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Monitoring the behaviour of Massingir dam in Mozambique

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Abstract

Massingir, an earth fill dam with nearly 5 km long and a maximum height of 47 m is the 2nd largest dam in Mozambique. The reservoir has a maximum storage capacity of about 2,800 hm³ and the flooded area, at the Normal Water Level (NWL), is 138 km².
Dam construction took place between 1971 and 1977, but because of armed conflict and funding issues, the construction was incomplete, missing the installation of the spillway gates (six gates) and the hydro-power station.
During the first filling in 1977 it exhibited serious behaviour problems on the foundation, namely because piping have occurred in large extensions of the right bank dike. Its exploitation was thus, limited, and only after the rehabilitation works completed in 2006, the normal exploitation of the reservoir begun, although still without the power house. The rehabilitation included the following tasks: i) partial treatment of the foundation, by grouting, ii) construction of a stabilizing berm downstream of the dam's right dike, iii) construction of relief wells along the right bank of the dike and the main valley; iv) the lift of 1 m of the crest elevation; v) construction of a concrete parapet on the crest; vi) rehabilitation of the rip-rap; vii) the installation of the spillway gates, viii) repair of hydro-mechanical equipments of the bottom discharges, ix) rehabilitation of the monitoring system and some other minor works.
As a result of the re-filling, a major accident occurred at the exit of the bottom outlet discharges, reconditioning again the exploitation of the reservoir. Currently the dam is undergoing a new intervention to increase the capacity of the spillway flood discharges and to repair the bottom outlets exit. The dam has a monitoring system installed, from which several physical quantities are obtained, on the basis of which it is possible to establish statistical models used for assessing its behaviour and to predict its response for different scenarios. Furthermore, the models allow to establish criteria for attention and alert to adopt in future exploration.
Keywords: fill dam, accident, rehabilitation, monitoring

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1 Description of Massingir dam

Massingir, an earth fill dam, with nearly 5 km long and a maximum height of 47 m is the 2nd largest dam in Mozambique. The dam consists of three different zones. The first one, is a 47 m high embankment dam, barring the larger bed of the river. The two other zones are lower embankments, one very long on the right margin (RM), and other in the left margin (LM). The dam has a spillway at the top of the left margin, followed by a channel that develops along the shore to the river bed, a conduit taking water to the hydro power station and two bottom discharges, located between the main dam and the dike on the right bank (Figures 1 and 2).
The reservoir has a maximum storage capacity of about 2,800 hm³ and the flooded area, at the Normal Water Level (NWL), is 138 km².
The dam site, near the ancient village of Tiobine, lies in the river valley of the Elephants river downstream of the former headquarters of the Administrative post of Massingir. The throat is where the river is about 400 m wide, with margins of low height, of about 20 m. The low flow channel of the river is located between the elevations 82.00 and 85.00 m. The flow channel as two levels, the first between elevations 85.00 and 87.00 m and the second between 90.00 m and 91.00 m, with extensive wetlands upstream and downstream section of Tiobine.
In this section, chosen for implementation of the dam, the slopes of the valley are quite steep. In the right bank the slope is of about 45°, up the level 110 m. From this level, the asymmetry of the valley on both sides becomes quite marked. The right edge extends in a length of 3 km, with levels lower than 110 m, with a dike at elevation 99 m, with almost 2 km of length. This plateau ends abruptly against the foot of the hills that rise rapidly to a height of 130 m.

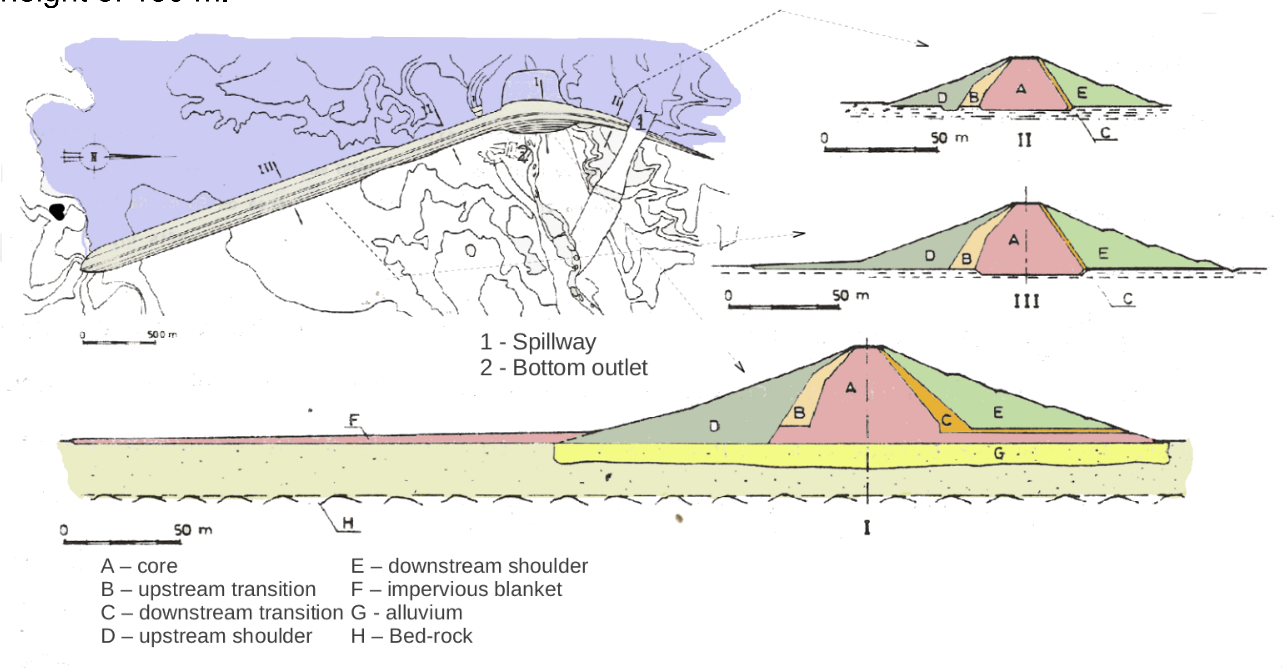

Figure 1 - Plan and cross sections of Massingir dam

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Figure 2 - View of the upstream slope (from the right margin) of the Massingir dam

On the left bank, the rise of the terrain is regular, reaching a distance of less than 1 km, the elevation 125.00 m. The main valley is filled with sandy alluvium, sometimes with coarser levels or lenticules with the same pebble and boulder or, conversely, horizons or finer passages, sometimes silts and clays. The "bed-rock" is a about 26 m depth. The foundation of the left margin and part of the right bank has a similar structure formed by gravel materials of variable thickness and more or less coarse or silty-clay, overlying the stoneware complex and marl with limestone counter tops. This complex is more or less weathered at the surface. The thickness of the gravel is between 0.5 and 3.5 m while the depth of the weathering of the clay stoneware could reach 2 to 3 m. On the right bank, between the topographic step that constitutes the hills and the level at elevation 107 m to 104 m, there are clay or silty alluvium whose thickness, in places, is about 30 m.
Due to its length, the dam has different solutions for cross sections as a function of the height in each zone and the conditions of the foundation. The dam has a maximum height above foundation of about 47 m and a length of 4650 m. The crest, with 12 m wide, is at elevation 131 m. The maximum water level (MWL) and the of maximum flood level(MFL) are 125.0 and 128.5 m, respectively.
Given the morphology of the terrain, availability of materials and characteristics of loan and nature of foundations, 3 types of cross-sections where adopted (Figure 1):
Profile I - Corresponding to the main valley area where the foundation is formed by sandy alluvium with low compactness and high permeability. The alluvium was treated by vibroflotation and a upstream “impermeable” clay blanket (Figure 3) with a length of about 220 m was placed. In addition relief wells at the downstream where also provided. The profile is characterized by having a central core of massive clayey material (Material A) and shoulders made of sand and sandy clay soils (D) in the upstream side, and originated from the excavations (E) in the downstream side. The transitions between the shoulders and core are made by means of gravelly materials (B and C). In the downstream zone, a drainage blanket extends downstream to the toe of the dam.

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Profile II - corresponding to the zone of the dam laying based on silty-clay materials. It comprises a clayey core (A) surrounded and shoulders made of sand and sandy clay soils (D), and originated from materials of the excavations. The transition between the core and the shoulders are, as in profile I, in the upstream side made with gravelly materials (B). Downstream of the core there is a layer of filter material (about 2 m in thickness). In the foundation, a cut-off trench cross-water of about 3 m deep exists to improve the tightness of the dam. In the upstream toe, a layer of granular materials with about 40 m length, provides protection to prevent alteration of the foundation soil and minimize the effects of swelling of the same soil, when in contact with water.
Profile III - Corresponding to areas of the dam built on gravel or on fractured rock. It is a profile similar to the profile II, differing from this, above all, for not having the upstream blanket nor the downstream drainage layer.

Figure 3 - Aerial view of the Massingir dam at the end of construction phase (Personal archives of Alvaro Carmo Vaz)

2 HISTORY OF THE DAM

2.1 First filling

Dam construction took place between 1971 and 1977, during a difficult period, corresponding to the armed conflict in Mozambique. Because of this and funding issues, the construction was incomplete, missing the installation of the spillway gates (six gates) and the hydro-power station. These were postponed. Nevertheless, in November 1977, the first filling was initiated. As a result of filling the reservoir, several deficiencies were detected in the behavior of the foundation, both in the area of the main dam, and more intensively in the dike on the right bank, where piping have occurred. The exploitation of the reservoir was then conditioned by these situations. For security reasons, the exploration of the reservoir was restricted to level 110 m, 15 meters below the Maximum Water Level (MWL). During the period preceding the rehabilitation, the water level in the reservoir (WL) only exceeded level 110, sporadically, due to the occurrence of significant floods. The higher level was achieved during the 2000 floods where, for a period of several hours, the water was at level 124 m. There are reports that, there was a significant increase in the percolation downstream of the right bank dike, with the notable increase in flooded areas and also zones where the water emerged to the surface, bubbling.

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2.2 Rehabilitation

From April 2004 to December 2006, Massingir dam was submitted to several rehabilitation works, based on a design from WAPCOS 1994, revised by Coyne et Bellier (as the consultant of ARA-SUL the owner). The rehabilitation included mainly the following tasks: i) sealing the foundation in certain areas, by grouting, ii) construction of a stabilizing berm downstream of the dam's right dike, iii) construction of 85 relief wells along the right bank the dike and the on main valley, with a drainage ditch to drain the water collected; iv) the lift of 1 m of the crest elevation; v) construction of a concrete parapet on the crest of the dam to increase security against extreme flood events; vi) rehabilitation of the rip-rap; vii) the installation of the spillway gates, viii) repair of hydro-mechanical equipments of the bottom discharges (cofferdams gates, sector gates, servomotors and downstream cofferdams), ix) rehabilitation of the monitoring system and other minor works (Figure 4).

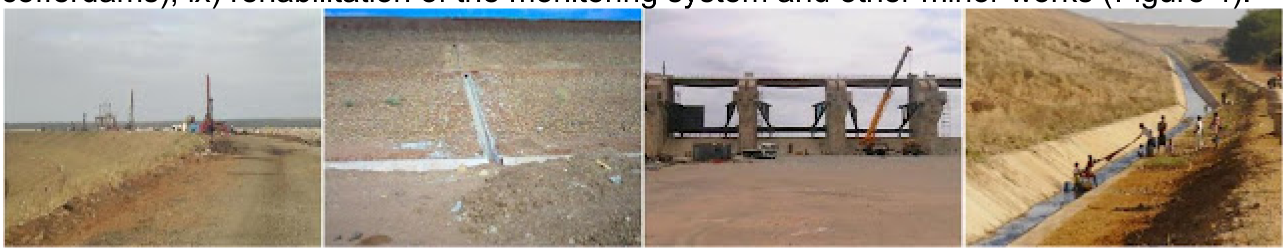

Figure 4 - Rehabilitation works: injections, stabilizing berm, gates and relief wells and ditch

2.3 Filling after the rehabilitation and accident in the bottom discharges

The refill of the reservoir, after the completion of the rehabilitation, followed a plan with three intermediate levels: 120, 122.5 and 125 m (MWL) (Coyne et Bellier, 2007). Filling was, however, interrupted by the accident that occurred in bottom discharges on 22 May 2008 when the water in the reservoir was at level 122.6 m (Figure 5). The analysis of possible causes of the accident is outside the scope of this paper and the subject will not be addressed. There is interest, however, to mention the consequences of the situation. Indeed, since then, the dam operation is very limited, because the bottom discharges are virtually inoperative. The level of the reservoir is controlled above the level 115 m, by the partial opening of the floodgates of the spillway.

Figure 5 - Accident in the dam (May 2008) and final state of the central area and exits of bottom discharges

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Below this elevation the dam can not be operated unless they proceed to the opening of the floodgates of the bottom discharges, with all the inconveniences that might ensue, namely very strong vibrations.
Moreover, according to the most recent hydrological studies, the total capacity of the spill-way, roughly 6x900m³/s, plus the bottom outlets (2x900m³/s) don't not have sufficient flow capacity for the maximum probable flood (MPF). For this reason a emergency flood spillway is now under construction. Thus, in the situation where the bottom discharges, which are essential to aid the flow of MPF, are not operating, the operation is even more constrained.

3 Monitoring system

Following the rehabilitation works, the observation system was reinforced and a organized registration system allowed the collection of valuable data from the devices installed. In a recent mission to the dam the authors have undertaken a comprehensive survey of the status of all the devices installed and have also analysed the results. In summary, the observation system currently installed in Massingir dam, following the construction and rehabilitation comprises devices for monitoring : a) water levels in the reservoir (WL), b) the surface displacement by leveling marks (MN), c) internal displacements, horizontal and vertical using inclinometers and settlement gauges; d) pore pressures in the dam body and foundation (using standpipe piezometers), e) flow from the relief wells, and, f) the total flow collected by the various wells in the right margin.
In addition to these quantities, the monitoring system of the dam also includes a weather station, also essential in the water balance of the reservoir.
The observation system, is suitable to monitoring the dam, as deduced by applying the methodology recommended in Portuguese dam safety legislation. However, it is considered that either the amount, the functionality or even the arrangement of devices along the dam revealed some shortcomings. For example, the system to monitor the surface displacements is insufficient and non-operational. The same scenario stands for the internal displacements. The pore pressures inside the dam are followed in very few cross sections. All the other aspects may be considered as adequate.

4 Statistical models

4.1 Introduction

Safety judgment of operating dams should be done by comparing the actual behaviour (observed) with the expected behaviour, the latter normally established by means of models. Models used in the comparisons, may be of the conceptual type or have mathematical nature. According to Oliveira (2000), the safety analysis can be done by: analytical models, usually limited to very simple cases; experimental models, usually more expensive and sometimes hard to make; numeric, when the equations that define the behavior of the dam are solved by numerical methods and; semi-empirical or statistical, in which relations between the actions and the responses are established in order to incorporate the monitoring data from the dam itself.
Obviously, the various approaches presented have different fields of application and different costs and should generally be considered as complementary. Here, it was decided to use the semi-empirical models, based on the available monitoring data.

This models attempt to relate the variables of the problem (typically the actions or independent variables) with the response (or dependent variables), translated by displacement, pressure, flow, etc., by a simple expression of the type:

$$f(x_1, x_2, \ldots, x_n) = \beta_1 \psi_1(x_1) + \beta_2 \psi_2(x_2) + \ldots + \beta_n \psi_n(x_n) \qquad \text{(Equation 1)}$$

where $f(x_1, x_2, \ldots, x_n)$ is the function which represents the statistical model, β_i are the parameters to be determined and $\psi_n(x_n)$ functions of the independent variables.

The unknown coefficients β_i can be computed by Equation 2:

$$\{\beta\} = ([X]^T [X])^{-1} [X]^T \{y\} \qquad \text{(Equation 2)}$$

where $\{\beta\}$ represents the vector of β_i , $\{y\}$ the vector of the dependent variables (actual values) and $[X]$ is the matrix of functions $\psi_n(x_n)$.

4.2 Model for flow totalizer

The flow obtained by the various wells installed downstream of the dam's right bank (Figure 1), is collected by a ditch for the lower zone located near profile 0+625. The measurement of total flow is done in a flow weir. Figure 6 shows the results from the adjustment made to the flow totalizer data. Two expressions were tested:

$$F = \beta_1 + \beta_2 WL \qquad \text{(Equation 3)}$$

$$F = \beta_1 + \beta_2 N_d + \beta_3 WL \qquad \text{(Equation 4)}$$

where F stands for flow, N_d the number of days from an arbitrary date and WL for water level in the reservoir. According to the results, (Equation 7) is sufficient to explain (model) the flow data. Furthermore, there is not a clear trend which the flow varies over time. In practical terms, the flow collected in the totalizer can be estimated by:

$$F[l/s] = -2420{,}9 + 23{,}22\, WL(m) \qquad \text{(Equation 5)}$$

In the computations, 51 data points were used, all available from 2006, yielding a correlation coefficient R of 0.98. It is considered that this model is reliable to represent the behaviour of the foundation.

4.3 Pore pressure in the foundation of the right bank dike

Several relationships between the reservoir level and the pressure on each piezometer were tested. To assess the possible variation in time, regardless the other variables, a time-related term was considered.

The following equations were considered (where β_i are the unknowns to be determined, *p* is the pressure and N_d the number of days that have elapsed since an arbitrary date (the date of the 1st observation was chosen):

$$p = \beta_1 + \beta_2 WL \qquad \text{(Equation 6)}$$

$$p = \beta_1 + \beta_2 N_d + \beta_3 WL \qquad \text{(Equation 7)}$$

$$p = \beta_1 + \beta_2 N_d + \beta_3 WL + \beta_4 WL^2 \qquad \text{(Equation 8)}$$

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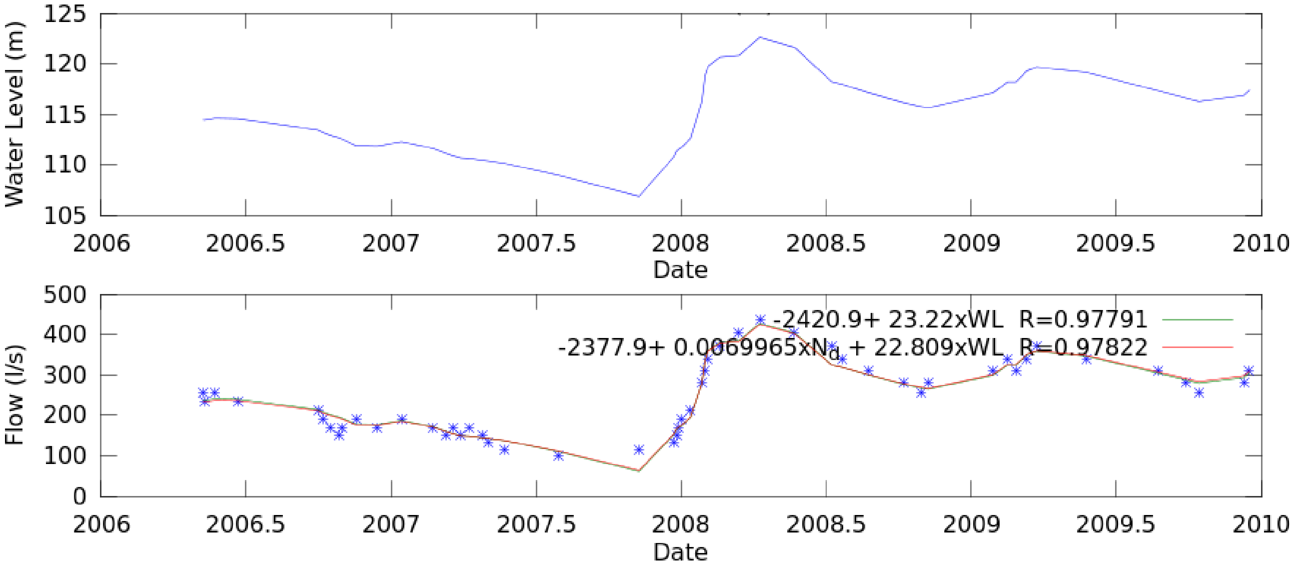

Figure 6 - Diagram setting expression (Equation 5) to flow totalizer

The analysis of diagrams of reservoir level versus piezometric pressure allows seeing a clear dependency on the two variables. The relation is approximately linear. In general, Equation 6 is sufficient to model the pressure in the piezometers. Figure 7 (left), corresponding to the adjustment from June 2005, illustrates one of the diagrams obtained. In this diagram it is clear that in the first readings there is a mismatch between the model and the actual data.

This may have various reasons but, most probably, corresponds to a period of (re) saturation of the foundation after a long period in which the reservoir was empty. Because it is a situation found in almost all devices of this type, it was decided that the model should not include these points, corresponding roughly to the first 40 data points. Thus, the adjustment was made for the readings from 20 March 2006. Figure 7 (right) shows the fit obtained. The resulting improvement is evident. Indeed, in this case, the correlation coefficient R increased from 0.84 to 0.98, as expressed in the plots.

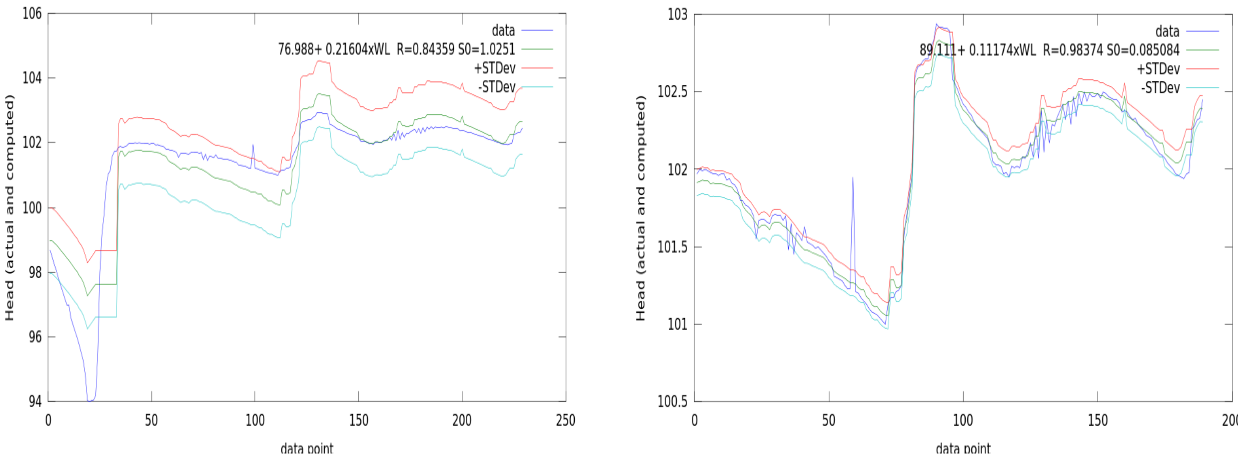

Figure 7 - Example of linear regression between reservoir level and piezometric pressure with/without initial readings (Piezometer 30-1, 1 profile +562.50)

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Analysing all the parameters obtained for the linear equation for all piezometers of the drainage ditch (no presented here), the average correlation coefficient is about 0.94. The lower value of 0.89 occurs in only two piezometers. All other devices have excellent adherence to the linear model proposed.

4.4 Flow in the relief wells

Interspersed with the piezometers, the dam has relief wells, whose purpose is to reduce the hydraulic gradients outside of the foundation in order to prevent piping or hydraulic lift. As recommended in the monitoring plan the flow in the relief wells is measured with the same frequency as the piezometers. About the measurement system, it should be noted that the measurement of the flow is made by the time needed for filling a container of a given volume. Initially a container of 5 liters was used. However, in many wells, when water rose above level 115 m, there are quite significant flows, which entailed the use of a higher volume. Currently, a container with 10 liters is used, but still, in many wells, when higher reservoir levels occur, it is difficult to measure the flow with the required accuracy. Because of this significant scatter was expected in the data.

For the establishment of analytical models, several relationships between the reservoir water level (WL), the rain fall (R_N) and the flow measured in each well (F) were considered. To assess the possible variation in time, regardless of any one of the other variables, a time dependent term was also considered. As respect to rainfall, it is important to consider the sum of the rainfall over a period of some days because it is reasonable to assume that the rain which has fallen on days prior to measurement will influence the flow rate. Time intervals of 1 to 7 days were considered.

In adjustments made, although the overall quality is not optimal (low correlation coefficient of about 0.6), it can be seen that the reservoir level is, unsurprisingly, the most important factor. Dependence on rainfall, although present, is very weak and relatively independent on the number of days in the accumulation rainfall. The best fit corresponds to a relatively high period of days of rainfall. However, in this case, the coefficient that affects the rainfall, is negative, and therefore with no physical meaning. Considering only the positive values the best fit corresponds to about 2 days of accumulated rainfall. However, as noted, the variation is insignificant and it is, perhaps, inappropriate to draw conclusions of this kind.

The following formulas were considered (where β_i are the unknowns, F the flow and R_N the rainfall over "N" days - 1-7 days)

$$F=\beta_1+\beta_2 WL \quad \text{(Equation 9)}$$

$$F=\beta_1+\beta_2 N_d+\beta_3 R_N+\beta_4 WL \quad \text{(Equation 10)}$$

$$p=\beta_1+\beta_2 N_d+\beta_3 WL+\beta_4 WL^2 \quad \text{(Equation 11)}$$

As expected, because of the errors due to the measuring system, some of the values in the data used have significant errors. In particular, very often, between consecutive readings of the flow, the change is of the same magnitude of the actual reading, without apparent reason for this to happen. Apart this aspect, there are interruptions in the readings, in particular when the reservoir level is higher, which relates to the fact that some wells become drowned, preventing the measurement. The available results, nevertheless, allow the realization of forecasts.

Figure 8 shows measurements of flow corresponding to reservoir level 122.53 m on 8 April 2008. Some of the wells were not measured because they were drowned. The sum of the measured flow (in the wells) in this date is 361 l/s. The predicted value, according to the model established in section 4.2 is 424 l/s. Thus, it is assumed that the wells that have not been measured, between the profile 825 and 1100, could have a flow corresponding to the difference, ie about 63 l/s. This value is compatible with other dates on which measurements were made in these wells. This was the case, for example in May 2010, where in the same set of wells (except those situated to profile 850, 950 and 1075) it was obtained a total flow of 68 l/s, although for a slightly lower water level (120.66 m) .

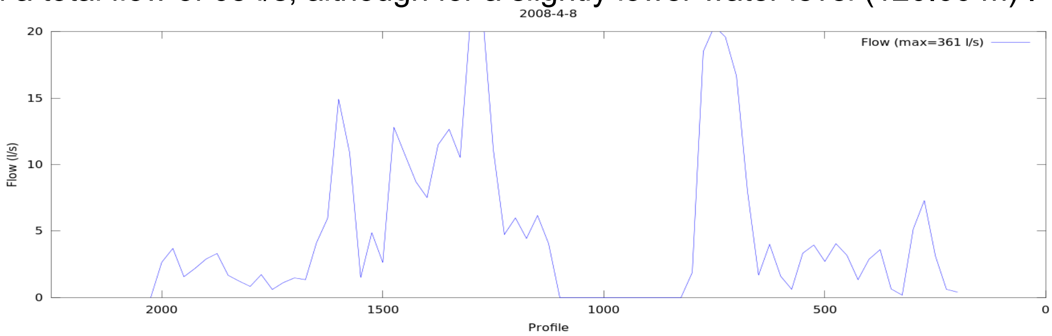

Figure 8 - Flow measured in the relief wells on 08/04/2008

5 Modelling future behavior

In Massingir dam, because of its history, it is of great importance the monitoring of the behaviour of the foundation of the right bank dike. Based on the models presented before, one can estimate the behavior of the piezometry in that area. Indeed, assuming that for higher reservoir levels, the models are still applicable, which could not happen due to very high gradients, one can predict the operation of the foundation in respect to water pressure, for example.

Figure 9 illustrates the situation observed for the lower levels of the reservoir - WL at elevation 110 m (in yellow, the limit of the areas treated with foundation grouting). The pressure head is always below the terrain elevation. Figure 10 illustrates the prediction for the maximum water level. In a considerable extension of the foundation, the water head is above the terrain. The worst situation, with a head of about 3.7 m above the foundation, is close to the profile 0+862.50, near the limit of an area treated with grouting. Although the hydraulic gradient is not very high - about 0.17 - this gradient is computed over 23 m (the length of the piezometer). Given the amount of wetlands that occur to the highest levels of the reservoir, probably the gradients are substantially higher.

Figure 11 shows the estimate corresponding to the maximum flood level. In these circumstances the maximum head height is 4.6 m above the ground and artesianism occurs in about 67% of the length of the dike. At the full storage level, about 50% of the foundation has artesianism.

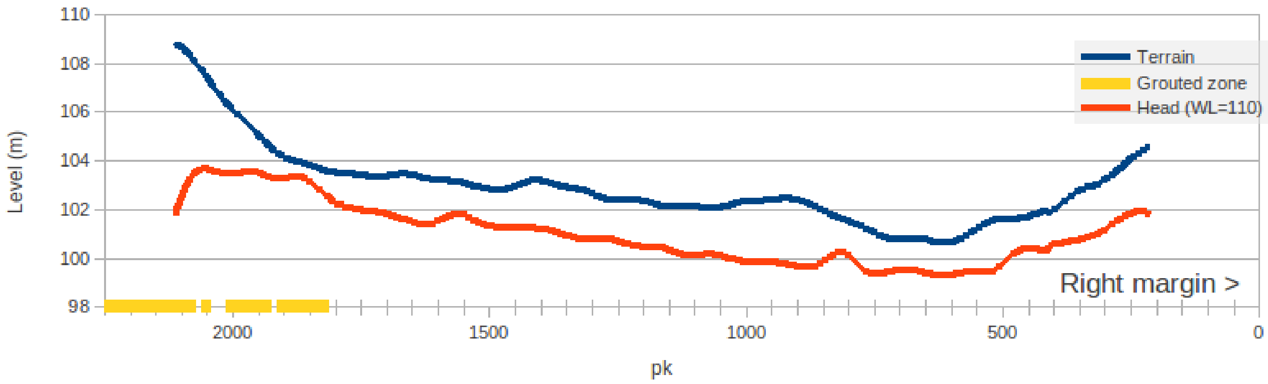

Figure 9 - Prediction of pressure in the foundation of the right bank dike for water level = 110 m

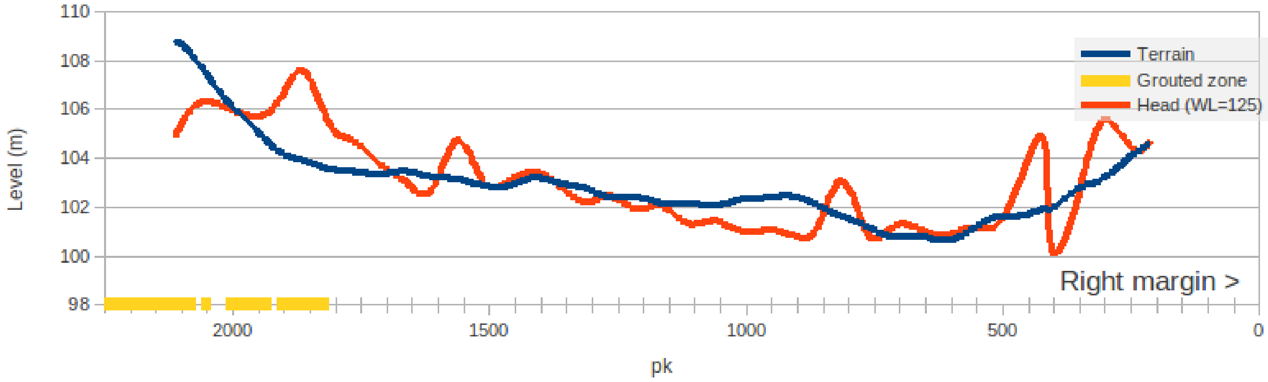

Figure 10 - Prediction of pressure in the foundation of the right bank dike for water level = 125 m

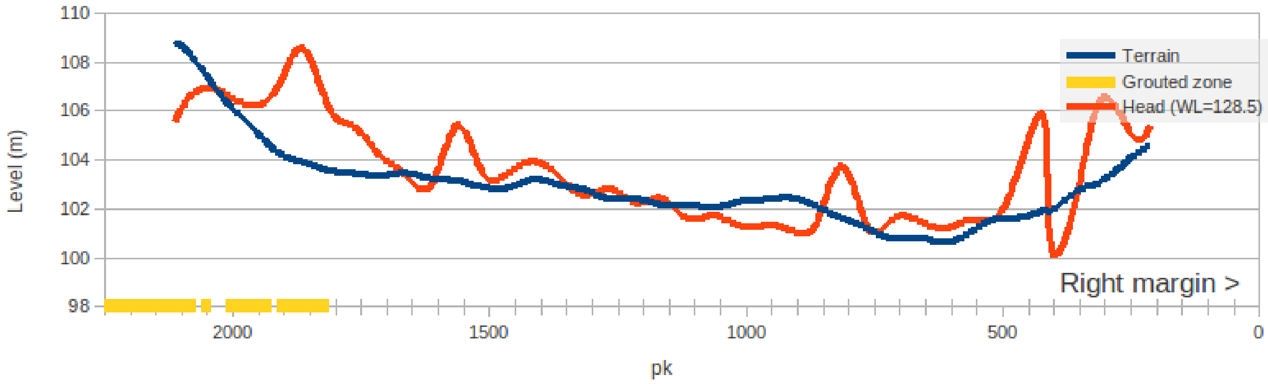

Figure 11 - Prediction of pressure in the foundation of the right bank dike for water level = 128.5 m

6 FINAL REMARKS

The results obtained by observation of the Massingir dam, particularly those related to the behavior of the foundation of the dike on the right bank, indicate that, even after the rehabilitation works, the dam is not fully safe. In fact, there are strong indications that the high flows and excessive pressures through the foundation are not yet controlled. The treatment with relief wells has shown to be ineffective to reduce the uplift and piping possibility and, the high flow indicate the occurrence of high seepage velocities. For higher levels of the reservoir, there is widespread flooding of the foundation.

Solutions to consider for the resolution of such problems should seek to satisfy two criteria: a) the dam and foundation waterproofing in the upstream side and b) the downstream pressure relief. The downstream drainage should have filter elements that control (prevent) particle erosion. In the case of Massingir dam, the foundation sealing has proved difficult to achieve. First, the foundation of the reservoir near the dike is not sufficiently waterproof and the soil reclamation near the toe probably exacerbated the situation. Secondly, the alluvium of the foundation, very permeable (permeability of about 10^{-1} cm/s) and heterogeneous, are hard to treat, particularly through partial grouting treatments such as those executed in the rehabilitation. The existence of a partial treatment redirects the flow to the untreated areas, as seems to happen in this case. Since the amount of flow through the foundation is not particularly important, for example, 500 l/s in a year corresponds to only 0.6% of the volume of the reservoir, the most likely economic solution is to ensure that the flow rate is delivered safely. This corresponds to minimize seepage velocity and to filter the output of the flow area. Thus, the implementation of additional drainage wells may not only be ineffective, as can aggravate the current situation. Instead, the installation of a stabilizing bank with a inverted filter, can provide a safe situation for the dam. Using the models presented before, one can predict that the downstream stabilizing berm should have a height of 5.0 m. This of the height sufficient so that in a situation of maximum reservoir level the water head is still under the berm level. Naturally, this is a illustrative value and more detailed analyses have to be made.
Figure 12 shows the proposed solution. The extent of the berm should be established according to the area to be protected. The existing drainage ditch may be replaced by a duct of suitable diameter and the existing wells have to be fitted to allow access to the interior.

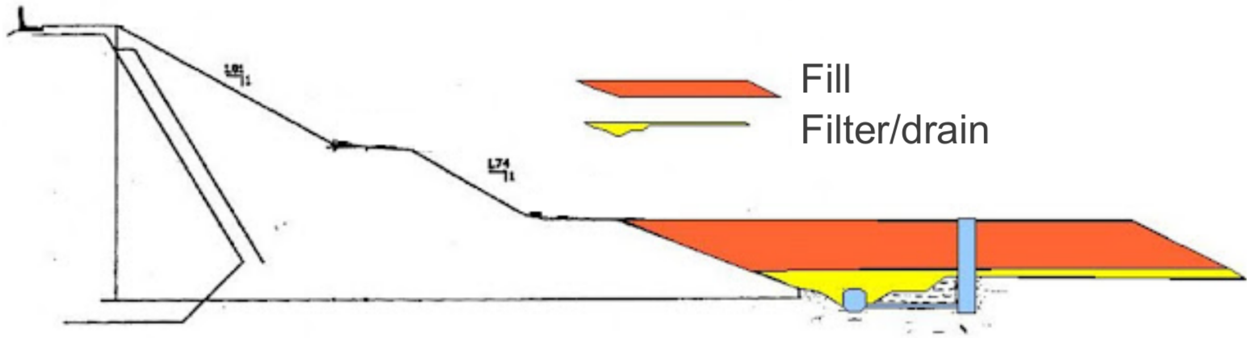

Figure 12 - Proposed stabilizing bank with a inverted filter for the right bank dike of Massingir dam

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