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PRELIMINARY IDENTIFICATION OF THE STRUCTURAL EFFECTS OF CONCRETE SWELLING IN CHICAMBA DAM (MOZAMBIQUE)



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ABSTRACT

This paper presents the relevant results of the monitoring data analysis of the Chicamba dam (Mozambique), which confirmed the existence of an ongoing concrete swelling process in the dam's body. The performed analysis was based in the qualitative evaluation of the monitoring results complemented with the quantitative interpretation of some physical quantities.

The Chicamba dam is a concrete dam, 75 m high, composed by two independent arch structures linked by an artificial abutment, built in two stages in the decades of 50 and 60 of the last century. The monitoring system had the last large rehabilitation and strengthening in 2010 and 2011. The safety control of the dam is based on the data provided by the monitoring system and on regular inspections.

About ten years ago, the first signs of alkali aggregate reactions were identified by tests performed in concrete samples extracted from the dam's body for the installation of rod extensometers in the foundation.

The available monitoring results, namely the displacements measured through plumb lines and by geodetic methods, were properly treated to estimate the swelling structural effects in the last 10 years. These effects are moderate and the apparent cracking is limited to specific areas of the structure, but the accumulated swelling effects over time are not known. To evaluate the development of this pathology, an experimental study using cores to be extracted from the dam's body is planned to be carried out.

Keywords: Chicamba dam, Monitoring, Structural behaviour, Swelling effects.

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1. INTRODUCTION

Chicamba dam, located in the central region of Mozambique, is integrated in a hydroelectric scheme [1,2], operated by Electricidade de Moçambique (EDM). A dedicated team of EDM is responsible for the dam's maintenance and observation.

In 2005 LNEC began to provide regular support to EDM in the activities of observation and safety control of Chicamba, Mavuzi, Lichinga and Cuamba dams. This support started with detailed inspections and diagnosis of the dams. In the case of Chicamba dam, two detailed inspections were performed in 2005 and 2008 and a safety assessment study of the dam was carried out in 2006 [3,4,5,6]. This study, that was supported by a finite element model of the dam and its rock mass foundation, included a static analysis and interpretation of the dam's structural behaviour over time and an analysis of the dam's seismic behaviour. The seismic analysis was done following the concerns raised by the Espungabera earthquake, which occurred on February 23, 2006, with epicenter located 150 km south from the Chicamba dam site and a magnitude of 7.0 (in the worst affected region twenty-seven people were injured, four people died and 160 buildings were damaged).

Over the years EDM has been improving the dam's observation procedures, despite the company's financial constraints. In 2001 and in 2010-2011 significant improvements and rehabilitations of the monitoring system were made [7]. Some of these works were done with the support of LNEC and Hidroeléctrica de Cahora Bassa (HCB), which also ensures the geodetic surveys. Nowadays, the dam's monitoring system, which has been properly operated by EDM, allows to monitor the main actions and the corresponding structural effects. The safety control of the dam is based on this data and on regular inspections.

This paper presents a numerical analysis of the dam's displacements, measured by plumb lines and geodetic methods, which was used to interpret the dam behaviour. The performed analysis allowed to estimate the effects of the concrete swelling on the dam behaviour, over the last 10 years, which are moderate and do not affect the operation of the spillway gates. The achieved results advise to carry out experimental studies to characterize the dam's concrete swelling pathology.

2. CHICAMBA DAM CHARACTERISTICS

2.1. Brief description of the dam

The Chicamba dam is composed by two independent arch structures (Fig. 1 and Fig. 2), linked by a middle artificial concrete abutment. The main arch, located in the valley of the old riverbed, is 75 m high, is formed by parabolic arches and its crest is at the elevation 625.00 m. The secondary arch, which closes a natural depression on the right bank, is 45 m high, is defined by circular arches and its crest is at the elevation 625.50 m (Table 1 and Fig. 2). The artificial concrete abutment was placed near the intermediate downstream quarzitic outcrop, having a maximum height of 25 m on the upstream side. The main and secondary arches were built with 16 and 11 blocks, respectively, and the intermediate abutment was divided into 3 blocks, which makes a total of 30 blocks for the entire dam [1,2].

The normal water level (NWL) and maximum flood level (MFL) are at the elevations 624.00 m and 625.00 m, respectively. The reservoir, for the NWL, has a volume of about 2000 hm^3.

Fig. 1 – Chicamba dam. General view from the outcrop top at the left bank

Table 1 – Characteristic dimensions of Chicamba dam

	Main arch	Secondary arch
Maximum height above foundations	75 m	45 m
Crest length	225 m	115 m
Thickness at the central cantilever bottom	11 m	5 m
Thickness at the central cantilever top	3 m	1.5 m

The dam has been built in two stages. In the first construction stage (June 1956 to December 1959), the main arch reached a maximum height of about 60 m. The crest reached the elevation of 610.25 m in the secondary arch and the elevations of 609.50 m and 602.50 m in the main arch, in the lateral zones and in the central section where the spillway was inserted, respectively. The construction of the powerhouse, downstream of the dam, began in 1965 and ended in 1968, having started operating in that year. The raising of the dam, up to the actual elevation, started in 1968 and ended in late 1969. The spillway gates were installed in 1970.

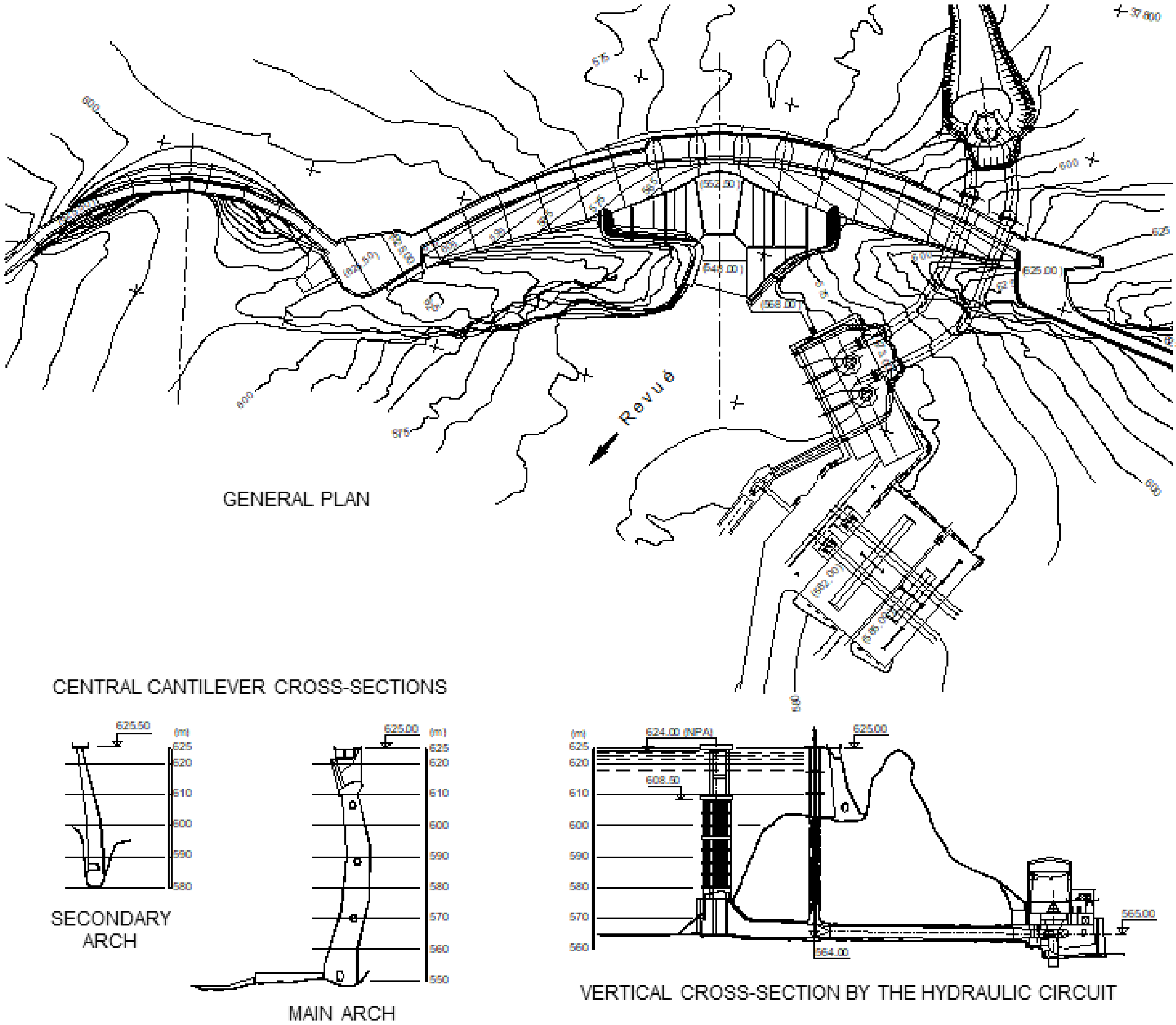


Fig. 2 – Chicamba dam. Plan and vertical cross-sections

The geological formations of the dam's foundation are of gneisses and quartzites types. At the site, these two formations constitute a complex structure in which the quartzites break through the gneissic rock mass, creating interfaces and strangulation strips, outcropping through spurs. In the strangulation zones the rock mass is highly weathered.

2.2. Dam's concrete

The composition of the concrete used in the first construction stage of the dam included [5]: i) Portland cement; ii) crushed aggregates from a quarry located about 2.5 km from the dam site, in an area of granite outcrops; and iii) an admixture with air introducing properties, commercially known as Darex AEA, used to achieve a good concrete workability and increased strength.

Table 2 shows the compounds of the cement used in the first construction stage of the dam. The cement alkali content (sodium and potassium oxides) is not known.

Table 2 – Chemical characteristics of the cement used in the first construction stage of Chicamba dam

Compounds	Percentage
Calcium oxide (Ca O)	63.7%
Silicon dioxide (silica) ($Si O_2$)	20.8%
Aluminum oxide (alumina) ($Al_2 O_3$)	6.7%
Iron oxide ($Fe_2 O_3$)	2.9%
Magnesium oxide (Mg O)	1.9%
Sulfur trioxide ($S O_3$)	1.7%
Other compounds	2.3%
TOTAL	100%

Table 3 presents the composition of the dominant concrete used in the first construction stage of the dam and Table 4 shows the compressive strength of this concrete, wet screened by a 38 mm sieve, referred to 20 cm edge cubes.

Table 3 – Concrete composition used in the first stage of construction of Chicamba dam

Components	Content
Portland cement	200 kg/m^3
Coarse aggregates (150 mm – 5 mm)	1750 kg/m^3
Fine aggregates (< 5 mm)	450 kg/m^3
Admixture DAREX AEA	0,4 cm^3 / kg of cement
Water/cement rate (w/c)	0.52

Table 4 – Compressive strength of concrete used in the first stage of construction of Chicamba dam

Age	Compressive strength of wet screened concrete (sieve of 38 mm), referred to 20 cm edge cubes
7 days	14.5 MPa
28 days	21.4 MPa
90 days	24.6 MPa

Relatively to the second stage of construction, it was not possible to obtain elements about the concrete applied on the dam raising.

2.3. Monitoring system

The monitoring system, rehabilitated and improved in 2001, 2010-2011 and 2015, allows to measure the physical quantities related with the main loads and the thermal, structural and hydraulic responses (Table 5).

Table 5 – Monitoring, inspections and tests: physical quantities, devices and frequencies

Physical Quantities	Methods/devices	Situation until 2010		Installation in 2010-2011	Installation in 2015	Frequencies
		Method	Condition			
Hydrostatic pressure	Water level in the reservoir	Staff gauges and water lever recorder	Good	-	-	Daily
Uplifts	Piezometers	6 piezometers of single chamber	Good (rehabilitated in 2007), except one (PZ7A)	5 news piezometers and 1 piezometer recovered (PZ7A)	-	Biweekly
Air temperature	Thermometer	Thermometer (maximum and minimum daily)	Good	-	-	Daily
Absolut displacements of the dam's body	Geodesy (planimetry)	Alignments	Disabled	Triangulation network downstream	-	Annual
	Geodesy (altimetry)	-	-	2 levelling lines at the crest	-	Annual
	Plumb lines	3 direct plumb lines	Satisfactory (rehabilitated in 2001)	Inverted plumb line in the foundation, linked with the central direct plumb line	-	Biweekly
Rotations of the dam's body	Clinometers	22 bases for clinometers	Disabled	-	-	-
Absolut displacements of the foundation	Rod extensometers	-	-	4 rod extensometers in the foundation	-	Biweekly
Joint movements	Embbeded devices	56 devices of the Carlson type	Good (except 34 damaged or with dubious operation)	-	-	Monthly
	Deformeters (2D and 3D)	18 2D-deformeters	Good (installation in 2001)	45 3D-deformeters (3 old 2D-deformeters were disabled)	-	Monthly
Crack movements	Deformeters 2D	-	-	-	2 deformeters	Monthly
Concrete temperatures	Embbeded and surfade thermometers	51 thermometers of the Carlson type	Good (except 34 damaged or with dubious operation)	-	-	Monthly
Discharged and infiltrated flows	Partial	98 drains of the foundation	Good (general rehabilitation in 2007)	-	-	Monthly
	Total	9 seepage measuring weirs	Good (installation in 2001)	-	-	Biweekly
Visual inspections	Routine	Monitoring technicians	Irregular	-	-	Biweekly
	Detailed	-	-	-	-	Biennial
	Exceptional	-	-	-	-	After exceptional events

						(floods, earthquakes, etc.)
Analysis of water from the reservoir and from drains	Physical and chemical analysis	-	-	-	-	Annual

2.4. Reservoir water level variation in the last 15 years

Figure 3 shows the evolution of the reservoir water level from the beginning of 2005 to August 2019. Between 2005 and 2008 the average water level in the reservoir was close to 618 m, showing the typical seasonal variations (rise in the rainy season, in the first semester, and fall in the dry period, in the second semester). After 2008, the average annual water level decreased gradually until 2012. In that year, in which the usual level rising during the rainy season did not occur, the reservoir reached the minimum level of the last 15 years (600.70 m on 12/06/2012). In 2013 and 2014 the level rose again, standing at around 610 m at the end of 2014. In 2015 the level decreased, having remained at elevations close to 607 m during the final part of 2015 and during 2016. In the beginning of 2017, the water level rose about 11 m, reaching the elevation 618.43 m in 19/04/2017. From there on the average annual level increased gradually. In early May 2019 the level reached 621.14 m, close to the maximum of the last 15 years, which occurred in early 2008 (622.35 m). From that date, the level gradually decreased, having been, at the end of August 2019, at 620.51 m.

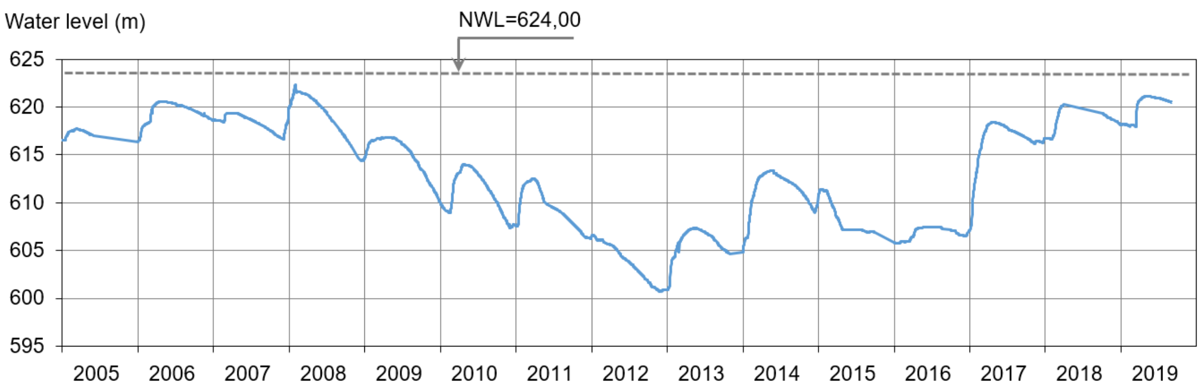


Fig. 3 – Evolution of the reservoir water level from the beginning of 2005 to August 2019

3. FIRST SIGNS OF SWELLING REACTIONS

3.1. Testing performed in 2011 and 2012

The results obtained in tests carried out at LNEC between June 2011 and December 2012, on concrete samples extracted from the body of the main arch when drilling for the installation of rod extensometers in the foundation, indicate the development of swelling reactions in the dam's concrete [8,9]. The obtained values for the residual expansions, $667\text{x}10^{-6}$ and $906\text{x}10^{-6}$, in the two samples that had sufficient dimensions to carry out this type of tests (Fig. 4), can be considered high in terms of the remaining expansive potential [9].

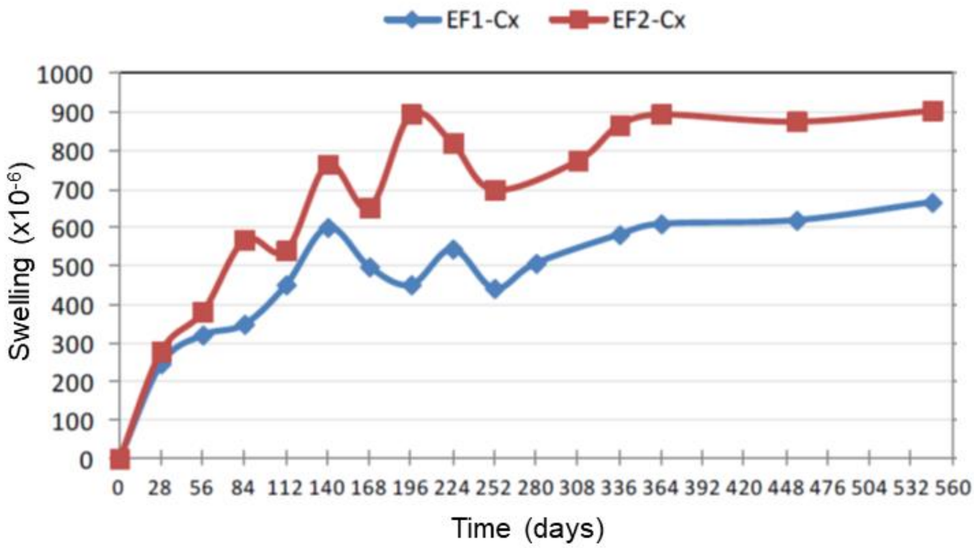


Fig. 4 – Results of the accelerated expansion tests, carried out at LNEC in 2011-2012, on concrete samples extracted when drilling the rod extensometers EF1 and EF2

3.2. Relevant results of visual inspections

In the last visual inspections cracks were detected on the downstream surface of the two arches, close to the dam-foundation interface and parallel to this surface, being more expressive in the secondary arch, in the lower section on the right bank side [10,11]. The main cracks are described in Table 6.

Table 6 – Cracking identified in inspections carried out by LNEC, on the downstream surfaces of the arches next to the dam-foundation interface

Main arch (downstream surface)	Secondary arch (downstream surface)
Cracks parallel to the dam-foundation interface, on the right bank side, on block G-H, near the G joint Cracks parallel to the dam-foundation interface, on the left bank side, near the P joint Horizontal crack near the dam-foundation interface, on the left bank side, on block Q-R (Fig. 5)	Horizontal cracks with a maximum opening of about 5 mm, between joints IV and VII Cracks parallel to the dam-foundation interface, on the right bank side, between joint II and the middle of block IV-V, with an opening between about 1.5 mm and 2.0 mm (Fig. 6)

Fig. 5 – View of the horizontal crack near the dam-foundation interface, on the downstream surface of the main arch, on the left bank side, on block Q-R

Fig. 6 – Views of cracks parallel to the dam-foundation interface, on the downstream surface of the secondary arch, on the right bank side, between joint II and the middle of block IV-V

These cracks might be related to the existence of concrete expansions. In order to monitor its opening-closing movements, two deformeters were installed on cracks of the secondary arch in 2015. The observation period for these devices, which is still very short, coincided with the high rise of the water level in the reservoir, that occurred in the last years. The observed movements were of closure, so it is not possible, yet, to associate the opening-closing movements of the cracks with the development of expansions in the dam’s concrete.

4. INTERPRETATION OF THE MONITORING DISPLACEMENSTS

4.1. Methodology of displacement analysis and interpretation

The displacements are physical quantities that integrate the deformations of the structure, so they reflect its global behaviour. Thus, the vertical and horizontal displacements, measured by precision levelling and plumb lines, respectively, were considered in the analysis of the dam's behaviour.

The vertical displacements of the crest are only slightly influenced by the hydrostatic load, depending essentially on the seasonal thermal variations and on the irreversible time effects. In Mozambique, the amplitude of the annual thermal wave is rather small, therefore, the time effects dominate the vertical displacements, and for this reason, can be obtained, with enough approximation, directly from the monitoring results.

On the other hand, the interpretation of the horizontal displacements should be supported by techniques allowing to perform effects separation, since the effects of hydrostatic pressure, seasonal thermal variations and time are all significant. The effect splitting can be done using quantitative interpretation techniques, which are based on semi-empirical relations between the observed structural effects and the main loads [12]. Usually, the following hypotheses are assumed: i) the dam behaviour for the hydrostatic load and for the seasonal temperature variations is reversible; ii) all non-reversible effects are a function of the time variable; and iii) the observed values are the sum of the reversible with the non-reversible parts.

For an observation survey j, the following generic expression is considered,

$$E_j(h,t',t) = E_h(h_j) + E_T(t_j') + E_t(t_j) + K + r_j \tag{1}$$

in which E_j stands for the observed response in the mentioned j survey, h is the difference between the upstream and downstream water levels, t' is the time (in days) since the beginning of the current year, t is the time, also in days, since the beginning of the process under analysis, E_T represents the thermal effect, E_t is the time effect, K is a constant and r_j is a residue (difference between the monitored value and the computed value).
In order to represent the effect of the hydrostatic pressure, a polynomial function is usually considered; the effects of the thermal seasonal variations are normally simulated by an annual sinusoidal function; and the effects of time can be represented by a sum of functions, according to the effects aimed to me considered (creep, swelling and other effects).

$$E_h(h) = \sum_{i=1}^{N} a_i h^i \tag{2}$$

$$E_T(t') = b_1 \cos\frac{2\pi t'}{365} + b_2 \sin\frac{2\pi t'}{365} = b_1 \cos\theta + b_2 \sin\theta \tag{3}$$

$$E_t(t) = \sum_{i=1}^{M} c_i (t-t_p)^i + d_1.\log\left((t-t_f) + \frac{1}{(t_p-t_f)}\right) + d_2\left(1 - e^{-\frac{t^n}{\beta}}\right) \tag{4}$$

In the previous equations *N* and *M* are the degree of the polynomials. If the analysis considers a long period of the dam lifetime, the creep and the swelling effects are usually represented by logarithmic and exponential functions, respectively, where t_p, t_f, n and β are parameters, previously chosen to calibrate the time evolution of these effects. If the analysis is done for a short period of time and the creep effects are already negligible, generally only a polynomial function is needed.

By assuming that the variable E has a normal distribution, characterized by a dispersion σ^2, being σ the standard deviation, then, the coefficients a_i, b_1, b_2, c_i, d_1 and d_2 can be computed by linear regression through the least square method, which by minimizing the residues, r_j, ensures a unique solution for the problem. The residues vector is associated with the uncertainty of the physical quantities, with the lack of fitting of the functional relation and with the redundancy due to the fact that the monitoring periods largely exceed the number of parameters to be determined.

If the variations of the reservoir water level are small and if no other important time effects are present, this formulation generally allows to separate, in an acceptable way, the elastic and viscous effects of the hydrostatic pressure. Otherwise, the dependency between the elastic and the creep effects should be considered [12,13].

4.2. Vertical displacements of the crest measured by precision levelling

Fig. 7 shows the dam's plan with the location of the measuring points of the two precision levelling lines installed on the crest of the dam (elevation 225.00 m) and on the open gallery under the crest of the main arch (elevation of 222.50 m).

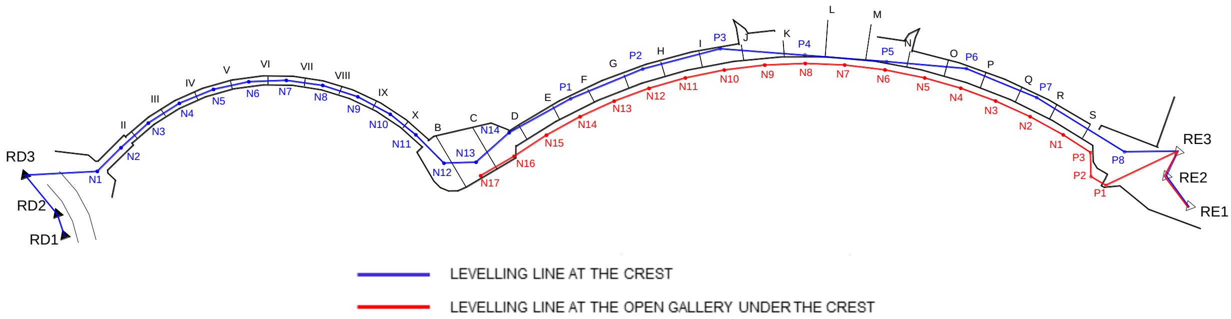


Fig. 7 – Plan of the two precision levelling lines installed on the open gallery under the crest of the main arch (elevation of 222.50 m) and on the crest of the secondary arch (elevation 225.00 m)

Fig. 8 and Fig. 9 present the vertical displacement along the levelling lines obtained on 11 surveys performed between 2010 and 2018. The obtained movements are consistent with seasonal thermal variations and with the existence of moderate expansions in the concrete.

The evolution of the vertical displacements is shown in Fig. 10 and Fig. 11. For better appreciate the progression of the vertical displacements, the results obtained in the warm and in the cold seasons were plotted in separated charts. Permanent upward vertical displacements of about 5,5 mm and 6 mm were observed on the central measuring points of the main and secondary arches, respectively, between December 2010 and November 2017, which correspond to average annual expansion rates of about $11\text{x}10^{-6}$ and $19\text{x}10^{-6}$, respectively (Table 7). It seems that the expansions effects are more significant in the secondary arch, which is consistent with the more expressive cracking existing near the interface of this structure with the foundation, downstream.

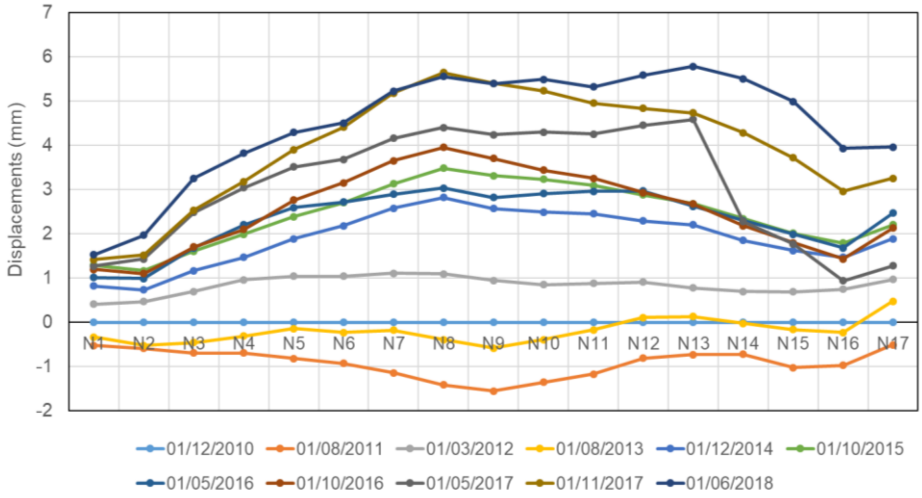


Fig. 8 – Vertical displacements measured by precision levelling on the open gallery under the crest of the main arch, at the elevation of 222.50 m, between 2010 and 2018

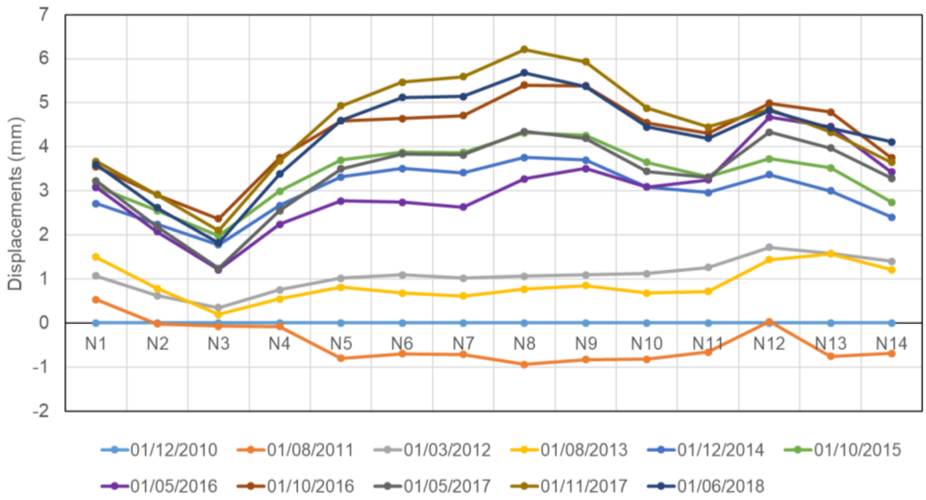


Fig. 9 – Vertical displacements measured by precision levelling on the crest of the secondary arch, at the elevation of 225.00 m, between 2010 and 2018

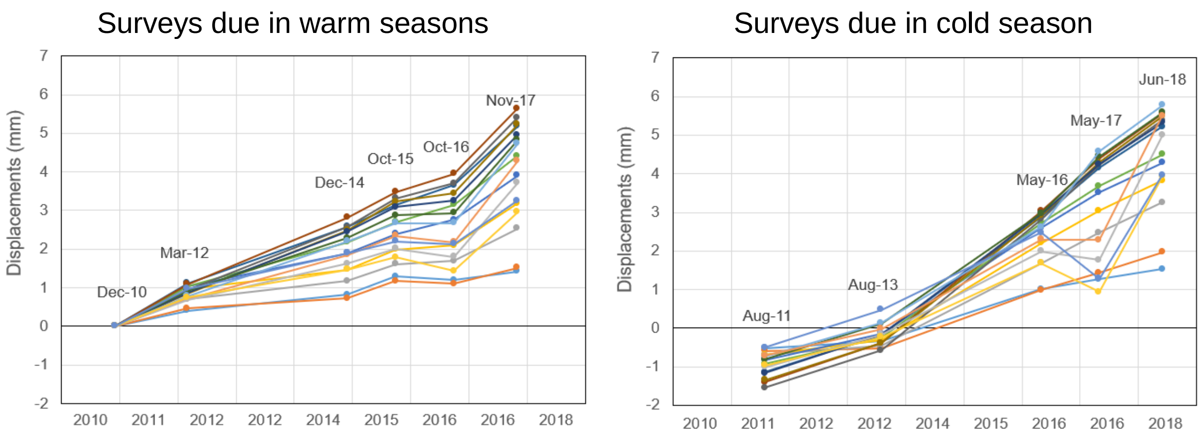


Fig. 10 – Evolution of vertical displacements measured by precision levelling on the open gallery under the crest of the main arch, at the elevation of 222.50 m, between 2010 and 2018

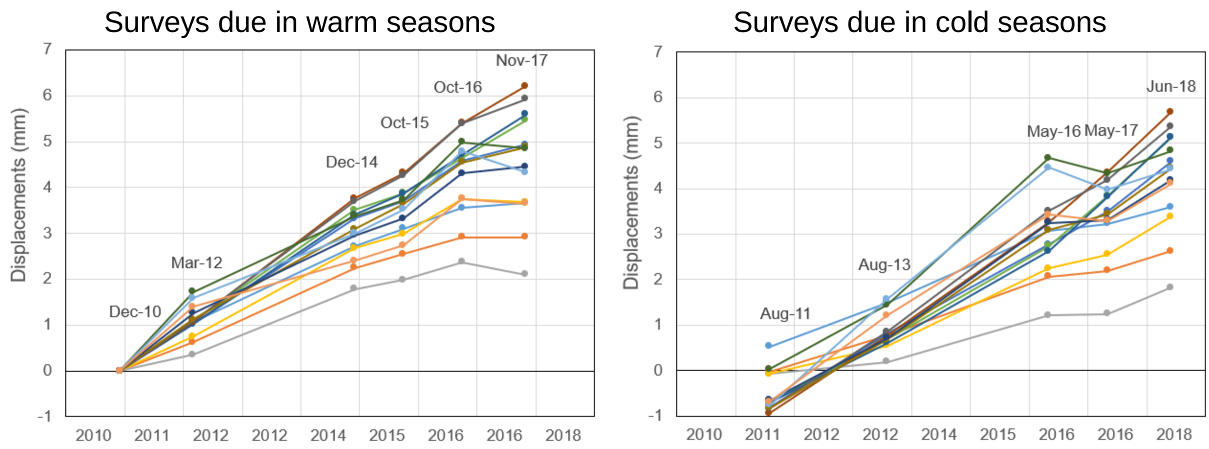


Fig. 11 – Evolution of vertical displacements measured by precision levelling on the crest of the secondary arch, at the elevation of 225.00 m, between 2010 and 2018

Table 7 – Maximum swelling measured by precision levelling of the crest, between December 2010 and November 2017, and average annual expansion estimation

	Main arch (height of 75 m)	Secondary arch (height of 45 m)
Maximum swelling (mm)	5,5	6,0
Average annual expansion rate*	11 x 10^{-6}	19 x 10^{-6}

*The values were estimated dividing the maximum upward vertical displacement by the dam maximum height and by the number of observation years

4.3. Horizontal displacements measured by plumb lines

Fig. 12 shows a dam's elevation view with the location of the devices to measure displacements.

First, it should be referred that the horizontal displacements observed by geodetic triangulation are consistent with those observed in plumb lines. Given the limited number of available geodetic surveys (9), quantitative interpretations of these displacements were not performed.

To carry out the quantitative interpretation of the radial displacements the following functions were adopted: i) a polynomial with a fourth degree term, to represent the line of influence of the reservoir water level effects; ii) the sum of sine and cosine functions, to represent the effect of seasonal thermal variations; and iii) a linear function to approximate the effects of the expansions.

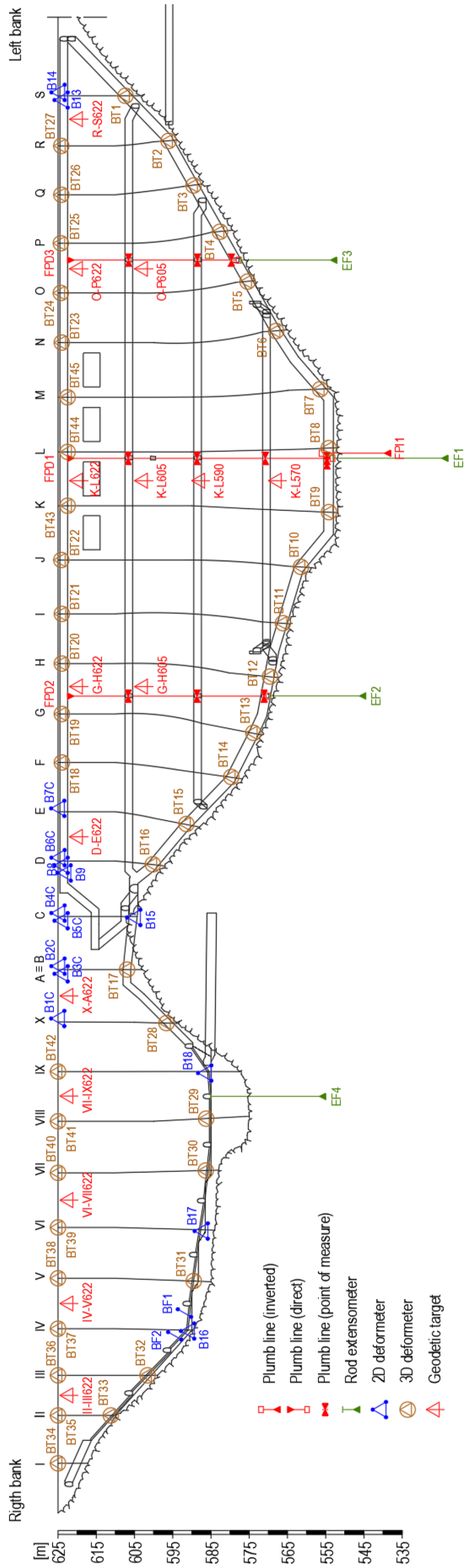


Fig. 12 – Dam's elevation with the location of devices to measure displacements

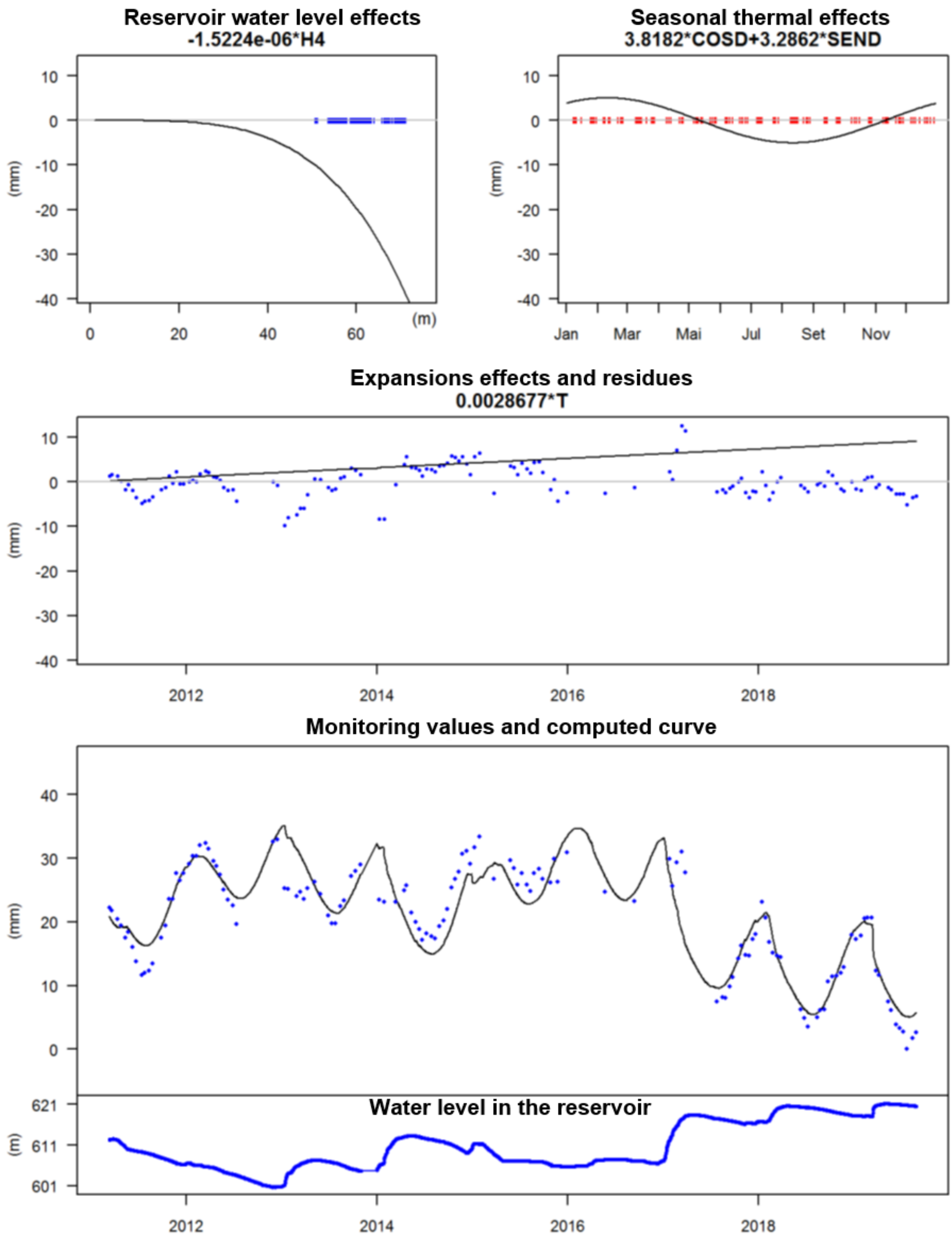
Table 8 summarizes the main results obtained by quantitative interpretation of the radial displacements measured on the three plumb lines of the main arch (at elevations 622.50 m and 606.54 m). It was found that the upstream displacements associated with the expansions are already significant, particularly in the plumb line FP3, in which the irreversible upstream displacements due to the expansions (20.1 mm, at the elevation 622.50 m, over a period of about 15 years) have a magnitude similar to the downstream displacements due to the hydrostatic pressure (21.6 mm at the elevation 622.50 m).

Table 8 – Parcels obtained by quantitative interpretation of the radial displacements measured in the main arch plumb lines

Plumb line points of measure	**Reservoir water level (h=622.35 m)** (1)	**Annual thermal variations**	**Swelling effects**	
	Displacement downstream (mm)	Amplitude (mm)	Displacement upstream (mm)	Annual rate (mm/year)
FP1 (622.50 m)	41.7	10.0	9,1 (2)	1.0
FP1 (606.54 m)	34.6	6.1	5.4 (2)	0.6
FP2 (622.50 m)	24.5	4.1	15.8 (3)	1.2
FP2 (606.54 m)	15.2	2.4	11.6 (3)	0.8
FP3 (622.50 m)	21.6	8.3	20.1 (3)	1.5
FP3 (606.54 m)	13.3	4.3	13.5 (3)	1.0

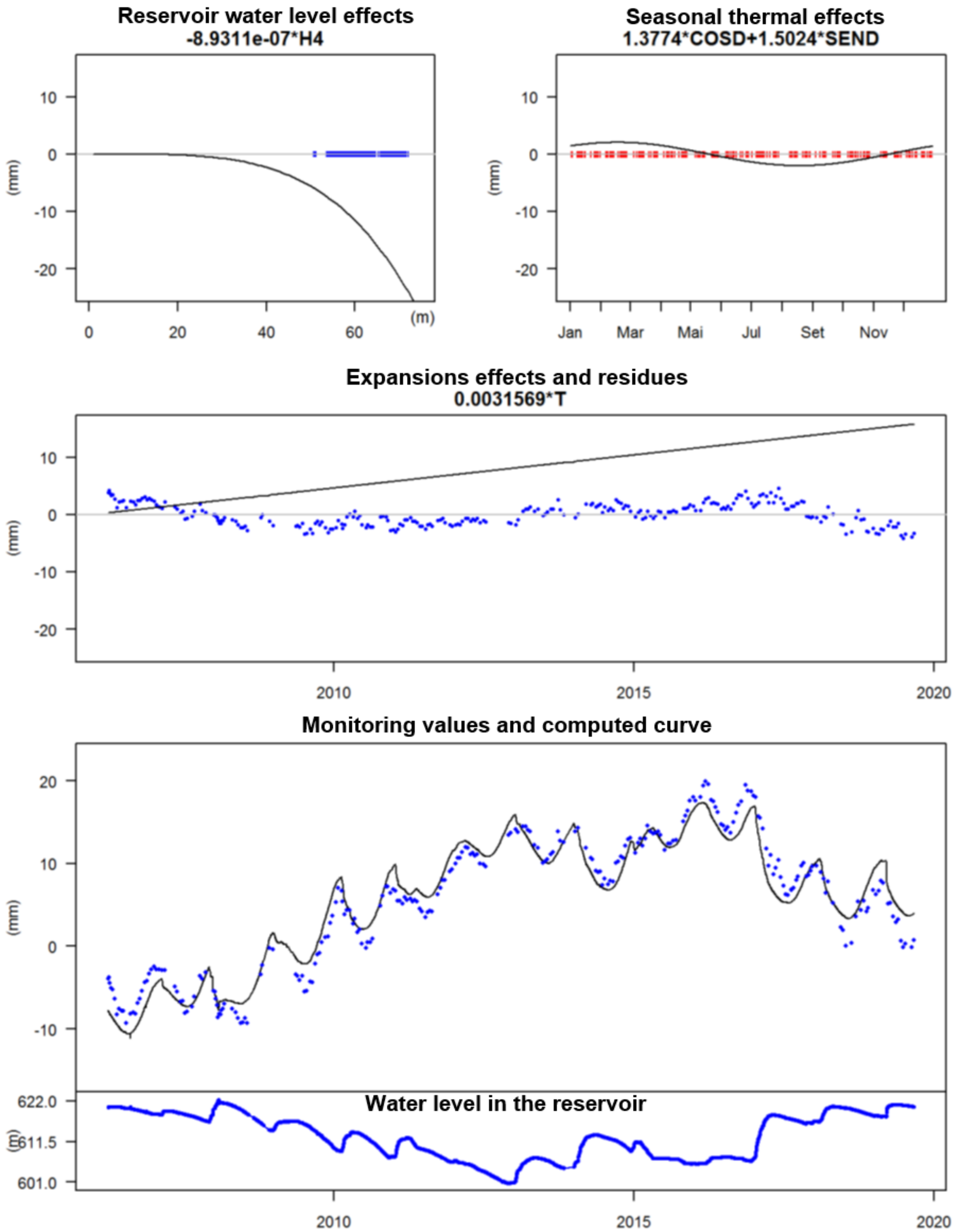
(1) Maximum reservoir water level recorded between January 2006 and September 2019
(2) Between March 2011 and September 2019
(3) Between January 2006 and September 2019

Fig. 13 to Fig. 15 show the graphical results obtained by quantitative interpretation of the radial displacements measured on the upper points of the three plumb lines (elevation 622.50 m). A good agreement between the observed results and the ones obtained by quantitative interpretation techniques is observed, which correspond to determination coefficients (R^2) close to the unity in the lateral plumb lines FP2 and FP3 (0.94 and 0.97, respectively) and of 0.82 in the central plumb line.



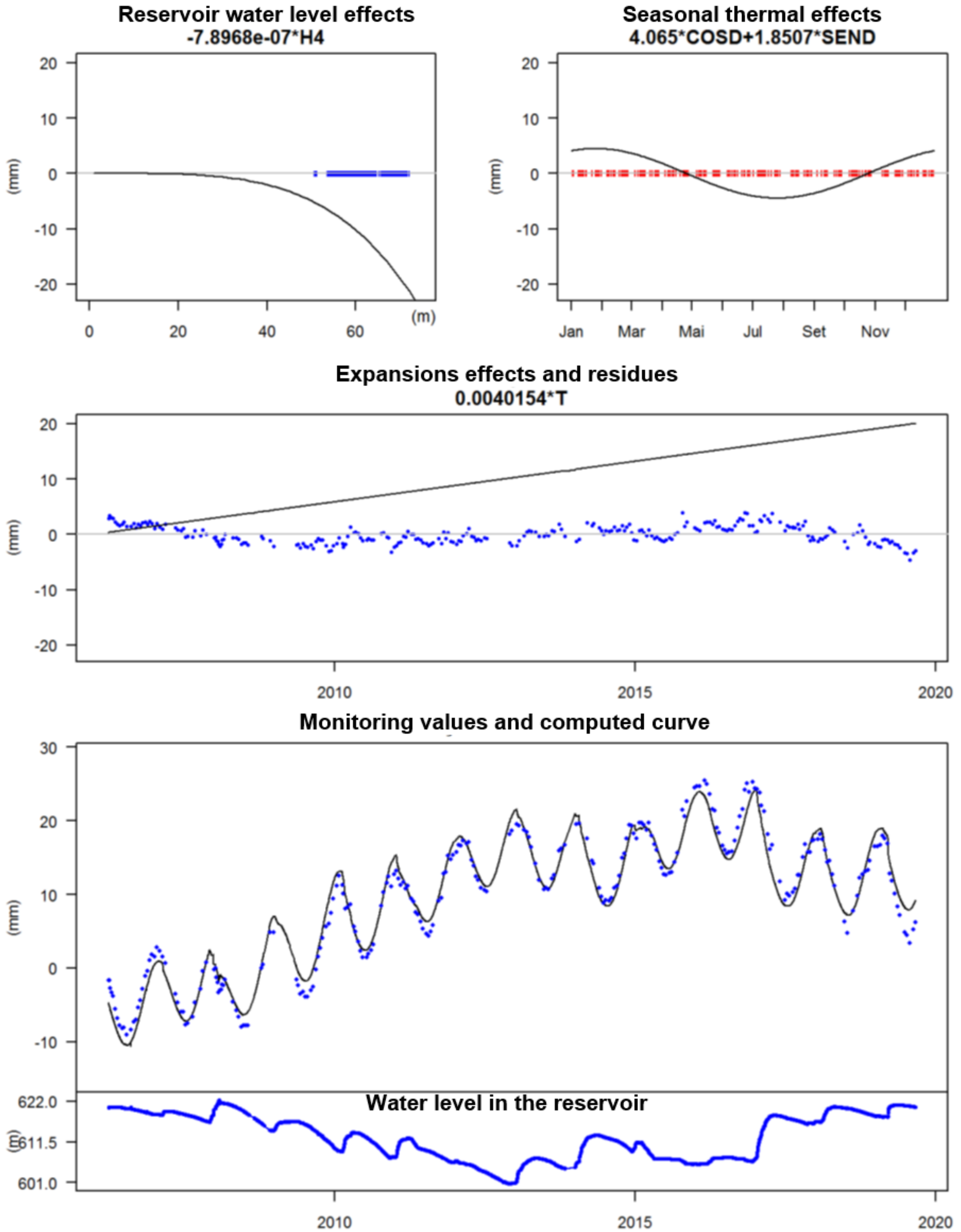
K	R^2	σ (mm)
39	0.82	4.48

Fig. 13 – Quantitative interpretation of the radial displacements observed in the main arch at the measuring point at elevation 622.50 m of the plumb line FP1, between 2011 and 2019



K	R^2	σ (mm)
12	0.94	1.92

Fig. 14 – Quantitative interpretation of the radial displacements observed in the main arch at the measuring point at elevation 622.50 m of the plumb line FP2, between 2011 and 2019



K	*R*²	σ (mm)
12	0.97	1.59

Fig. 15 – Quantitative interpretation of the radial displacements observed in the main arch at the measuring point at elevation 622.50 m of the plumb line FP3, between 2011 and 2019

4.4. Notes about other monitored displacements

The relative displacements observed in the contraction joints are, in general, very low, which confirms the structural continuity of the dam. The general closing state of the contraction joints is also consistent with the existence of expansions in the dam’s concrete. In some contraction joints moderate opening movements were observed, near the crest and the spillway openings, which decrease with the water level rising in the reservoir.

The vertical displacements observed in the foundation rod extensometers are also consistent with the existence of moderate expansions in the dam´s concrete. The observed displacements are larger in the secondary arch. In fact, upward displacements of about 2 mm were measured between March 2011 and April 2019 in the rod extensometer installed downstream of this arch, while in the main arch the maximum displacements were less than 0.5 mm.

5. CONCLUSIONS

5.1. Dam´s structural behaviour

The analysis of the dam's behaviour allows to consider that the structure remains complying the necessary operational and safety conditions for its normal operation. As mentioned, a progressive component over time was detected in some monitored quantities, which is not explained by the variation of the water level in the reservoir neither by the seasonal thermal variations, being consistent with the development of moderate expansions in the dam’s concrete.

The apparent effects of the expansions in the dam are still not very significant, being the visible cracking limited to some downstream areas next to the dam-foundation interface of both arches. However, the interpretation of the horizontal and vertical displacements, observed in plumb lines and through precision levelling, respectively, indicate that the expansions have already a significant influence in the dam’s structural behaviour, being expected that this action will become dominant in the future, if the expansion rates remain constant in the coming years.

5.2. Next steps forward the characterization of the concrete’s pathology

To characterize the properties of the swelling processes on the dam’s concrete, an experimental study is planned to be carried out in laboratory, which will require the previous extraction of a set of concrete cores from the dam’s body (Fig. 16).

The laboratory tests will allow to assess the magnitude of the current expansions and the eventual depreciation of the concrete properties, and, additionally, to forecast the remaining expansion potential of the dam’s concrete.

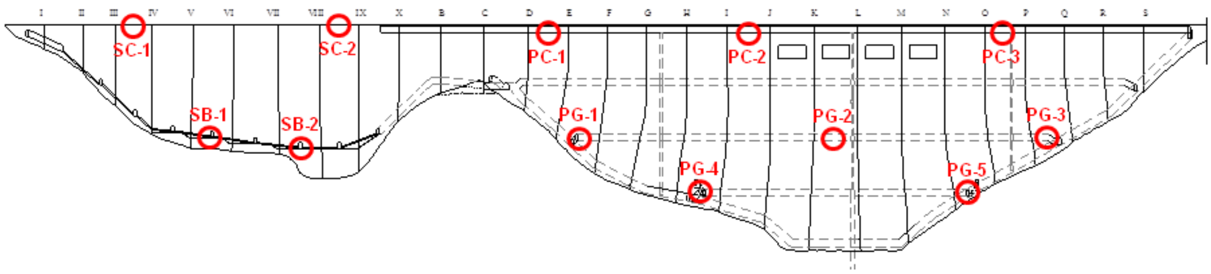


Fig. 16 – Elevation view of the dam with the planned locations of core extraction for the study of the concrete's pathology

5.3. Update of the dam safety assessment study

The laboratory testing results will provide relevant elements to update the study of analysis, interpretation and prediction of the dam's structural behaviour [5]. In fact, by considering the action corresponding to the evolution of the concrete expansions on the available finite element dam-rock foundation model, the stress fields in the dam's body and the dam's safety conditions can be assessed in the current and medium-term situations.

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