. The tunnel was opened on the river right bank, with no need of concrete lining in part of its length due to the characteristics of the rock mass. The powerhouse is located in a zone where the riverbed

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widens, being equipped with two power units of horizontal axis, with a nominal flow of 11.0 m³/s and capacity of 5000 kW, each.

The scheme was designed by COBA[1] and the current owner is Pebble Hydro, a company of the EDP group.

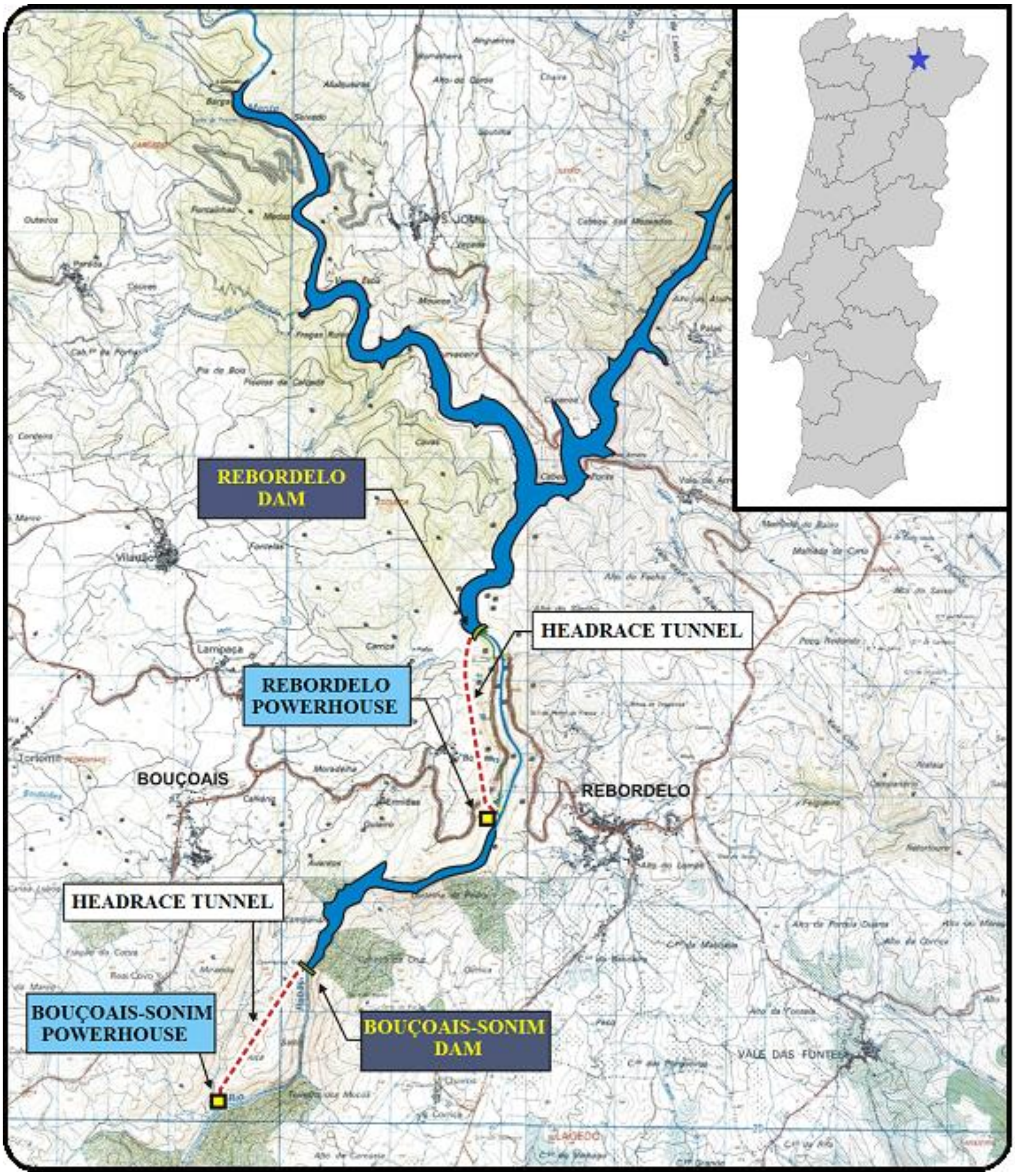

Figure 1: Location of the Bouçoais-Sonim and Rebordelo hydroelectric schemes

During the design phase, simplified geotechnical surveys were carried out finding a rock mass with adequate characteristics for the dam foundation between 4 and 15 m, in depth, along the insertion surface of the dam on the foundation. At the construction stage, greater depth was reached at the central zone of the valley which generating considerable volumes of excavations and the consequent increase of the dam high and of the site cast concrete volumes (see Figure 2).

The concrete dam is of the gravity type with straight axis, maximum high of 42.5 m, crest length of 91 m and thickness of 4.0 m at the crest. The downstream surface has a slope of 0.80 (H):1 (V), while the upstream surface is vertical in the 20 upper meters and has a slope of 0.30 (H):1 (V) from there to the dam heel. The crest and the retention water level (RWL) are located at levels 341.00 m and 334.00 m, respectively.

The central part of the dam's body contains the an uncontrolled spillway (Figure 3) with overflow ogee Creager profile that ends through a ski jump bucket for conveying the discharged water back to the river. The outlet discharge channel is convergent, in the horizontal plan, having a length of 45.25 m at the spillway crest and 32.00 m at the terminal section of the ski jump bucket. The spillway opening is divided in three spans by two piers that support a prestressed concrete walkway.

The reservoir has a volume of about 1.4 hm^3, for the retention water level.

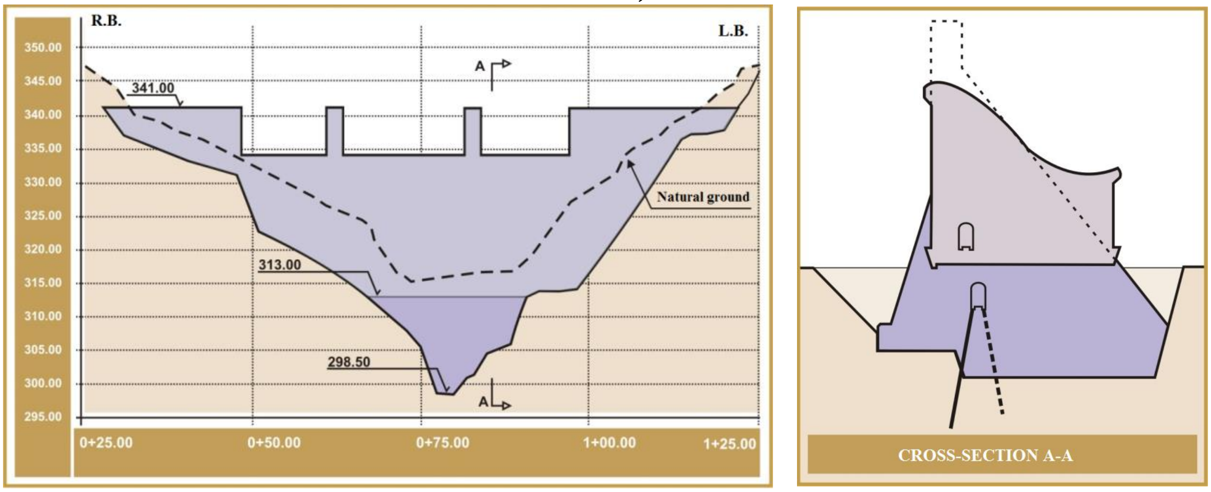

Figure 2: Bouçoais-Sonim dam. Schematic longitudinal profile and cross-section

Figure 3: Downstream view of Bouçoais-Sonim dam

The construction works of the Bouçoais-Sonim dam started at November 2002, with the opening of the accesses to the construction site, and have taken about two years of duration. The excavations were performed between May and September 2003, while the concreting of the dam blocks began at October 2003 and was concluded in August 2004. The foundation treatment, consisting in the execution of a grouting waterproofing curtain, have been performed between August and November 2004. The installation of the dam monitoring system has been finished late January 2005.

The dam concrete (C12/15 strength class) was produced with Portland lime-based cement, fly ashes, granitic aggregates (with maximum size of 100 mm) and the additive Melcret PF75, used for improving the concrete compactness and impermeability. The strengths obtained at the 180 days of age, by performing tests on cubic samples of 38 mm wet-screened concrete (with edges of 15 cm), were of 25.2 MPa and 20.1 MPa for the mean and characteristic values, respectively.

1.2 Monitoring and safety control

The dam monitoring system allows the evaluation of the actions and the determination of the structural and hydraulic responses, by measuring: i) the reservoir water level, by a staff gauge and a continuum water level recorder; ii) air temperatures, by a thermograph and a maximum-minimum thermometer (both located 3 Km upstream, at the Rebordelo dam); iii) uplifts, by seven piezometers and by five manometers installed in drain holes; iv) horizontal displacements, by two coordimeter bases installed in the gallery next to the extremes of the inverted and direct plumb lines; v) vertical displacements, by 5 marks of precision geometric levelling in the dam's crest, and by a two-rod extensometer installed from the gallery into the mass rock foundation; vi) joint movements using three joint meters; and vii) discharged and infiltrated flows, by drains and seepage measuring weirs.

The first filling plan defines the monitoring and safety activities for being performed during the filling period, which is a real load testing of the dam and its foundation, and requires a close continuum following up for the early detection of any misbehaviour. However, for dams with small water reservoirs in high flow rate rivers, the first filling plan is of very difficult execution. In fact, in November and December 2004, the river high flows have filled the Bouçoais-Sonim dam reservoir without control, due to the small release capacity of its bottom outlet (8.1 m^3/s). This uncontrolled filling did not cause any apparent damages in the structures and in the foundation. The first formal filling occurred later on, from the 1st to the 15th of March 2005, with the reservoir water at levels 323.00 m and 334.00 m (RWL), respectively. The filling occurred very rapidly in only 15 days, due to the river high flow rates[2].

The LNEC intervention on the Bouçoais-Sonim dam has begun during its construction phase in February 2004, when the concreting works were already ongoing. LNEC has participated in a set of meetings and surveys to the construction site, has revised the monitoring plan and has supported some works related to the installation of sensors, of the monitoring system. LNEC has followed up the first filling of the reservoir and has composed the digital archive of the monitoring data, which has been used for the safety control of the dam in the subsequent years of operation. In addition, the safety control of the dam has involved periodic surveys to the dam site and to the monitoring system. For analysing and interpreting the monitoring data, a thermal and structural finite element model was developed, whose main characteristics and obtained results are presented in the following sections[3].

2 STRUCTURAL AND THERMAL MODELS

2.1 Finite element models

The continuum finite element model used for interpreting the dam behaviour was elaborated by using software developed at LNEC, for both thermal[4] and structural[5] components.

The transient thermal model allows to compute the temperature distribution across the dam's body over time, by considering air and water temperatures in the external dam boundaries. No specific tests were performed for characterizing the thermal properties of the concrete, being its thermal diffusivity estimated of about 0.13 $m^2 day^{-1}$, based on the reference values of the constituent materials[6]. Also in the absence of specific tests, the concrete thermal expansion coefficient was taken equal to $1.0\text{x}10^{-5}$ $^{o}C^{-1}$.

For the structural model, the material of the rock mass foundation was considered linear-elastic with Young's modulus of 20 GPa and Poisson ratio of 0.2, while the concrete of the dam's body was simulated by using a viscoelastic model[7], characterized by Poisson ratio of 0.2 and by the Bažant Panula expression (1), plotted in Figure 4-a) for three ages of loading, which was estimated based on the data of the concrete composition together with the 28 days' compression strength results, obtained through laboratory tests on wet-screened concrete samples.

$$J(t,t_0) = \frac{1}{38.0}\left(1 + 5.2(t_0^{-0.44} + 0.046)(t - t_0)^{0.12}\right) \quad (\text{GPa}^{-1}) \tag{1}$$

By considering the creep function (1), the evolution of the elasticity modulus is given by expression (2), being graphically represented in Figure 4-b).

$$E(t_0) = \frac{38}{1 + 3{,}945(t_0^{-0.44} + 0{,}046)} \quad (\text{GPa}) \tag{2}$$

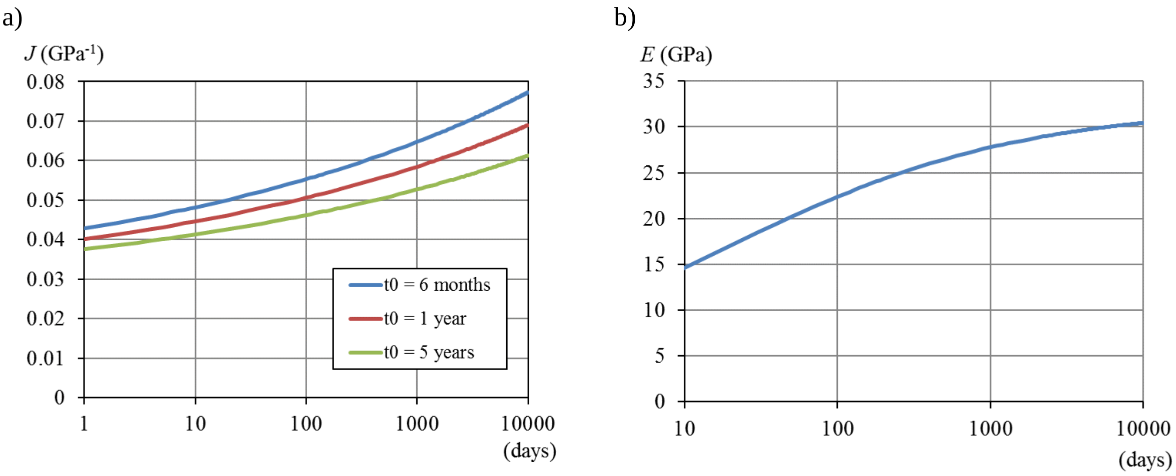

Figure 4: Estimated deformability of the concrete of dam's body: a) Bažant Panula creep function for three ages of loading; and b) evolution of the elasticity modulus

The used finite element mesh (Figure 5), for both the structural and the thermal models, has a total of 14929 nodal points for 3017 volumetric 20-node hexahedral finite elements, from which 902 belong to the dam's body and the remaining to the rock mass foundation. For the time discretization was used a time step of two-weeks.

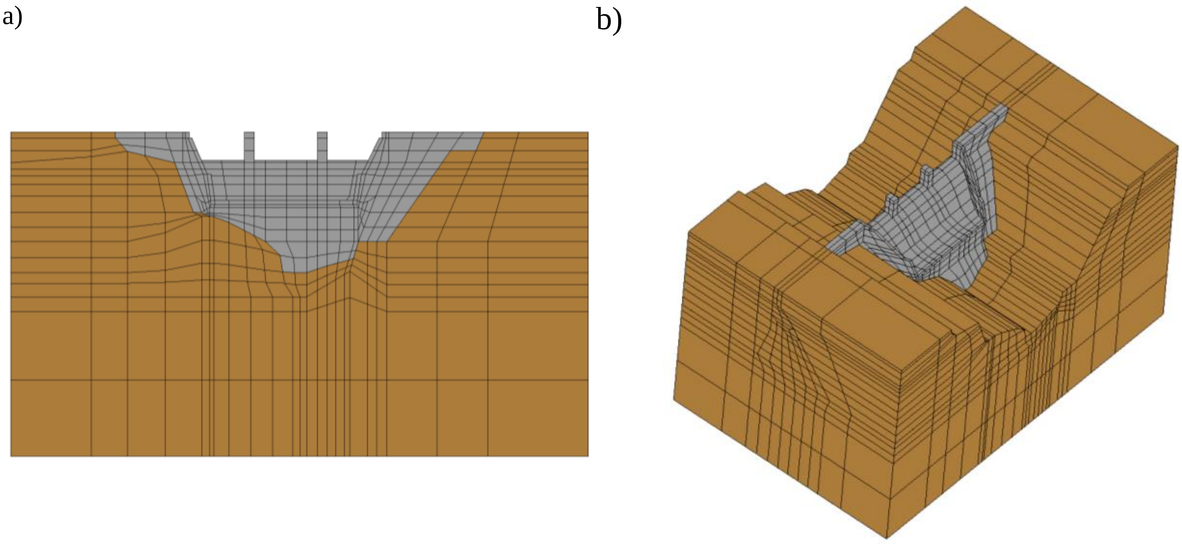

Figure 5: Finite element mesh of the dam's body and its rock mass foundation: a) Downstream orthogonal view; and b) 3D perspective

2.2 Loads

The dead weight of the dam's concrete was represented in the structural model by means of vertical body forces (γ_c=24 kN/m^3). The construction evolution was not considered in the analysis, therefore the dam's weight was applied from the beginning of the computations.

Regarding the water pressure, it was discretized in time according to the selected time step (two-weeks) and applied to the structural model by using surface loads (γ_w=10 kN/m^3), acting on the dam's upstream surface (Figure 6). No uplifts were considered in the insertion surface nor in the rock mass foundation because its variations were small over time.

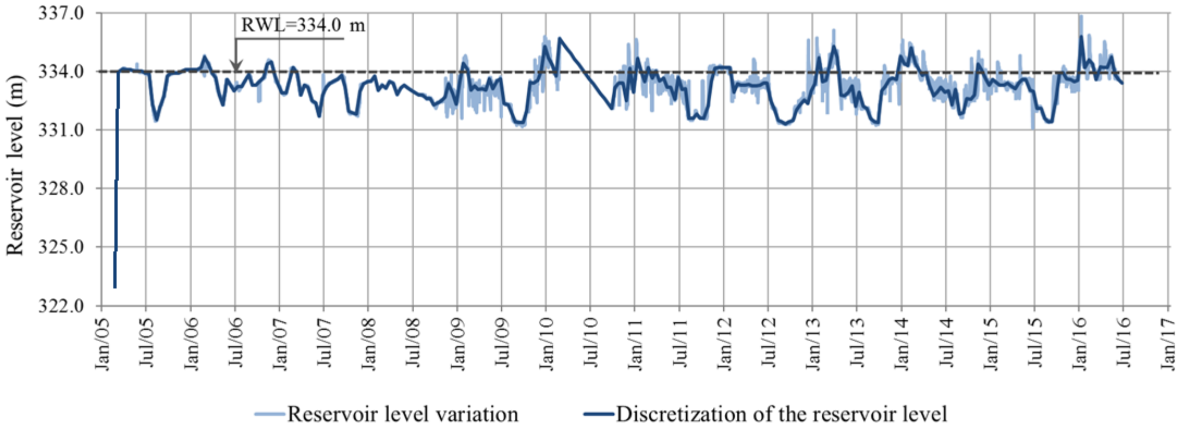

Figure 6: Variation and discretization of the reservoir water level

The thermal actions were represented by sinusoidal waves of annual period for the air and water. The wave parameters of the air temperature were numerically determined by the least square method based on the mean daily temperatures observed on air. The adjusted wave curve for the air temperature, plotted in Figure 7, is characterized by a mean annual temperature of 14.5 ºC and a semi-amplitude of 9.2 ºC.

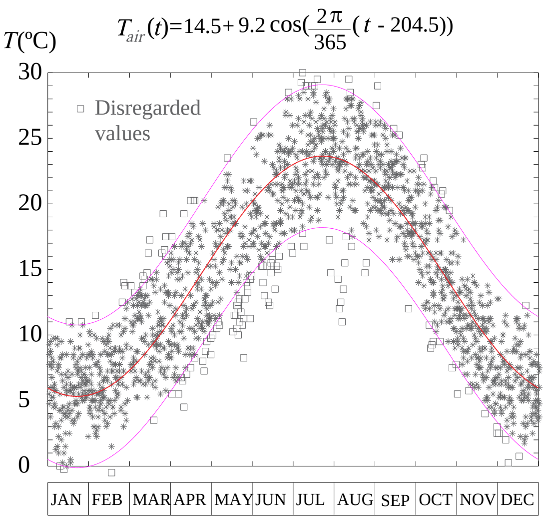

Figure 7:Annual thermal wave of the air evaluated from the monitoring data (is also represented a band defined by two standard deviations)

Regarding the water temperature, only two curves were considered in the thermal model, since the temperature gradient in depth was found to be small: i) at the surface, the water temperature was considered equal to the air temperature; and ii) for 5 m in depth, an additional curve was manually adjusted for best fitting the results of the measurements performed by LNEC in the reservoir water, between 2005 and 2014 (Figure 8). Therefore, in the thermal model, the temperature variations through depth were admitted linear from the surface to 5 m in depth and constant from there to the bottom of the reservoir.

For solving the transient thermal problem, the boundary conditions are updated at each time step according to the estimated sinusoidal waves for the air and water. Then, the computed temperature variations across the dam's body (obtained with the thermal model) were used in the structural model, at each time step, for computing their structural effect.

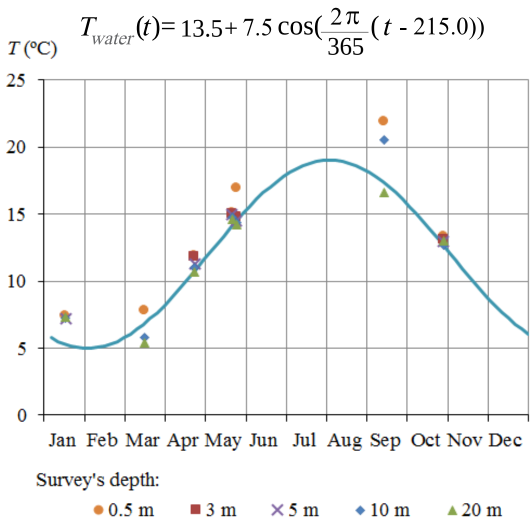

Figure 8: Annual thermal wave of the water at 5 m in depth, adjusted manually for best fitting the results of the measurements performed to the reservoir water at different depths

3 INTERPRETATION OF THE STRUCTURAL BEHAVIOUR

3.1 Dam thermal response

The thermal field at the beginning of the first filling (late February 2005) was predicted by performing a previous transient analysis in which the initial temperatures, across the domain, were considered equal to the mean annual air temperature. After obtaining stabilized values at the first filling beginning, the thermal field was computed, for each time step, until the end of 2016.

Because of the small variations of the reservoir water level and of the cyclical character of the thermal action, the computed fields of temperature are rather similar for the different years. Figures 9 and 10 show the results obtained on the 15th of March and the 15th of September 2015.

Both figures show that in zones close to the dam's core, the temperature variations over time are small. On the other hand, in zones close to the dam surfaces, the thermal gradients throughout the year are much higher. The upstream surface, being submerged during the larger part of the year, goes through smaller thermal variations relatively to the downstream surface.

a) **b)**

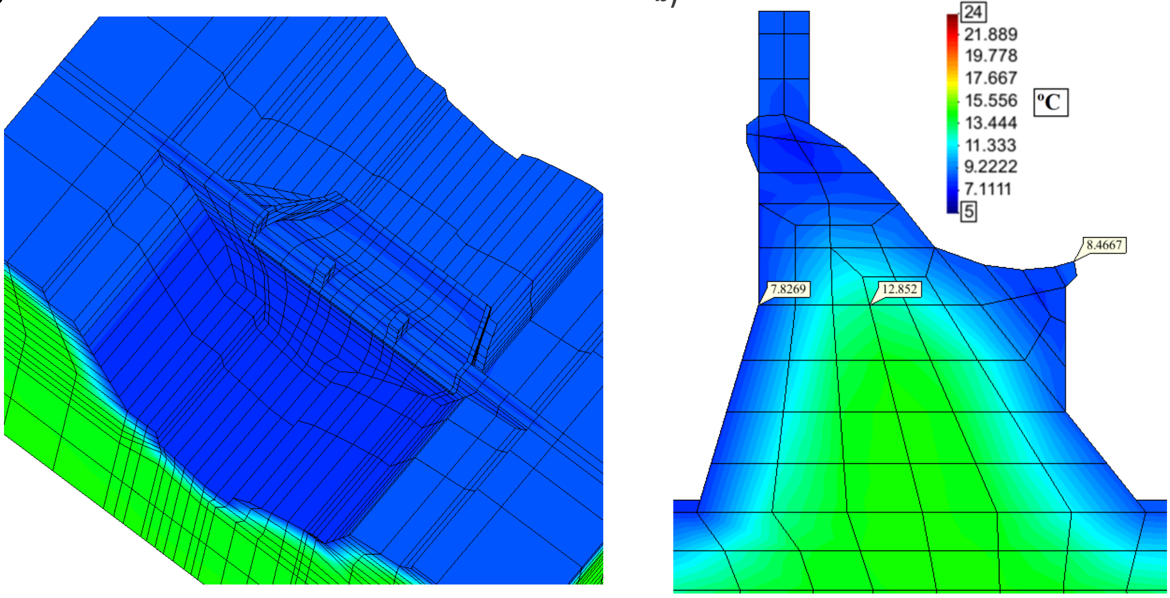

Figure 9: Thermal field on the dam's body by the 15th March 2015: a) 3D perspective; b) Cross section

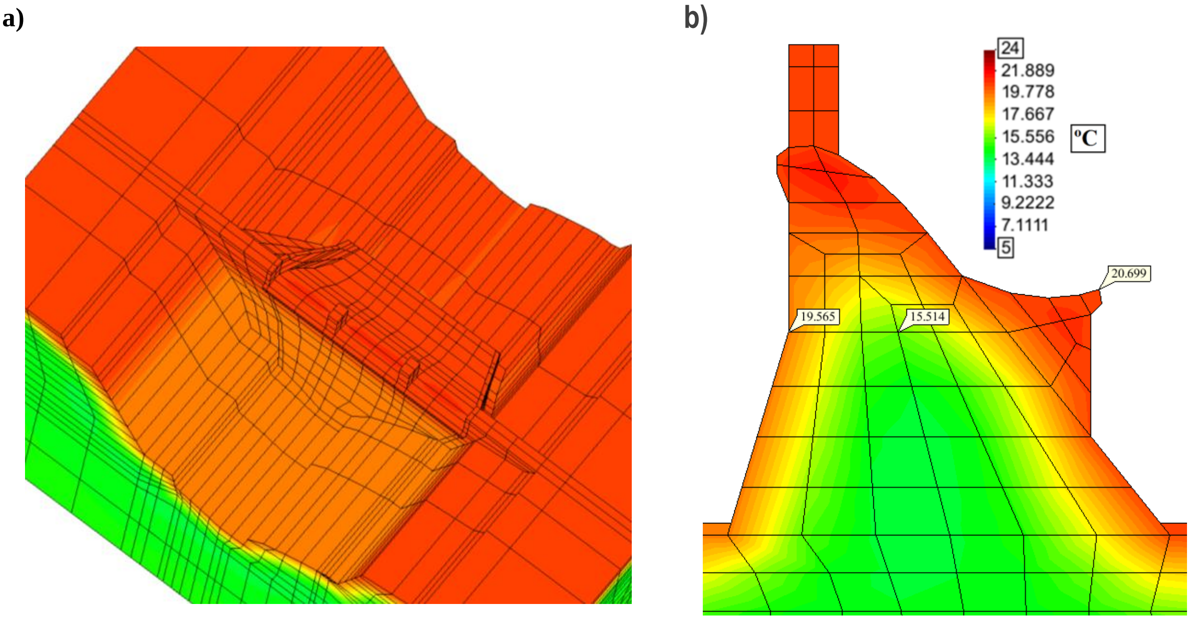

Figure 10: Thermal field on the dam's body by the 15th September 2015: a) 3D perspective; b) Cross section

The temperature fields computed with the thermal model for the period of analysis allow the computation of the temperature variations to be applied in the structural model, for each nodal point of the dam's discretization and for each time step.

3.2 Dam structural response

The results computed with the structural model are used for interpreting the data provided by the dam's monitoring system. Figure 11 shows the evolution of the displacements, in the riverbed direction, observed in the plumb-line at elevation 336.55 m, along with the values computed with the numerical model, considering the combined loads of the dead weight, the water pressure, the concrete thermal variations and their corresponding delayed effects.

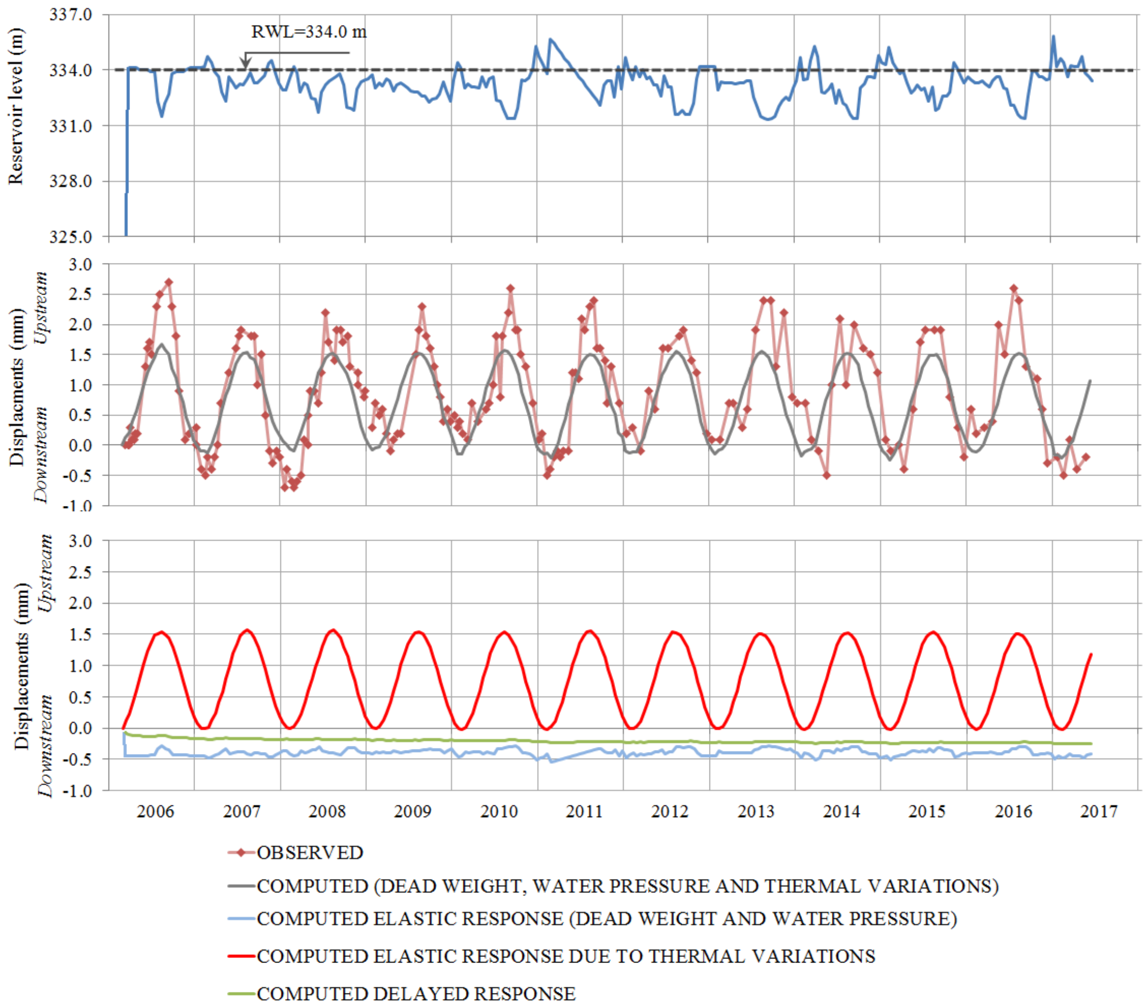

Figure 11: Horizontal displacement in the riverbed direction at 336.55 m elevation, measured on the plumb line and computed with the structural model

A global reasonable agreement between the computed and observed results was found, which means that the observed behaviour is well explained by the considered acting loads. In general, the computed values are lower than the observed ones. This difference, that in some phases of the analysis is considerable, might be attributed to the occurrence of time periods with higher temperatures in the summer and lower temperatures in the winter, than the ones simulated by the thermal waves. Additionally, the observed point, at elevation 336.55 m, is located close to the dam external surface (in the spillway pier), and by this reason, it might suffer from the influence of the daily thermal wave. Notice that in summer, the maximum daily temperatures overcome frequently the 35 ºC while the maximum value predicted by the thermal wave curve, for the air temperatures, is about 23.7 ºC.

The computed response is also depicted in the third plot of Figure 11, where the different effects are separately represented, so the relative weight of each effect on the dam behaviour can be appreciated. As expected for this type of gravity dam, the elastic displacements due to the concrete thermal variations are the dominant component on the dam response, with an amplitude of about 1.5 mm, followed by the elastic displacement due to the hydrostatic pressure, of about 0.5 mm towards downstream (for the full water reservoir), while the delayed response is smaller, of about 0.25 mm, also in the downstream direction.

4 STRESS FIELDS IN THE DAM

Figures 12 and 13 present the principal stresses on the upstream and downstream dam surfaces and also in two cross sections of the dam, due to the combined loads of the dead weight, hydrostatic pressure (h=334.00 m) and the maximum annual positive thermal variations.

The stress field is compressive near the dam's surfaces while in the core tensile stresses occur. This issue is consistent with the thermal gradients occurring along the riverbed direction, that are small in the dam's core and much larger near the surfaces, generating compressive and tensile stresses at the surfaces and at the core, respectively. At the surfaces, the compressive stresses are horizontal, since the displacements are restrained by the riverbanks. The stresses at the downstream surface are slightly higher than the ones occurring at the upstream surface, since this surface is submerged during the larger part of the year undergoing, by this reason, lower thermal variations.

The compressive stresses are about 4.0 MPa in the zone immediately beneath the spillway crest, while the maximum values, of about 6.3 MPa, occur near the right bank bearing, in a zone where stresses concentrate due to the dam geometry (Figure 12). The maximum tensile stresses reach about 1.2 MPa, at the upper part of the dam's core, and are approximately vertical (Figure 13).

Figure 14 presents the principal stresses on two cross sections of the dam due to the combined loads of the dead weight, hydrostatic pressure (h=334.00 m) and the maximum annual negative thermal variations. It can be observed that tensile and compressive stresses occur near the dam surfaces and in the core, respectively. The maximum tensile stresses reach about 2.2 MPa near the ski bucket and about 2.0 MPa at the central part of the upstream surface. The maximum compressive stresses are moderate, of about 1.3 MPa. The tensile stress near the ski bucket value is higher than the estimated concrete tensile strength ($\approx 0.1 \times 20.1 = 2.0$ MPa), which justifies the stabilized longitudinal crack existing in this zone (Figure 15).

The continuum structural model predicts tensile stresses in the longitudinal direction of the dam of about 6.0 MPa near the dam surfaces. However, this tensile stresses cannot occur at the site since the concrete dam blocks can shrink practically without resistance, due to the existence of the vertical contraction joints.

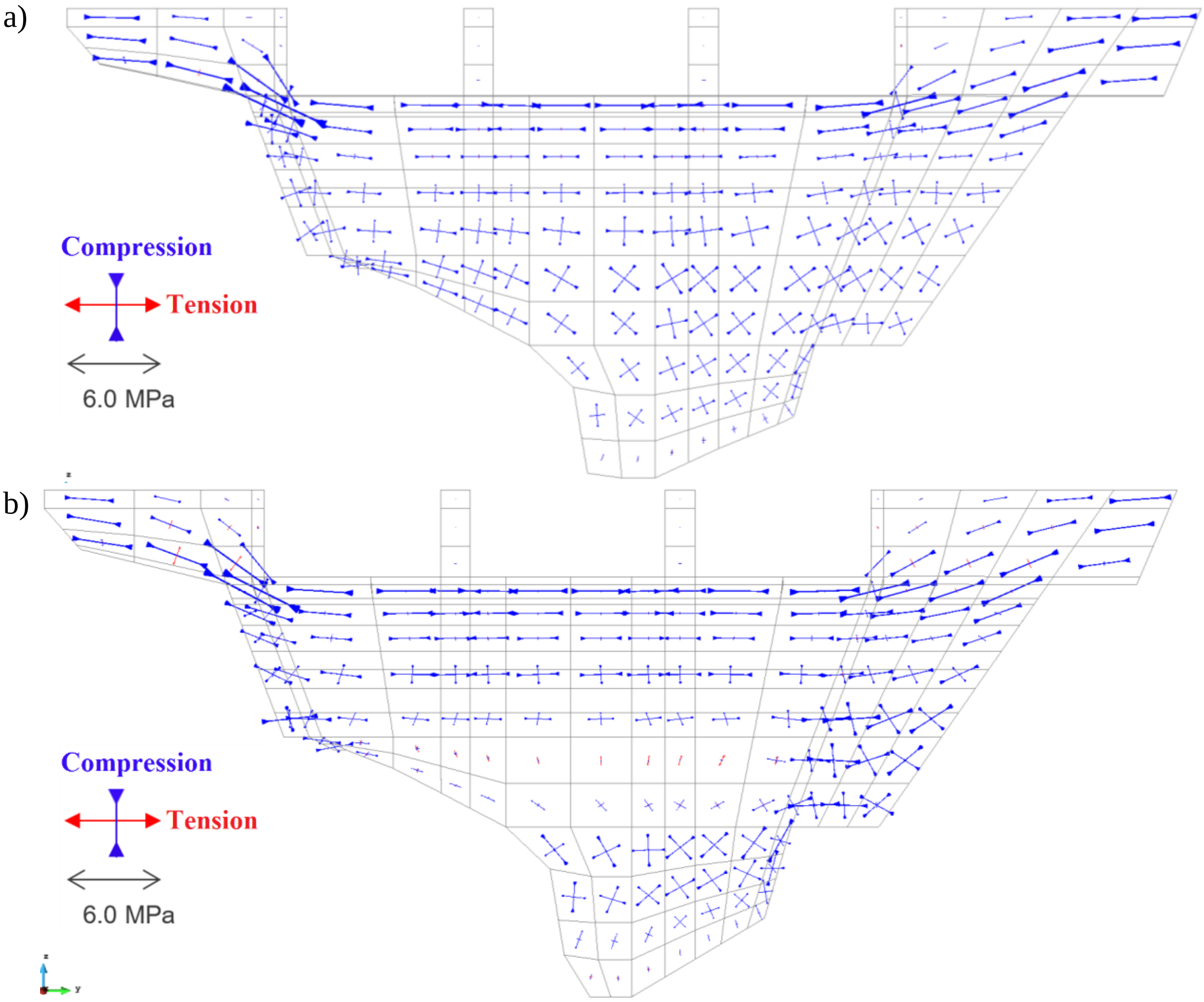

Figure 12: Calculated dam's principal stresses due to the combined loads of the dead weight, hydrostatic pressure (h=334.00 m) and maximum annual positive thermal variations: a) upstream surface; b) downstream surface

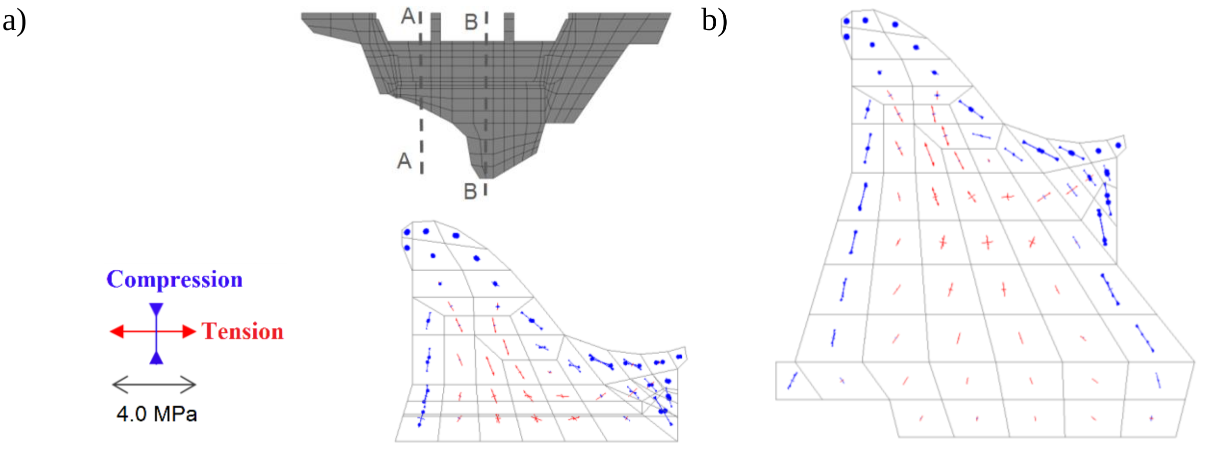

Figure 13: Calculated dam's principal stresses due to the combined loads of the dead weight, hydrostatic pressure (h=334.00 m) and maximum annual positive thermal variations: a) cross section A-A; and b) cross section B-B

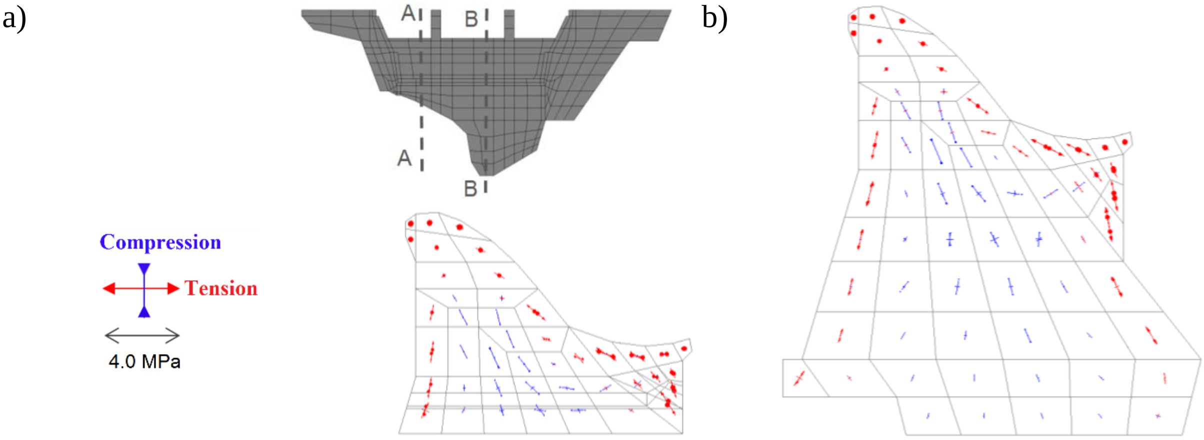

Figure 14: Calculated dam's principal stresses due to the combined loads of the dead weight, hydrostatic pressure (h=334.00 m) and maximum annual negative thermal variations: a) cross section A-A; and b) cross section B-B

Figure 15: Stabilized longitudinal crack on the ski jump bucket

5 CONCLUSIONS

The mathematical methodologies used for interpreting the structural behaviour of the Bouçoais-Sonim dam had shown to be adequate, having into consideration both the instantaneous and the delayed effects of the main loads acting in the dam: the dead weight, the hydrostatic pressure and the temperature variations.

The monitoring results, interpreted with the support of numerical models, allow to conclude that the dam behaviour during the first controlled filling and the first 10 years of operation was satisfactory. This was demonstrated by the good agreement obtained between the observed and calculated displacements, which are consistent with the loading actions and with the deformability estimated for the materials.

The computed stress fields allow obtaining the stress envelope that can occur at the site. The compressive stresses are moderate with generalized maximums of about 4.0 MPa, that reach 6.3 MPa in localized zones, much lower than the characteristic concrete compressive strength, at the 180 days of age, which was determined as 20.1 MPa. Across the dam section, maximum tensile stresses of about 1.2 MPa were computed, in the dam core, which, due to the moderate magnitude of these stresses, do not pose effective risk of cracking at these zones. Next to the downstream surface, near the ski jump bucket, maximum tensile stresses of about 2.2 MPa were calculated. As the tensile stresses are moderate, no significant cracking on the dam surface was observed.

The dam behavior is mainly conditioned by the environment thermal actions both for the dam relative and absolute displacements and also for the stress fields.

ACKNOWLEDGMENT

Thanks are due to Pebble Hydro (EDP group), the owner of Bouçoais-Sonim dam, for permission to present the dam monitoring results.

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