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ANALYSIS AND INTERPRETATION OF THE STRUCTURAL BEHAVIOUR OF ALQUEVA DAM DURING THE FIRST FILLING OF THE RESERVOIR

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Abstract. *This paper presents the most relevant monitoring data and the interpretation of the structural behaviour of Alqueva dam during the first filling of the reservoir, which took place between February 2002 and January 2010.*
The safety control and the interpretation of the dam's behaviour make use of the data provided by the monitoring system. In particular, the interpretation of the structural behaviour is based on: planimetric (radial and tangential) displacements, measured through plumb-line and geodetic methods; vertical displacements, measured with rod extensometers and obtained through geometric levellings; and temperatures, strains and stresses within the dam's body, measured with embedded electrical resistance devices.
A three-dimensional finite element model, considering the concrete viscoelastic behaviour and foundation specific characteristics, such as fault 22 located in the left bank, was developed for the numerical simulation of the dam structural behaviour. The time histories of the hydrostatic load and of the measured temperature within the dam's body were considered in the model. The dam's full-mixed concrete delayed behaviour was assumed to be represented by the Bažant *and* Panula*'s basic creep function, evaluated from both in situ and laboratory tests.*
The results of a quantitative interpretation of the monitored displacements during this period, using a viscoelastic formulation, are close to those provided by the structural analysis.

1 INTRODUCTION

The failure of dams with large reservoirs can be the cause of catastrophic accidents with very important losses of human lives and of economic and environmental beings. Thus, all over the world, the safety control of these projects is regulated and followed by national authorities, in general with specialized technical advice.

The safety control of these projects considers structural, hydraulic-operational and environmental aspects[1]. This work deals with structural safety issues, which refer to the capacity of the dam for meeting the behaviour requirements as regards actions and other factors associated with construction and operation and exceptional occurrences.

The structural safety control of major projects begins in the design phase and ends only with the decommissioning of the dam. During and after the first filling of the reservoir, this control is based in an almost continuous comparison between monitoring data, obtained through several devices installed according to a Monitoring Plan, defined during the design phase and actualized along the project lifetime, and results of models, usually numerical ones, considering the most important characteristics of the structures, such as the geometry and the materials' properties.

As the first filling of the reservoir is like a first load test of the structures under real operating conditions, the safety control activities during this period are important not only to avoid accidents and incidents, but also to acquire knowledge about the structural behaviour of the dam that will be a reference during the lifetime of the project. The safety regulations of almost all developed countries determine special monitoring programmes during this phase, involving the owner and national institutions with authority in dam's safety control, which, in Portugal, are the Water Institute (INAG) and the National Laboratory for Civil Engineering (LNEC).

This work presents the structural safety activities during the first filling of Alqueva reservoir. After a brief description of the project and of the most important aspects of its construction, the monitoring system, the monitoring plan during the first filling of the reservoir and some important results of tests carried out for the characterization of the materials' rheological properties are presented. The interpretation of the monitoring results is based on their comparison with results of a numerical model that represents the structural behaviour of the dam and of its foundation.

2 BRIEF DESCRIPTION OF THE PROJECT

Alqueva dam (Fig.1) is the main structure of a multipurpose project, located in the Alentejo region (southern Portugal), whose main objectives are the creation of a strategic water reserve in a region which has been showing a trend to desertification, the water supply to the population and to irrigation and the production of electricity. This project, owned by a public enterprise (EDIA – Empresa de Desenvolvimento e Infra-Estruturas do Alqueva), involves numerous structures, mainly dams, irrigation channels and pumping stations, and is changing the region through the development of new industries and services, such as agriculture and tourism.

Figure 1: Alqueva dam

Alqueva dam is a 96 m high double curvature thick arch dam, with two artificial abutments and a crest development of about 348 m, built on the Guadiana River. At the normal water level (el. 152 m), the reservoir is the largest artificial lake in Western Europe, with a total volume of 4150 hm^3 and covering an area of about 250 km^2.

Two surface spillways with a total of 3 spans (3×2100 m^3/s) with the crest at elevation 139 m, two bottom spillways crossing the dam at elevation 92 m (2×1750 m^3/s) and a bottom outlet (160 m^3/s) present a total maximum discharge capacity of 9960 m^3/s.

In a pumping station, located on the right bank, a few kilometres upstream of the dam, the water for irrigation is pumped to another reservoir, created at a higher elevation by three earthfill dams – Álamos dams – from which the water is supplied for the irrigation system by gravity.

The power plant, located at the toe of the dam, is equipped with two reversible Francis type turbine-pumps of 129.6 MW each. A new power plant, with two similar groups, is currently under construction on the right bank.

The dam foundation is a heterogeneous rock mass, consisting of green schists with good mechanical properties in the right abutment and in the bottom of the valley, and less competent phyllite in the left abutment. The interface between these two distinct geologic lithologies occurs along a fault zone, known as fault 22, whose relevance was acknowledged since early design stages. In the next chapter reference will be made to the works developed during the construction phase in order to improve the mechanical characteristics of this fault.

3 CONSTRUCTION PERIOD

The construction began in the end of 1997, with the excavations, followed by the dam's body construction between May 1998 and December 2001.

During the excavations, it was found that fault 22 was really a set of two faults, separating the schist and the phyllite, which develop according two almost parallel plans crossing the rock mass in a descent way from downstream to upstream (Figure 2). This finding required a new design for the treatment of this zone and retarded the construction of the concrete blocks of the left bank. The new treatment was based on the replacement of the disturbed material between the two faults by concrete, through a set of galleries opened in the rock mass.

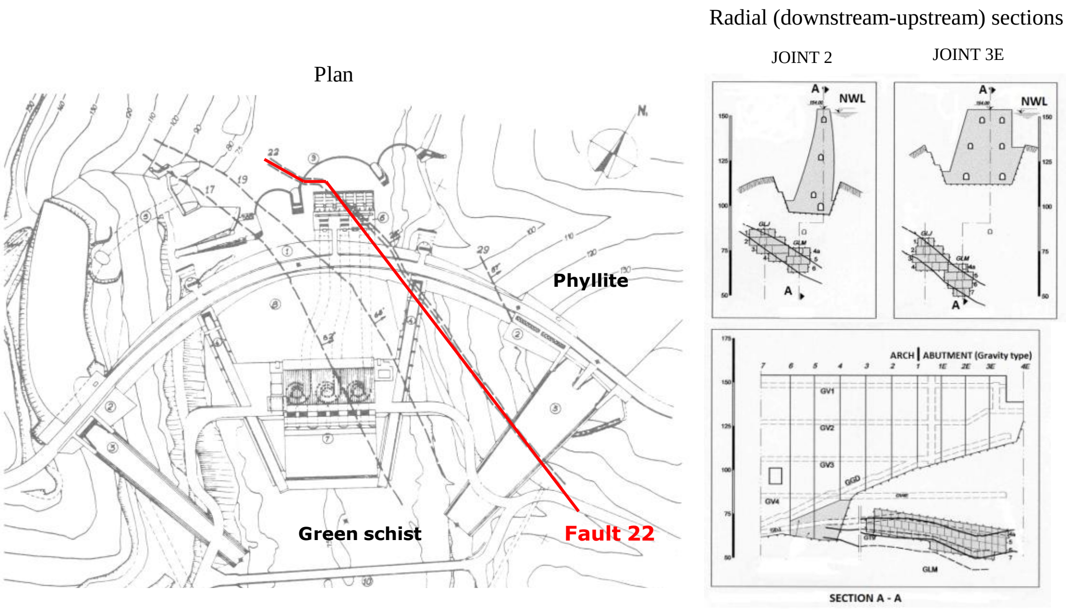


Figure 2: Location and treatment of fault 22

The first filling of the reservoir began in February 2002, but the full water level was only achieved in January 2010.

3 MONITORING AND FIRST FILLING PLANS

Under Portuguese dam safety regulations[1] the Monitoring Plan is a binding document that supports the structural safety control of dams and rules all the activities related with safety control, including instruments and methodologies that allow the characterization of the actions, the material properties and of the structural responses.

Alqueva dam Monitoring Plan[2] includes: i) the characterization of water, temperature and earthquake actions; ii) the measurement of representative structural response parameters, such as displacements, joint movements, stresses and strains; iii) the development of tests to characterize the properties of the materials; and iv) carrying out periodic visual inspections.

The main actions, due to the effects of the water upon the dam and its foundation and of variations in environmental temperature, are characterized by the measurement of the water level in the reservoir, of the water flow rates in the drainage net, of the pressures in piezometers located in the dam foundation and of the temperature in the air and within the dam's body. Earthquake actions are characterized by a group of synchronous seismometers placed in the dam and along the reservoir banks that allow accelerations to be measured and the epicentre location of seismic events to be obtained.

The structural response of the dam is monitored by the measurement of (Figure 3): i) dam horizontal displacements by means of 9 inverted plumb-lines; ii) vertical displacements in the dam foundation by 12 borehole (rod) and 6 incremental extensometers; iii) relative displacements between dam blocks by 178 electric resistance jointmeters embedded in the concrete and 84 three-dimensional devices in accessible zones (e.g. galleries); iv) concrete strains by 36 plane and 9 three-dimensional electric resistance extensometers groups; v) stresses in the concrete by 7 electric resistance stress meters; and vi) dam displacements by a geodetic monitoring system[3], which comprises 2 traverse lines along gallery GV4 and along the crest, connected to the inverted plumb wires for horizontal components, and one precision levelling line in each gallery, for the vertical component (Figure 4).

The monitoring plan outlined prior to the dam construction already considered the monitoring of specific items related to the foundation behaviour in the zone around fault 22. However, during the construction works it was necessary to adjust the primary design, which required the development of a specific monitoring plan concerning the observation of the behaviour of this foundation region[4]. According to this plan, specific devices for measuring displacements and stresses in the concrete were placed in well-defined cross sections of blocks 2-3 and 1-1E (Figure 5).

According to the Portuguese regulation, a special monitoring plan was developed to control the structural behaviour during the first filling of the reservoir[5]. This plan defined the set of observation campaigns to be carried out during this period and established 7 water levels (Figure 6) at which the filling of the reservoir was suspended in order to perform an evaluation of the dam behaviour.

Due to some delays during the construction, the filling of the reservoir began before the conclusion of the foundation treatment and the grouting of the contraction joints. Nevertheless, the water level in the reservoir could not rise above the level at which these works were concluded.

The first filling of the reservoir began on 7 February 2002, after carrying out an inspection to the structure and to the monitoring system[6]. The conclusion date of the foundation treatment and of the grouting of the contraction joints – May 2002 – was considered a reference date to the interpretation of the monitoring data.

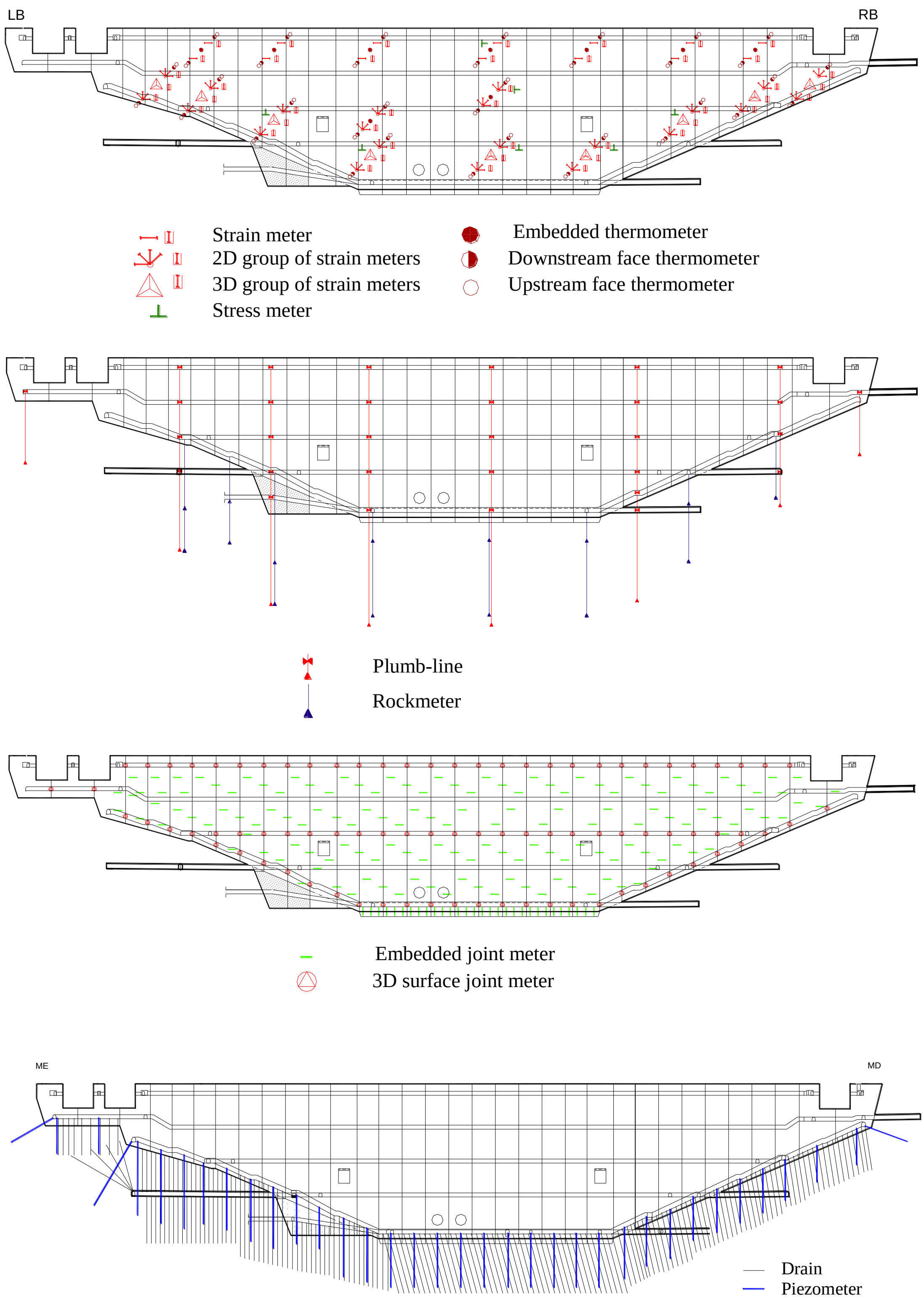
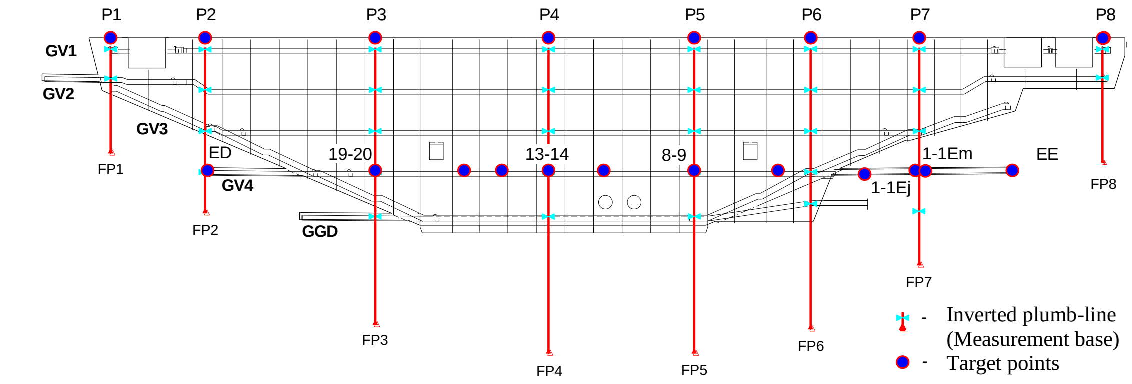
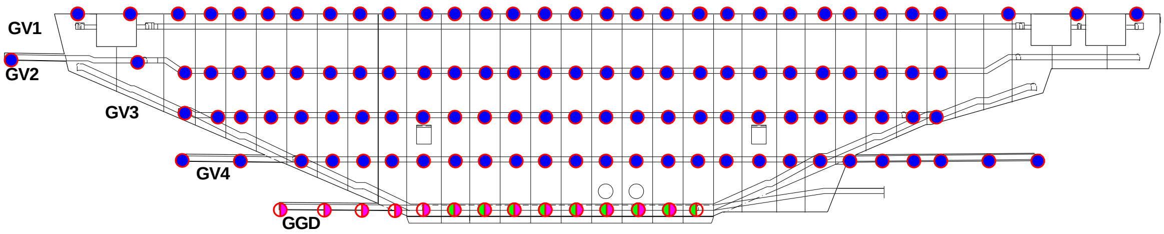


Figure 3: Relevant devices of the Alqueva dam monitoring system



(a) Traverse lines along the crest and GV4 gallery



(b) Levelling lines along horizontal galleries

Figure 4: Alqueva dam geodetic monitoring system

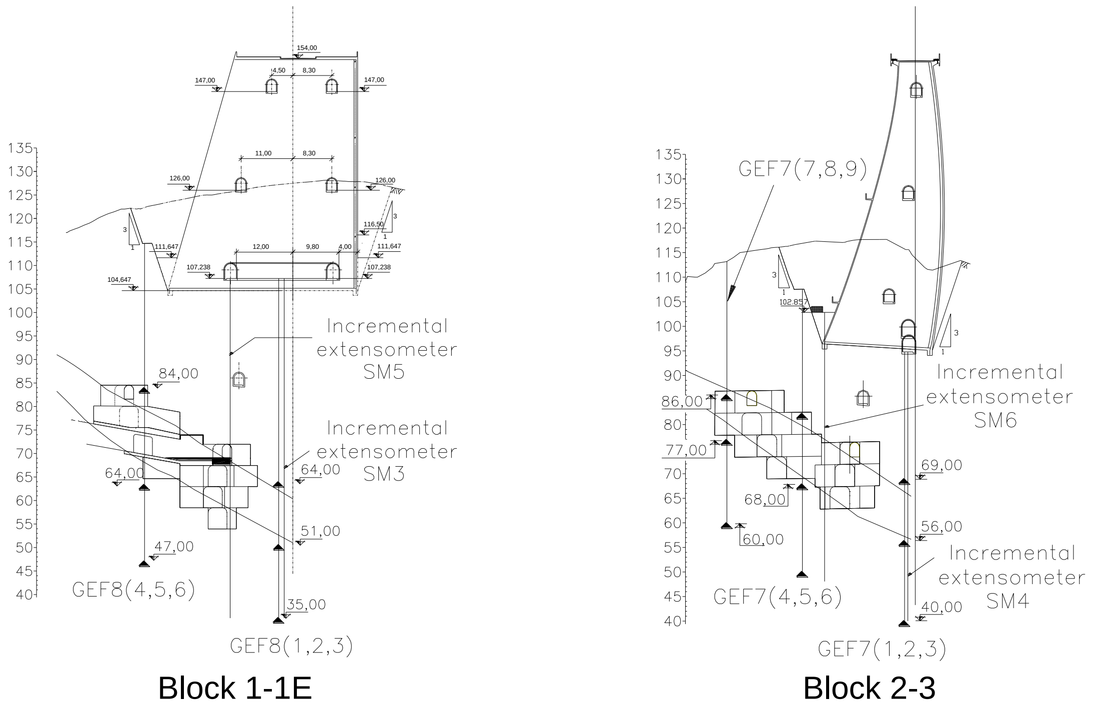


Figure 5: Monitoring devices in the zone of fault 22

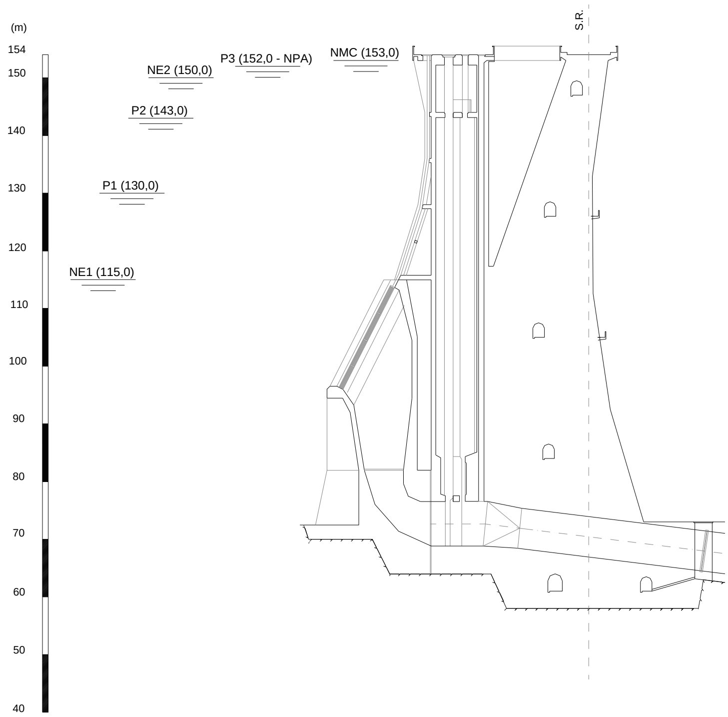


Figure 6: Suspension levels during the first filling of the reservoir

4 MATERIALS PROPERTIES

4.1 Dam foundation properties

Several geological and geotechnical investigations were made concerning the rock mass foundation of the dam, before and during the construction[7,8].

The foundation is heterogeneous and consists of good quality green schist in the right bank and the river bottom and of quite good phyllite, with a higher deformability, in the left bank (Figure 2). After the treatment, the foundation Young's modulus in the left bank varied from 6 to 20 GPa, but the foundation of the right bank and of the bottom of the valley had a Young's modulus greater than 20 GPa. The region of the phyllite is more fractured and is crossed by several faults, the most important being fault 22, which corresponds to the green schist–phyllite interface.

In general the schistosity is barely evident and sub-parallel to bedding planes. The phyllite has a very visible schistosity, while the green schist looks highly dense. There are several faults, aligned with some joints sets, especially in the left bank, but their opening is, in general, less than 10 cm, with the exception of fault 22.

Foundation treatment consisted of consolidation of faults, general consolidation of the rock mass in the area of the dam, contact grouting, and the execution of a grout curtain and a line of drainage boreholes (Figure 3). Due to the importance of fault 22 in the structural behaviour of the dam, a special treatment was carried out, which included the excavation of a set of galleries and the removal of the fault gouge and its replacement by concrete (Figure 2). After grouting, the majority of the permeability values obtained in Lugeon type tests were lower than 1.0×10^{-8} m/s.

4.2 Dam concrete properties

The majority of the dam's full-mixed concrete is C12/15 strength class, according to the European code. In average, the concrete's binder dosage was 200 kg/m^3, including 80% of cement (160 kg/m^3) and 20% of fly ash (40 kg/m^3), and the water-cement ratio (by weight) was about 0.59. The mix data of the full-mixed concrete is presented in Table 1.

Mix components	Content
Cement IV-32.5	160 (kg/m^3)
Fly ash	40 (kg/m^3)
w/c ratio (by weight)	0.59
Coarse aggregate	
150-75 mm	613 (kg/m^3)
75-38 mm	427 (kg/m^3)
38-19 mm	302 (kg/m^3)
19-10 mm	210 (kg/m^3)
10-5 mm	153 (kg/m^3)
5-2.5 mm	176 (kg/m^3)
Fine aggregate	
2.5-0 mm	424 (kg/m^3)
Plasticiser	0.86 (kg/m^3)

Table 1: Alqueva dam's concrete composition

The rheological properties of the concrete were estimated from the results on both *in situ* and laboratory tests, which were carried out for the control of its quality and for the characterization of its deformability[9,10].

The most important data for the interpretation of the dam structural behaviour are the results of the *in situ* and laboratory tests developed in the context of creep cells. Creep cells are sets of concrete cylinders embedded in the dam's body, isolated from the structural stress field with a steel frame that creates a gap between the cell and the dam's concrete and subjected to *in situ* thermohygrometric variations. In the core of each one of these cylinders there is a strain meter that measures its axial deformation. In each set there are one active cell subjected to an imposed load history and one non-stress cell subjected to free deformation. The load history is imposed by a loading system composed by a closed hydraulic circuit which controls the applied pressure on a flat jack located on the base of the active cell. In general, for each one of the most important concrete compositions, there are three sets of specimens: two cylinders within the dam's body and one prism in laboratory. One of the *in situ* sets is filled with the mass concrete used within the dam's body, called full-mixed concrete, and the other two are filled with concrete obtained retiring the larger aggregates from the full-mixed concrete.

These systems allow creep tests to be carried out, maintaining the load constant during long time intervals, and the determination of the modulus of elasticity, imposing quick load variations.

In all cases the results of these tests must be analysed taking into account the results of quality control tests for the determination of the concrete strength and of its Young's modulus.

In the case of Alqueva dam, the following Bažant and Panula (BaP) creep law[11] was predicted from compression strength tests results and adjusted to experimental results of modulus of elasticity obtained from creep cells (considering t-t_0 = 0.1 days)9.

$$J(t,t_0) = \frac{1}{58.0} + \frac{3.70}{58.0}\left(t_0^{-0.36} + 0.05\right)\left(t - t_0\right)^{0.15} \text{ (GPa}^{-1}\text{)} \tag{1}$$

$$\frac{1}{E(t_0)} = \frac{1}{58.0} + \frac{3.70}{58.0} \times 10^{-0.15}\left(t_0^{-0.36} + 0.05\right) \text{ (GPa}^{-1}\text{)} \tag{2}$$

Figure 7 shows the creep function for three loading ages and the corresponding relaxation curves, obtained by numerical inversion.

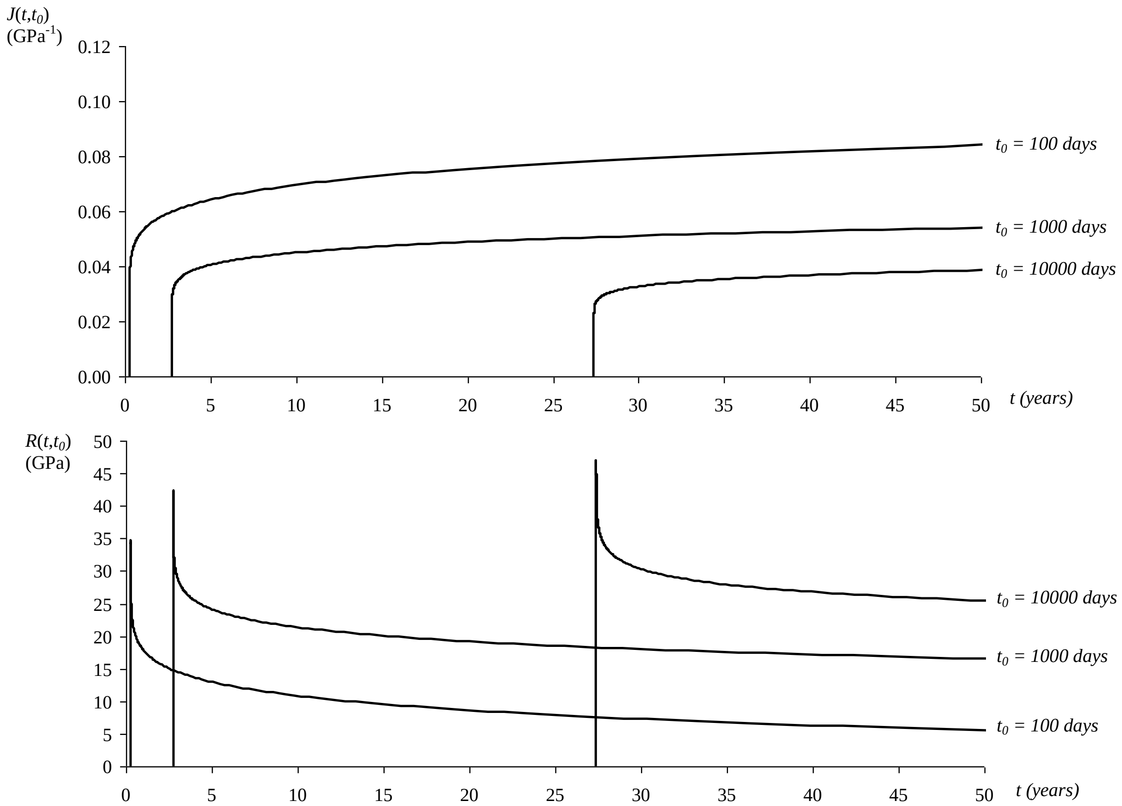


Figure 7: Creep and relaxation curves of Alqueva dam's concrete

5 ANALYSIS AND INTERPRETATION OF THE OBSERVED BEHAVIOUR

The first filling of the reservoir can be analysed in two different periods: before and after May 2002[12].

The first period was characterized by the conclusion of the foundation treatment, including grouting for consolidation and impermeabilization, the execution of the drainage boreholes at both banks higher elevations and by the injection of the contraction joints, which required a significant artificial refrigeration of the concrete. Moreover, there was yet significant effects related to the hydration heat of the gravity concrete blocks of the left bank which were concluded during the last months of 2001. Thus, it is very difficult to analyse the structural response of the dam during this period.

After the conclusion of all works in May 2002, the behaviour of the dam and of its foundation was affected mainly by the variations of the water level in the reservoir and in the concrete temperature related with the thermal field on this date and the environmental variations (in particular the air and water temperature). For this reason, the interpretation of the behaviour was based on a reference date established on 7 May 2002, when the water level in the reservoir was at elevation 111.7 m.

5.1 Structural model

The interpretation of the dam's behaviour was based on the results of a three dimensional structural model representing the dam and its foundation (Figure 8). The model was analysed by the finite element method, using 20-noded isoparametric brick elements with second degree shape functions[13], with homogeneous and isotropic properties.

The foundation was divided into three regions of different deformability: the left bank, the fault 22 and the right bank/bottom of the valley. The dam was divided into eight regions according to the date at which the concrete was placed, between the first semester of 1998 and the end of 2002.

The rock mass foundation was assumed to be an elastic medium and the concrete was assumed to be a viscoelastic material, whose deformability was characterized by equation 2. The main parameters considered for the material properties are presented in Table 2.

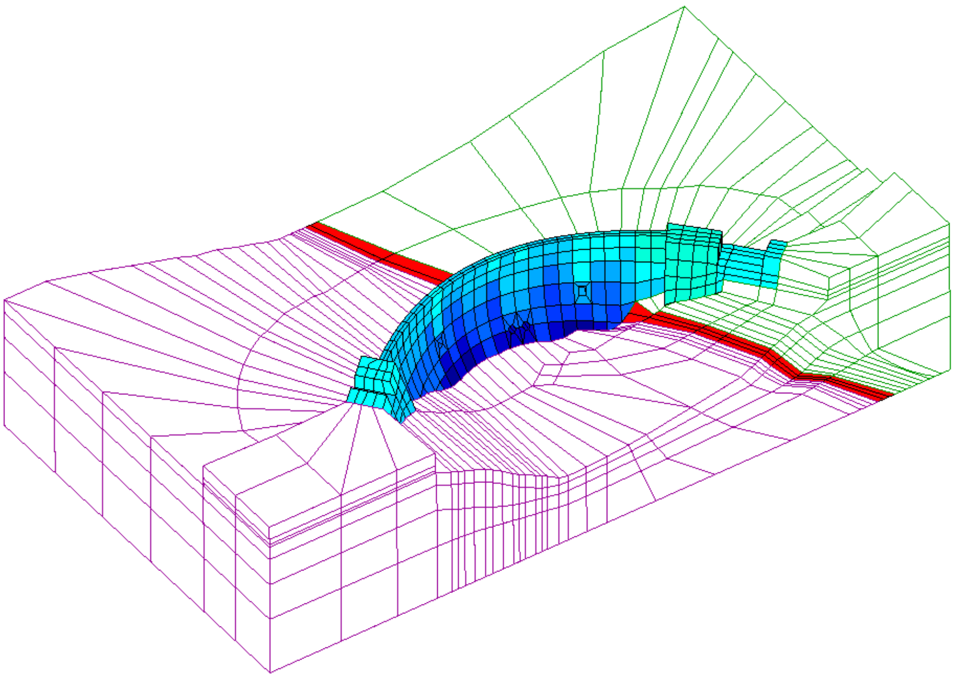


Figure 8: Alqueva dam and foundation. Finite-element mesh

	Finite element group	E (GPa)	ν	α (1/ºC)
	Dam concrete	(Eq. 2)	0.2	1×10^{-5}
Foundation	Left bank	15.0	0.2	0
	Right bank and bottom of the valley	20.0		
	Fault 22	10.0		

Table 2: Material properties considered in the finite element model.

Both the hydrostatic load on the upstream face of the dam and the time history of the observed temperatures within the dam's body were taken into account. In order to consider the delayed effects related with creep and relaxation, each one of these loads was discretized over time.

Regarding the boundary conditions, displacements were restrained in the outer borders of the foundation.

6.2 Evolution of the main loads

The main actions during the first filling of the reservoir are the hydrostatic pressure on the upstream face of the dam and the temperature variations measured within the dam's body.

Until May 2002 the increase of the water level was conditioned by the works in the foundation and in the contraction joints (Figure 9). After the 2003/2004 winter the increase rate of the water level was smaller, due to the use of water for irrigation purposes. The water level only reached elevations 150 m and 152 m, corresponding to the last phases of the reservoir filling, in 2007 and 2010, respectively.

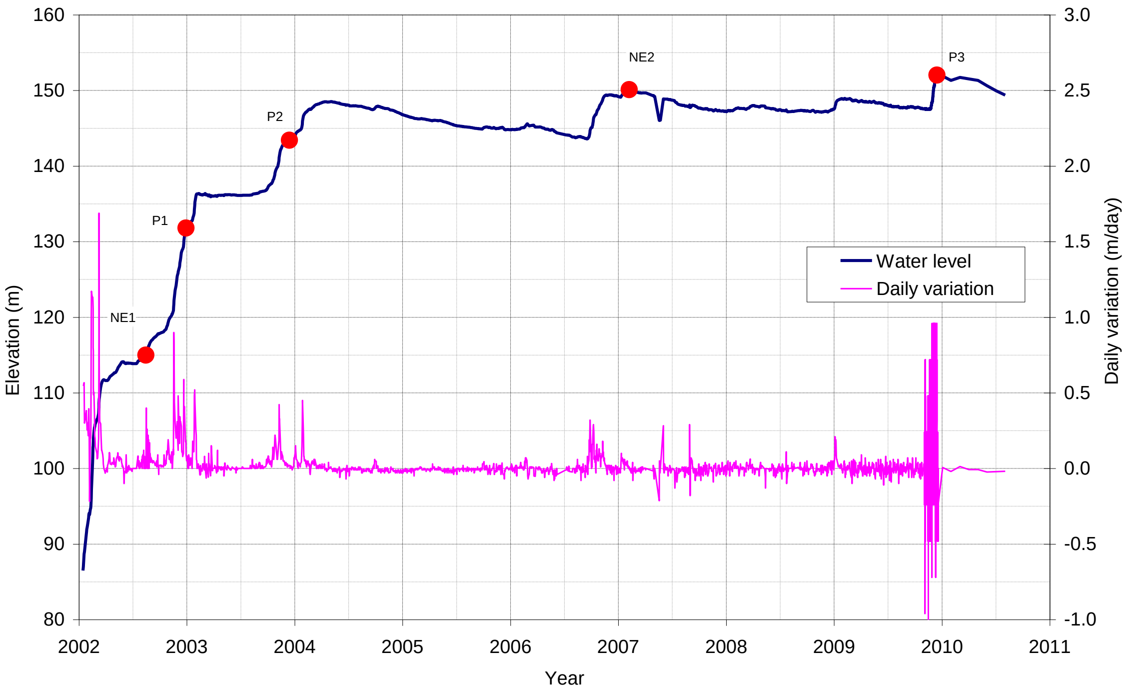


Figure 9: Time history of the water level in the reservoir

During the first months of the reservoir filling the concrete temperature variations were significantly affected by a forced cooling, carried out so as to allow a proper injection of the contraction joints (Figure 10). Thus, in May 2002, when this injection was concluded, the dam body was at an exceptional low temperature, of about 10 ºC, and only in 2005 the temperature correspondent to the average annual temperature was reached. As the dam's body was cooled in several phases, it was necessary to take into account the variations of the real thermal field in the dam. Figure 11 shows the isothermals calculated in some key dates during the reservoir filling.

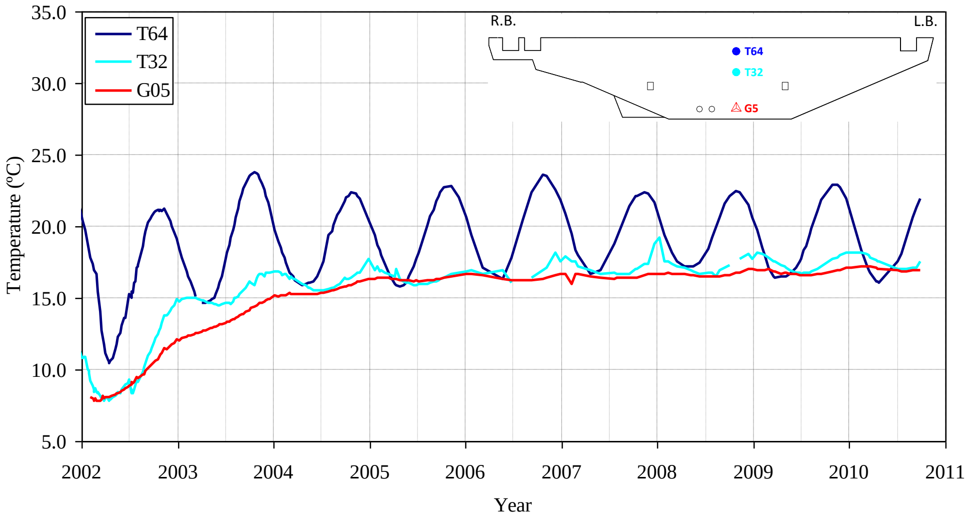


Figure 10: Time history of the measured temperature in some points of the central cantilever

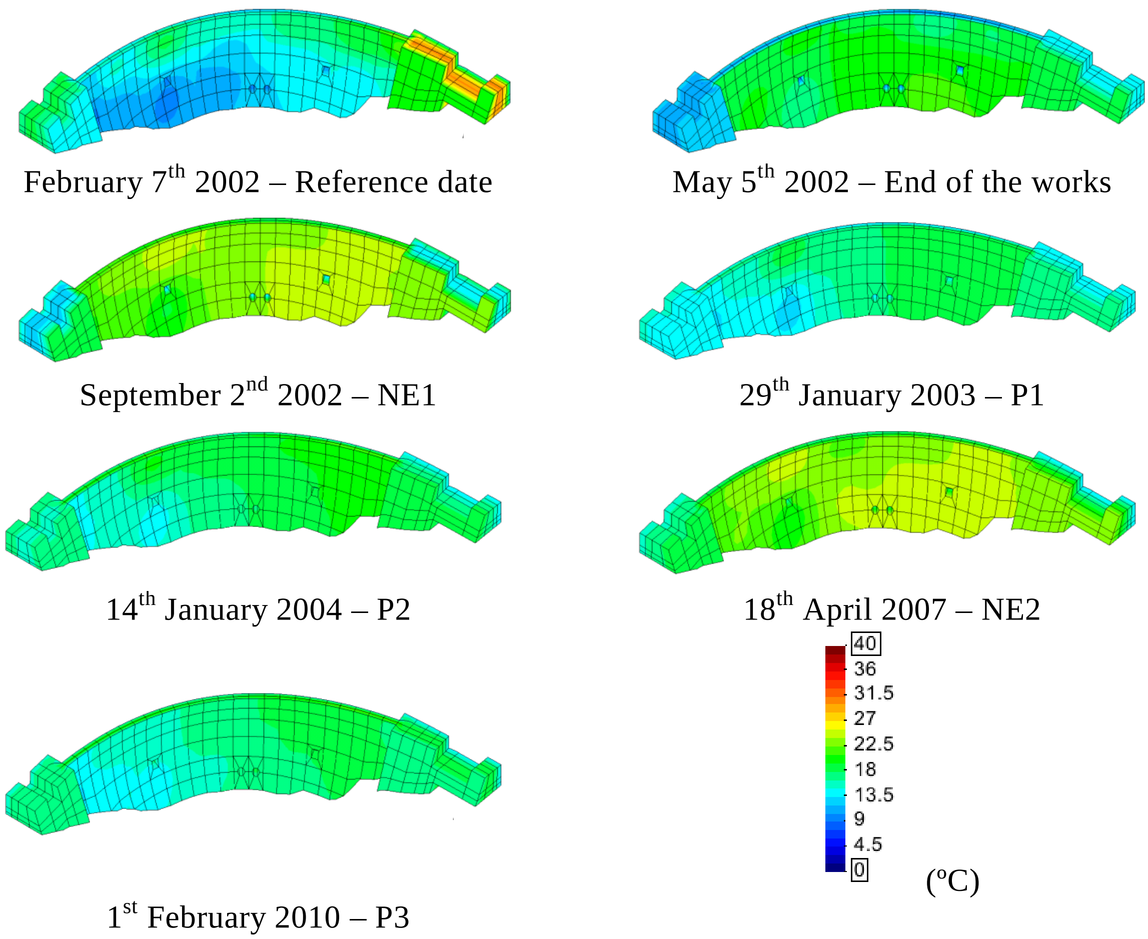


Figure 11: Calculated thermal field on different dates.

6.3 Joint movements

One important aspect to consider in this type of dams is the structural continuity, which can be verified through the analysis of the evolution of joint movements. In

Alqueva dam there are several devices which allow measurement of relative displacements between blocks, namely 3D joint meters in the galleries and embedded joint meters within the dam's body.

Data recorded in these devices is especially important after the injection of the joints, which was concluded in May 2002. Afterwards, the variations in both the water level in the reservoir and in environmental temperature are the main causes of these movements.

As expected, for higher water levels the joints were closed, even in the cold season, as was verified in January 2007 (Figure 12).

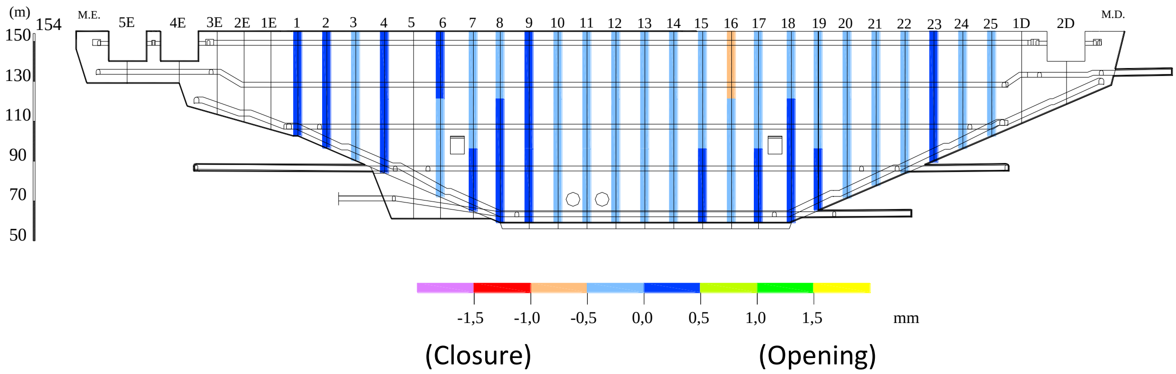


Figure 12: Joint displacements between May 2002 and January 2007 (H = 143 m), measured in the joint meters embedded at half-thickness of the dam

6.4 Horizontal displacements

The effects of the main loads in the horizontal displacements observed in the plumb-lines were separated through a quantitative analysis of the observed data. The empirical model used in this analysis is represented in equation 3, where h, θ and t represent the effects associated with the water level, the environmental temperature and the time period since the beginning of the observations, respectively.

$$\delta(h,\theta,t) = a_1(h-h_0)^4 + b_1\cos\left(\frac{2\pi s}{365}\right) + b_2\, sen\left(\frac{2\pi s}{365}\right) + c_1\sum_{i=1}^{n\,steps}\varphi(t,t_0)\left[h_{f,i}^4 - h_{i,i}^4\right] + k \qquad (3)$$

The hydrostatic and the environmental temperature effects were represented by a fourth degree polynomial function of the water height upstream, $(h-h_0)$, and by a sinusoidal function of the number of days, s, since the beginning of each year, respectively. The time history of the hydrostatic load was discretized into n steps, being $h_{f,i}$ and $h_{i,i}$, the final and the initial water level in each time interval i. $\varphi(t,t_0)$ represents the creep coefficient value at instant t, correspondent to the action of a constant unit load acting during the time interval between instants t_0 and t. The constants a_1, b_1, b_2, c_1 and k were determined through a linear regression using all the observed data during the period under analysis.

In order to eliminate the long time effects of the artificial cooling, only displacements observed after the beginning of 2004, when the temperatures within the dam's body presented only sinusoidal variations related with the environmental temperature variations, were used in the quantitative analysis.

As an example, Figure 13 shows the comparison between the measured values and the results of both the quantitative analysis and the finite-element model for the radial displacements observed at the higher coordinometer base of plumb-line FPI4, at elevation 148.25 m, close to the central cantilever.

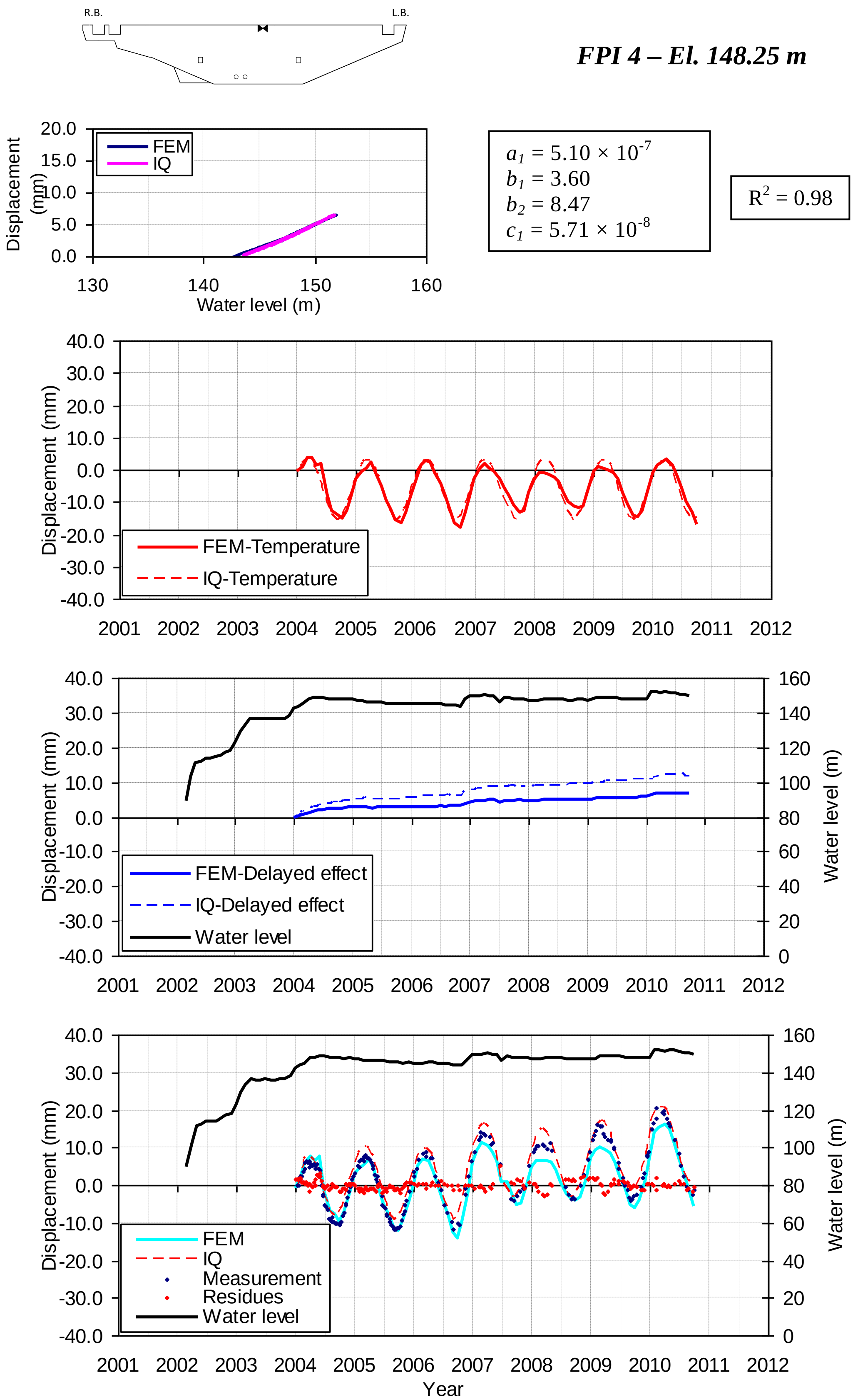


Figure 13: Comparison between observed radial displacements measured at FPI4 plumb-line (el. 148.25 m) and the results of both the quantitative analysis (IQ) and the FEM analysis (displacements towards downstream are positive).

The quantitative analysis results show that this model is a close representation of the observed displacements.

As shown, the hydrostatic effect estimated by the quantitative analysis is very close to the correspondent results of the finite element model.

In the quantitative analysis method, the thermal effects have the same sinusoidal evolution in all years during the analysed period. Thus, in years with environmental temperature variations very different from average temperatures in the period under analysis, the agreement results of the quantitative interpretation are not close to those calculated with the numerical model, which takes into account the observed temperature variations. This is what happens, for instance, in 2008 and 2009.

The time effect estimated by the quantitative analysis is more important than the correspondent results of the numerical model. Nevertheless, during almost all period, with the exception of 2008 and 2009, the observed displacements are close to those calculated with both the quantitative analysis and the numerical model. A reference must be made to the fact that the quantitative analysis considers only the measured displacements in each base, while the numerical model takes into account the structural behaviour of the dam/foundation set.

Using the displacements observed in all the bases of each plumb-line it is possible to extrapolate the displacements at the crest in that specific section. These displacements may then be compared with those measured by geodetic campaigns in the crest traverse. Figure 14 shows the good agreement between the displacements obtained by both methods in the higher blocks[14].

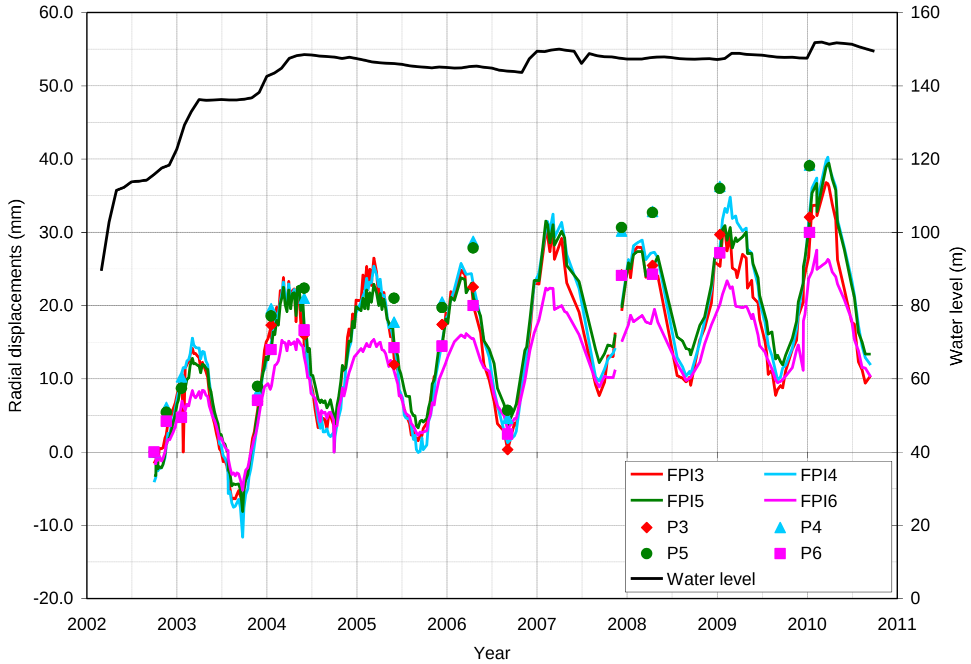


Figure 14: Radial displacements at the crest obtained through the plumb-lines measurements and by geodetic methods (positive displacements towards downstream)

Figure 15 shows the comparison between the horizontal displacements measured in January 2010 (H = 152.02 m) at the central plumb-lines by the two methods and calculated by both the quantitative analysis and the numerical model at the same points.

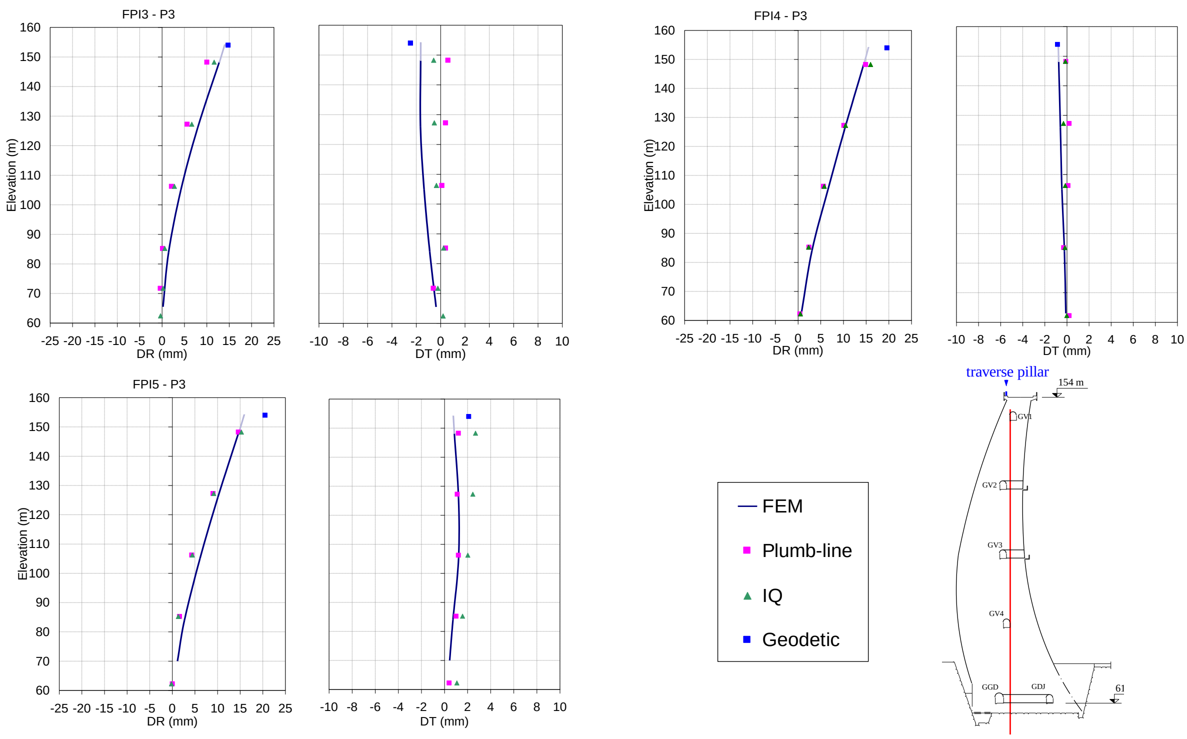


Figure 15: Radial (DR) and tangential (DT) displacements in the central plumb-lines at P3 water level (2010-01-12; H = 152.02 m)

6.5 Vertical displacements

According to the monitoring plan, vertical displacements are measured in rod extensometers installed in the foundation and by precision levellings in target points located along horizontal galleries.

As an example, Figure 16 presents the comparison between the displacements observed in the rod extensometer group GEF4, placed close to the central cantilever. GEF4 combines three rod extensometers: GEF4-1 and GEF4-2 are installed in a borehole drilled from the upstream drainage gallery and GEF4-3 is in another borehole drilled from the downstream drainage gallery. GEF4-1 is sealed at a depth of 60 m and the other two at a depth of 15 m. In the upstream side the vertical displacements are mainly upwards, while in the downstream side are mainly downwards, which corresponds with a rotation of the cantilever towards downstream, due to the hydrostatic pressure. Nevertheless, the observed displacements in the downstream extensometer do not show the seasonal variations calculated by the numerical model.

Figure 17 shows a similar comparison between vertical displacements observed in some dam blocks through precision levellings along GV3 gallery and the correspondent values determined by the numerical model.

6.6 Hydraulic behaviour of the foundation

Although there are a great number of drains, the total discharges are small, because the permeability of the rock mass foundation is very low. Figure 19 presents the accumulated discharges along the drainage gallery in the key dates of the reservoir filling. As can be seen in the figure, it is in the bottom of the right blank where the most important discharges are measured and, as expected, the measured values increase with the rise of the water level in the reservoir.

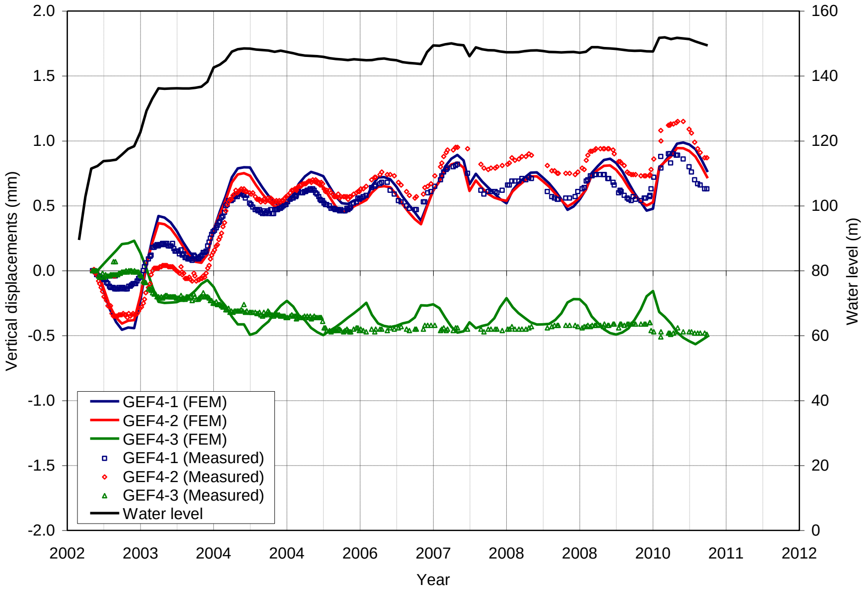


Figure 16: Vertical displacements measured in GEF4 rod extensometers and obtained from the FEM model (positive values upwards).

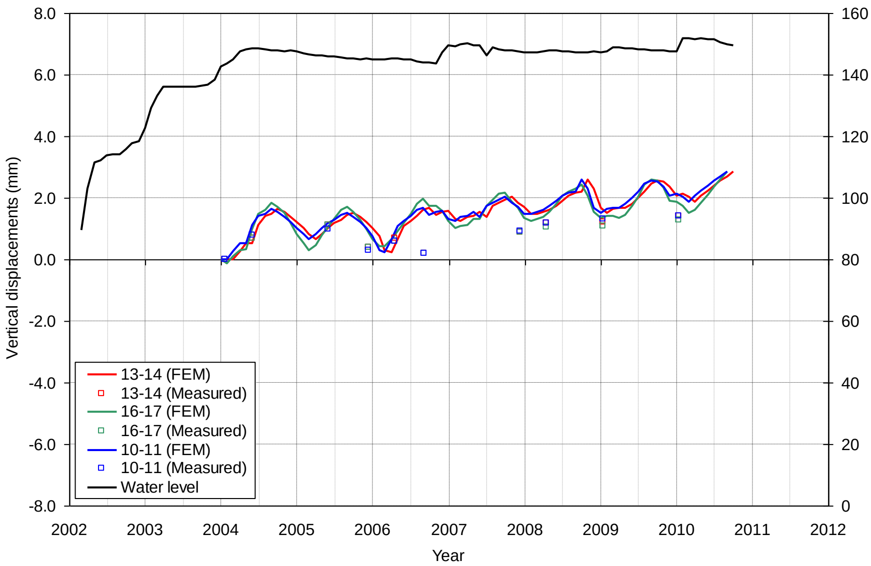


Figure 17: Vertical displacements on the GV3 gallery measured by geometric levelling and obtained from FEM analysis (positive values upwards).

In general, in Alqueva dam foundation the uplift pressures are not important. For instance, at the central cantilever, the uplift pressures upstream of the drainage curtain are lower than 60% of the hydraulic head, decreasing to values of about 10% immediately downstream of that curtain (Figure 19).

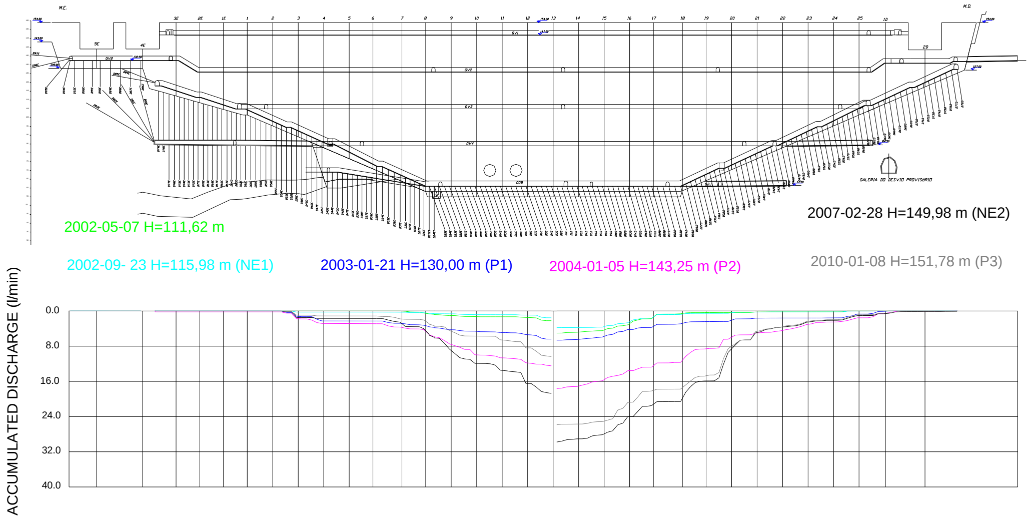


Figure 18: Accumulated discharges in the drainage gallery during the reservoir filling.

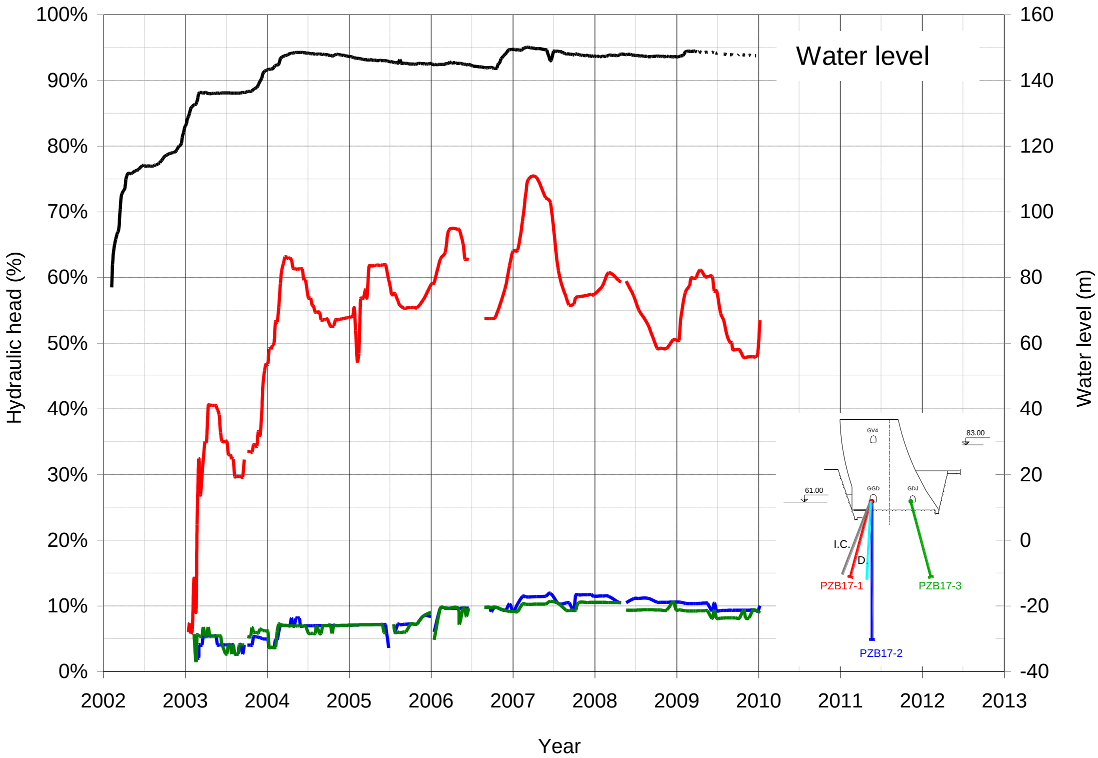


Figure 19: Uplift pressures in the central cantilever foundation in terms of percentage of hydraulic head.

7 CONCLUSIONS

Alqueva dam is the main structure of a large project for the development of Alentejo. As it creates a large reservoir, the activities related with the safety control and the interpretation of its behaviour are very important, in particular during the first filling of the reservoir.

The safety control of such a dam during the first filling of the reservoir requires the installation and proper use of the instrumentation system, the complete characterization of the main properties of the materials and the development of numerical models, whose results can validate the measured data.

In this paper a brief description of the main results concerning the interpretation of Alqueva dam behaviour during the first filling of its reservoir is presented. The numerical analysis of the structural behaviour of the dam shows that the observed response is well explained by the evolution of the main loads, taking into account the materials properties. Considering also that there is not any important anomaly concerning the hydraulic behaviour of the foundation, it can be concluded that the dam presents an adequate behaviour.

The overall results obtained during this period represent a reference state that allows the characterization of the dam's behaviour over time.

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