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

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ABSTRACT

This paper describes the main characteristics of Daivões dam and the construction period, presents the most relevant monitoring data and the structural behaviour interpretation of Daivões dam during the reservoir's first filling, between October 2020 and March 2022.

The safety control and the interpretation of the dam's behaviour are based on: contraction joint movements, measured in jointmeters; planimetric (radial and tangential) displacements, measured through plumbline and geodetic methods; vertical displacements, measured through geometric levelling; rock mass foundation displacements, measured with rod extensometers; seepage and leakage measured through the drainage system; uplift pressure in the rock mass foundation, measured in piezometers; and temperatures and strains in the dam's body measured with embedded electrical resistance devices.

A three-dimensional finite element model was developed for the numerical simulation of the dam structural behaviour during the first filling of the reservoir. The hydrostatic load and the measured temperature in the dam's body were considered in the model. The dam's concrete delayed behaviour was assumed to be represented by the Bažant e Panula's basic creep function, evaluated from laboratory tests.

Keywords: Daivões dam, reservoir first filling, monitoring activities, structural behaviour interpretation.

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1. INTRODUCTION

The failure of large dams can lead to disastrous incidents resulting in significant loss of human lives and severe economic and environmental damage. Consequently, safety monitoring for these structures is commonly governed and supervised by national organisations worldwide, typically with expert technical guidance. The safety oversight of these projects encompasses structural, hydraulic-operational, and environmental factors.

The structural safety concerns of major projects initiate during the design stage and continues until the dam is decommissioned. During and following the initial filling of the reservoir, this oversight relies on ongoing analysis and comparison between monitoring data collected from various instruments, alongside outcomes from models, typically numerical, reflecting key features of the structures like geometry and material properties.

Since the first filling of the reservoir acts as an initial load test of the structures under authentic operating conditions, safety monitoring during this phase is crucial not just for preventing accidents and incidents, but also for gaining insights into the dam's structural behaviour. The safety regulations in most developed nations stipulate distinct monitoring programs during this phase, involving both the owner and national agencies responsible for dam safety oversight, which in Portugal include the Portuguese Environment Agency (APA) and the National Laboratory for Civil Engineering (LNEC) [1].

This paper, in addition to describing the main aspects of the design and construction of the dam, with a special focus on structural safety observation and control activities, presents the assessment of the observation carried out, using the support of a numerical finite element model.

2. BRIEF DESCRIPTION OF THE PROJECT

The Daivões dam is part of the Daivões hydroelectric plant, which is located on the Tâmega River, a right-bank tributary of the Douro River, in Entre-os-Rios, between the Carrapatelo and Crestuma Lever dams. This development, which is part of the Tâmega Electric Production System, includes, in addition to the dam, two independent underground hydraulic circuits on the slope of the right bank and a semi-buried plant downstream. The dam, with abutments in the municipality of Cabeceiras de Basto, district of Braga (right bank), and in the municipality of Ribeira de Pena, district of Vila Real (left bank), is located approximately 80 km upstream of the confluence with the Douro river and approximately 3.5 km upstream from the road bridge that connects the districts of Braga and Vila Real, via the N206 national road, in the parish of

Cavez. Its reservoir extends to the municipalities of Vila Pouca de Aguiar and Boticas, with a total volume of around 55.5 hm^3, for the full storage level (FSL = 228.0 m).

The dam has a maximum height above the foundation of 77.5 m, measured between the lowest average plane of the insertion surface, at elevation 153.5 m, and the crowning floor, at elevation 231.0 m (Fig. 1). It is a conventional concrete structure, of the gravity arch type, with a vertical upstream facing and a downstream facing with a slope of 1V:0.5H up to a level of 222.25 m and vertical above this level. The crest has a width of 8.30 m and a total length of 265.0 m and 190.0 m between abutments. The geometric definition of the dam is based on circular arcs, with a reference surface having curvature radii of 140 m in the central section and 210 m in the lateral areas.

The dam is made up of 16 blocks, separated by vertical contraction joints, approximately 13.80 m to 18.30 m apart on the right bank (blocks B12 to B16), approximately 15.85 m in the central area (blocks B1 to B11) and from around 18.25 m to 18.95 m on the left bank (blocks B13 and B15). The construction of the dam took place between April 2018 and September 2020.

Fig. 1 – Daivões dam

3. MONITORING AND FIRST FILLING PLANS

Under Portuguese dam safety regulations, the monitoring plan is a binding document that supports the structural safety control of dams and rules all the activities related to safety control, including instruments and methodologies that allow the characterisation of the actions, the material properties and the structural responses.

In the case of the Daivões dam, it is recommended that the observation of most of the quantities foreseen for monitoring the actions and responses of dams be considered [2, 3]. Thus, the observation of the following quantities was planned: i) air temperature and humidity; ii) reservoir water level; iii) concrete temperatures; iv) relative displacements in contraction joints (Fig. 2); v) absolute and relative displacements (Fig. 3); vi) strains in concrete; vii) uplifts (Fig. 4); and viii) seepage and leakage (Fig. 4). Given the specificities of geodetic observation systems, a geodetic observation plan [4] was developed, which defines the characteristics of the system and the methods to be used to determine displacements.

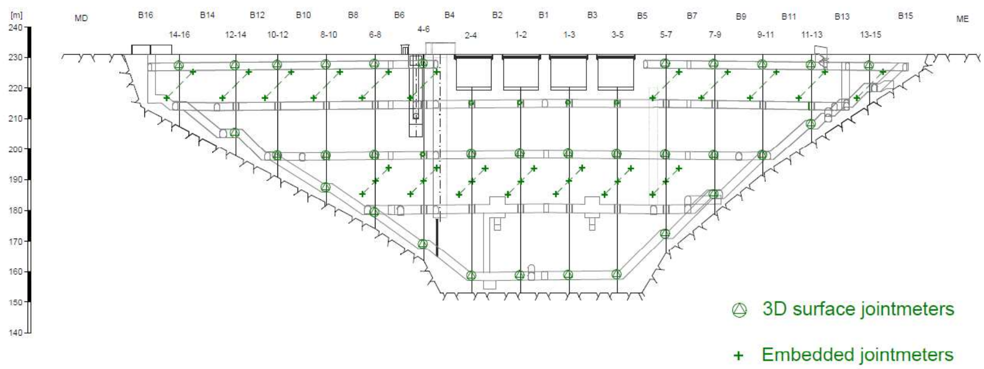


Fig. 2 – Dam monitoring system – Embedded jointmeters and 3D surface jointmeters

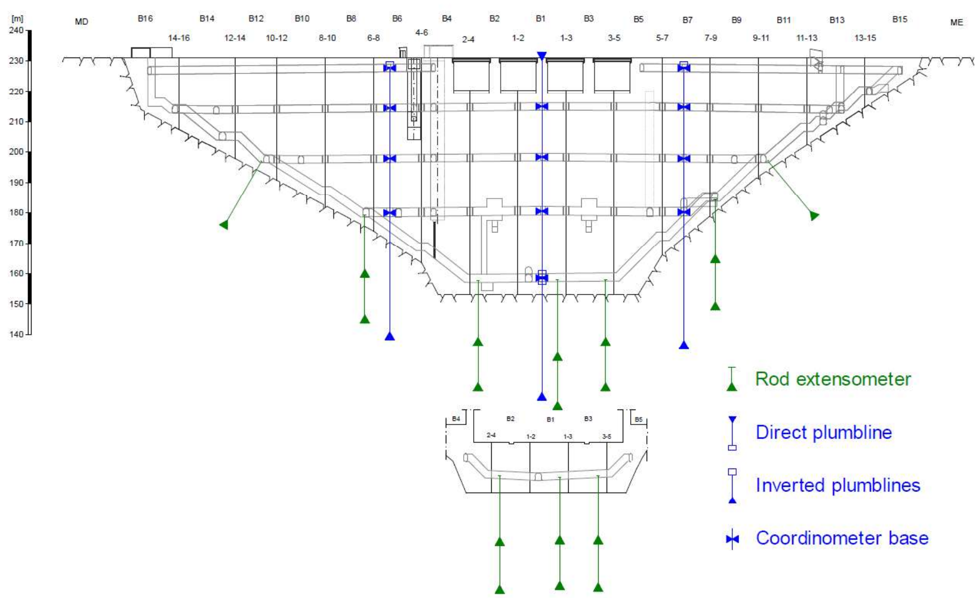


Fig. 3 – Dam monitoring system – Plumblines and rod extensometers

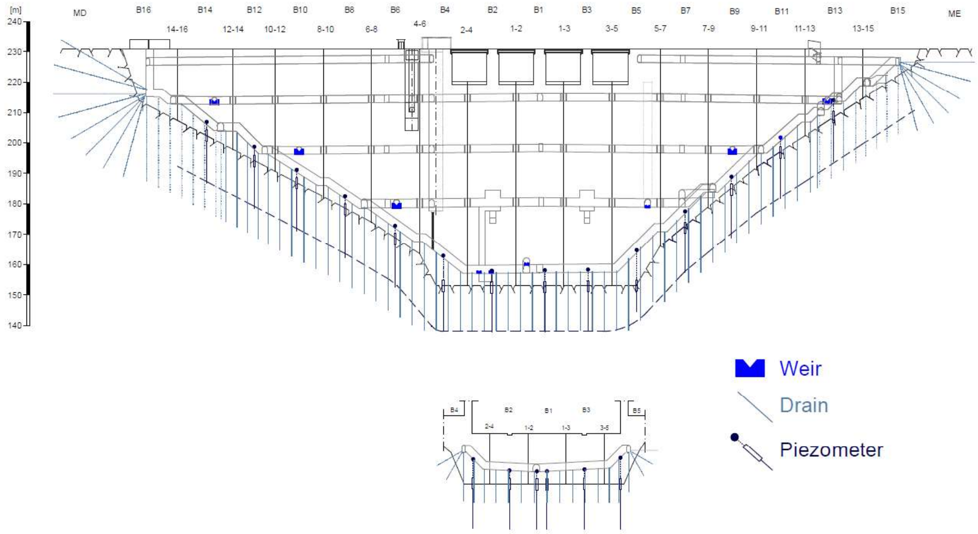


Fig. 4 – Dam monitoring system – Drainage system and piezometers

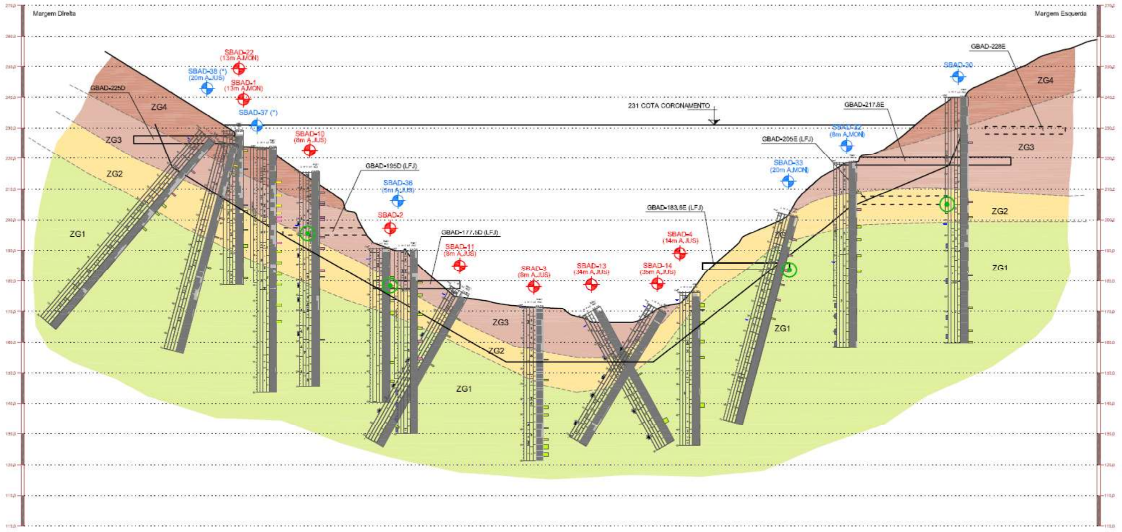
The stabilization thresholds and levels for the first filling of the Daivões dam reservoir were defined in the First Filling Plan. The plan included five observation levels (P) and a stabilization level (NE), namely, P1, P2, P3, NE1, P4 and P5-FSL, establishing that safety conditions would be assessed at the observation levels based on complete observation campaigns and visual inspections. At each of the stabilization thresholds or levels, it was established that the reservoir level would be maintained for a minimum period of 7 days, with constant hydraulic load, in order to allow the stabilization of the dam's behaviour and the execution of observation campaigns.

4. FOUNDATION AND CONCRETE PROPERTIES

4.1. Dam foundation properties

The rock mass foundation comprises two-mica granites, medium-grained and medium-to-coarse-grained. On each bank, 4 families of approximately vertical discontinuities and 2 families associated with slope decompression phenomena were identified. The rock mass foundation appears altered and fractured superficially, decreasing the degrees of alteration and fracture with depth, although differently along the insertion. Local observation, execution of surveys and field and laboratory tests were carried out to characterize the rock mass foundation, considering conventions defined by the International Society for Rock Mechanics (ISRM) [5].

Fig. 5 represents the geomechanical zoning of the foundation mass in which four zones were considered, with modulus of deformation ranging from 0.5 to 20 GPa. The rock mass presents low modules of deformability in the surface areas of the slope on the right bank and in the upper area of the slope on the left bank. The areas with the highest modulus of deformation are the bottom of the valley and the lower and intermediate sections of the left bank slope.



Geomechanical zoning	Meterioration degree	Fracture degree	Rock quality designation RQD (%)
ZG4	W4-5	F5	0-60
ZG3	W3-2	F3 a F4 (F5, F2)*	40-100
ZG2	W2	F2 a F1 (F4, F5)*	70-100
ZG1	W1	F1 a F2 (F4, F5)*	100 (10-90%)*

Fig. 5 – Geomechanical zoning of the foundation rock mass (view from downstream) [6]

4.2. Dam concrete properties

The concrete of Daivões dam used VLH IV/B (V) 22.5 cement, which incorporates c.a. 45% of its mass in fly ash. The aggregates used in the concrete are granitic. The fine aggregates were divided into two classes, 0/1.2 mm and 1.2/5 mm, and the coarse aggregates were divided into four classes: 5/15 mm, 15/30 mm, 30/70 mm, and 70/150 mm. The admixture Muraplast FK 88i was used as a plasticizer.

The experimental work was focused on the characterisation of the deformability properties of both the dam core concrete (DAM150) and the concrete wet-screened through a 38 mm sieve (SCR38), which was obtained from the previous one. Table 1 shows the values related to the particle size distribution of the aggregates used in the dam and screened concretes. The composition of the SCR38 was estimated from the dosages of the DAM150 by removing the approximate volumes of aggregates larger than 38 mm and the volume of the surrounding mortar lost during the screening process [7]. Table 2 shows the compositions of the two types of concrete and their main proportions by mass. Note that c.a. 3/4 of the added water corresponds to ice, and the aggregates were pre-cooled.

Table 1 – Particle size distribution of the aggregates used in the dam concrete (DAM150) and in the respective screened concrete (SCR38)

Concrete type	Fine aggregate (kg/m^3)		Coarse aggregate (kg/m^3)				
	0/1.2 mm	1.2/5 mm	5/15 mm	15/30 mm	30/38 mm	38/70 mm	70/150 mm
DAM150	305.3	270.2	312.1	385.3	480.1		496.9
SCR38†	423.7	367.9	495.6	611.8	152.3	-	-

† Estimated value

Table 2 – Dosages of the dam concrete (DAM150) and the respective screened concrete (SCR38)

Type of concrete	Cement VLH IV B (V) (kg/m^3)	Added water (kg/m^3)	Aggregate water (kg/m^3)	Total water (kg/m^3)	Additive (Muraplast FK 88i) (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)	Water-binder ratio
DAM150	180.0	88.5	14.9	103.4	1.7	575.5	1676.1	0.57
SCR38†	251.9	127.4	23.6	151.0	2.5	790.8	1262.2	0.60

† Estimated value

The obtained values of modulus of elasticity reveal greater stiffness of the screened concrete in relation to the integral one (Fig. 6), despite the latter's higher aggregate dosages. This result may be related to the higher binder dosage in screened concrete and the better compaction conditions that screened concrete allows (Table 2). It should be mentioned that the average values of compressive strength at 90 days of age, which were 26.9 MPa and 31.5 MPa in the tests of cylindrical specimens of integral concrete ϕ30×44 cm and of cylindrical specimens of screened concrete ϕ15 ×30 cm, are greater than the characteristic value of 22 MPa defined in the project. For one-year-old concrete, these values reach 36.5 MPa and 45.3 MPa (Fig. 7).

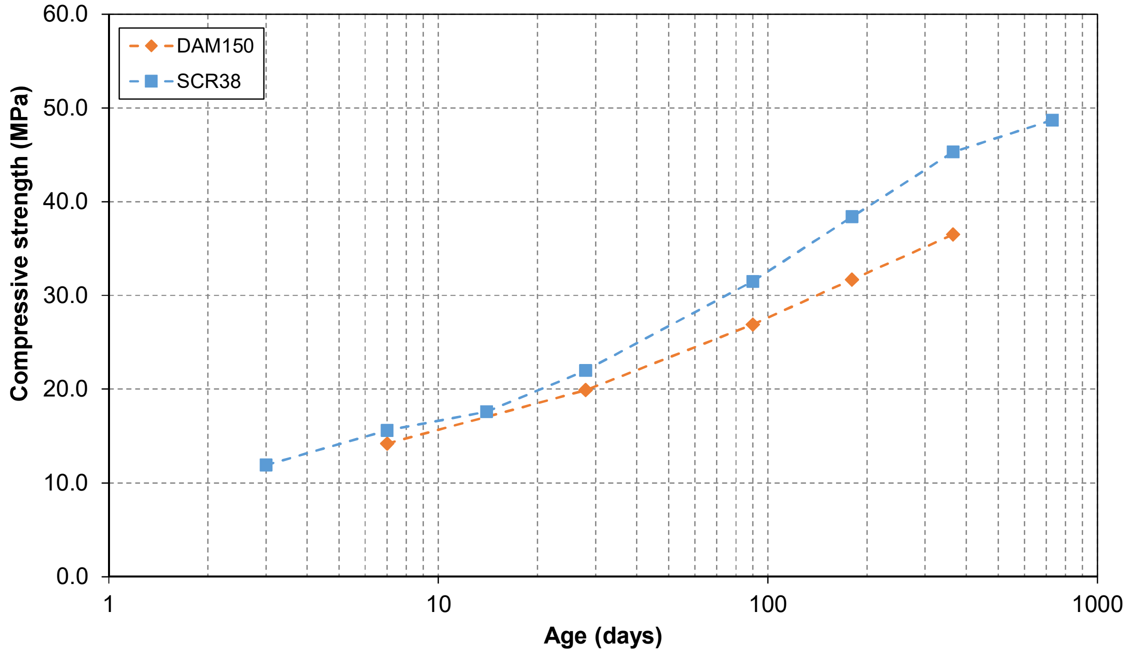


Fig. 6 – Evolution of the modulus of elasticity estimated from laboratory tests for quality control

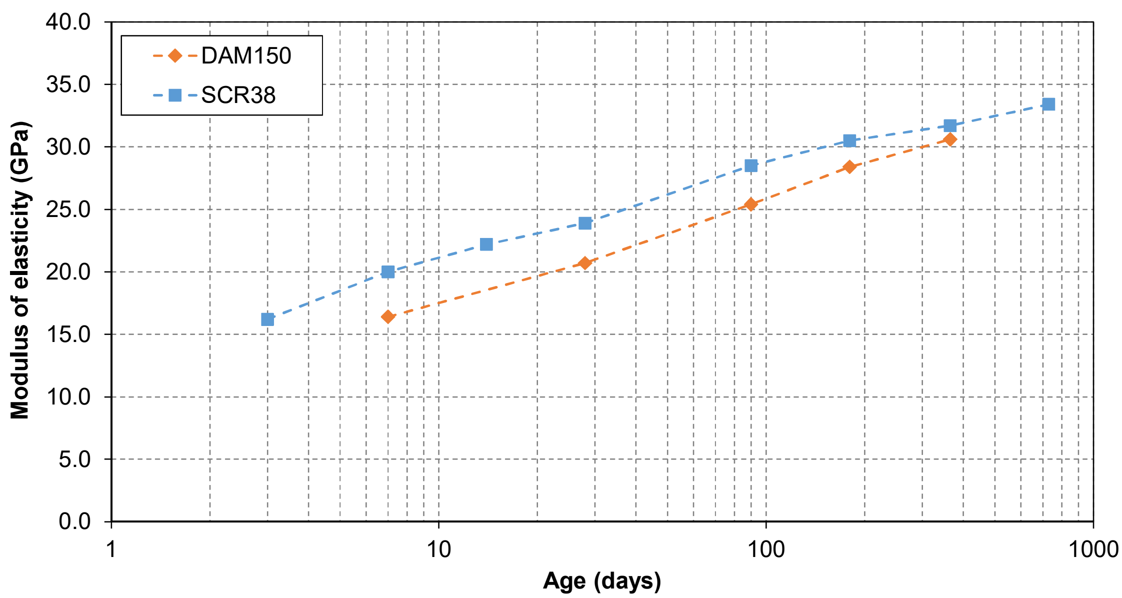


Fig. 7 – Evolution of compressive strength obtained from cylinders in the laboratory tests for quality control

5. ANALYSIS AND INTERPRETATION OF THE OBSERVED BEHAVIOUR

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 (Fig. 8). The model was analysed by the finite element method, using 20-noded isoparametric brick elements with second-degree shape functions, with homogeneous and isotropic properties.

The foundation was divided into two regions of different deformability: the upper areas in both banks and the bottom of the valley and in depth. The dam was divided into several regions according to the date on which the concrete was placed, between May 2018 and January 2020.

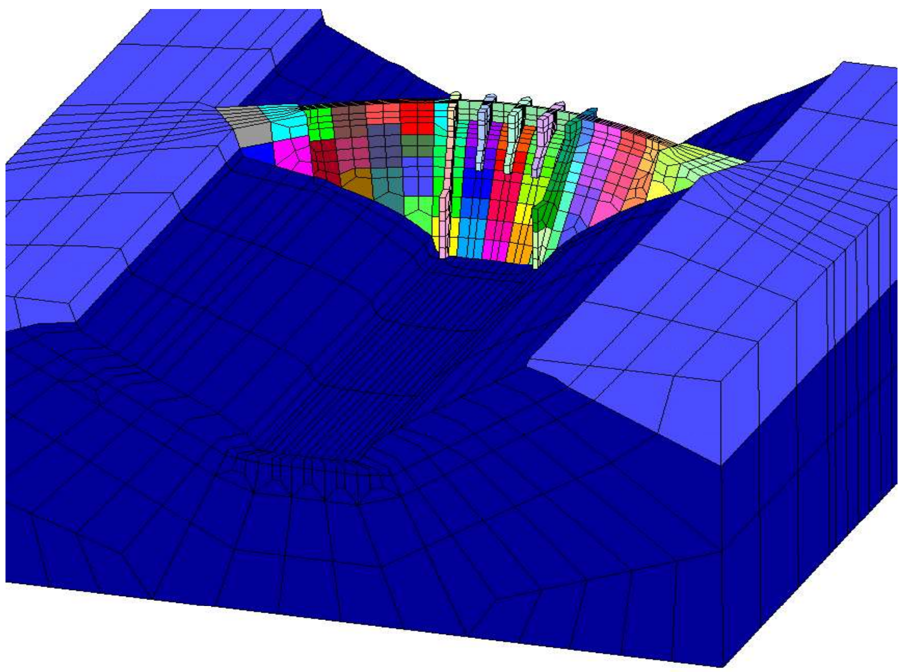


Fig. 8 – Representation of the finite element mesh of the dam-foundation set, considering the zoning of the dam and the rock mass foundation

The rock mass foundation was assumed to be an elastic behaviour and the concrete was assumed to be a viscoelastic material, whose deformability was characterised by equation 1. The main parameters considered for the material properties are presented in Table 3.

Poisson's ratio (ν) was considered equal to 0.20 for the dam concrete and the foundation mass.

Based on the temperatures and strains observed in the correcting strain gauges installed in the dam, a linear thermal expansion coefficient (α) equal to 1.0×10^{-5} $^{o}C^{-1}$ was obtained for the structural concrete.

Table 3 – Material properties considered in the finite element model

Type of material	E (GPa)	ν (-)	α (1/ºC)
Dam concrete	(Eq. 2)	0.2	1.0×10^{-5}
Superficial at elevated ground	12	0.2	0
Right bank and bottom of the valley	18		

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. To consider the delayed effects related to 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.

Creep prediction models are associated with functions that have specific characteristics, chosen to represent the physical phenomenon of concrete creep. The parameters of these functions, typically determined from known material properties and, when available, adjusted to experimental test results, provide an estimate of creep strain values within certain assumptions. In the Bažant and Panula (BaP) model [8], the creep function, $J(t, t_0)$, is given by the sum of the elastic component, $1/E_0$, basic creep, $C_0(t, t_0)$, and drying creep, $C_d(t, t_0, t')$. One of the distinctive aspects of this formulation is the consideration of the concrete's maturation process in the basic creep component by multiplying the power of the age at loading, t_0, by the power of the time under load, t-t_0.

The creep function coefficients were adjusted to the results of tests carried out in the laboratory with screened concrete, Equation (1). Fig. 9 and Fig. 10 present, respectively, the adjustment of the creep function to the experimental results of specific extension and the experimental results of elastic modulus obtained in the laboratory tests.

$$J(t,t_0) = \frac{1}{E_0} + \frac{\phi_1}{E_0}(t_0^{-m} + \alpha)(t - t_0)^n = \frac{1}{34.3} + \frac{4.04}{34.3}(t_0^{-0.42} + 0.00)(t - t_0)^{0.11} \quad (GPa^{-1}) \qquad (1)$$

The prediction of the elasticity modulus over time can be obtained by considering the time under load, t-t_0, as equal to 0.1 days,

$$\frac{1}{E(t_0)} = \frac{1}{E_0} + \frac{\phi_1}{E_0} 10^{-n}(t_0^{-m} + \alpha) = \frac{1}{34.3} + \frac{4.04}{34.3} 10^{-0.11}(t_0^{-0.42} + 0.00) \qquad (GPa^{-1}) \qquad (2)$$

where E_0, φ_1, m, α e n are parameters dependent on the intrinsic characteristics of the concrete. Since, in the case of dams, the concrete does not, in practice, lose water to the outside, the drying creep component may be considered negligible compared to the basic creep values.

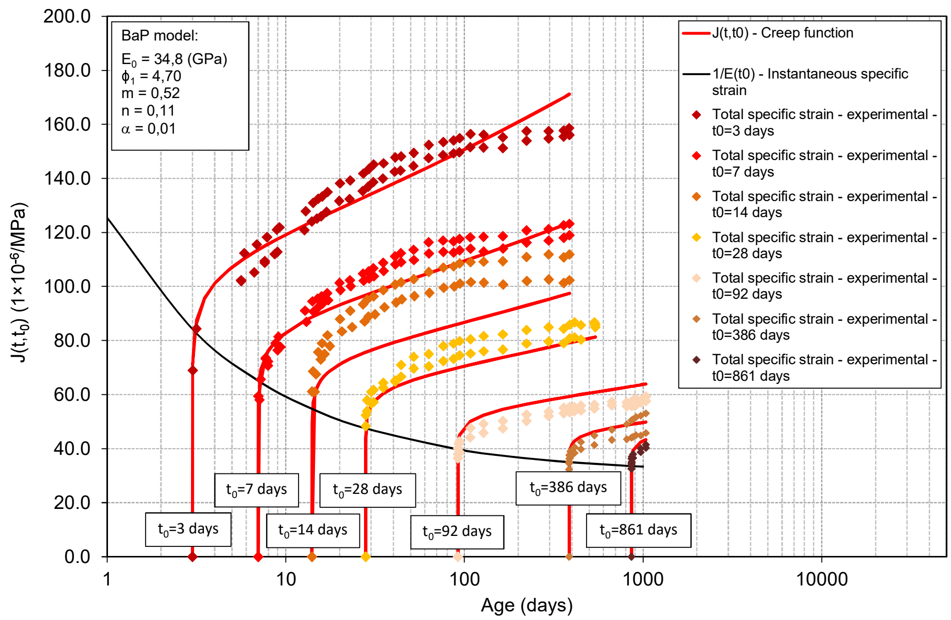


Fig. 9 – Representation of the creep function fitted from specific strains obtained experimentally for various loading ages

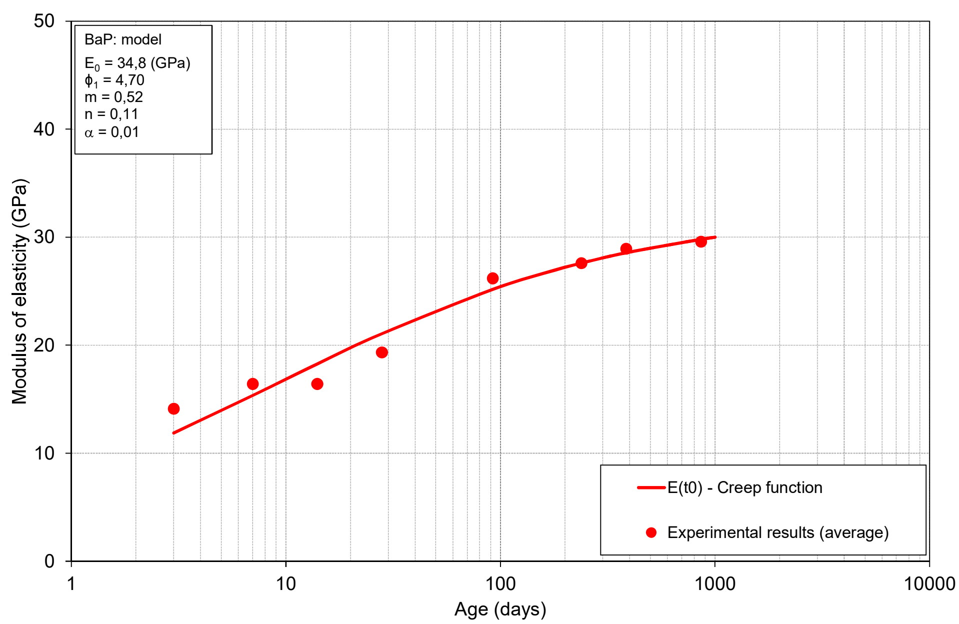


Fig. 10 – Evolution of modulus of elasticity obtained from experimental tests and from the creep function adopted

5.2. Evolution of the main actions

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.

Fig. 11 shows the reservoir water level evolution, as well as the daily level change rate and the accumulated daily precipitation measured at the meteorological station. The water level rose rapidly in February 2021 due to an intense period of rain. After that, the level rose gradually, considering all the predefined levels for safety control assessment, as defined in the first filling plan.

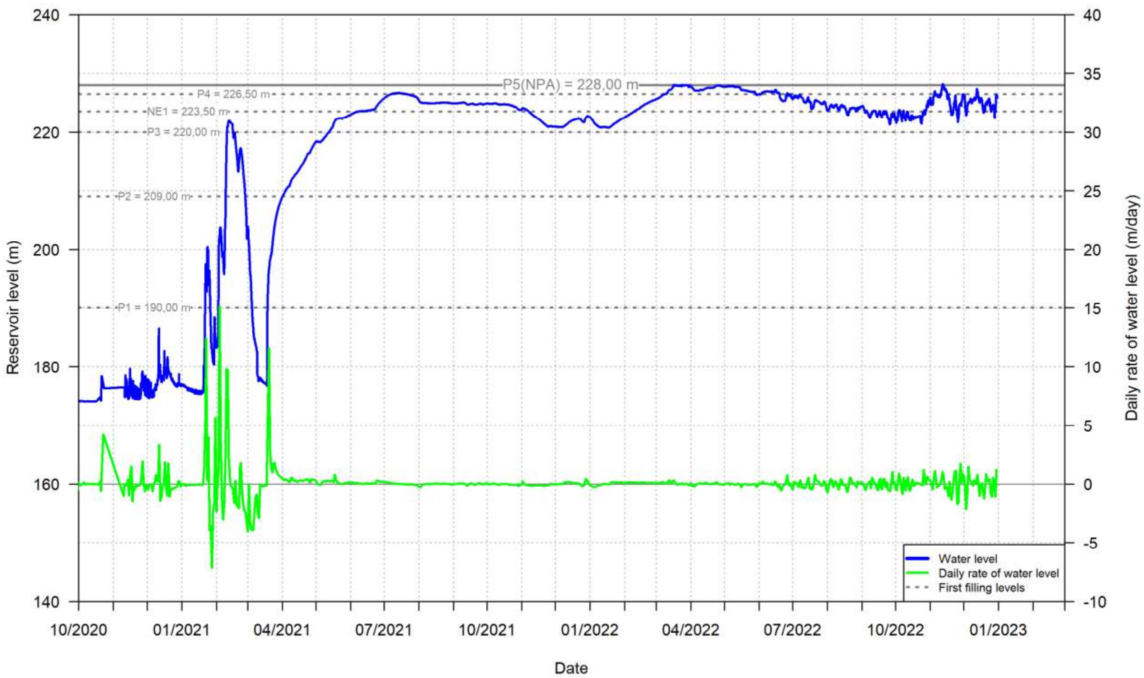


Fig. 11 – Reservoir level and daily level rate

The hydrostatic pressure on the upstream face of the dam was simulated considering a specific weight of 10 kN/m^3 for the water. The effects of uplift in the insertion between the dam and the foundation, and the hydrostatic pressure downstream, were not considered. The hydrostatic pressure in the gates was transmitted along the height of the pillars, at the intersection between the gates and the pillars in the spillway area.

The temperature variations considered in the finite element model were estimated from the temperatures observed in thermometers and electrical resistance strain gauges embedded in the dam body, considering the predominant thermal flow in the radial direction and using an interpolation method in the thickness of the dam and along reference planes. Fig. 12 shows the isothermals calculated on some key dates during the reservoir filling.

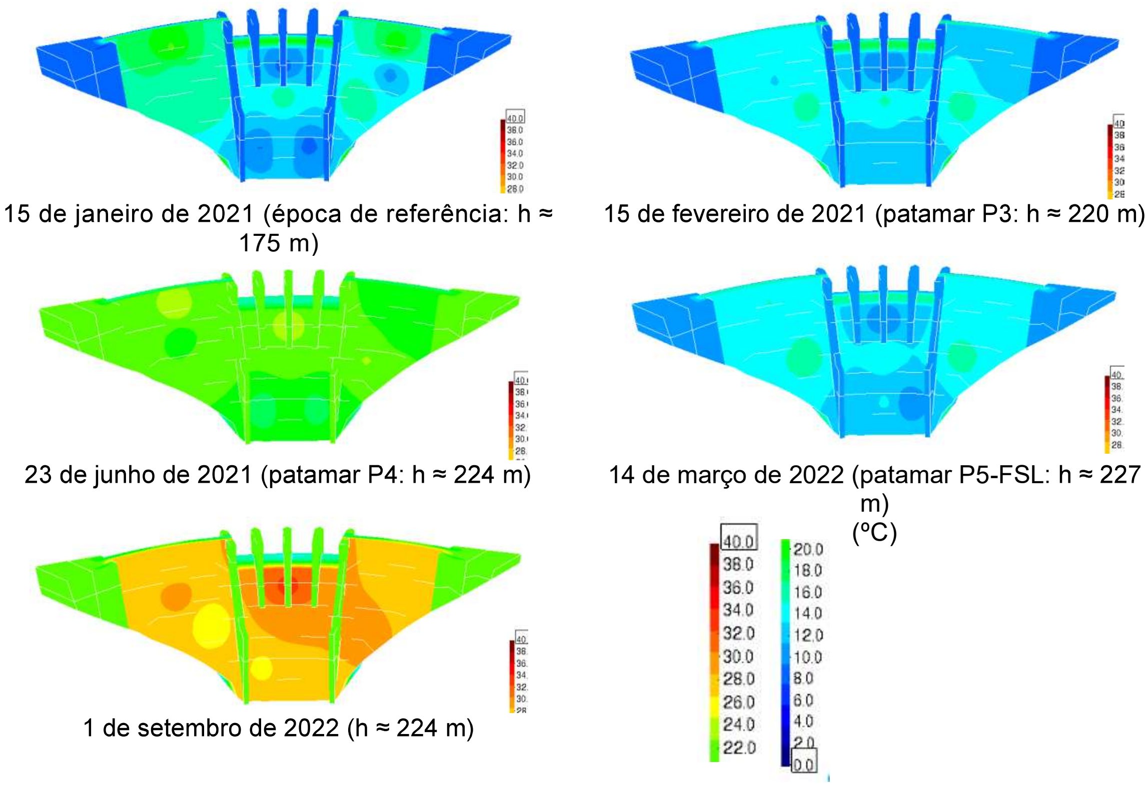


Fig. 12 – Calculated thermal field in the dam body on different dates during the first filling of the reservoir

5.3. Joint movements

Joint movements are generally small and are correlated with temperature variations in the dam body and the reservoir water level, with the greatest variations being observed in joints at higher elevations, close to the abutments. From the second half of March 2021 until the beginning of July 2021, coinciding with the refilling of the reservoir and the temperature increase of the dam body, movements were observed towards the closure of the joints. Between July 2021 and mid-January 2022, there were minor movements towards closing the joints followed by movements towards opening compatible with variations in concrete temperature. Between the second half of January and March 2022, the increase in the reservoir level up to the FSL contributed to movements towards the closure of the joints, so that the effects of level and temperature (with opposite directions) led to small movements (Fig. 13).

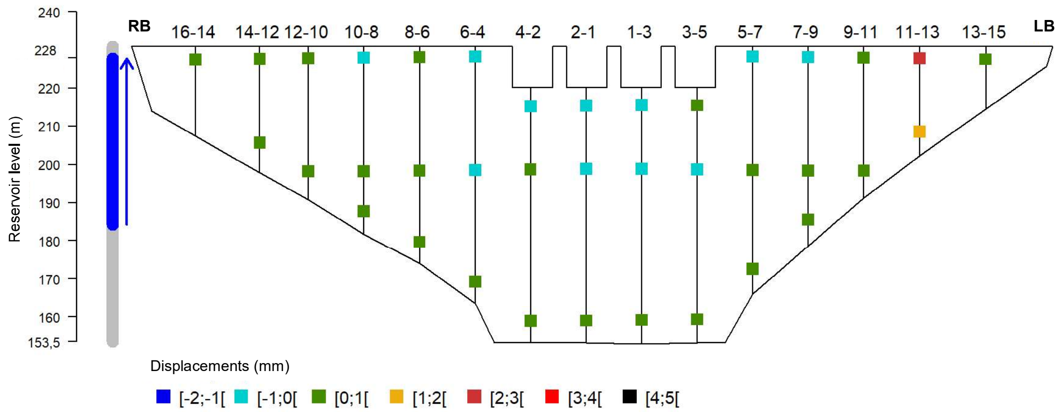


Fig. 13 – Opening and closing movements observed in 3D surface jointmeters on 03/16/2022 (P5-FSL level, h = 228 m)

5.4. Horizontal displacements

The radial and tangential displacements observed in various coordinometer bases of the central plumb line are presented (Fig. 14, the vertical dashed line corresponds to the reference date used in this analysis, January 15, 2021). The sign convention determines that positive values correspond to radial displacements towards the upstream and tangential displacements towards the left bank.

The numerical model confirms the pattern of observed behaviour, for the combined effect of the reservoir level and the temperature variations of the dam concrete. In general, the calculated results are like the results observed at the points located at the highest elevations (227 m and 231 m) and at the lowest elevations (214 m), for the plumb lines on the right bank side, central block and on the left bank side. Fig. 15 shows an example of the comparison between observed and calculated displacements at the highest elevation in the central block and the magnitude of each effect over time, namely, the hydrostatic pressure and the temperature variations.

The separation of the effects of water level and temperature highlights the magnitude of the displacements for each of these actions. At the upper level of the central zone of the dam (block B1, level 231 m), the maximum radial displacements due to the action of water upstream were around 10 mm downstream, in March 2022 with the FSL level, while the displacements Maximum radial values due to thermal action were about 10 mm upstream, estimated during the last warm season, in September 2022.

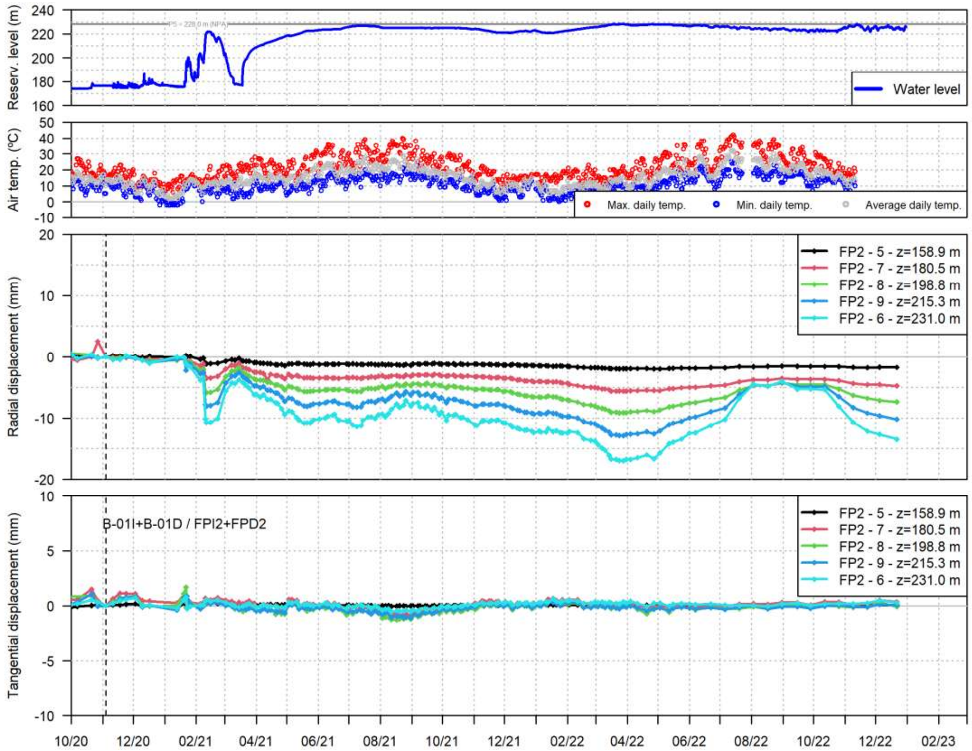


Fig. 14 – Radial and tangential displacements observed in plumb lines B-01I and B-01D / FPI2 and FPD2, located in block B1

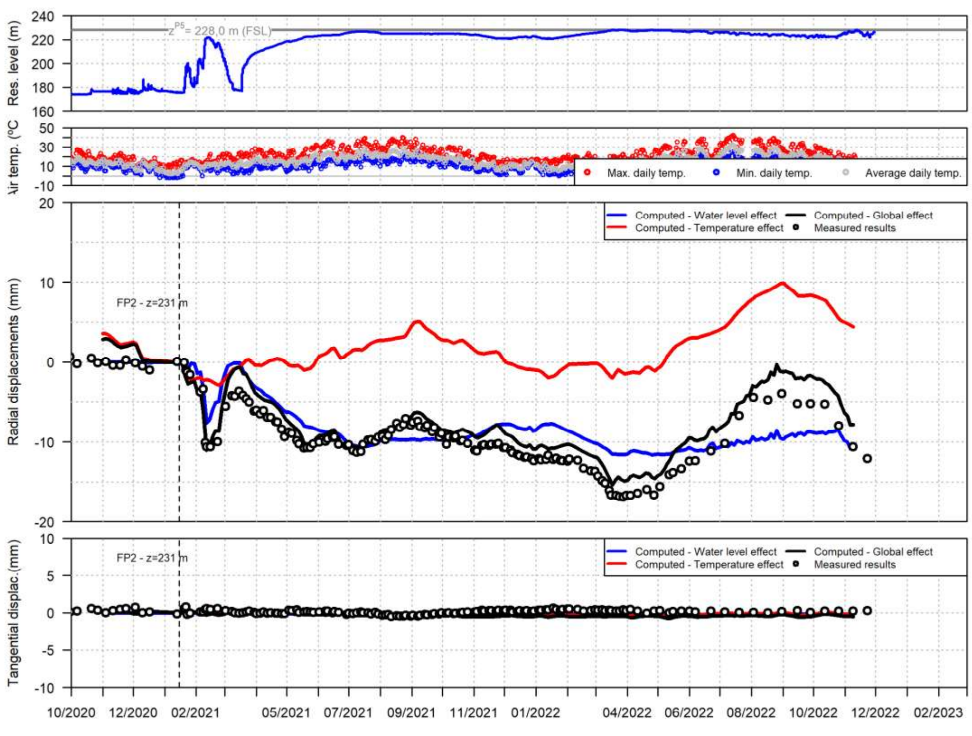


Fig. 15 – Comparison of the calculated and observed radial and tangential displacements at elevation 231 m on the FP2 plumb line, in block B1

5.5. Vertical displacements

A comparison is presented between the results obtained with the numerical model and the results of observation in rod extensometers located in the foundation, in the main drainage gallery and in the downstream gallery (Fig. 16). The sign convention determines that positive values correspond to bulging of the foundation mass and that negative values correspond to settlements of the foundation mass.

The results show a good agreement between the vertical displacements calculated with the model and the vertical displacements, considering the order of magnitude of the values (less than 1.5 mm). It is noted that the numerical model used considers the rock mass as an equivalent continuous elastic medium, and does not consider the non-linear displacements that may occur in the foundation.

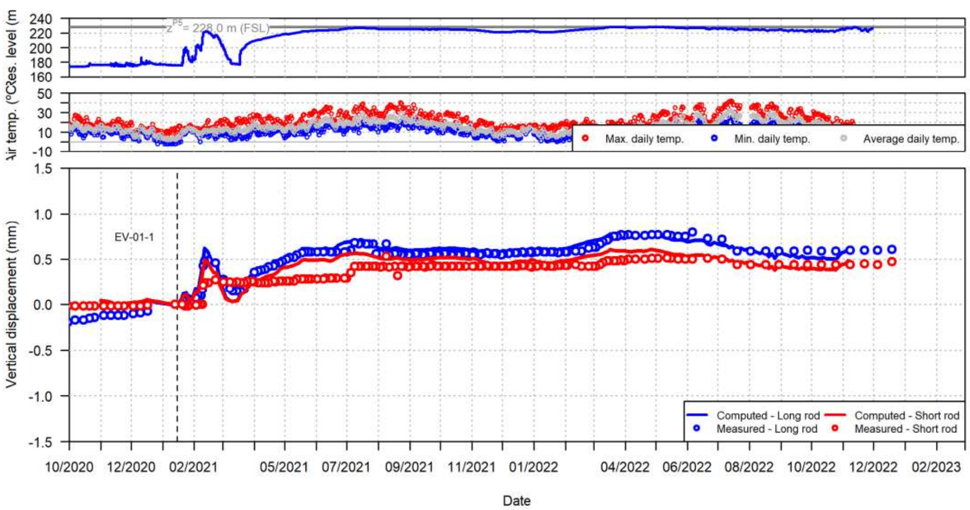


Fig. 16 – Comparison of vertical displacements calculated with the finite element model and those observed in the rod extensometer EV-01-1, located in the drainage gallery - block B1

5.6. Hydraulic behaviour of the foundation

Flows into the drainage network showed reduced values in all drains until February 2022, a date from which some drains recorded significant increases in flow, a unique situation that was monitored by increasing the frequency of measurements (Fig. 17). It was observed that a specific drain registered an increase in flow directly related to the increase in level up to the FSL, reaching a maximum of 53.9 l/min on March 18, 2022, with the level at 228 m. On this date, the average value of the drained flow in blocks B6 and B8 was 16.3 l/min and 5.9 l/min, respectively, which correspond to acceptable values for the type of dam.

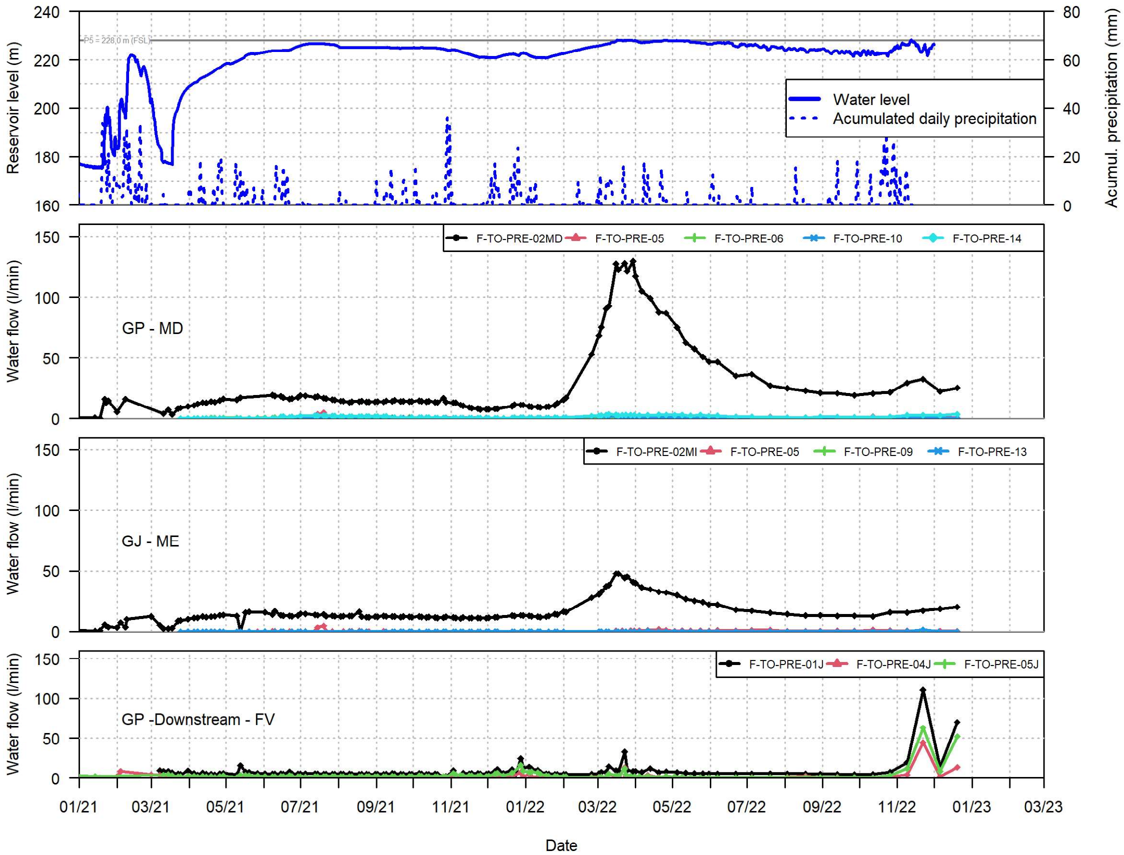


Fig. 17 – Total flows (drained and infiltrated) observed in weirs located in the drainage gallery, in the radial gallery and in the downstream gallery

Analysing globally the values of the piezometric levels observed along the drainage gallery between the months of February 2021 and March 2022, the uplifts present reduced values up to the P4 level, and from the level rise in February 2022 and with the transition to the cold season, usually associated with some decompression in the upstream area, the referred uplifts increased significantly. During the period of stabilisation of the level at the P5 level (FSL), there was stabilisation and even a slight reduction in the uplift values in the piezometers P-03-1 and P-04-1 (Fig. 18). The other piezometers showed low uplift, with percentages of hydraulic load appropriate to the type of work.

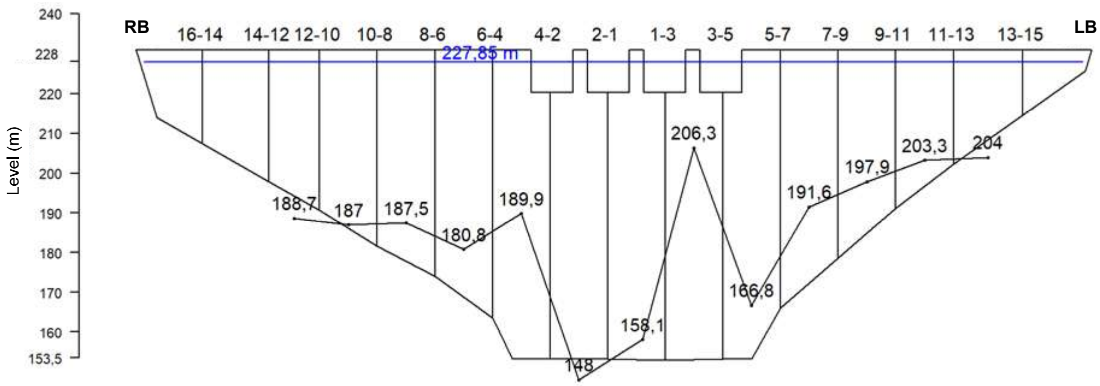


Fig. 18 – Piezometric elevations corresponding to the uplift observed in the drainage gallery on March 21, 2022, with the reservoir at approximately 228 m

6. CONCLUSIONS

The safety control of Daivões dam during the first filling of the reservoir required the installation and proper use of the instrumentation system, the complete characterisation 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 Daivões dam behaviour during the first filling of its reservoir was 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, considering the material's properties. Most of the devices of the monitoring system presented lower water flows and uplift pressures, indicating adequate hydraulic behaviour of the foundation. In summary, it can be concluded that the dam presented adequate structural and hydraulic behaviours.

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

7. ACKNOWLEDGEMENTS

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REFERENCES

[1] RSB (2018). Dam Safety Standard (in Portuguese). Decree-Law nº 21/2018, Diário da República. Lisbon.

[2] DTA (2018). Technical documents supporting dam safety standard (in Portuguese). Lisbon.

[3] LNEC (2011). Daivões Dam. Observation and first filling plans (in Portuguese). Report 53/2011 – NO. LNEC. Lisbon.

[4] LNEC (2020). Daivões Dam. Plan of the geodetic observation system (in Portuguese) . Report 203/2020 – DBB/NGA. LNEC. Lisbon.

[5] International Society for Rock Mechanics (1981). Basic geotechnical description of rock masses (BGD). International Journal of Rock Mechanics and Mining Sciences and Geomechanics. 18:1 (1981) 85–110.

[6] ATACI (2011). Daivões Hydroelectric Plan. Design (in Portuguese).

[7] Serra, C. (2017). Prediction of dam concrete structural properties based on wet-screened test results and mesoscale modelling. Faculdade de Ciências e Tecnologia. Universidade Nova de Lisboa, 2017, PhD Thesis, 368 p. Lisbon.

[8] Bažant, Z. P., and Osman, E. (1976). Double power law for basic creep of concrete. Materials and Structures (RILEM, Paris), 9(49), 3–11.