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CONSTRUCTION AND FIRST FILLING OF PINHÃO CONCRETE FACE ROCKFILL DAM

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Abstract. *The Pinhão dam is located in the municipality of Vila Pouca de Aguiar in the district of Vila Real, intersecting the Pinhão River. Its primary purpose is to create an urban water supply reservoir of 4.24 x10^6 m^3.*

The dam has a profile of granite riprap with a concrete face as the impervious curtain. The maximum height of the dam is 22 m. Construction took place between September 2006 and March 2008 when, after authorisation was given by INAG, first filling started.

This paper is intended to provide basic data on the dam and its construction phase and on the monitoring activities undertaken by the ATMAD in close collaboration with LNEC and INAG to ensure the safe control of this dam.

1 INTRODUCTION

The Pinhão dam (Figure 1) is located in the municipality of Vila Pouca de Aguiar in the district of Vila Real, intersecting the Pinhão River. Its primary purpose is to create a water reservoir of 4.24 x10^6 m^3 for urban supply.
The dam has a homogeneous profile made of granite rockfill. The water tightness is ensured by a concrete face casted over a cushion layer also made of (finer) rockfill.
The maximum height over the foundation is 22 m. The freebody relative to the maximum water level is 2.0 m and to the normal water level is 3.5 m. The dam crest level is 682.5 m and, consequently the:

- maximum flood level (MFL) is 680.5 m for a reservoir storage of 5.19 hm^3,
- normal water level (NWL) is 679 m for a reservoir storage of 4.24 hm^3, and
- minimum operating level is 671 m to a stored volume of 0.64 hm^3.

The crest is straight with a length of 285 m and a width of 7 m. The upstream slope has a gradient of 1:1.4 (v:h) and the downstream slope has a 1:1.5 gradient with a bench of 3.0 m

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at an elevation of 672.5 m.
The dam is located on a plateau zone of a very open and asymmetrical valley. The abutments have gentle slopes of 1:8.8 (v:h) for the left abutment and 1:4.2 (v:h) for the right abutment.
According to geological and geotechnical characterisation, the dam foundation essentially consists of medium and coarse grained, sound granite.
The dam foundation has been subjected to a general cleaning procedure, removing loose blocks, soils and organic matter. After this phase, it was decided to level the foundation surface, taking away any sharp rocks and filling any holes in the rock with concrete.
During this phase, and despite it being carried out during the summer, several springs were detected. All of these springs were captured and diverted to a drain where a weir was constructed in order to evaluate the total flow due only to the natural springs.
This measuring system was separated from the one that is used to measure the seepage flow of the dam. Therefore, in the monitoring system two separate flow measurements are taken.

Figure 1: View of Pinhão dam from the left abutment

On the plinth area, two distinct rock quality zones occurred, both of which were good enough to allow an adequate foundation for this important part of the dam.
The excavation of the foundation for the plinth was done using a hydraulic hammer. The structure of the plinth, with a base width of 5.5 m and a minimum thickness of 0.60 m, was secured to the foundation through a series of nailing rods of Ø32 mm spaced at 2.5 m. The joints between the plinth and the panels, as well as the joints between panels themselves, incorporated water-stop joints as sealing elements. From the plinth, a grout curtain, consisting of injections for consolidation with an average depth of 5 m, and injection of holes with depths of 15 to 20 m, was created.

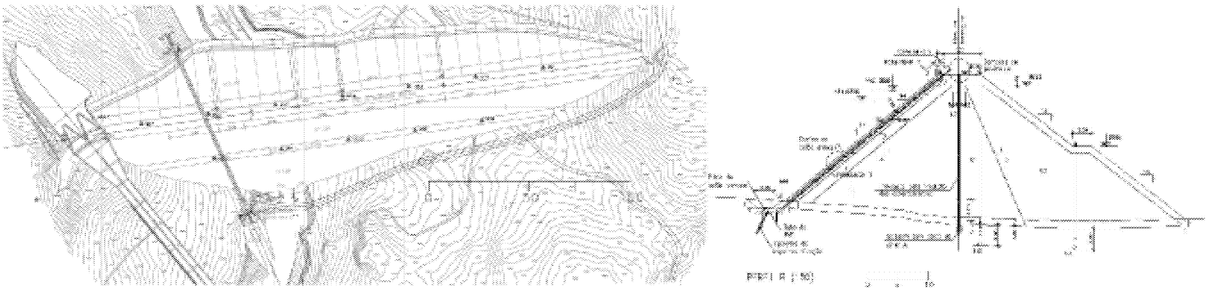


Figure 2: Plan and cross-section of the Pinhão dam

2 MONITORING SYSTEM

The monitoring plan was conceived by LNEC (2007a)[1], taking in consideration the typical behaviour observed in this type of dam and the devices needed to monitor the data relevant to the safety control of the dam.

The monitoring system includes devices for measuring the reservoir water level, meteorological data, surface and internal displacements, water pressure in the foundation and the total flow discharge.

In respect to surface displacements, the dam has 12 surface benchmarks, allowing the measurement of vertical displacements (by high accuracy levelling) and the measurement of horizontal displacements.

Seven of those benchmarks are located in the dam's crest and the rest in the downstream bank.

To measure the internal displacements, 8 inclinometers where placed in 4 cross-sections. Each cross section comprises a vertical inclinometer located near the dam axis, and a inclined one located beneath the concrete face. This last inclinometer is crucial in order to evaluate the integrity of the impervious membrane.

The water pressure in the foundation is measured in the same profiles where the inclinometers are located. In each profile 2 standpipe piezometers are placed. The first is near the dam axis and the other in the downstream bench. The two piezometers allow for a better understanding of the water pressure dissipation at the foundation of the dam. If the grout curtain is behaving properly, both piezometers (located upstream and downstream) should only have residual pressure.

Seepage flows are measured in two V shaped weirs located downstream near the dam toe. There are two weirs. The first, measures the flow from the springs captured during construction. The second one measures the flows that pass through the dam and foundation. Due to the design of the dam and the nature of the dam, this weir also captures water from rainfall.

The frequency of measurements established in the monitoring plan, comply by all the regulations (NOIB, 1993)[3], but were also established for taking into account the need for an effective security control of the dam during its life. Furthermore, it was considered important to maximise the amount of data collected during the early stages of the dam's life in order to get valuable data to feed and calibrate statistical models that allow for safety control.

Table 1 shows the frequencies recommended in the monitoring plan for the first filling phase. It is worth noting that, except for rainfall, the actual frequencies were higher than those recommended.

Table 1: Monitoring frequencies during the first filling (LNEC 2007b)[2]

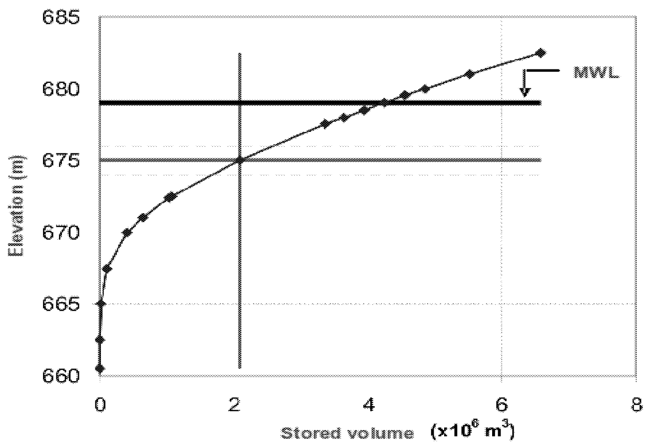
Variable	*Minimum frequency*	*Mandatory measurements*
Reservoir level	Daily	Not applicable
Surface displacements	Annual	a)
Internal displacements		
Flow	Biweekly	a)
Piezometric levels	Monthly	a)
Rain fall	Daily	Not applicable
Routine visual inspection	Monthly	a)
Special visual inspection	Annual	a)
Exceptional visual inspection	After event	Not applicable

a)Beginning, end of stairs, and end of first filling or rapid drawdown

3 FIRST FILLING

As required by Portuguese Dam Safety Regulations, the first filling of the dam was carried out according to a plan: The first filling Plan (FFP) (LNEC 2007b)[2], which took into account all relevant aspects of the dam design and construction. Thus, it was recommended that the filling should be conducted in two stages, with an intermediate level at the end of which, after a performance evaluation of the behaviour during the first phase, the filling could proceed to the MWL.

The intermediate level was set by taking into account the need to maximise the amount of water available for consumption within the safety aspects. It was considered that it would be desirable to have access to approximately 50% of the reservoir volume, so the level was established at an elevation 675 m (Figure 3).



Filling step	*Elevation (m)*	*% height*	*Storage x10^6 m^3*	*% of maximum volume*
Ideal value	675	65.9	2.09	49.3
Minimum value	674	61.4	1.69	39.8
Maximum value	676	70.5	2.6	61.3

Figure 3: Intermediate level design

3 BEHAVIOUR MODEL

The monitoring plan of the dam proposes performance models for seepage through the foundations and for the stress-strain behaviour during the construction phase. These models, established by using the finite element method (FEM), were done to predict the distribution of water pressure in the dam foundations, the seepage flow rates and finally settlement due to the construction.
The models where developed using the FEM code Code-Aster, released under GPL by EDF (Électricité de France). The finite element mesh and the post-processing where carried out using SALOME (http://www.salome-platform.org). Figure 4 shows the finite element mesh used in the calculations.
As the calculations were made prior to construction, two different scenarios regarding the permeability of the foundations were considered. The first scenario, which was more optimistic, corresponds to a more waterproof foundation, while the second corresponds to a more permeable foundation. The difference between the two calculation scenarios corresponds to a variation in the permeability of the foundation by a factor of ten.

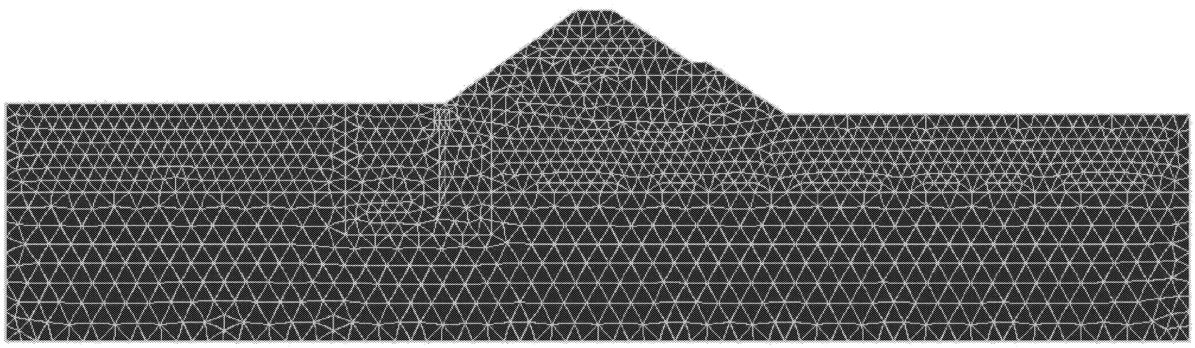


Figure 4: Finite element mesh (2383 triangular elements and 4875 nodal points)

Table 2 summarises the parameters considered. According to the calculations for the MWL, the expected flow rate should be between 0.079 m^3/day/m and 0.692 m^3/day/m. For the entire dam these values correspond to 11 and 98 m^3/day, which means in one year 0.1% and 0.86% of the reservoir volume. These values are considered acceptable for this dam.
Concerning the mechanical behaviour of the dam, the Monitoring Plan establishes a model of behaviour based on the mechanical characteristics considered in the project. These characteristics are summarised in Table 2.
According to the model developed, the vertical displacements expected for the dam due to the construction phase should be less than 3 cm (Figure 3).

Table 2: Parameters for the seepage analysis

Material	kx= ky (m / s)	kx= ky (m/day)
Dam body* (rockfill)	10^{-4}	86.4
Upper foundation	10^{-7} to 10^{-6}	$8.64x10^{-3}$ a $8.64x10^{-2}$
Grout curtain **	$10^{-8}$to 10^{-7}	$8.64x10^{-4}$ a $8.64x10^{-3}$
Lower foundation	$10^{-8}$to 10^{-7}	$8.64x10^{-4}$ a $8.64x10^{-3}$

* nominal value, chosen to represent a highly permeable material
** established as 10 times less than the upper foundation

Table 3: Parameters for the stress-strain model

Material	*E (MPa)*	*ν*	*γ (kN/m3)*
Bedrock	500	0.27	25
Rockfill	60 to 80	0.27	22

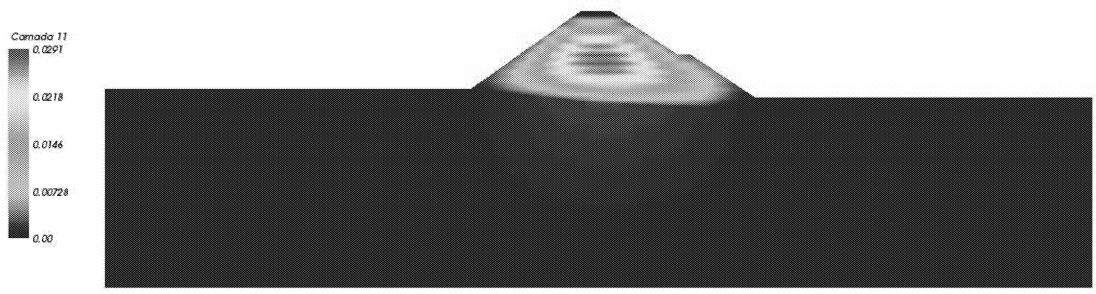


Figure 5: Regions of equal settlement at the end of the construction phase

4 MONITORING RESULTS

4.1 Water level

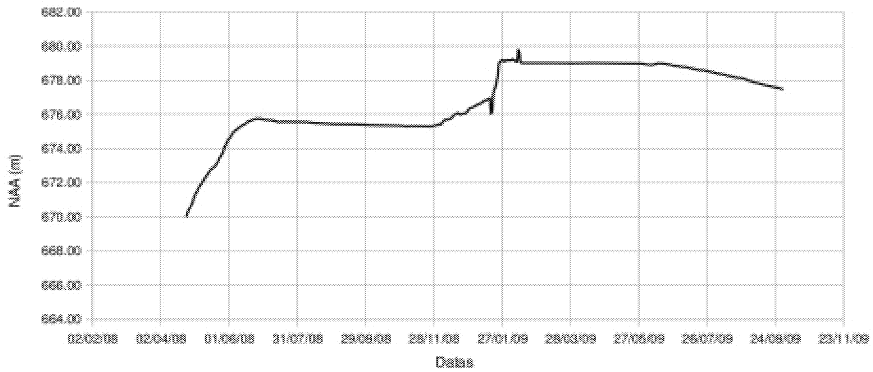
The main action to be considered in the dam's behaviour is the water level in the reservoir. In agreement with the first filling plan, the variations in the reservoir level were regulated. Figure 6 presents the variation of the reservoir level during the period of first filling. During the first filling up to the maximum water level height, achieved on the 02/10/2009, the reservoir rose at a rate of 9 cm/day until the first filling stair, and after a stationary period at that level, the water level has rose 6 cm/day until the first discharge in the spillway on the 01/24/2009.

4.2 Internal horizontal displacements

As previously stated, the dam has 8 inclinometer tubes installed in 4 cross-sections. Figs. 7 and 8 show the diagrams with the displacements recorded in the taller cross sections. It is worth noting that the displacements in the upstream curtain correspond to a slanted profile. The analysis of the results presented in these these figures show that, to date, the horizontal displacements recorded in the dam have a reduced expression.

At the concrete face, the maximum displacement occurs, as expected, in the upper zones of the dam, at profiles PB (150) and PC (175), reaching at the crest values of around 20 mm. At profiles PA (100) and PC (200) (not represented) the maximum values recorded are about half that amount.

In general, the displacements recorded by the vertical inclinometers are low. The inclinometers IV2, IV4 and IV8 have displacements of less than 2 mm. The only exception occurs in the inclinometer IV6 (profile C (175)) where the displacement obtained in July 2009 is higher, especially in the towards the right bank. It should be noted that on that date, the water level was 678.64 m while in September 2008 the water level was 677.67 m. Therefore, a relationship seems to exist between these the two quantities



Máximo=	679.8	em	10/02/09	Média=	676.61
Mínimo=	670	em	24/04/08	Desvio Padrão=	2.26

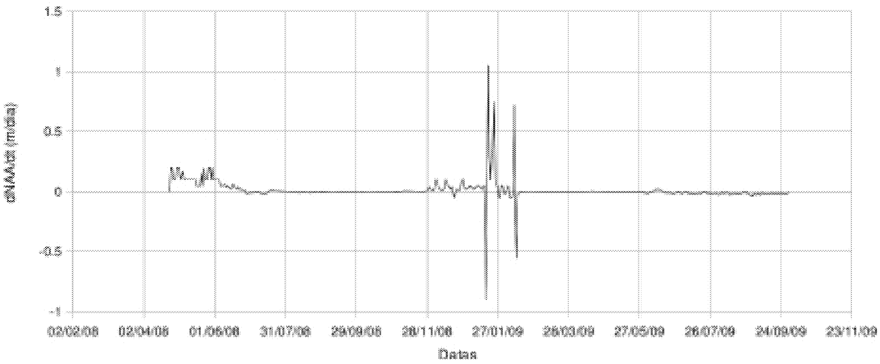


Figure 6: Reservoir water level and daily variations

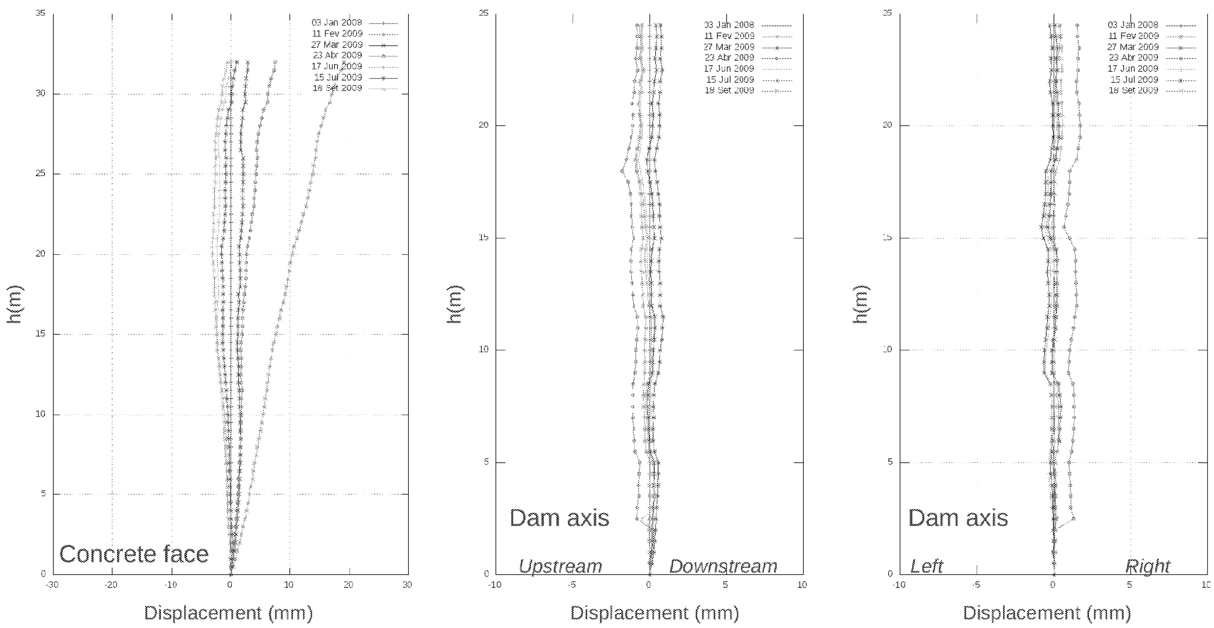


Figure 7: Horizontal displacements at cross-section B (150)

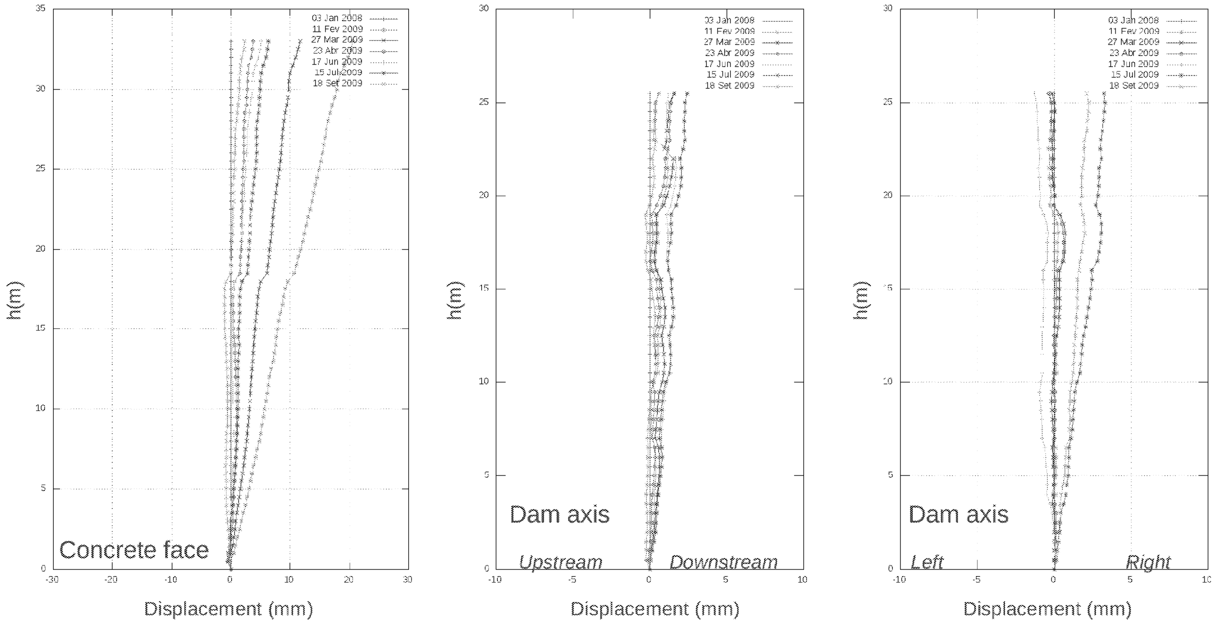


Figure 8: Horizontal displacements at cross-section C (175)

4.3 Internal vertical displacements

Regarding the measurement of settlements during first filling, the values measured in the settlement gauges (in the same tubes as the inclinometers) may be considered as residual. The diagrams shown in Figure 9 indicate that the settlement occurs mainly in the lower 5 meters, remaining fairly constant up to the surface. It is also worth noting that the dispersion which occurs in various diagrams is related to the precision of the method.

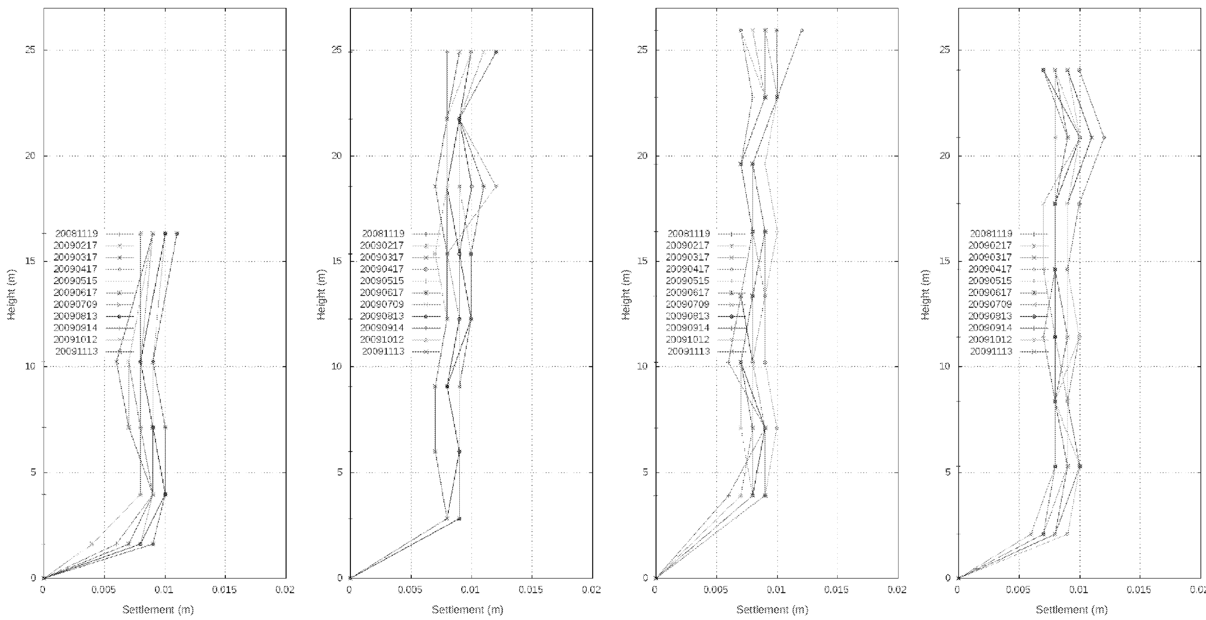


Figure 9: Settlements in recoded by the settlement gauges

4.4 Water Pressure in the foundation

The dam has 8 piezometers installed to measure the pressure drop along the foundations. The location of these devices is presented in Figure 2. The percolation model (Figure 10) shows, as one would expect, that the head losses were concentrated in the grout curtain. Therefore, in normal conditions of the foundations, the pressures in the upstream and downstream piezometers should be low and almost equal. Figure 11 presents the measured pressures in two of the instrumented cross-sections. As can be seen, the piezometers have very low pressure and with little changes due to the variation of the reservoir level. The values obtained are very close to the forecast made in the Monitoring Plan (LNEC, 2007a)[1] where it is estimated that for the cross-sections PB and PC, corresponding to the highest cross-sections, one would have, for the piezometers installed in the alignment of the crest, roughly half of the dam base length, a maximum head of 5 m. The measured value is about 3 m.

Still, according to this model, the head loss should be very low between the upstream and downstream piezometers. The highest value that has been recorded to date was 0.5 m between PH3 and PH4.

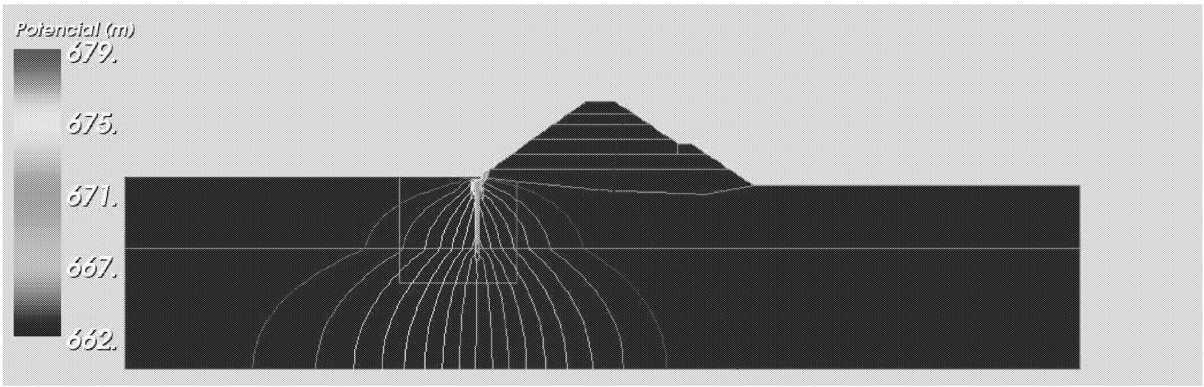


Figure 10: Head contours given by the seepage FEM model

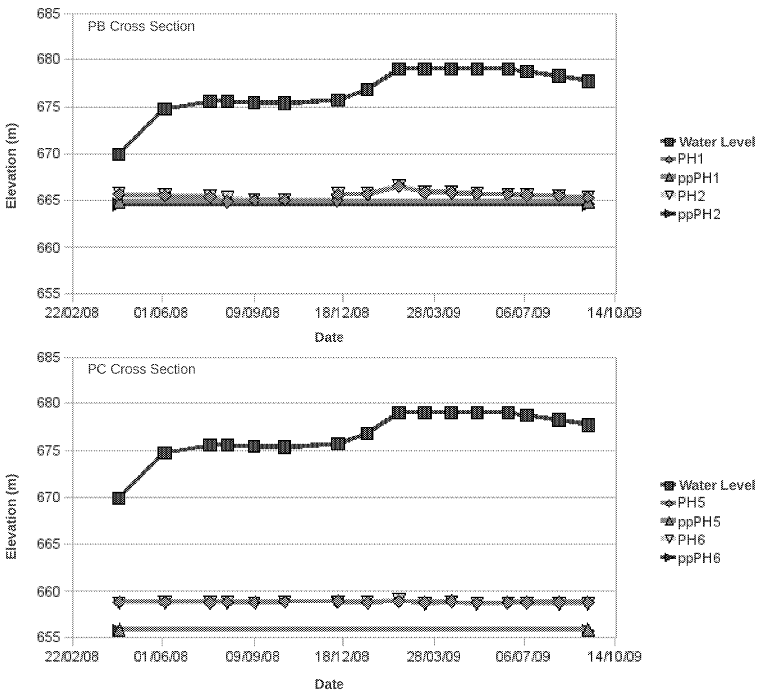


Figure 11: Recorded head values in the taller cross sections

4.4 Flow meters

Pinhão dam has several devices for measuring flow rates. The most relevant are those used to measure the flow from the dam foundations and the flow from the springs, captured during the construction.

Figure 12 shows the weirs and the diagrams with a graph of the measurements taken from the start of first filling. There is a clear correlation between the water level in the reservoir and the flow. This flow is also influenced by rainfall, but unfortunately those readings are not currently available. Using a statistical model, it would be possible to establish the relationship between the reservoir level, the rainfall and the flow. To overcome the lack of rainfall data, one can admit that rainfall follows a sinusoidal law over the year, therefore allowing an approximate relationship between the quantities.

The relationship considered has the following form:

$$f(x_1, x_2, \ldots, x_n) = \beta_1 \psi_1(x_1) + \beta_2 \psi_2(x_2) + \ldots + \beta_n \psi_n(x_n) \quad (1)$$

where $f(x_1, x_2, \ldots, x_n)$ is the wanted function, β_i are the coefficients that minimise the error between the actual values and the model, and $\psi_n(x_n)$ are arbitrary functions of the independent variables x_n .

The model coefficients (β_i) are determined by:

$$\{\beta\} = \left([X]^T [X]\right)^{-1} [X]^T \{y\} \quad (2)$$

where $\{\beta\}$ represents the vector containing the unknowns β_i , $\{y\}$ represents the vector of dependent variables (measured values) and $[x]$ the matrix with the functions $\psi(x_n)$.

Testing different models, a good agreement was obtained for the expression:

$$F(l/s) = 2.149 - 0.00133N + 0.3046 f(M) - 0.3537(WL - 670) + 0.451(WL - 670)^2 \quad (3)$$

where $f(M)$ is a sinusoidal function of the month in question. Because very high rainfall occurred during January and February 2009, some data points were not considered for the determination parameters.
The model obtained is represented in Figure 13.

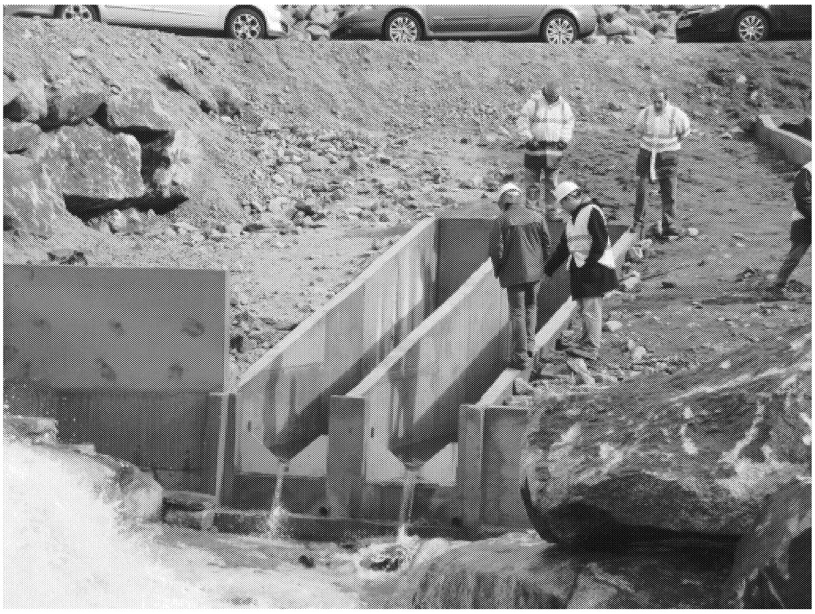


Figure 12: Flow meters near the toe of the dam. The left one measures the seepage and the right one the flow from the natural springs

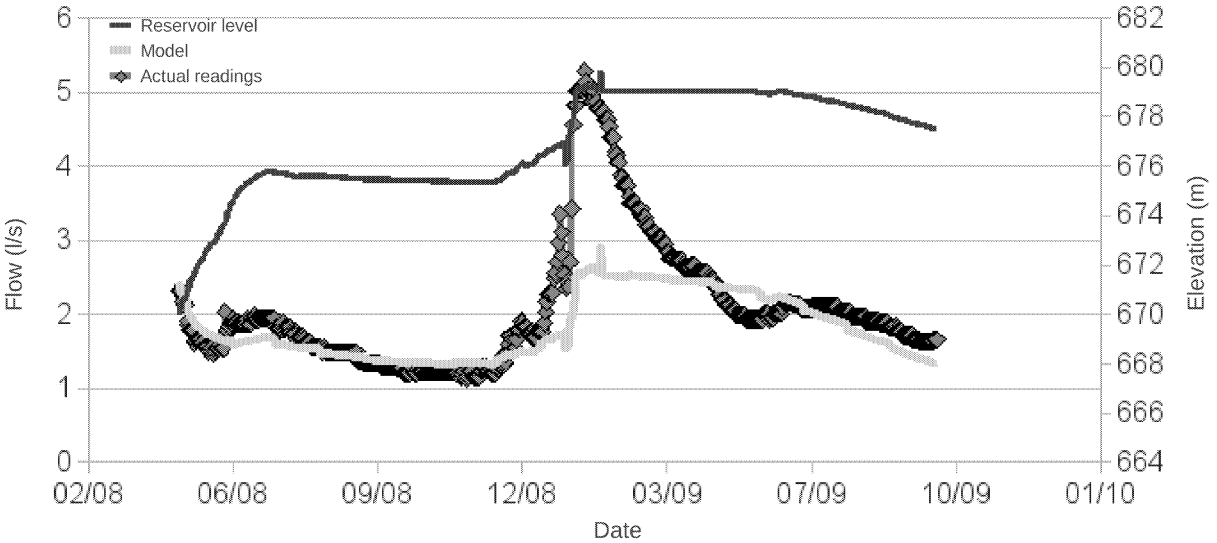


Figure 13: Statistical model for the flow

The analysis of Figure 13 suggests the following conclusions:

- the measured flow rate at the lowest level of the reservoir (the beginning of filling) does not match the historical minimum, which occurred later when the reservoir level was at the first filling stair;
- the historical minimum was reached on 9 November 2008, corresponding to a value of 1.14 l/s, which is, probably, the value corresponding to natural drainage without rainfall;
- the maximum measured value of 5.28 l/s was obtained on 30 January 2009, which coincided with a period of heavy rain (although there are no measurements of rainfall, there is a qualitative record of the weather conditions), and consequently was probably influenced by the rainfall on the exposed surface of the dam;
- analysis of the data suggests that the flow corresponding to the MWL, without influence of rainfall, should be around 2.5 l/s; this value can also be obtained from the statistical flow model.

Finally, one can state that the statistical model gives an acceptable compromise considering the available data. It appears that there is no trend towards increased flow over time. This can be deduced, given that in the model the coefficient that affects the number of elapsed days since the beginning of the filling is virtually zero (actually it is even negative).

In the Monitoring Plan (LNEC, 2007a) an estimation of the seepage flow through the dam foundation is presented. Two different scenarios were considered, corresponding to different qualities of the upper foundation rock. In the case of a more permeable foundation, the estimated flow rate was 98 m^3/day. The measured value (2.5 l/s) minus natural drainage (1.14 l/s) corresponds to about 118 m^3/day, a value that can be considered to be in good agreement, taking into account the uncertainty in quantifying the permeability of the foundation.

5 FINAL REMARKS

This paper presents a brief description of the Pinhão dam and of the monitoring activities undertaken during the first filling stage of the reservoir.

This important phase of a dam's life was conducted in strict compliance with the first filling plan and, at every moment, safety was guaranteed.

The dam presents a behaviour that can be classified as good, within the forecasts made prior to construction, both for the hydraulic behaviour of its foundations and for the stress-strain behaviour.

Continuation of the monitoring activities will soon allow the preparation of models for the validation of the monitoring results and its integration into safety control systems.

As in other endeavours, one of the key factors for success is the commitment and attention provided by the various entities engaged in the project, and of these the human resources are crucial as it is with their input and involvement in solving problems, that issues are being surmounted and solutions arise.

Since its conception to completion and initial operation, the Pinhão dam had the dedicated support of all its stakeholders, from the designer (PROSISTEMAS SA), the licensing authorities (CCDR-N and INAG), the contractor consortium (SOMAGUE SA / Chupas e Morrão SA), supervision team (PROSPECTIVA) the owner (ATMAD - Águas de Tras-os-Montes and Alto Douro group ADP) and LNEC.

In fact, since the start of this project, ATMAD has established a true partnership with LNEC that has contributed greatly to the success of the project throughout its different phases (design, construction, and now the first filling operations).

This partnership has allowed the owner to create their own essential skills for the operation and maintenance of such infrastructure, and LNEC to be able to have important case studies.

6 References

[1][1]LNEC (2007a) Plano de observação da barragem do Pinhão. *Rel 137 DG/NBOA*

[2][2]LNEC (2007b) Plano de primeiro enchimento da barragem do Pinhão. *Rel 201 DG/NBOA*

[3][3]NOIB (1993) Normas de observação e inspecção de barragens. *Anexo à Portaria nº 847/93*