HYDROMECHANICAL ANALYSIS FOR THE SAFETY ASSESSMENT OF A GRAVITY DAM

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ABSTRACT

This paper presents a study on seepage in a gravity dam foundation carried out with a view to evaluating dam stability for the failure scenario of sliding along the dam/foundation interface. A discontinuous model of the dam foundation was developed, using the code UDEC, and a fully coupled mechanical-hydraulic analysis of the water flow through the rock mass discontinuities was carried out. The model was calibrated taking into account recorded data. Results of the coupled hydromechanical model were compared with those obtained assuming either that the joint hydraulic aperture remains constant or that the drainage system is clogged. Water pressures along the dam/foundation interface obtained with UDEC were compared with those obtained using the code DEC-DAM, specifically developed for dam analysis, which is also based on the Discrete Element Method but in which flow is modelled in a different way. Results confirm that traditional analysis methods, currently prescribed in various guidelines for dam design, may either underestimate or overestimate the value of uplift pressures. The method of strength reduction was used to estimate the stability of the dam/foundation system for different failure scenarios and the results were compared with those obtained using the simplified limit equilibrium approach. The relevance of using discontinuum models for the safety assessment of concrete dams is highlighted.

INTRODUCTION

Gravity dams resist the thrust of the reservoir water with their own weight. The flow through the foundation, in the upstream-downstream direction, gives rise to uplift forces, which, in turn, reduce the stabilizing effect of the structures' weight. Due to the great influence that uplift forces have on the overall stability of gravity dams, the distribution of water pressures along the base of the dam should be correctly recorded, in operating dams, and as accurately predicted as possible, using numerical models, at the design stage or for dams in which additional foundation treatment is required.

Stability analysis of gravity dams for scenarios of foundation failure is often based on simplified limit equilibrium procedures. Equivalent continuum models of the rock mass foundation can be employed to assess the safety of concrete dams, complemented with

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interface elements to simulate the behaviour of joints, shear zones and faults along which sliding may occur. More advanced analysis, however, is carried out with discontinuum models which simulate the hydromechanical interaction, which is particularly important in this type of structure. These models take into account not only shear displacements and apertures of the foundation discontinuities, but also water pressures within the dam foundation. Discrete element techniques, which allow the discontinuous nature of the rock mass to be properly simulated, are particularly adequate to assess the safety of concrete dams.

This study was carried out with data obtained from Pedrógão gravity dam (Figure 1), the first roller compacted concrete (RCC) dam built in Portugal, located on the River Guadiana. The dam is part of a multipurpose development designed for irrigation, energy production and water supply (Miranda and Maia 2004). It is a straight gravity dam with a maximum height of 43 m and a total length of 448 m, of which 125 m are of conventional concrete and 323 m of RCC. The dam has an uncontrolled spillway with a length of 301 m with the crest at an elevation of 84.8 m, which is the retention water level (RWL). The maximum water level (MWL) is 91.8 m. The foundation consists of granite with small to medium-sized grains and is of good quality with the exception of the areas located near two faults in the main river channel and on the right bank, where the geomechanical properties at depth are weak. The construction of the dam began in April 2004 and work was concluded in February 2006. The controlled first filling of the reservoir ended in April 2006.

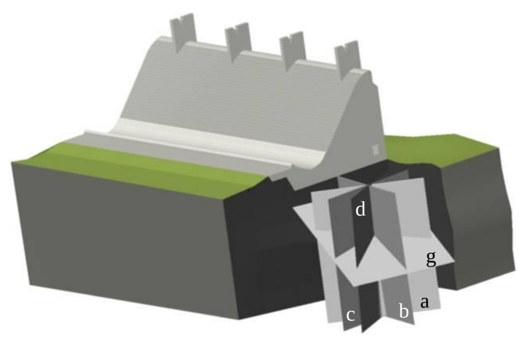


Figure 1. Pedrógão dam. Downstream view from the right side of the uncontrolled spillway and average position of the main sets of rock joints in relation to the dam.

In order to analyse seepage in some foundation areas and to interpret recorded discharges, a two-dimensional equivalent continuum model was developed, in 2006, in which the main seepage paths, identified with *in situ* tests, were represented (Farinha 2010; Farinha et al. 2007). This model allowed recorded discharges during normal operation to be accurately interpreted and thus it was used to calibrate the parameters of the discontinuous hydromechanical model of Pedrógão dam foundation presented in this paper. Analysis was carried out with the code UDEC (Itasca 2004), in which the medium is represented as an assemblage of discrete blocks and the discontinuities as boundary conditions between blocks. Water pressures along the dam/foundation interface obtained with UDEC were compared with those obtained using the code DEC-DAM, which is being developed as part of a PhD thesis currently being written by the second author, for the safety assessment of gravity dams. This code is also based on the Discrete Element Method but the flow is modelled in a different way. Results of the coupled hydromechanical model were compared with those obtained with a simple hydraulic model, in which the joint hydraulic aperture remains constant. The method of strength reduction was used to estimate the stability of the dam/foundation system for different failure scenarios, and the results were compared with those obtained using the simplified limit equilibrium approach.

HYDROMECHANICAL DISCONTINUUM MODEL

Fluid flow analysis with both UDEC and DEC-DAM

The code UDEC allows the interaction between the hydraulic and the mechanical behaviour to be studied in a fully-coupled way. Joint apertures and water pressures are updated at every timestep, as described in Lemos (1999) and in Lemos (2008). It is assumed that rock blocks are impervious and that flow takes place only through the set of interconnecting discontinuities. These are divided into a set of domains, separated by contact points. Each domain is assumed to be filled with fluid at uniform pressure and flow is governed by the pressure differential between adjacent domains. Total stresses are obtained inside the impervious blocks and effective normal stresses at the mechanical contacts.

Flow is modelled by means of the parallel plate model, and the flow rate per model unit width is thus expressed by the cubic law. The flow rate through contacts is given by:

$$q = -k_j \, a^3 \, \frac{\Delta p}{l} \qquad (1)$$

where k_j = a joint permeability factor (also called joint permeability constant), whose theoretical value is $1/(12\ \mu)$ being μ the dynamic viscosity of the fluid; a = contact hydraulic aperture; Δp = pressure differential between adjacent domains (corrected for the elevation difference); l = length assigned to the contact between the domains. The dynamic viscosity of water at 20°C is 1.002×10^{-3} N.s/m^2 and thus the joint permeability factor is 83.3 Pa^{-1}s^{-1}. The hydraulic aperture to be used in Equation 1 is given by:

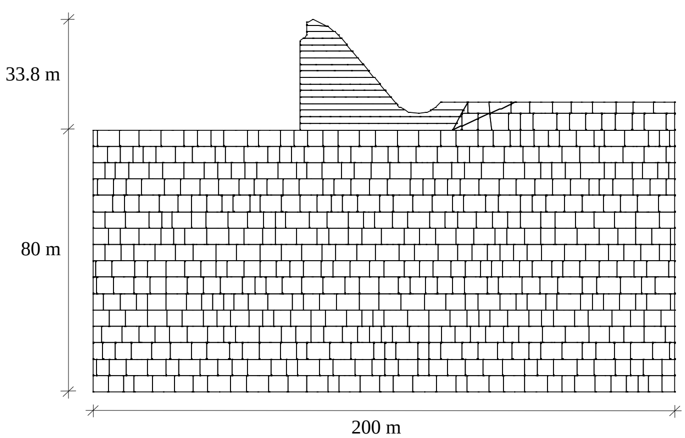
$$a = a_0 + \Delta a \qquad (2)$$

where a_0 = aperture at nominal zero normal stress and Δa = joint normal displacement taken as positive in opening. A maximum aperture, a_{max}, is assumed, and a minimum value, a_{res}, below which mechanical closure does not affect the contact permeability.

The code DEC-DAM allows both static and dynamic analysis by means of the Discrete Element Method, and has been used to investigate failure mechanisms of reinforced gravity dams (Bretas et al. 2010). In both of the above-mentioned codes, the medium is assumed to be deformable and the flow is dependent on the state of stress within the foundation. The main difference between both codes relies on the hydraulic-mechanical data model, mainly on the representation of block interaction. Regarding modelling of the hydraulic behaviour, DEC-DAM considers flow channels, where the flow rate is determined, and hydraulic nodes, where water pressures are calculated. The flow channels correspond to the mechanical face-to-face contacts, while the hydraulic nodes correspond to the sub-contacts where the mechanical interaction between blocks takes place. The main advantage of the approach used in DEC-DAM is that the mechanical actions of the water are obtained from the integration of a trapezoidal diagram of water pressures (rectangular diagrams are used in UDEC), allowing greater accuracy even when a coarse mesh is used. Both the above-mentioned codes allow the modelling of grout and drainage curtains, which is necessary in order to study seepage in concrete dam foundations.

Model description

The discontinuous model developed to analyse fluid flow through the rock mass discontinuities is shown in Figure 2. In a simplified way, only two of the five sets of discontinuities identified at the dam site were simulated: the first joint set is horizontal and continuous, with a spacing of 5.0 m, and the second set is formed by vertical cross-joints, with a spacing of 5.0 m normal to joint tracks and standard deviation from the mean of 2.0 m. The former attempts to simulate the sub-horizontal set of discontinuities g) and the latter the sub-vertical set b), both of which are shown in Figure 1. An additional rock mass joint was assumed downstream from the dam dipping 25° towards upstream, necessary to the stability analysis for failure scenarios of sliding along foundation discontinuities. The foundation model is 200.0 m wide and 80.0 m deep. The dam has the crest of the uncontrolled spillway 33.8 m above ground level and the base is 44.4 m long in the upstream-downstream direction. In concrete, a set of horizontal continuous discontinuities located 2.0 m apart was assumed to simulate dam lift joints. The UDEC model has 611 deformable blocks divided into 2766 zones, and 3451 nodal points, and the DEC-DAM model has 611 deformable blocks.



Concrete:
- unit weight = 2400 kg/m^3
- Young´s modulus = 30 GPa
- Poisson's ratio = 0.2

Foundation blocks:
- unit weight = 2650 kg/m^3
- Young´s modulus = 10 GPa
- Poisson's ratio = 0.2

Foundation discontinuities:
- k_n = 1 or 10 or 100 GPa/m
- k_s = 0.5 k_n
- φ = 30°

Figure 2. Discontinuum model of Pedrógão dam foundation and material properties.

Both dam concrete and rock mass blocks are assumed to follow elastic linear behaviour, with the properties shown in Figure 2. Discontinuities are assigned a Mohr-Coulomb constitutive model, complemented with a tensile strength criterion. In a base run, a joint normal stiffness (k_n) of 10 GPa/m, a joint shear stiffness (k_s) of 5 GPa/m, and a friction angle (φ) of 35° were assumed at the dam lift joints, at the foundation discontinuities and at the dam/foundation interface. Both at the dam lift joints and at the dam/foundation interface cohesion and tensile strength were assigned 2.0 MPa. In rock joints, cohesion and tensile strength were assumed to be zero.

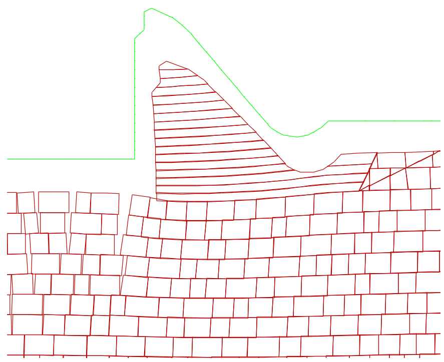


Figure 3. Block deformation (magnified 3000 times) due to dam weight, hydrostatic loading and flow.

To take into account the uncertainty in joint normal stiffness, new analysis was carried out assuming rock masses with different deformability (k_n 5 times higher and 5 times lower than that assumed in the base run). Using the following equation,

$$\frac{1}{E_{RM}} = \frac{1}{E_R} + \frac{1}{k_n\, s} \tag{3}$$

where E_R is the modulus of deformation of the rock matrix, k_n is the fracture normal stiffness, and s is fracture spacing, the rock mass in which the normal stiffness of discontinuities is assumed to be 2 GPa/m has an equivalent deformability of 5 GPa, that with k_n = 10 GPa/m an equivalent deformability of 8.33 GPa and the stiffest foundation, with k_n = 50 GPa/m, an equivalent deformability of 9.6 GPa.

Sequence of analysis

Analysis was carried out in two loading stages. Firstly, the mechanical effect of gravity loads with the reservoir empty was assessed. In the UDEC model, an in-situ state of stress with an effective stress ratio σ_H/σ_V = 0.5 was assumed in the rock mass. The water table was assumed to be at the same level as the rock mass surface upstream from the dam. Secondly, the hydrostatic loading corresponding to the full reservoir was applied to both the upstream face of the dam and reservoir bottom. Hydrostatic loading was also applied to the rock mass surface downstream from the dam. In this second loading stage, mechanical pressure was first applied, followed by hydromechanical analysis. In both stages, vertical displacements at the base of the model and horizontal displacements perpendicular to the lateral model boundaries were prevented. Regarding hydraulic boundary conditions, joint contacts along the bottom and sides of the model were assumed to have zero permeability. The drainage system was simulated assigning a hydraulic head along the drains equal to one third of the sum of the hydraulic head upstream and downstream from the dam. On the rock mass surface, the head was 33.8 m upstream from the dam, and 5.0 m downstream. Figure 3 shows a detail of dam and foundation deformation due to the simultaneous effect of dam weight, hydrostatic loading and flow.

Hydraulic parameters

The model hydraulic parameters (a_0 and a_{res}), which correspond to an equivalent permeability of the rock mass of 5.0×10^{-7} m/s, were adjusted from a two-dimensional equivalent continuum model previously developed, which had been calibrated taking into account recorded discharges (Farinha et al. 2007). It was assumed that the grout curtain was 10 times less pervious than the surrounding rock mass. The *in situ* borehole water-inflow tests performed (test procedures described in detail in Farinha et al. (2011)), led to the conclusion that the main seepage paths crossed the drains at between 3.0 and 8.0 m down from the dam/foundation interface. In order to simulate this area where the majority of the flow is concentrated, it was assumed that the horizontal discontinuity located 5.0 m below the dam/foundation interface was 8 times more pervious than the other rock mass discontinuities, in the area underneath the dam and crossing the grout curtain.

In every run, with different joint stiffnesses, the same a_{max} and a_{res} were assumed and a_0 was that which, in each analysis, led to the recorded discharge (a_0 = 0.1313 mm for k_n = 50 GPa/m, a_0 = 0.17 mm for k_n = 10 GPa/m, and a_0 = 0.4287 mm for k_n = 2 GPa/m and a_{res} = 0.05 mm). In this way, the same situation is simulated with different models, which enables comparison of water pressures and apertures along the base of the dam or along other rock mass discontinuities.

RESULTS ANALYSIS

Fluid flow analysis

Results of fluid flow analysis carried out with the UDEC model, with the reservoir at the RWL, both with constant joint hydraulic aperture and taking into account the hydromechanical interaction are shown in Figures 4 and 5. Figure 4 shows the percentage of hydraulic head contours within the dam foundation (percentage of hydraulic head is the ratio of the water head measured at a given level, expressed in metres of height of water, to the height of water in the reservoir above that level). In Figure 5, the line thickness is proportional to the flow rate in the fracture.When the coupling between stress and flow is taken into account, the loss of hydraulic head is concentrated at the grout curtain's area, below the heel of the dam, and the maximum water pressure is around 10 % higher (Figure 4 a) and b)). Without drainage, the hydraulic head decreases gradually below the base of the dam (Figure 4 c)).

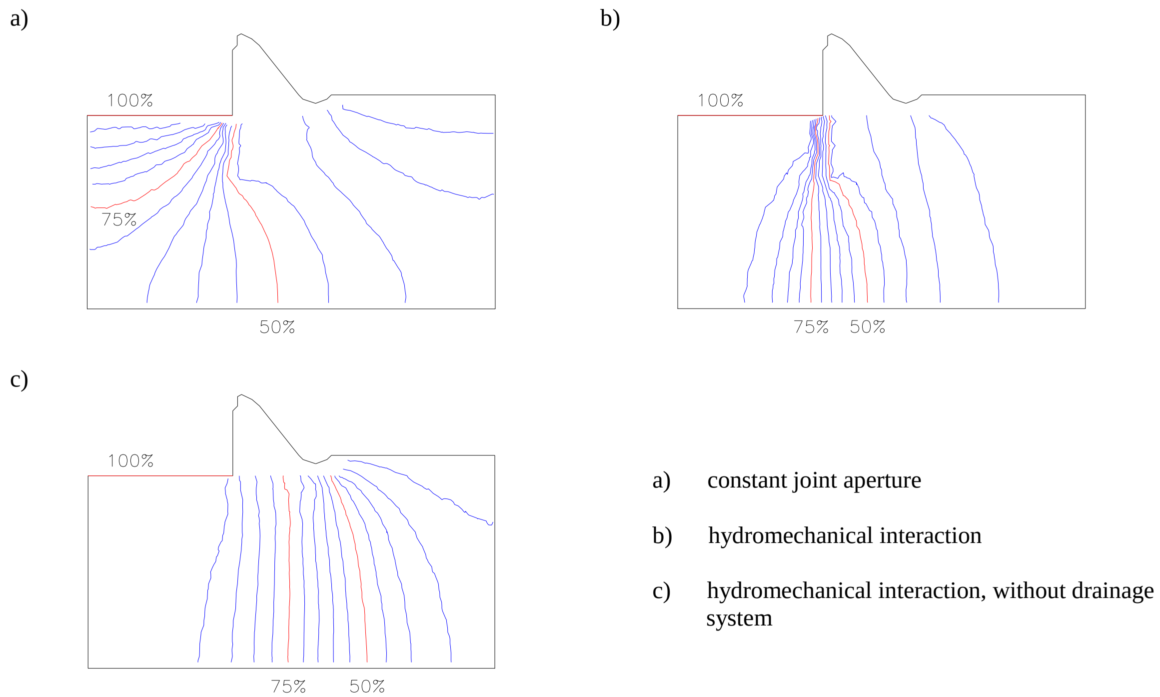


Figure 4. Percentage of hydraulic head contours for full reservoir.

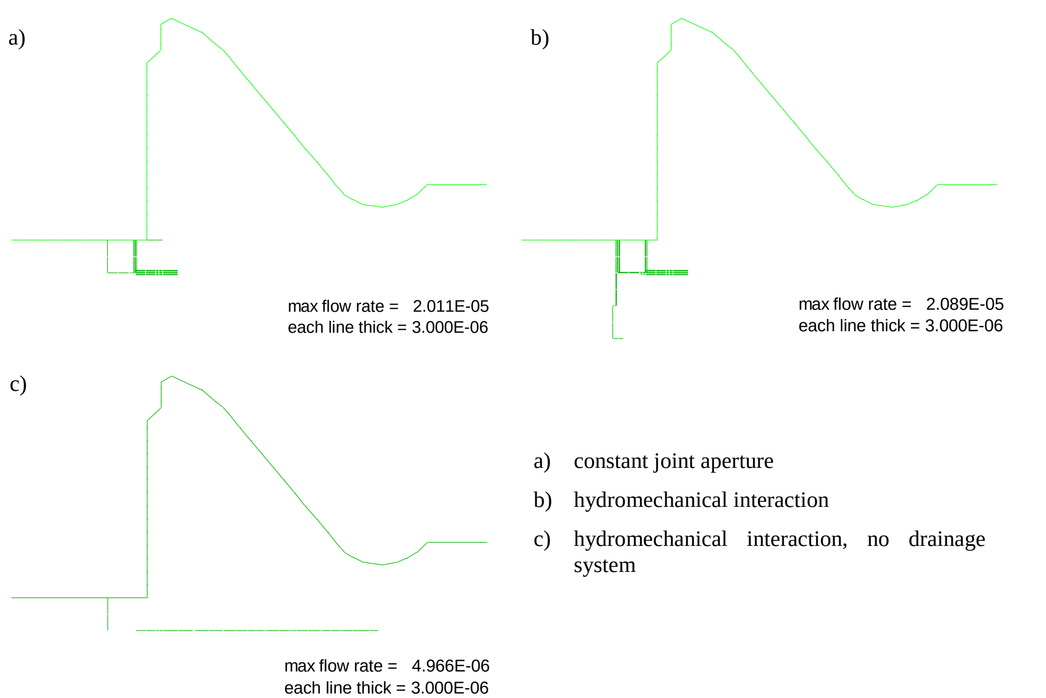


Figure 5. Flow rate for full reservoir (flow rate is proportional to line thickness; flow rates below 3.0×10^{-6} $(m^3/s)/m$ (0.18 (L/min)/m) are not represented).

Figure 5 shows that the majority of the flow is concentrated in the first two vertical joints upstream from the heel of the dam, and that this water flows towards the drain, or towards downstream in the foundation with no drainage system, along the joint of higher permeability that crosses the grout curtain, which simulates the main seepage paths. When the hydromechanical interaction is taken into account, flow rates are higher at lower levels and a higher quantity of water flows into the model through the second vertical joint upstream from the heel of the dam, rather than through the first as is the case in the run where joint aperture remains constant. This depends on the increase in water pressure in a given vertical joint, which causes the closure of adjacent vertical joints. The maximum flow rate is slightly higher when the interaction is taken into account (it varies from around 1.21 to 1.25 (L/min)/m). The quantity of water that flows through the model in the analysis with no drainage system and constant joint aperture is 0.57 (L/min)/m. This increases by around 248 %, to 1.40 (L/min)/m, in the case of the most deformable foundation, and decreases by around 26 %, to 0.42 (L/min)/m, in the case of the stiffest foundation.

Water pressures along the dam/foundation joint

The variation of water pressures along the dam/foundation joint is shown in Figure 6, along with a comparison of water pressures along the dam/foundation joint with both bi-linear and linear uplift distribution, usually used in stability analysis of dams with and without drainage systems, respectively. Results obtained with the foundations of different deformability are presented. In the hydraulic analysis in which the HM effect is not taken into account, variations in uplift pressures along both the interface and the foundation discontinuities are the same regardless of the foundation deformability, because the joint hydraulic aperture remains constant. Figure 6 shows that variations in water pressures are highly dependent on the pressure on the drainage line. Upstream from this line, water pressures are higher for more deformable foundations. Downstream from the drainage line, on the contrary, water pressures are higher for stiffer rock masses. Along the dam/foundation joint, if all the drains are clogged, the highest water pressures are obtained with the stiffest foundation, and the lowest with the most deformable rock mass.

In the case of drained foundations, the water pressure curves are close to the bi-linear distribution. In this case, computed water pressures between the heel of the dam and the drainage line are lower than those given by the bi-linear distribution, whereas between the drainage line and the toe of the dam they are higher, except for the most deformable foundation. In the case of the stiffest foundations with no drainage system, calculated uplift pressures are lower than those obtained with the linear distribution, to a distance of around 8.0 m from the heel of the dam, and downstream from this point they are considerably higher. At the dam/foundation joint end close to the toe of the dam, UDEC water pressures are higher than those assumed with the linear distribution of pressures, due to the presence of the rock wedge downstream from the dam. For the most deformable foundation, the linear distribution of uplift pressures greatly overestimates pressures along the base of the dam, with the exception of an area with a length of around 6.0 m, close to the toe of the dam.

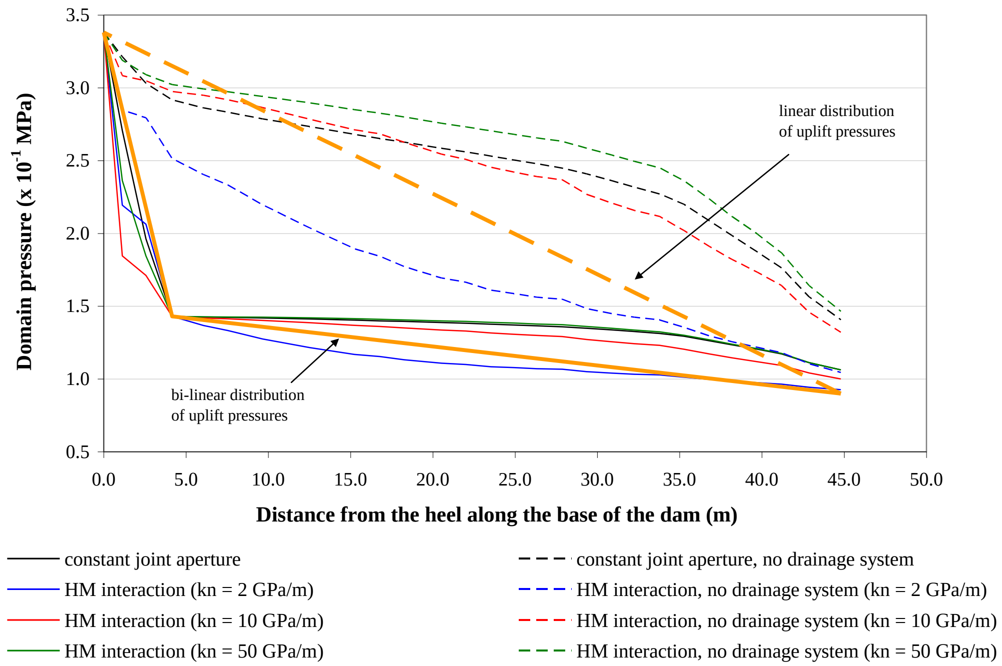


Figure 6. Water pressure along the dam/foundation joint and comparison with both bi-linear and linear distribution of water pressures.

Figure 7 shows the comparison between water pressures along the dam/foundation interface calculated with both UDEC and DEC-DAM, for the case of joint normal stiffness (k_n) of 10 GPa/m and of both operational and non-operational drainage systems. In the former case, there is an overall good match between the curves, except in the vicinity of the drain due to the small difference in the location assumed in the numerical representation.

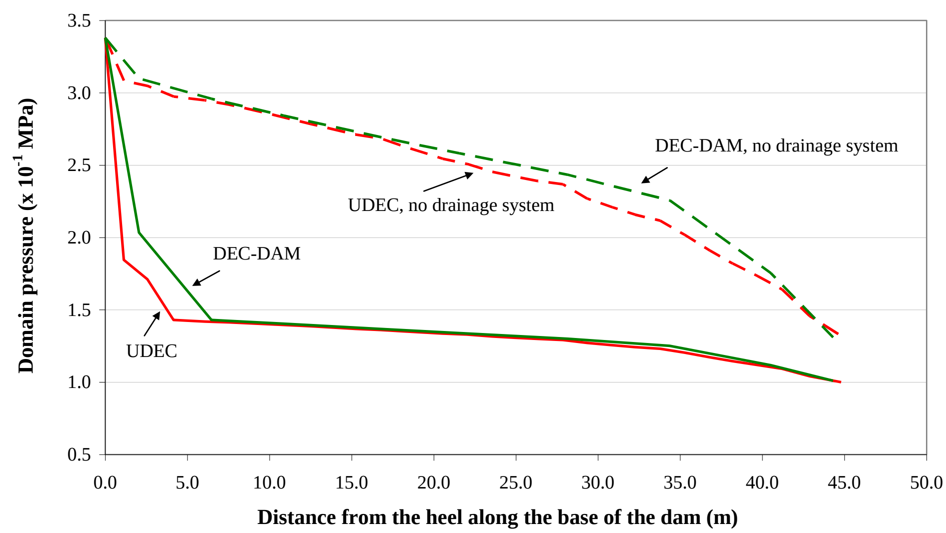
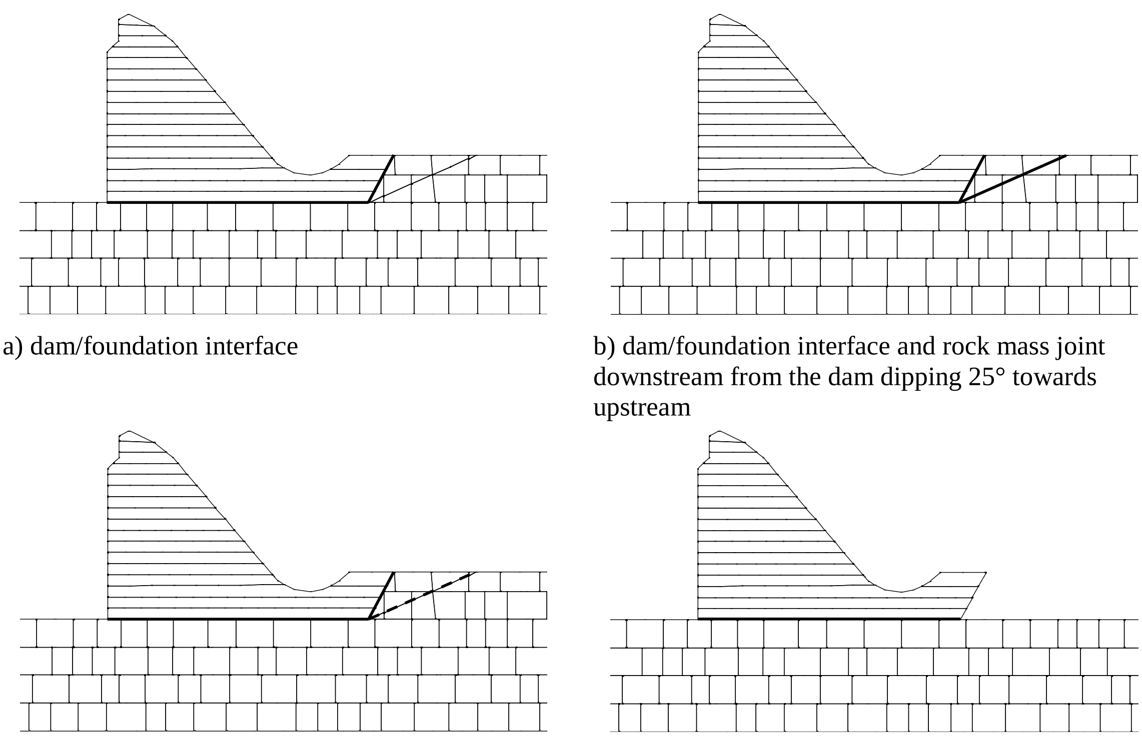


Figure 7. Water pressure along the dam/foundation joint, calculated with both UDEC and DEC-DAM.

STABILITY ANALYSIS

Strength reduction method

The UDEC model developed, with joint normal stiffness of 10 GPa/m, was used to assess the stability of the dam/foundation system for the four different possible sliding failure scenarios shown in Figure 8. Scenarios a) and d) concern only the dam/foundation joint. Sliding along this interface is the most probable failure scenario in dam foundation rock masses containing widely spaced discontinuities, none of which are unfavourably oriented. Pedrógão dam is embedded in the foundation, and therefore the resistance to sliding is high. Scenario d) neglects the resistance of the rock wedge at the toe of the dam, in order to take into account a possible excavation downstream, close to the toe of the dam. Scenario b) involves both the dam/foundation joint and the rock mass joint dipping 25° towards upstream, which was purposely included in the model for stability analysis. This hypothetical situation may simulate a combined mode of failure, where the failure path occurs both along the dam/foundation interface and through intact rock, in geology where the rock is horizontally or near horizontally bedded and the intact rock is weak (USACE 1994). In scenario c), sliding along the inclined rock mass joint is prevented, assuming that the behaviour of this joint is elastic.



a) dam/foundation interface

b) dam/foundation interface and rock mass joint downstream from the dam dipping 25° towards upstream

c) dam/foundation interface, preventing slip on the rock mass joint downstream from the dam dipping 25° towards upstream

d) dam/foundation interface, neglecting the resistance of the rock wedge at the toe of the dam

Figure 8. Analysed failure modes.

Analysis was carried out with the method of strength reduction, typically applied in foundation design. An initial friction angle of 35° was assigned to the rock mass discontinuities, dam foundation interface and dam lift joints, and zero cohesion and zero tensile strength were assigned to the dam/foundation joint, involved in the failure modes. The model was first run until equilibrium, then the fluid flow analysis was switched off and, from this step, water pressures were kept constant. For each failure scenario, the friction angle of the discontinuities highlighted in Figure 8 was gradually reduced until failure (the reduction coefficient was applied to tan φ). The failure indicator was the horizontal crest displacement. Analysis was carried out assuming that the reservoir was at the RWL or at the MWL, and that the drainage system was either operational or non-operational. Stability analysis results are shown in Figure 9 and in Table 1. In Figure 9, friction angles in the x-axis are shown in reverse order, for ease of analysis.

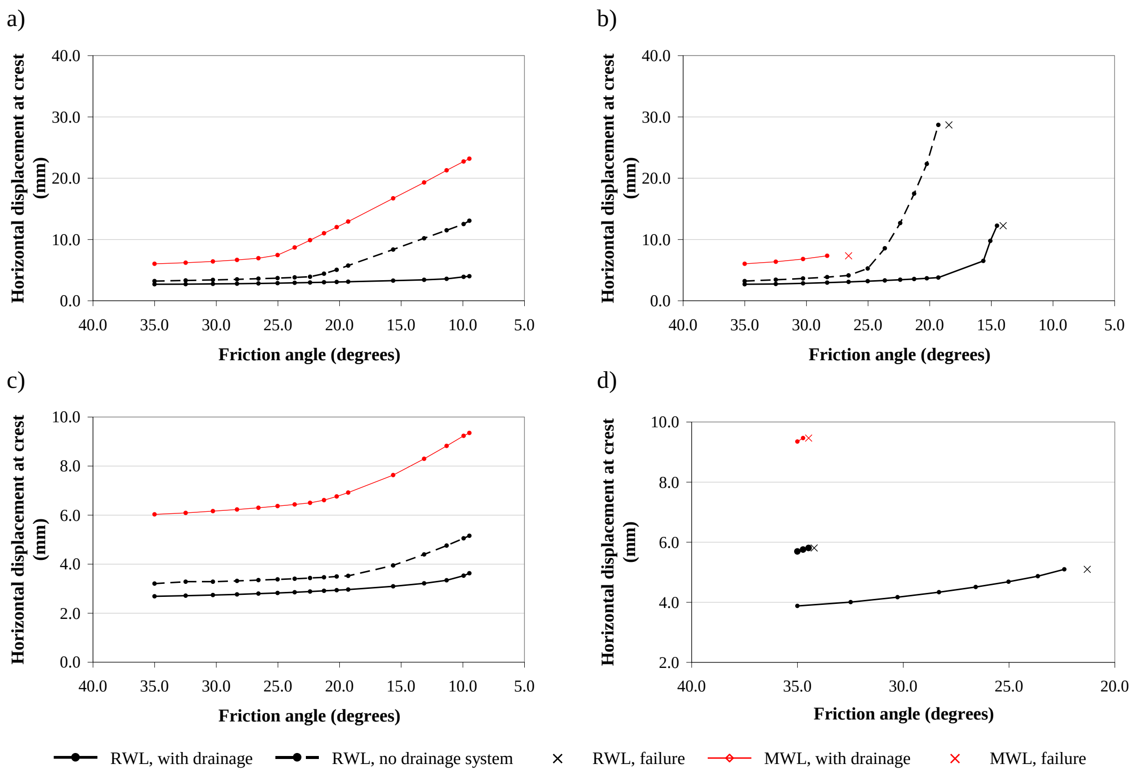


Figure 9. Variation in crest horizontal displacement due to reduction of the friction angle on highlighted joints, for the failures modes shown in Figure 7.

In the four analysed failure modes, the dam foundation system is unstable when the reservoir is at the MWL and the drainage system is non-operational, and therefore, these situations are not shown in Figure 9. For the same reservoir level, in both scenarios a) and c) the dam/foundation system remains stable when the drainage system is working properly, while in scenario b), as shown in Figure 9, failure occurs for a friction angle of around 27.5° (safety factor F = 1.4). In scenario d) the dam is unstable for friction angles lower than 34.5° when the reservoir is at the MWL (F = 1.01).

Table 1. Comparison of friction angles for which failure occurs calculated with the hydromechanical model and with the limit equilibrium method.

$H_{upstream.}$ (m)	$H_{downstream}$ (m)	Drainage system	River bottom downstream from the dam *	Friction angle		
				Limit equilibrium **	UDEC	
					failure	last stable
84.8	60.0	not operative	1)	27.8°	34.2°	34.5°
(RWL)			2)	11.1° - 22.6°	18.4°	19.3°
		operative	1)	21.2°	21.3°	22.4°
			2)	8.2° - 17.1°	14.0°	14.5°
91.8	67.8	not operative	1)	45.6°	unstable	
(MWE)			2)	27.8° - 40.6°	unstable	
		operative	1)	32.4°	34.5°	34.7°
			2)	18.2° - 28.1°	26.6°	28.3°

* Downstream from the dam the river bottom is: 1) at the same level as the dam/foundation interface (51.0 m) – scenario d)
2) at its actual level (59.5 m) – scenario b)

** For failure scenario b), results are shown considering full passive force or only 1/3 of the passive force

Comparison of the UDEC results with those obtained using the limit equilibrium method

Table 1 shows the comparison between the UDEC results and those from the equilibrium method, for failure modes b) and d). In the analysis in which the stabilizing effect of the rock wedge downstream from the dam is taken into account, the study was done assuming either full development of passive pressure, which is improbable as it requires large structure displacements, or the development of one-third of the passive pressure, which is more realistic. Results show that the dam is stable when the reservoir is at the RWL, even when the drainage system is inoperative. When the reservoir is at the MWL, the safety factor is lower than 1 when: i) the drainage system is inoperative and the resistance from the rock wedge downstream from the dam is neglected (F = 0.69); and ii) the drainage system is inoperative and only one third of the passive force is considered in the analysis (F = 0.82).

Failure mode d) is the only one which enables UDEC analysis to be verified, as the same results must be obtained for similar loads with both the UDEC and limit equilibrium analysis. Indeed, when the reservoir is at the RWL and the drainage system is operative almost the same friction angles were obtained (21.2° in the limit equilibrium analysis and between 21.3° and 22.39° in the UDEC analysis). A difference as low as around 2° is obtained in similar conditions, but with the reservoir at the MWL (32.4° in the limit equilibrium analysis and between 34.47° and 34.73° in the UDEC analysis). However, when the drainage system is inoperative, the friction angles obtained in the UDEC analysis (34.21° - 34.47°) are higher than that given by the limit equilibrium method (27.8°). This difference can be explained by the higher uplift pressures obtained in the UDEC analysis, when compared with those given by the linear distribution of water pressures between the reservoir and the tailwater, assumed in the limit equilibrium analysis. This difference in water pressures is shown in Figure 10. A limit equilibrium analysis carried out assuming a resultant of the uplift pressure 24 % higher than that given by the linear distribution of water pressures would lead to the same friction angle at failure as the UDEC analysis (assuming that in the UDEC model failure occurs for a friction angle of 34.3°).

In the analysis in which it is assumed that downstream from the dam the reservoir is at its actual level, the UDEC results are within the range of friction angles given by the limit equilibrium method, when only part or full passive force is considered, but are closer to those obtained for one third passive force.

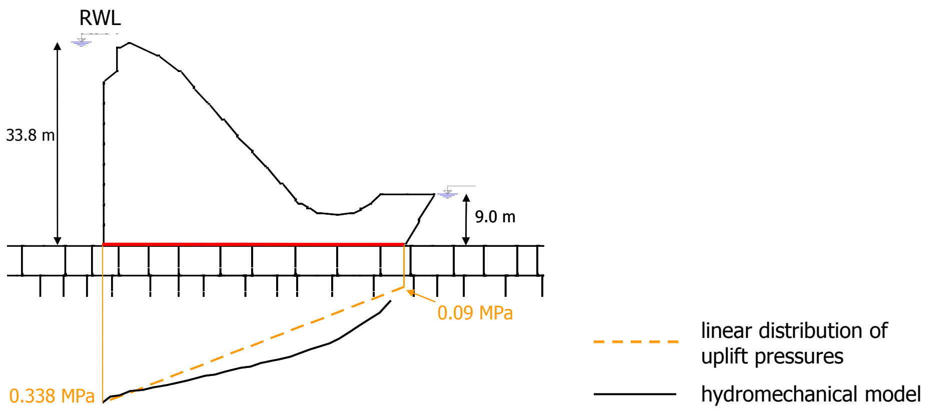


Figure 10. Comparison between the UDEC results and those from the limit equilibrium method.

CONCLUSION

This paper presents a study on seepage in Pedrógão dam foundation using a discontinuum model, which was developed taking into account recorded data and information provided from tests carried out *in situ*. Analysis of seepage was done using both UDEC and DEC-DAM codes, which take into account the coupled hydromechanical behaviour of rock masses. Stability analyses were carried out for different failure scenarios and with different assumptions about uplift pressures and joint shear strength. Some of the analyzed scenarios are highly unfavourable hypothetical situations, as in this dam the resistance to sliding is high. Results allowed us to quantify the influence of water pressures on the stability of the dam. This result draws attention to the importance of using recorded water pressures for the sliding safety assessment of existing dams, as recommended by the European Club of ICOLD (2004).

The uplift water pressure along the dam base is always of concern to the stability of concrete dams and is usually prescribed in design codes assuming a bi-linear uplift distribution to account for the relief drains. The study presented here shows that results depend mainly on the joint normal stiffness and on joint aperture. The comparison between the results obtained with the codes UDEC and DEC-DAM showed that there is a good match between water pressures calculated along the dam foundation joint, with both operational and non-operational drainage systems.

Discontinuum models are difficult to apply in most practical cases, because jointing patterns are very complex and there is usually a lack of data on hydraulic properties of the discontinuity sets. Among these parameters are the orientation and spacing of discontinuities, and the hydromechanical characterization data, namely joint normal stiffness, joint apertures and residual aperture, which is not readily available. However, such models which simulate the hydromechanical interaction are relevant in stability analysis, and the uncertainty in the different parameters, can be overcome by performing stability analysis assuming that each parameter may vary within a credible range.

Flow in fractured rock masses is mainly three-dimensional. However, in dam foundations the flow is mainly in the upstream-downstream direction, and therefore 2D analysis may be considered adequate in most cases. For arch dams, 3D analysis is necessary, but coupled fracture flow modelling of an arch dam foundation would imply representing a network of joints from various sets, which would be computationally prohibitive. The alternative is to use 3D mechanical models, in which only the discontinuities involved in possible failure modes are represented, and the water pressures are obtained from 3D equivalent continuum models.

In dam stability evaluation, the main advantage of using a 2D hydromechanical discontinuum code instead of the limit equilibrium method is that it allows the study of a wider range of failure modes. In addition, this type of code enables displacements to be calculated in seismic analysis, in contrast to what happens with the limit equilibrium approach. This type of analysis is particularly useful when the foundation contains more than one material or is made up of a combination of intact rock, jointed rock and sheared rock, as, in these cases, the overall strength of the foundation depends on the stress-strain characteristics and compatibility of the various materials. It is also relevant in those cases in which controls of maximum displacement, needed to ensure proper function and safety, may prevail over safety factor requirements. In 3D, discontinuum models are particularly adequate for scenarios of foundation failure, as limiting equilibrium procedures, like those proposed by Londe (1973)**,** make basic assumptions about the forces acting on the independent volumes of rock that may become kinematically unstable, and are thus much simplified.

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