Influence of water pressure field on foundation displacements of Alqueva arch dam – A numerical approach

M. L. Braga Farinha[1], J. V. Lemos[1] and E. Maranha das Neves[2]

[1] LNEC – National Laboratory for Civil Engineering, Av. do Brasil 101, 1700-066 Lisboa, Portugal
[2] IST – Instituto Superior Técnico, Av. Rovisco Pais 1, 1049-001 Lisboa, Portugal

E-mail: lbraga@lnec.pt

Summary

A global three-dimensional model of a large concrete arch dam and foundation was developed, with which it is possible to perform both mechanical and hydraulic analyses. Joint water pressures are a relevant factor in the safety of this type of structure. This paper presents a study of the influence of different fields of water pressures within the foundation rock mass on both dam behaviour and foundation displacements. The model simulates various discontinuities, such as the dam/foundation interface, the dam contraction joints, and three of the five main foundation sets of discontinuities. In this model, both the grout and drainage curtains are represented. Analysis is done using the discrete element method (DEM) and the code 3DEC, and applying a methodology which has been used to assess the safety of arch dams regarding ultimate limit states involving the dam foundation. Results analysis showed that the orientation of rock mass discontinuities ensures that failure is very unlikely. The main objective of the study was to analyse the influence of different ways of modelling the water pressure field within the foundation rock mass on calculated displacements. Strength characteristics of the foundation discontinuities were gradually reduced to very low values, and it was this situation which showed that for very high water pressures the largest displacements are observed in a rock mass wedge located at around mid-height of the right bank, immediately downstream from the dam. The influence of the water pressure within the foundation on the stability of this rock wedge is assessed by comparing the results obtained without uplift pressures with those obtained using the hydraulic model or assuming simplified water pressure fields, defined in terms of a water table compatible with the water levels upstream and downstream from the dam and the valley slopes.

Introduction

Water pressures within the dam foundation have to be considered in the safety assessment of concrete arch dams, in order to express the failure criterion in terms of effective stresses. Three-dimensional (3D) discrete element models are particularly adequate for these studies [1], as they allow us to take into account not only the existence of the rock mass discontinuities and their shear and normal displacements, but also the water pressure field.

In dam foundation rock masses the majority of the flow takes place through the foundation discontinuities. However, in practice, it is not possible to use 3D discontinuum models in order to analyse seepage through the foundation and so obtain the water pressure field to be considered, because the jointing pattern is very complex and there is usually a lack of information regarding the hydraulic properties of the foundation joints. One possible alternative is to impose simplified water pressure fields, taking into account the reservoir water level, the existence of the drainage curtain and of a water table downstream from the dam compatible with the valley slopes. This was the procedure recently used in the foundation safety assessment of various dams of the "National Programme for Dams with High Hydroelectric Potential", launched in 2007 by the Portuguese Water Institute, which are presently either at the design stage or at the beginning of construction [1]. Another alternative to obtain the water pressures within the foundation, suggested by Lemos [2], is to perform hydraulic analysis using equivalent continuum models, which allow water pressures to be obtained in a more realistic way.

The study here presented takes into account the characteristics of Alqueva dam, and was carried out following another study in which the stability of the dam/foundation interface was analysed [3]. For that study a global modal of the dam and foundation was developed, in which both the grout and drainage curtains were represented and the discontinuities corresponding to the dam/foundation interface and to the dam contraction joints were simulated. The model, which was calibrated with field data, was used to perform a non-linear mechanical analysis (behaviour is linear before failure). The same model was used to perform hydraulic analysis, based on equivalent continuum concepts, which allowed the water pressures within the dam foundation to be obtained. These water pressures were applied on the discontinuities involved in the possible failure mechanism of sliding along the dam/foundation interface, and the dam safety was evaluated using the process of progressive reduction of strength characteristics of the discontinuities.

Results were compared with those obtained with the usual bi-linear uplift pressure distribution at the base of the dam, commonly used in concrete dam design.
In this study, three of the five main foundation sets of discontinuities were introduced into the above mentioned model. These sets of discontinuities are those which create rock mass blocks thought to have the potential to slide. In order to simplify the analysis, the sets of foundation discontinuities were only represented on the right bank and the possibility of failure was analysed for given mechanisms. The seismic action was not taken into account. Results obtained showed that, for the considered mechanisms, the orientation of the rock mass discontinuities ensures that failure is highly improbable. The model was used to study the relative influence of water pressures within the rock mass on its strength capacity.

Alqueva dam

Dam

Alqueva dam (Figure 1) is located on the River Guadiana, in the southeast of Portugal, and is the main structure of a multipurpose development designed for irrigation, energy production and water supply [4]. Alqueva dam creates the largest artificial lake in Western Europe.

Figure 1: Views of Alqueva dam

The double curvature arch dam has a maximum height of 96 m and a total length of 348 m between the abutments at the crest elevation (154 m). The dam width is 7 m at the crest, while at the base it varies from 30 m at the central cantilever to 33 m at the abutments. The powerhouse is located at the toe of the dam with a concrete dam-wall downstream. This dam-wall protects the powerhouse and the substation of the reservoir of Pedrógão dam, located around 23 km downstream from Alqueva dam, which allows the creation of an area downstream from the dam with an adequate water height for the turbines of Alqueva dam to work in a reverse motion, by pumping water from the downstream to the upstream reservoir. In the valley bottom there is an impervious slab between the arch and the dam-wall (substation slab), and thus, in this area, the dam length in the upstream-downstream direction is 140 m. The construction of the dam began in 1997 and ended in 2003. The first filling of the reservoir began in February 2002 and finished in January 2010, when the reservoir reached the retention water level (RWL), at 152.0 m.

Rock mass

The foundation consists of green schist of good quality on the right bank and the river bottom and of quite good phyllite, with a higher deformability, on the left bank. According to *in situ* tests carried out prior to dam construction the foundation Young’s modulus in the left bank varied from 6 to 20 GPa, and the foundation of the right bank and of the bottom of the valley had a Young’s modulus greater than 20 GPa.
Various sets of rock joints were identified, of which the most prevailing are: **A**) N40˚-65˚W, 12˚-65˚NE; **B**) N75˚W, S75˚W; 65˚N-90-55˚S; **C**) N65˚-90˚W,12˚-55˚NNE; **D**) N20˚-42˚W, 65˚-90˚ENE and **E**) N25˚-55˚W, 65˚SE-90˚ [5]. Figure 2 shows the average position of the main sets of rock joints in relation to the dam.

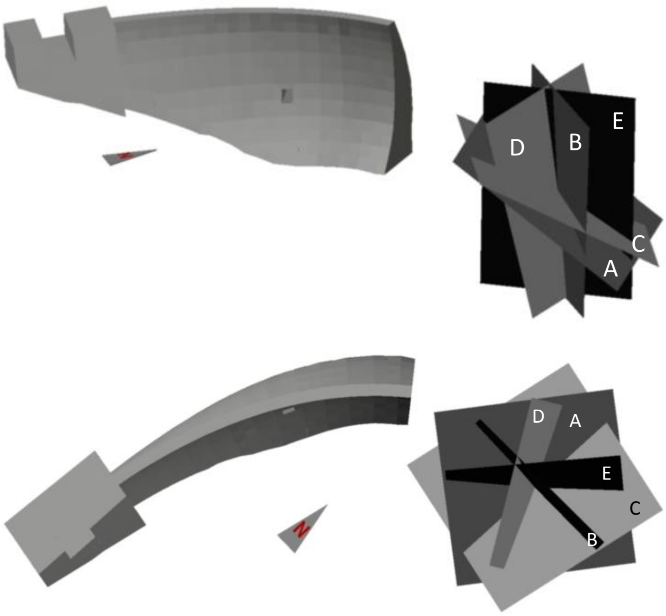


Figure 2: Average position of the main sets of rock joints in relation to the dam (perspective and view from above)

There are various faults in the rock mass, particularly in the left bank, the most important being fault 22, which corresponds to the green schist/phyllite interface. The faults identified at the dam site show attitudes (dip and dip direction) that, in the majority of the cases, allow the inclusion in the above mention sets of discontinuities. Sets of discontinuities observed in the green schist and in the phyllite, although with the same attitudes, show markedly differences in their geotechnical characteristics, namely in spacing between joints, persistence, filling, rugosity, aperture, and sidewall weathering [5]. Foundation set of discontinuities **A** is subparallel to schistosity. Table 1 shows the average cohesion and friction angle of discontinuities in the two different foundation areas.

TABLE 1: AVERAGE COHESION AND FRICTION ANGLE OF DISCONTINUITIES [6]

Discontinuities		Cohesion (MPa	Friction angle (˚)
Green schist	Along schistosity	0.10	24
	making an angle < 15° with schistosity	0.17	38
	making an angle > 15° with schistosity	0.18	43
phyllite	Along schistosity	0.11	22
	Subvertical and subhorizontal	0.13	29
	Between subvertical and subhorizontal	0.13	36

Foundation treatment consisted of consolidation of faults, general consolidation of the rock mass in the area of the dam and of the downstream dam-wall, contact grouting, and the installation of a grout curtain and a line of drainage boreholes. Due to the importance of fault 22 in the structural behaviour of the dam, a special treatment was carried out which included the excavation of a set of galleries, the removal of the material inside the fault and its replacement with concrete.

Numerical analysis

Model

Numerical analysis of both concrete dam and rock mass was carried out with the code 3DEC [7], based on the discrete element model, which allows the analysis of the mechanical behaviour of both structures and media with discontinuity surfaces and of the hydraulic behaviour, assuming that flow takes place either through the discontinuities or through equivalent continuum media.

The model used in the study presented in this paper is based on a model previously developed [8], which was used to analyse failure along the dam/foundation interface. In that model, shown in Figure 3, discontinuities simulating the dam contraction joints, the dam/foundation interface (foundation joint), and two hypothetical joints between the grout curtain and the rock mass, at the upstream and downstream faces of the grout curtain, respectively ("grout curtain/rock interface") were considered. These latter hypothetical joints were introduced in order to simulate the opening of vertical fissures within the dam foundation close to the upstream face of the dam, caused by the existence of tensile stresses that usually develop within the rock mass below the heel of the dam, due to the filling of the reservoir. In this model neither the faults nor the rock joints within the rock mass were considered, and thus the rock mass was assumed as a continuum medium. However, the model includes the location of fault 22, in order to simulate the area of lower modulus of elasticity, where the phyllite occurs.

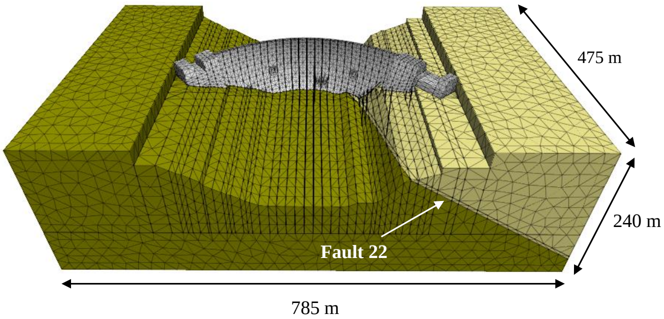


Figure 3: Geometry of the block system developed to analyse failure along the foundation joint and finite element mesh within the rock mass blocks

The geomechanical model was calibrated taking into account the monitoring results, and, for different reservoir levels, calculated dam displacements and displacements at the dam/foundation interface were very close to those recorded. The model developed also allowed an adequate simulation of variations of stresses within the dam foundation due to reservoir filling, and of variations of the aperture in the discontinuities through which water flows [3, 9].

In order to analyse ultimate limit states involving foundation discontinuities it was assumed that it was adequate to introduce into the model three of the main sets of discontinuities: sets **A**, **B** and **D**, represented in Figure 2. These sets of discontinuities, one of which is subhorizontal, are those which create rock mass blocks thought to have the potential to slide. It was decided not to represent the faults, which have the same attitude as the sets of discontinuities, and to simplify the model, representing the sets of discontinuities only on the right bank.

Each set of discontinuities was represented by a series of planes, and the spacing between these planes was assumed to be 30 m. Thus, only some of the discontinuities were represented, close to the arch. It should be mentioned that discontinuities were represented as continuous planes, disregarding the non-persistence of the joints, i.e. disregarding the existence of rock joints. Figures 4 and 5 show views of the model used in the study presented in this paper. Figure 5 shows that the set of subhorizontal discontinuities **A**, dips towards upstream, which does not favour failure mechanisms.

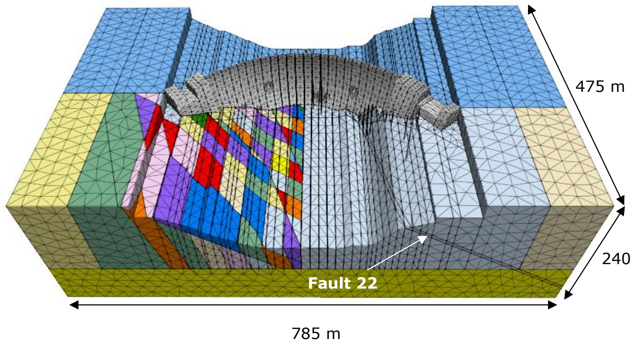


Figure 4: Perspective of the model developed to analyse the stability of the rock mass in the right bank, showing the finite element mesh within the rock mass blocks

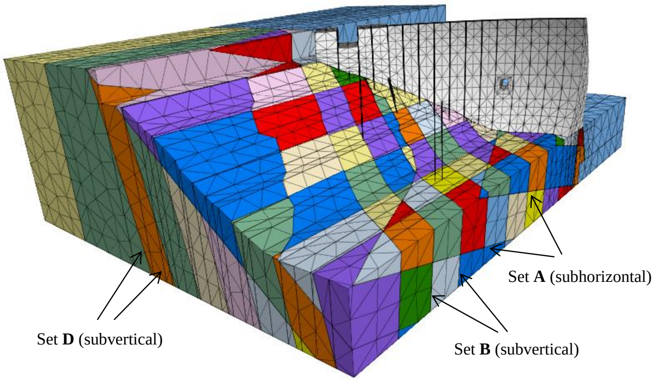


Figure 5: View of the right bank of the model, showing the finite element mesh within the rock mass blocks and the considered sets of discontinuities

The joint between the grout curtain and the rock mass, at the upstream face of the grout curtain, was extended laterally to the model boundaries, and downwards, to the block at the base of the model. Therefore, rock mass blocks located upstream from the dam stay separate from the remaining blocks. This joint marks the upstream limit of the planes which represent sets of discontinuities, which have been extended to the boundaries with the area at the base of the model and with the block lateral model, on the right bank.

It was assumed that blocks of both the dam and the rock mass have a linear elastic behaviour, with the properties shown in Table 2, and that all the discontinuities may have failure caused by either tensile or shear stress, using the Mohr-Coulomb failure criterion.

Regarding the behaviour of discontinuities, the same normal and shear stiffness was assumed in all the discontinuities, and both cohesion and shear strength were neglected in dam contraction joints and in the joints between the grout curtain and the rock mass, thus they resist only by friction. For the set of discontinuities **A**, cohesion and friction angle were assumed to be those of the discontinuities along schistosity in the green schist area, shown in Table 1. In a simplified way, for the subvertical sets of discontinuities (sets **B** and **D**), cohesion and friction angle were assumed to be those corresponding to discontinuities in the green schist area making an angle < 15° with schistosity (Table 1).

TABLE 2: ELASTIC PROPERTIES OF BOTH CONCRETE AND ROCK MASS

Material	Density (kg/m^3)	Young modulus (GPa)	Poisson's ratio
Dam	2400	20	0.2
Rock mass: green schist	2650	10	0.2
Rock mass: phyllite	2650	5	0.2
Grout curtain: green schist	2650	10	0.2
Grout curtain: phyllite	2650	5	0.2

Regarding the foundation joint, various studies have presented strength parameters determined experimentally [10], but the results are widely scattered. In this study it is assumed that the foundation joint friction angle is 45° and that cohesion is 2 MPa. The hypothesis of brittle failure was assumed for this joint, and thus, either when the normal stress reaches the tensile strength, or when it reaches the Mohr-Coulomb straight line, the model assumes cohesion and tensile strength to be zero, and then the joint resists only by friction. Table 3 shows the mechanical properties of the joints assumed in the numerical model.

TABLE 3: MECHANICAL PROPERTIES ASSUMED IN THE NUMERICAL MODEL

Discontinuities (k_n = 10 GPa/m; k_s = 0.5 k_n)*	Friction angle	Cohesion (MPa)	Tensile strength (MPa)
Joint set A	10	20	0.2
Joint sets B and D	10	10	0.2
Grout curtain/rock mass joint	2650	5	0.2
Dam/foundation joint	2650	10	0.2
Dam contraction joints	2650	5	0.2

* k_n – normal stiffness; k_s – shear stiffness

The hydraulic model was developed taking into account field data and the results of several in situ tests which allowed the main seepage paths to be identified [8, 9]. The grout curtain is modelled adjacent to the upstream edge, not underneath the dam itself, and the drainage system is simulated in a simplified way by a hypothetical continuous trench with the same depth as the drains. In reality, the grout curtain was drilled from the drainage gallery, which is located around 7 m downstream from the upstream face of the dam. However, the small shift upstream of the grouted area is an acceptable approximation considering the uncertainty regarding both the exact extent and the permeability of the grouted area.

The existence of vertical fissures at the heel of the dam is simulated by a near-surface area of higher permeability upstream from the grout curtain, in the valley bottom and at the base of each slope. Horizontal layers of higher permeability between the above-mentioned near-surface area and the drainage curtain are assumed close to the concrete/rock mass interface to simulate the main seepage paths (Figures 6 and 7). In the foundation of some of the dam blocks located in the valley bottom, the permeability of the horizontal layers between the near-surface area of higher permeability and the drainage curtain was adjusted in order to obtain numerical discharges close to average discharges recorded with the reservoir at the RWL (152 m) and the water downstream from the dam-wall at an elevation of 85.6 m (reservoir and tailwater levels recorded on the 8th January 2010). The concrete/rock mass interface and the bottom and lateral boundaries are assumed to be impervious. A zero pressure is assumed at the drains' head, which corresponds, at the bottom of the valley, to a hydraulic head of around 61.0 m along the drainage boreholes.

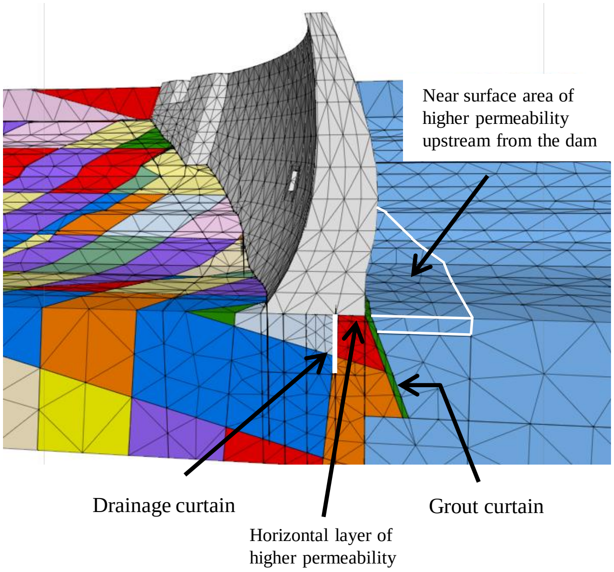


Figure 6: Global foundation model: cross section with simulated grout and drainage curtains

The model has 4184 blocks (of which 35 are dam blocks) with 100956 nodal points, and 95138 contact points where interaction between blocks occurs. The average edge lengths of the tetrahedral finite elements of the dam foundation are: i) 4 m in the blocks surrounding the drainage curtain, ii) 8 m in the vicinity of the above-mentioned blocks, including the grout curtain area, iii) 12 m in the areas close to the dam in the upstream and downstream direction, and iv) 20 m in the remaining blocks, including the block at the base of the model. The dam blocks are divided into 442 finite elements of the second degree, with 6999 nodal points, and the rock mass blocks are divided into 163122 tetrahedra elements, with 93957 nodal points.

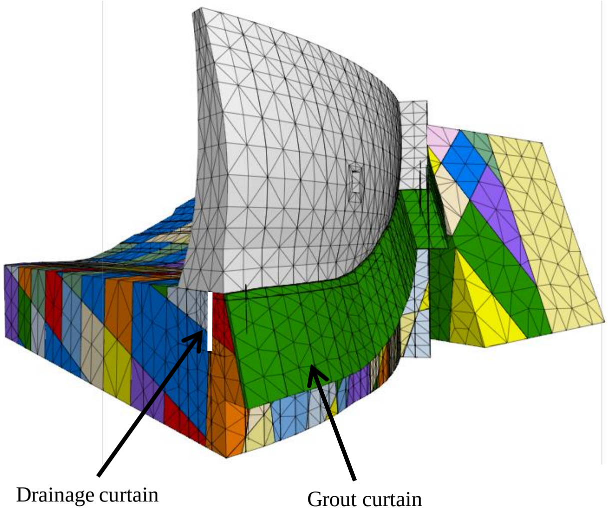
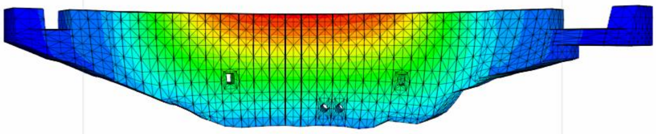


Figure 7: Detail of the global model showing the simulated grout and drainage curtains

Sequence of analysis and numerical results

The sequence of analysis included: i) calculation of in situ stresses due to the weight of the rock mass; ii) consideration of dam weight; iii) application of hydrostatic loading on the upstream face of the dam; and iv) application of water pressures on the dam/foundation interface and on the foundation discontinuities. Each one of these actions was kept constant during numerical analysis.

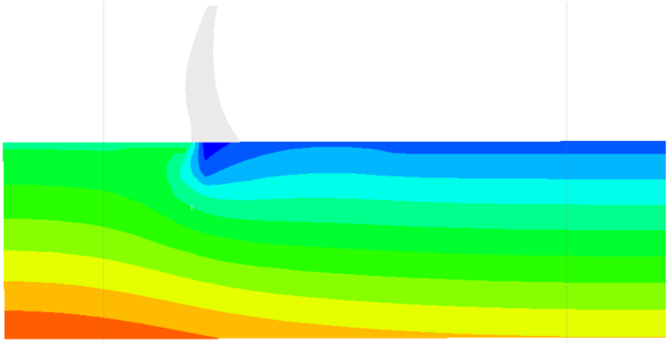
To validate the model, displacements obtained at the phases of application of the dam weight and of hydrostatic loading on the upstream face of the dam were compared with those obtained with the model which had been previously developed [3], shown in Figure 3. In both phases of analysis, the maximum displacements were around 16 % higher, due to the foundation higher deformability. Figure 8 shows the displacements at the downstream face of the dam due to the simultaneous effect of both the dead weight and hydrostatic pressure at the RWL, 94 m above the foundation level.



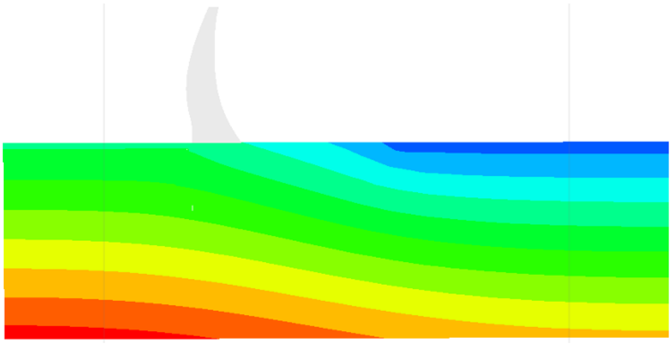
(Min. displacement = 0.3 cm; max. displacement = 4.9 cm)

Figure 8: Displacements at the downstream face of the dam due to the simultaneous effect of dead weight and water pressure at the RWL

Regarding water pressures, it was assumed first that the drainage system was operating properly and then that it was non-operational. Figure 9 shows the calculated water pressure fields in both situations, in a cross section along the valley bottom, in the upstream-downstream direction. It is observed that with a non-operational drainage system the water pressures in the valley bottom, between the arch and the downstream dam-wall, are significantly higher than when the drainage system is assumed to be operational. Indeed, a non-operational drainage system is, in this particular case, highly unfavourable for the possible sliding mechanism along the dam/foundation interface, because, as mentioned in [3], tailwater head is only assumed downstream from the dam-wall, more than 100 m away from the toe of the dam, and thus near full reservoir head is calculated along the base of the dam.



(min. water pressure = 0.0 MPa; max. water pressure = 2.2 MPa)



(min. water pressure = 0.0 MPa; max. water pressure = 2.3 MPa)

Figure 9: Hydraulic head contours for the reservoir at the retention water level, with operational and non-operational drainage systems

With the aim of assessing the safety of the dam foundation, a sequence of analysis was performed, in which the strength characteristics of the discontinuities were progressively reduced. In accordance with the Portuguese regulation, cohesion was neglected, which means that it was assumed that foundation discontinuities resist only by friction, and thus the reduction coefficient was only applied to the tangent of the friction angle.

Results showed that the safety of this dam is very high, as for very low strength parameters failure involving the simulated foundation discontinuities is never reached.

Concerning the deformation of the rock mass, results showed that for very high water pressures, when a non-operational drainage system is assumed, the highest displacements are observed in a rock wedge located at around mid-height of the right bank, immediately downstream from the dam. This rock wedge involves the subhorizontal foundation set of discontinuities simulated in the model.

The main objective of this study is to analyse the influence of different ways of modelling the water pressure field within the foundation on the numerical results. In order to highlight that influence, the strength characteristics of both the subhorizontal and subvertical discontinuities were gradually reduced to very low values, corresponding to friction angles of 5.1° in subhorizontal discontinuities (set **A**) and 8.9° in subvertical discontinuities (sets **B** and **D**). These values would never correspond to a real situation, but, as previously mentioned, the aim is to analyse the influence of water pressures on the displacements of a rock wedge.

Influence of water pressures on the displacements

Water pressure fields

The study was carried out assuming both operational and non-operational drainage systems, using the water pressures calculated with the hydraulic model, shown in Figure 9, and assuming simplified water pressure fields, defined in terms of a water table compatible with the water levels upstream and downstream from the dam, and which, downstream from the dam approximately follow the slopes of the valley, although at a slightly lower level at the top of the valley. This reduction in the water level aims to simulate the effect of foundation grouting works.

In the simplified water pressure fields it was assumed that: i) upstream from the dam, the water pressure on the foundation discontinuities corresponds to the reservoir level; ii) on the foundation discontinuity and on the rock mass discontinuities below the dam, the distribution of water pressures is bi-linear or linear, with and without drainage system, respectively; iii) in the bottom of the valley downstream from the arch the water level is 27.6 m above the minimum level of the dam/foundation interface; iv) in the direction perpendicular to the river channel, the phreatic level increases with a gradient lower than the dip of the slopes; v) close to the abutments, at the top of the valley slopes, there is a reduction of 4 m in the water level and vi) from the abutments area, the phreatic level decreases gradually towards downstream, with a gradient of -0.25 m/m. It should be noted that using these simplified water pressure fields, the pressure pattern immediately downstream from the dam is, with a non-operational drainage system, significantly different from that calculated using the hydraulic model, shown in Figure 9. This is due to the fact that the existence of the impervious substation slab between the arch and the downstream dam-wall was not taken into account.

In order to have results which allowed us to make accurate comparisons, two additional hypothetical situations were considered. The first (reference situation) corresponds to a situation with the reservoir full (at the RWL), but without water pressures in the rock mass below and downstream from the dam. This situation, which is completely unreal, would correspond to the existence of a totally impervious grout curtain, drilled to a great depth below the heel of the dam. In the second hypothetical situation it is also assumed that the reservoir is at the RWL, but it is assumed that below and downstream from the arch, to the tailwater level, the phreatic surface coincides with the ground surface level. This situation, in which the submerged weight of the rock mass blocks is considered, would correspond to a completely drained rock mass, below and downstream from the dam.

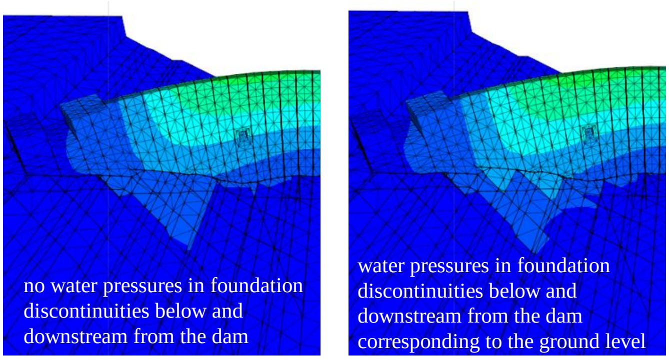
Dam and rock wedge displacements

Figures 10 to 12 show the field of displacements at the downstream face of the dam and in the rock mass immediately downstream from the dam, in the six analysed situations, for the previously mentioned very low friction angles. These values correspond to a reduction factor fr = 5.0. Figures are all at the same scale, in order to make comparisons easier. Figure analysis shows that the rock mass wedge downstream from the dam located approximately at mid-height of the right bank, immediately downstream from the dam, is visible even in the hypothetical situation of no water pressures below and downstream from the dam (Figure 10). Calculated displacements in that rock wedge are, however, significantly different in the various situations.

The maximum arch displacement is 5.3 cm in the reference situation (in which it is assumed that there are no water pressures in foundation discontinuities below and downstream from the dam). When the submerged weight of the blocks below and downstream from the dam is considered, there is a slight increase of 2.6 % in the maximum displacement. Considering the water pressure fields calculated using the hydraulic model, the maximum displacement with an operational drainage system is 15.4 % higher than the reference value, and 156 % higher with a non-operational drainage system. Assuming simplified fields of water pressures, displacements are 71.8 % and 137.6 % higher than the reference value, with operational and non-operational drainage systems respectively.

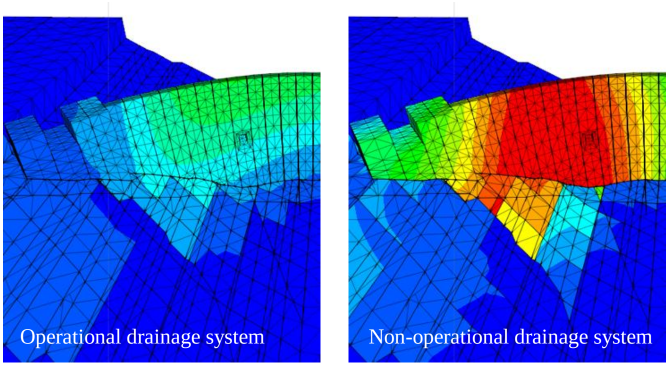
The water pressures at the base of the rock wedge, for the different analysed situations, are shown in Figures 13, 14 and 15 (the reference situation is not shown in these figures, because in this case uplift pressures at the base of the rock wedge are equal to zero). It should be highlighted, as noted before, that these water pressures remain constant during the process of reduction of the strength characteristics of the foundation discontinuities.

Rock wedge displacements, in the different situations, are shown in Figures 16, 17 and 18. The same figures show the displacement vectors. Displacements are all at the same scale, but this is not the case with displacement vectors, with which we only want to show that displacements are in the upstream-downstream direction. Table 4 shows the rock mass wedge maximum displacements and the increase in displacements in relation to the reference situation.



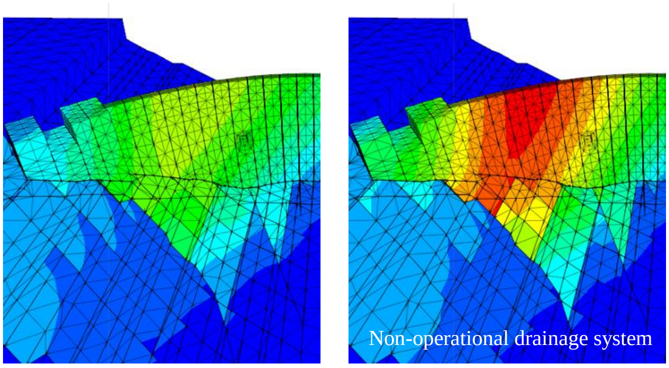
(max. displacement = 5.3 cm) (max. displacement = 5.4 cm)

Figure 10: Displacements obtained assuming two hypothetical situations



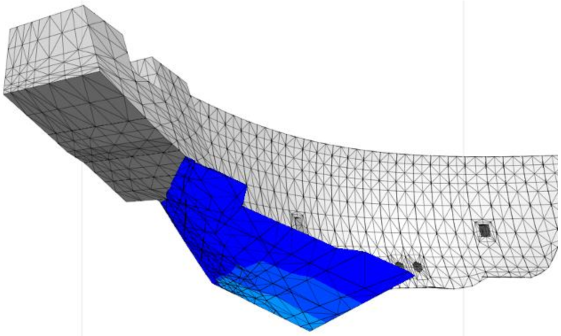
(max. displacement = 6.1 cm) (max. displacement = 13.4 cm)

Figure 11: Displacements obtained with the calculated water pressures



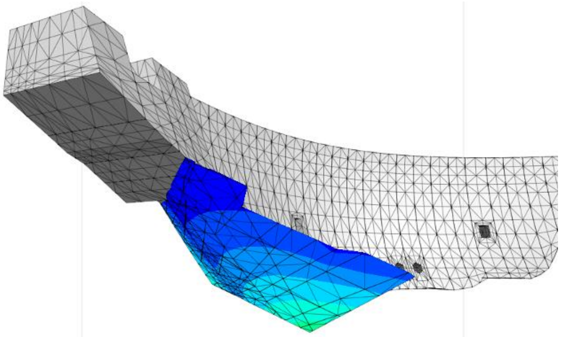
(max. displacement = 9.0 cm) (max. displacement = 12.5 cm)

Figure 12: Displacements obtained with the simplified water pressure fields

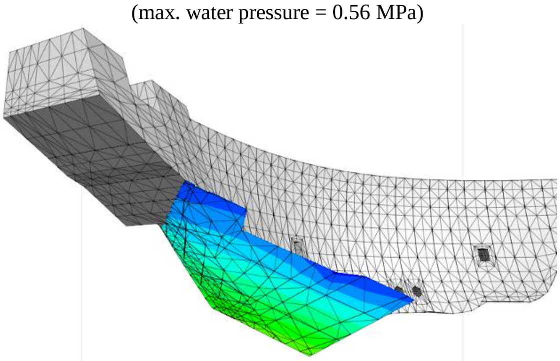


(max. water pressure = 0.26 MPa)

Figure 13: Water pressures at the base of the rock wedge, with the water downstream from the dam at the ground level

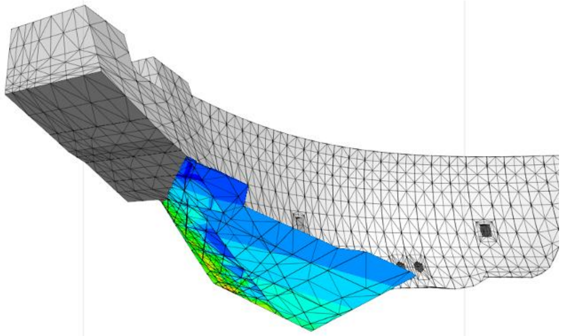


(max. water pressure = 0.56 MPa)

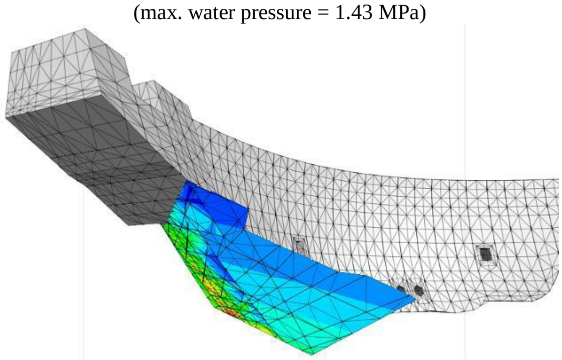


(max. water pressure = 0.90 MPa)

Figure 14: Water pressures at the base of the rock wedge, calculated using the hydraulic model, with operational and non-operational drainage system

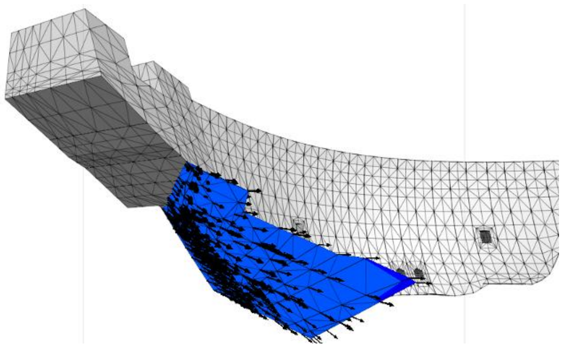


(max. water pressure = 1.43 MPa)

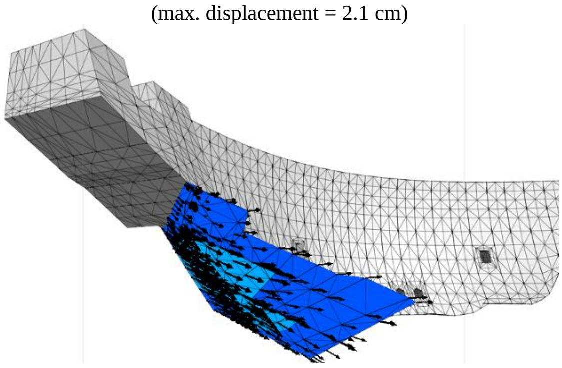


(max. water pressure = 1.43 MPa)

Figure 15: Water pressures at the base of the rock wedge obtained assuming simplified water pressure fields, with operational and non-operational drainage system

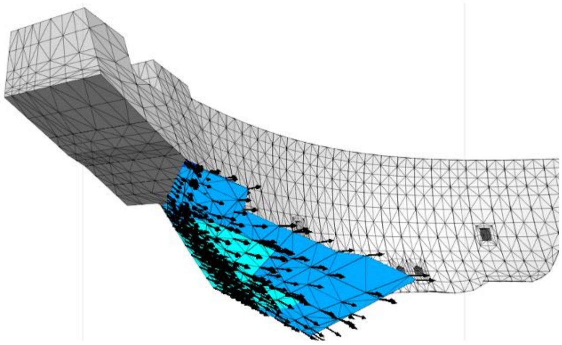


(max. displacement = 2.1 cm)

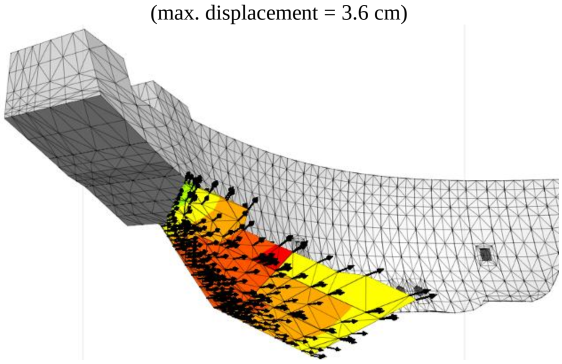


(max. displacement = 2.5 cm)

Figure 16: Displacements of the rock wedge, for the hypothetical situations of no uplift pressures (top), and water pressures below and downstream from the dam at the ground level (bottom)

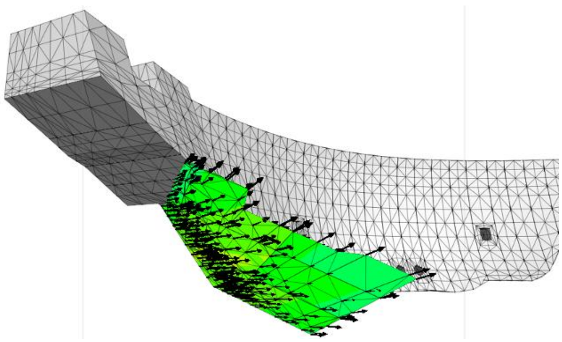


(max. displacement = 3.6 cm)

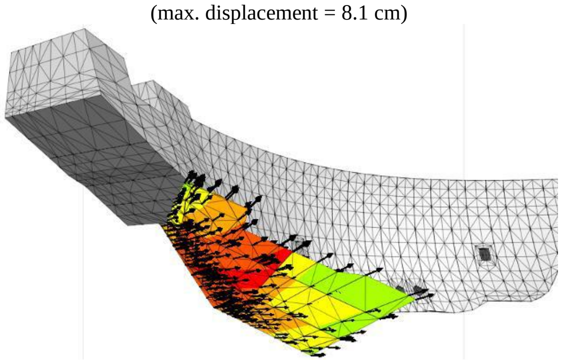


(max. displacement = 12.7 cm)

Figure 17: Displacements of the rock wedge obtained using calculated water pressures, with operational drainage system (top), and with non-operational drainage system (bottom)



(max. displacement = 8.1 cm)



(max. displacement = 12.3 cm)

Figure 18: Displacements of the rock wedge obtained with the simplified water pressure fields, with operational drainage system (top), and with non-operational drainage system (bottom)

TABLE 4: MAXIMUM ROCK WEDGE DISPLACEMENT AND INCREASE IN DISPLACEMENTS IN RELATION TO THE REFERENCE SITUATION

Analysed situation	Maximum displacement (cm)	Increase in maximum displacement
Reference	2.12	-
Tailwater at ground level	2.47	+ 16.5 %
Hydraulic model, with drainage	3.60	+ 69.9 %
Hydraulic model, without drainage	12.73	+ 499.8 %
Simplified water pressure field, with drainage	8.10	+ 281.6 %
Simplified water pressure field, without drainage	12.33	+ 480.9 %

Analysis of Figures 16, 17 and 18 and of the results presented in Table 4 shows that the rock mass wedge is relatively superficial, and thus the increase in displacements when it is assumed that the phreatic surface below and downstream from the dam is at the ground level is relatively small (+16.5 %), in relation to the reference situation.

When it is assumed that the drainage system is operating properly there is a significant difference between the maximum calculated displacement assuming the water pressures obtained using the hydraulic model and those obtained using the simplified water pressure field. In the former case, the maximum displacement is around 70 % higher than that obtained in the reference situation, while in the latter it is around 282 % higher.

For a non-operational drainage system, the highest displacements are obtained with the water pressures calculated using the hydraulic model, which are around 500 % higher than those obtained in the reference situation. Using the simplified water pressure field, displacements are around 480 % higher.

Conclusion

The foundation hydraulic and mechanical behaviour of Alqueva dam had already been studied using a 3D model, which had been calibrated taking into account monitoring results, and in which not only both the grout and drainage curtains had been simulated, but also the discontinuities corresponding to the dam/foundation interface and to the dam contraction joints [9]. The same model was used to analyse the stability of the foundation joint [3]. In this latter study analysis was carried out assuming both the uplift pressures calculated with the hydraulic analysis, based on equivalent continuum concepts, or corresponding to bi-linear or linear pressure distributions at the base of the dam, commonly used in concrete dam design. Results were coherent with those obtained by Pereira Gomes [11], who developed a physical model of both the dam and foundation of Alqueva dam in order to analyse ultimate limit states involving the dam/foundation interface due to an exceptional rise in the reservoir level, and who prepared and interpreted the test results with a numerical model and numerical simulations carried out with 3DEC.

In a previous study [3] ultimate limit states involving the dam/foundation interface were evaluated. Two different situations were analysed, with and without a drainage curtain, and comparison between the results obtained using the calculated uplift pressures and those commonly used in concrete dam design (bi-linear and linear uplift pressure distributions) led to the conclusion that they may be quite different.

In the present study, three of the main sets of foundation discontinuities were introduced into the above-mentioned model. These sets are those which could possibly create foundation blocks with the potential to slide. The possibility of failure associated with those mechanisms was then evaluated. In the considered discontinuities, mainly due to their orientation, even for very low strength values, no ultimate limit states were obtained.

The model was also used to analyse the dam displacements and the displacements in a rock wedge downstream from the dam, obtained with different water pressure patterns. It was confirmed what had previously been shown, that the difference in calculated displacements considering water pressures calculated with the hydraulic model or based on an assumed location for the phreatic surface may be significant. Therefore, it can be concluded that a set of boreholes drilled downstream from the dam in the abutments and in the valley slopes, where the water level could be measured, would be crucial to validate these models.

The study presented here shows that realistic water pressure patterns may be obtained with hydraulic analysis (taking into account the detailed studies concerning the foundation hydraulic behaviour carried out in this dam), using models that simulate both grout and drainage curtains and that take into account the different hydrologic and geotechnical conditions at the base of each dam block. The use of this models is particularly important in dams with complex hydraulic boundary conditions, for instance in dams with more than one grout or drainage curtain, with drainage galleries in the abutments or adjacent structures, like the powerhouse at the toe of Alqueva dam.

The success of this model has shown how important it is for further work to be carried out in order to continue developing the potential of this 3D modelling technique. Indeed, further work is currently under way to improve the simulation of the grout curtain and to try to simulate the progressive aperture of the dam/foundation interface, with the continuous updating of the built up water pressures. This latter model development will be particularly relevant for very high arch dams.

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