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MONITORING PLANIMETRIC DISPLACEMENTS IN CONCRETE DAMS

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Abstract: Structural safety control of concrete dams is based in the analysis of the response of the structure, characterized by the evolution of several variables representative of its behaviour. Among these variables are absolute and relative displacements of points of the structure and of the foundation, uplift and flow rates in the foundation, and deformations and stresses in the concrete.

In particular, the analysis of the displacements is very important, because they reflect the global structural behaviour of the dam. Therefore, monitoring plans of concrete dams usually consider the measurement of displacements of points of the structure and of its foundation.

In the more important concrete dams, displacements measurement involves the simultaneous use of different methods, such as rockmeters, pendulums and applied geodesy. For the measurement of planimetric displacements, two methods have long been applied: one uses pendulums, placed inside the structure; the other makes use of applied geodesy.

These monitoring methods are complementary, and coexist in many Portuguese large concrete dams since the 1940's. Pendulums have the advantage of being more precise and, nowadays, can easily be included in automatic data acquisition systems. Geodetic methods can give information not only on the dam but also on the foundation and surrounding terrain. Because they are more expensive, geodetic campaigns are much less frequent, but are very important for the validation of the pendulums readings.

In this paper a comparison between these two methods is made using the monitoring data of a Portuguese large concrete dam and some considerations about the LNEC experience on their use are presented.

1. INTRODUCTION

Nevertheless large dams are associated with important economic benefits, it is well known that their collapse can originate the liberation of huge quantities of water and be the cause of catastrofic losses along the river valleys, in terms of human lives and economic goods. Due to this fact, in many countries there is public legislation to regulate the safety control of large dams, based on specific technical rules for their design, their construction and their monitoring.

The base of safety control is the comparison between calculated and observed responses of the structures. The calculated responses – displacements, stresses, flow rates, etc. – are, in the more important projects, determined through the use of numerical models. The observed responses are determined through the monitoring of the structure, using specific devices to measure quantities that can characterize the dam's behaviour. Usually the more important quantities shall be monitorized using different and independent methodologies to avoid systematic errors related to a given methodology.

Planimetric displacements of dam's points are one of the more important quantities to characterize the behaviour of the structure. To evaluate the evolution of such quantities along time, two independent methodologies are usually used in large dams: one uses plumb lines, or pendulums; the other is based on the use of geodetical methods. In this work, a comparison between the two methodologies is presented using the monitoring results of a Portuguese large concrete dam. This paper includes the description of the dam, of its monitoring systems and of the two methods used for the measurement of the planimetric displacements. It is also included a comparison between the results obtained by the two methods.

2. ALQUEVA DAM PLANIMETRIC MONITORING SYSTEMS

As a case history, it will be used the monitoring system of planimetric displacements of Alqueva dam. Alqueva dam, located in river Guadiana, is the core of a large multi-purpose hydraulic scheme planned for irrigation, water supply and electric power generation in Alentejo, a region of the South of Portugal. The main structure is an arch concrete dam, with a maximum height of 96 m, a crest length of 348 m and a concrete volume of 687,000 m^3 (Fig. 1). Its hydrologic basin has an area of 48,500 km^2, and its reservoir, with the water at the full storage level (elevation 152 m), presents a water volume of 4.15×10^6 m^3 and covers an area of 250 km^2. The dam was built between 1998 and 2002 and the first filling of the reservoir begun in February 2002.

Figure 1 –Alqueva dam.

The installation of the monitoring equipments and the safety control of the structures of dam and power plant were carried out during the construction according to the monitoring plan (LNEC, 1997). This plan includes also the safety procedures to be followed along the different periods of the dam life, according to the Portuguese regulations (RSB, 2007), and was complemented by different specific plans, included the ones concerning the geodetic monitoring (LNEC, 2000 and 2001).

A total number of about eight hundreds monitoring equipments were installed in the dam. In these equipments are included those used for measuring planimetric displacements: inverted pendulums (a direct measuring device) and pillars included in two traverses (an indirect measuring system, making use of horizontal angles and distances measured by geodetic equipment).

2.1. INVERTED PENDULUMS SYSTEM

Alqueva dam has eight inverted pendulums in the main structure (Fig. 2). Each pendulum consists of a steel wire anchored in the firm rock beneath the structure, at a depth that, in the higher blocks, is of about 65 m. The wire is installed in vertical shafts and, on its upper end, near the higher inspection gallery, is suspended by a float in a water reservoir (Fig. 3). The impulse forces tensions the wire in such a way that it remains vertical. Along the wire, whenever the shaft crosses a gallery, there is a reading station, where the position of the line with respect to the structure is measured by a micrometer microscope along two directions: the radial (orthogonal to the downstream face of the block) and the tangential (parallel to the mentioned face).

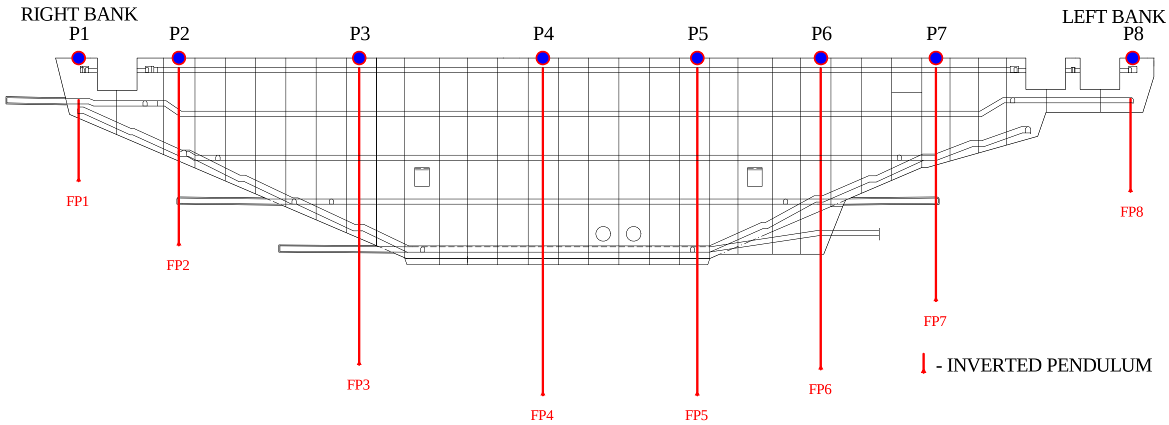
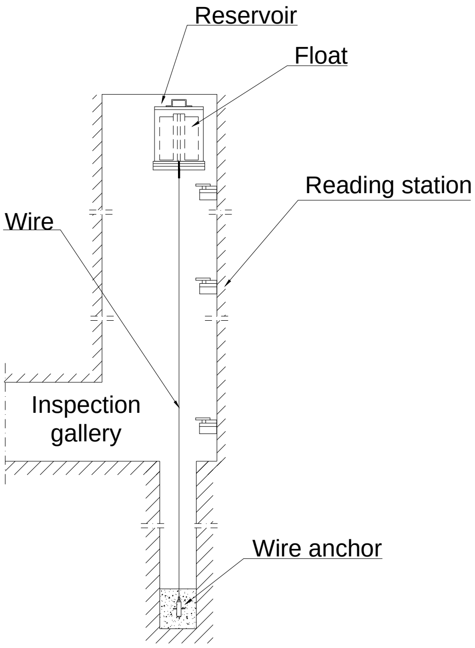


Figure 2 –Inverted pendulums in Alqueva dam

2.2. GEODETIC MONITORING SYSTEM

To control the horizontal displacements, the geodetic monitoring system includes two traverses, one along the crest, and another along the inspection gallery number 4. In this paper will be only presented the results of the upper traverse.
The traverse along the crest has, at the present, fourteen points, as seen in Fig. 4: the eight points P1 to P8 are object points; the three points PD, PD1 and PE, are the reference points; the three points on the left abutment (P9, P10 and P11) are auxiliary points. All points are materialized by forced centring pieces (Wild type) in the top of concrete pillars (Fig. 5). The reference pillars have large concrete foundations on rock. The eight pillars on the crest of the dam are placed in the same profiles that the inverted pendulums of the dam (see Fig. 2).



(a) Scheme

(b) Reading station

Figure 3 – Inverted pendulums

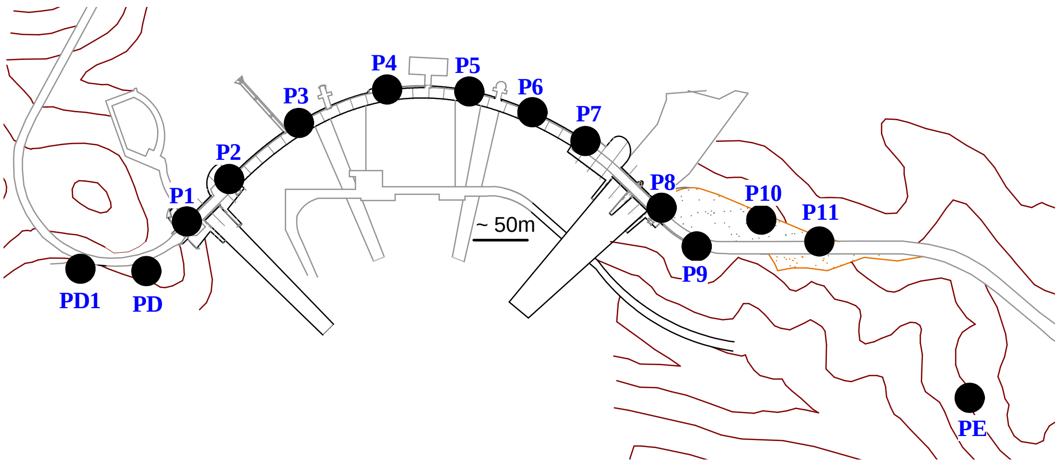


Figure 4 – Traverse on the crest

The equipment used to made the measurements was a motorized tacheometer Leica TCA2003 (Fig. 5), equipped with an automatic target recognition (ATR) system and precision circular retro-reflectors (Fig. 6), each mounted on a carrier with a built in tubular bubble.

Figure 5 – Tacheometer TCA2003

Figure 6 –Retroreflector GPH1P

3. THE CAMPAIGNS

The first geodetic campaign was made in February 2002, during the week after the beginning of the first filling of the reservoir. As the measurements of all pendulums started a few weeks later, in this study the reference campaign will be the one made in the beginning of October 2002, the first one that has “geodetic” displacements and “pendulums” displacements. The pendulums measurements had a weakly periodicity in the first years, reduced to two or even once a month after the year of 2005. Concerning the geodetic campaigns, they were undertaken usually twice a year in the first years; after 2006, once a year. In Table 1 is presented the date and the water level in the reservoir for each epoch that will be analysed in this paper.

Epoch	Date	Water level (m)	Epoch	Date	Water level (m)
01 (ref)	2002-10-01	116.76	06	2004-06-01	148.51
			07	2005-05-31	145.98
02	2002-11-19	118.33	08	2005-12-13	144.95
03	2003-01-22	129.25	09	2006-04-19	145.42
04	2003-11-25	138.85	10	2006-09-04	143.83
05	2004-01-20	143.49	11	2007-12-11	147.45

Table 1 – Epochs: Date and water level

4. THE DISPLACEMENTS

The pendulums displacements are determined directly, by calculating differences from the readings. To estimate the displacements from the geodetic measurements is applied a mathematical model (Casaca, 2001; Henriques et al, 2003) that combines a functional model, which relates displacements of the points to variations of observables variables (herein horizontal angles and distances), with a stochastic model that allows the performing the quality control of the observables.

Is possible to compare the displacements obtained by the two methods since the geodetic object points are in blocks that have pendulums. In the next paragraphs are presented the results concerning the 11 epochs (Table 1) that have the “pendulums” and the “geodetic” displacements.

To compare de displacements calculated from the traverse measurements with those from the pendulums, one must take into account the fact that the crest (elevation 154 m) is 6 m above the first inspection gallery (elevation 148 m), where is the higher reading station of each pendulum.

As along each plumb line are made several measurements (one by each gallery crossed by the pendulum, see Fig. 7), it is possible to fit two lines, one constrained to the radial displacements, the other to the tangential displacements. Each line is described by a first or by a second degree polynomial (the degree is function on the number of reading points in the pendulum). The coefficients of each polynomial were estimated using the function *polyfit* of *MATLAB* (Mathworks, 2008), making possible to estimate the displacements at the level of the crest. In the graph of Fig. 8 are presented the two components (radial and tangential) of the displacements measured along the pendulum 4 (FP4 in Fig. 2) in two epochs (09-01 and 10-01). It's also represented, using larger lozenges, the displacement estimated at the crest elevation (154 m).

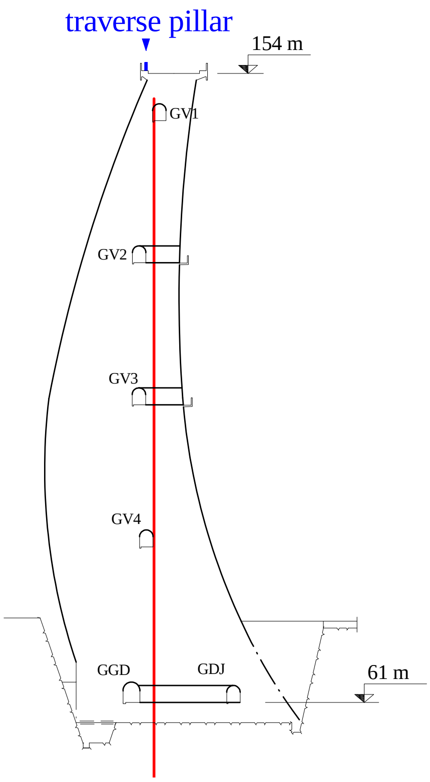


Figure 7 – Cross section of the inverted pendulum in the central block

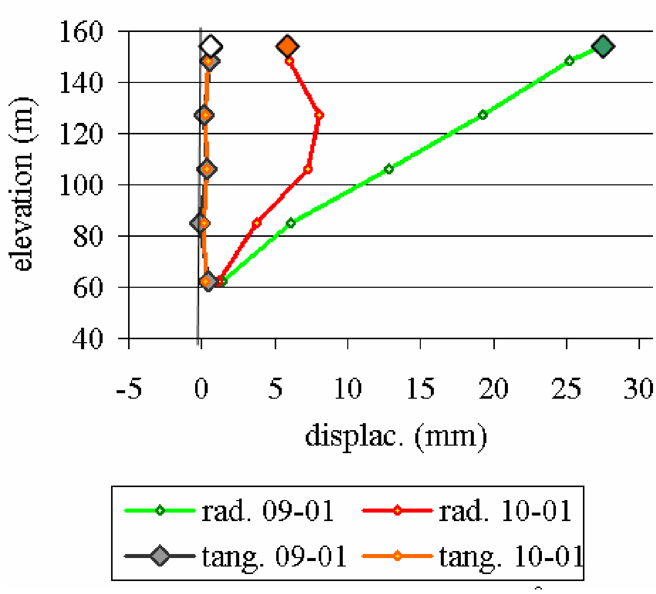


Figure 8 – Radial and tangential displacements measured along the pendulum 4. Larger lozenges: displacements calculated at the level of the crest

The displacements calculated from the geodetic measurements (points P1 to P8) as well as the displacements calculated at the crest based on the measurements made at the pendulums (curves FP1 to FP8; there were no readings in pendulum 7 in the epoch of reference) are showed in Fig. 9.

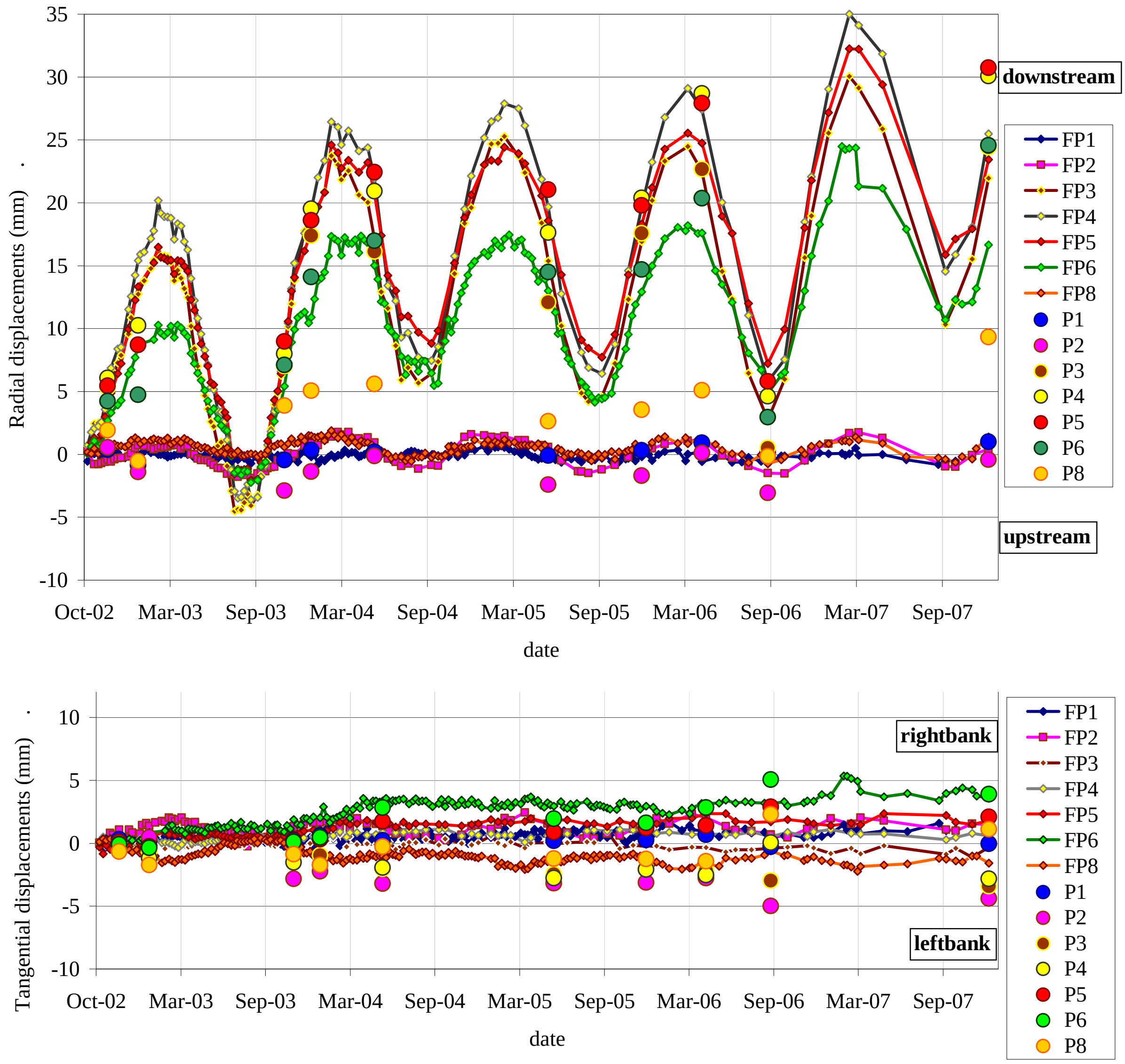


Figure 9 – Radial and tangential displacements at the crest. Reference 2002-October

5. TESTING THE DISPLACEMENTS

The displacements at the level of the crest, the pendulum and the geodetic ones, are calculated by two independent methods, making it easy to test if they are significantly different or not. The test here proposed is based in Hotelling T^2 statistic (Morrison, 1990). Let $M=[\delta R, \delta T]$ be the matrix of differences of the radial displacements and of the tangential displacements for a point. The null hypothesis of the test is, for each point,

$$H_0: \quad \mu = \begin{bmatrix} \mu_{\delta R} \\ \mu_{\delta T} \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \tag{1}$$

where $\mu_{\delta R}$, for instance, stands for the mean of the differences of the radial displacements. If *N* independent observations are made, is possible to calculate $\bar{x}$ and S, which are estimates of the mean μ and of the variance-covariance matrix Σ,

$$\bar{x} = \begin{bmatrix} \bar{x}_{\delta R} & \bar{x}_{\delta T} \end{bmatrix}, \quad S = \begin{bmatrix} S^2_{\delta R} & cov(\delta R, \delta T) \\ cov(\delta R, \delta T) & S^2_{\delta T} \end{bmatrix} \tag{2}$$

Hotelling's T^2 statistic is defined, in the general case, as

$$T^2 = N(\bar{x} - \mu_0)^T S^{-1} (\bar{x} - \mu_0) \tag{3}$$

If the null hypothesis is true then

$$F = \frac{N-p}{p(N-1)} T^2 \quad \in \quad F(p, N-p) \tag{4}$$

The null hypothesis is accepted if

$$T^2 \le \frac{p(N-1)}{N-p} F_{\alpha;\, p, N-p} \tag{5}$$

In the above example of Alqueva dam we have μ_0=[0,0], *N*=10 (10 sets of displacements) and *p*=2 (each displacement has two components). Establishing the significant level, here set as α=0.05, it can be seen that whenever $T^2 \le 10.4$ the hypothesis that the pendulum displacements and the geodetic displacements are not significantly different is accepted. In Table 2 are presented the values of T^2 for the 7 points presented in the graphs and if the null hypothesis is accepted (A) or rejected (R).

Point	P1	P2	P3	P4	P5	P6	P8
T^2	5.4 (A)	53.3 (R)	58.0 (R)	45.4 (R)	2.5 (A)	5.1 (A)	11.1 (R)

Table 2 – Hotelling T2 statistic values

A quick analysis shows that the majority of the geodetic displacements are significantly different from pendulum displacements.

6. A NEW REFERENCE EPOCH

The displacements presented in Figure 10 have as reference the epoch 1 (October 2002). The configuration of the traverse is the one presented in Figure 10 (configuration A). When the pillars of the auxiliary points (P9, P10 and P11) and of the reference point PD1 (see Fig. 3) were erected during the year of 2003, it was decided to use a new configuration (here called B). In Figure 10 are presented the two configurations.

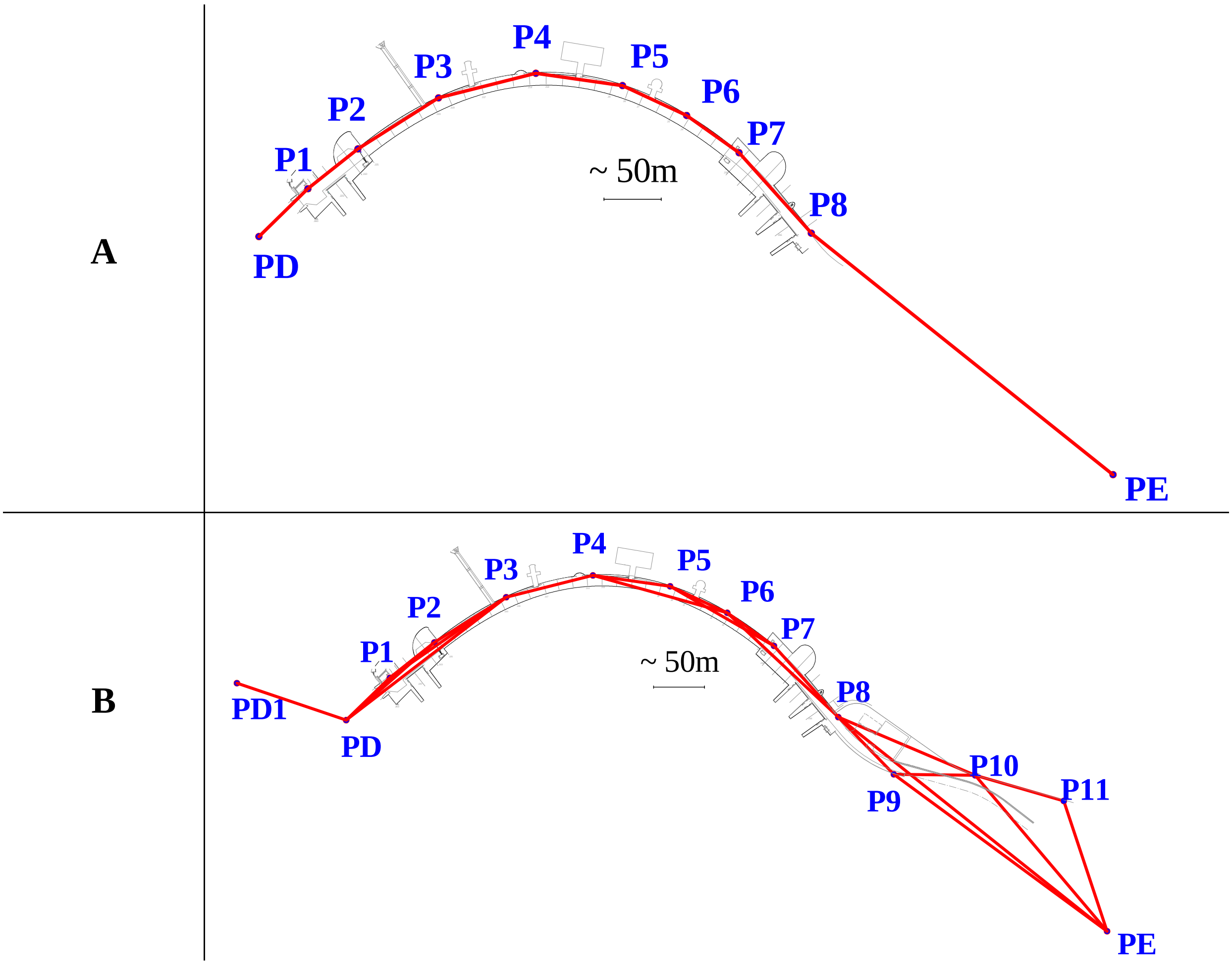


Figure 10 – Configuration A (prior November 2003) and configuration B

The configuration A has the usual configuration of a traverse. Configuration B has much higher redundancy, being therefore more robust. In a robust network the results are much less affected by observation errors than in a network with lack of redundancy. Observation errors can be detected during the quality control of the observations. The robustness of a network can be quantified by the local redundancy numbers (LRN) that are connect with each observable, numbers that can undertaken values in the interval [0,1]. It is usual to consider that the redundancy of one observable is insufficient when its LRN is smaller than 0.5, sufficient when its LRN is in the interval [0.5, 0.8] and good in the remaining cases. In Table 3 is presented, for configurations A and B, the percentage of angles and distances in the three classes previously described. It is to highlight that, with configuration B, is possible to control the majority of the angles.

	Configuration A		Configuration B	
redundancy	angles	distances	angles	distances
insufficient	100%	12%	16%	0%
sufficient	0%	88%	53%	83%
good	0%	0%	31%	17%

Table 3 – Percentage of observables in the three classes of redundancy

It was decided to consider a new epoch of reference for the displacements: the first geodetic campaign with configuration B (Nov. 2003). The results are presented in Figure 10. Due to the shift of the reference epoch is now possible to include the seventh pendulum and point P7.

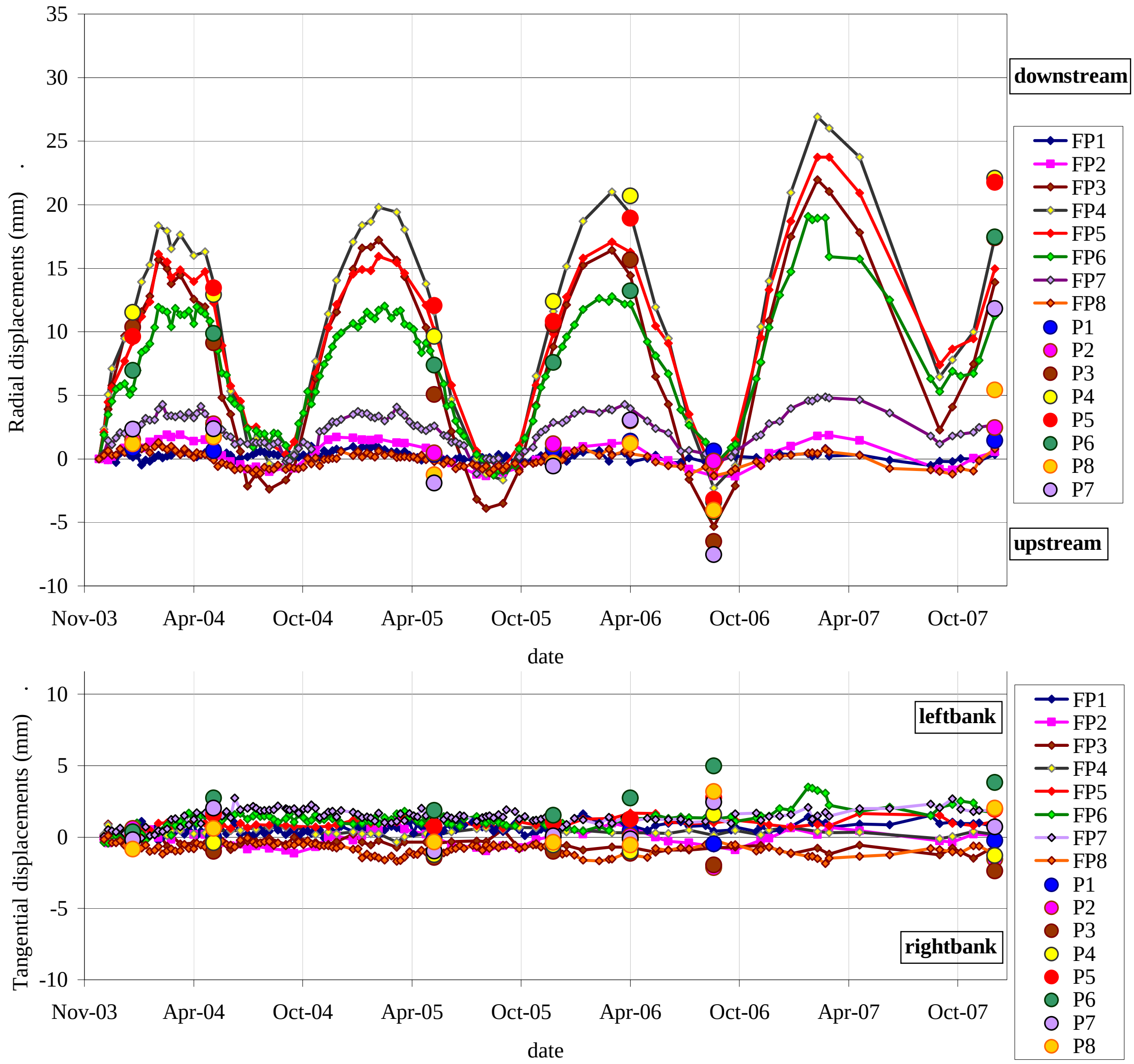


Figure 10 – Radial and tangential displacements at the crest: reference 2003Nov

Performing the test to analyse if the pendulum and geodetic displacements are significantly different, based in Hotelling T^2 statistic we have the values presented in Table 4. N has now the value 7, the other variables have obviously the same values and if $T^2 \leq 13.9$ the null hypothesis is accepted.

Point	P1	P2	P3	P4	P5	P6	P7	P8
T^2	22.5 (R)	9.3 (A)	7.8 (A)	3.3 (A)	2.5 (A)	7.0 (A)	3.6 (A)	9.5 (A)

Table 4 – Hotelling T^2 statistic values. Configuration B

The new values show that, with the exception of point P1, it can be accepted that the geodetic displacements are not significantly different from the pendulum displacements.

The rejected cases presented in tables 2 and 4 are related with points that have, for each component, differences that are or always positive or always negative. This situation is a result of the existence of systematic errors.

7. CONCLUSIONS

In this paper is presented a comparison between the planimetric displacements observed in a large concrete dam through pendulums and geodetic methods.

The use of two independent methodologies is important to analyse and validate the results of each one. The results obtained by the geodetic methods, that are referred to the first campaign were not as good as the ones obtained after the improvement of the geodetic network. The first geodetic campaign was made during the final stages of the construction of the dam, being usual to have to interrupt the observations due to the works undergone on the crest. After the conclusion of the dam, which has allowed the inclusion of new points in the traverse, along with the increase of the number of measurements made in the network, the values obtained by the two methods are not significantly different.

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