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SAFETY EVALUATION OF CONCRETE GRAVITY DAMS SLIDING CONSIDERING THE VARIABILITY OF ROCK MASS FOUNDATION HYDRAULIC AND MECHANICAL PROPERTIES

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Abstract. *In many countries, including Portugal, the design of concrete dams is based on the use of a global safety factor. Adopting a probabilistic safety approach, the Eurocodes are based on the principles of the limit states design in conjunction with the partial safety factors method. Thus, to develop a compatible approach for the design of concrete dams, it is important to understand the contribution of the uncertainty of the hydraulic and mechanical rock mass properties to the safety of concrete gravity dams for the design loads, including high intensity seismic actions. In this work, a set of Portuguese gravity dams are assessed in a reliability framework, combining both design data and data collected in the service lifetime. Geometrical data, as well as the dead-weight action, the hydrostatic pressure and the maximum design earthquake were considered as deterministic variables. The coefficient of friction and the uplift, quantified by means of a factor under the drainage line, were considered as probabilistic variables. Using the Monte Carlo method, the frequency of violating the safety criteria under extreme seismic actions was evaluated for six dams. An annual probability of failure of 10^{-5} was obtained for a high intensity seismic occurrence, which is the value defined in the national appendix of Eurocode 8 for a return period of 1000 years.*

1 INTRODUCTION

Portuguese concrete dams are designed based in the national guidelines on dam safety[1,2]. Following the proposal of ICOLD, these guidelines are based, regarding to safety evaluation, on the global safety factor method. However, most other civil infrastructures are evaluated using the partial safety factors method, established in the Eurocodes[3]. Considering the new demands on structural safety, studies on probabilistic approach to dams safety have been developed in the last few years[4 5].

For concrete dams design, some loads are well-known (eg, dead-weight actions, hydrostatic pressure and thermal variations). On the other hand, the uplift pressures, seismic actions and the foundation resistance are evaluated with large uncertainty. In the last decades, studies on hydromechanical behavior of concrete dams foundations have been carried out, but the knowledge on physical uncertainty of actions which are associated with the seepage is

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still scarce. In this paper, the contribution of the uncertainty in the uplift and concrete-rock interface resistance to the safety conditional on occurrence of a strong earthquake is studied. The proposed methodology is applied to the analysis of six Portuguese gravity dams.

2 EVALUATION OF GRAVITY DAM SAFETY

2.1 General considerations

Due to their dimensions, internal stresses in gravity dams are, in general, much smaller than the concrete strength. The applied loads, including water pressure and seismic actions, tend to destabilize the dam as a rigid-body while the dead-weight is a stabilizing force. The two main modes of failure, for gravity concrete dams, are sliding and overturning. Typically, the critical safety verification is related to sliding of the dam. In this work, only this case is considered.

The Portuguese design codes establish that it should be guaranteed that the resistance actions (stabilizing loads) have to be greater than the driving forces (destabilizing loads):

$$F_R \geq F_A \Leftrightarrow \frac{F_R}{F_A} \geq 1 \Leftrightarrow FS \geq 1 \tag{1}$$

where FS is the safety factor. In the design of gravity dams, three sets of actions must be considered (Figure 1): *i)* gravitic actions, given by the material dead-weight, represented by its resultant W; *ii)* water actions, namely the hydrostatic pressure on the upstream and downstream faces (with resultants I_w and I_j, respectively) and the uplift pressure on the concrete-rock interface or along critical surfaces in foundation (with resultant U); and *iii)* seismic actions, that can be considered, approximately, as equivalent inertia forces applied in the dams' center of mass (with horizontal and vertical resultants F_{sh} and F_{sv}, respectively) and the hydrodynamic action on the upstream face (with resultant I_{ws}).

The resistance force results from the contribution of friction and cohesion along natural discontinuities in foundation (with resultant R) and on a passive resistance on the downstream face (with resultant I_p).

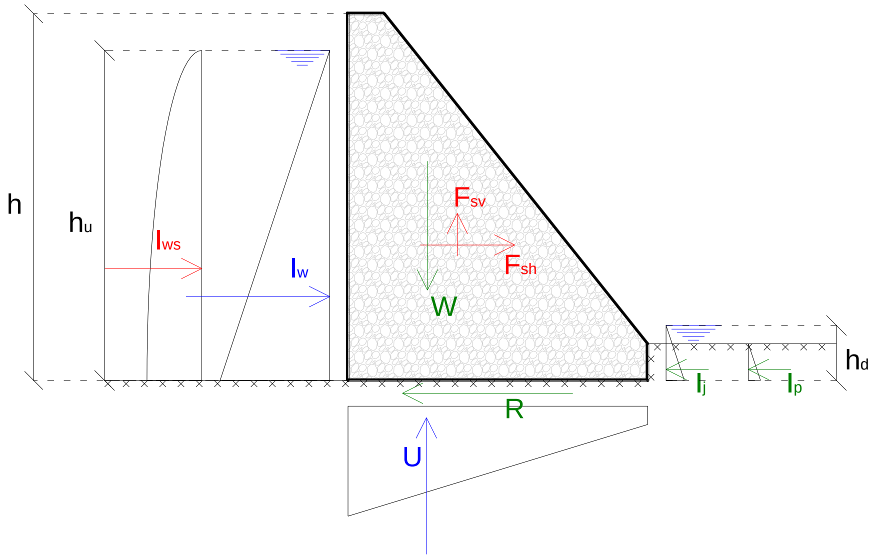


Figure 1. Destabilizing and stabilizing forces involved in sliding safety evaluation.

In the sliding failure mode, the dam slides above the concrete-rock interface or along natural rock discontinuities near the dam base. The real sliding surface is extremely difficult to predict. Thus, it has been usual to consider the worst scenario, the sliding along natural horizontal or subhorizontal discontinuities in foundation. The stability condition that describes the sliding failure mode is given by:

$$R + I_j + I_p = I_w + F_{sh} + I_{ws} \Leftrightarrow (R + I_j + I_p) - (I_w + F_{sh} + I_{ws}) = 0 \qquad (2)$$

Equivalent to the concept of ultimate and serviceability limit states, the Portuguese codes[1 2] establish that one should evaluate the dam safety and functionality, respectively, related to failure and current scenarios. In this paper, the probability of failure is evaluated only for the failure scenario. According to Eurocode 0[3], the definition of ultimate limit state is related to the occurrence of extreme events. In the Portuguese codes[1], and using the French provisory codes' terminology[4], the dam safety should be verified for the extreme hydrostatic loads combination (occurrence of a flood) and for the accidental seismic loads combination (occurrence of a high intensity seismic action). In the Portuguese dams design codes, the worst scenario is the accidental seismic loads combination that combines an occurrence of a high intensity seismic action and a normal water level in the reservoir.

2.2 Uplift

The seepage through foundation induces hydraulic pressures in the discontinuities surfaces. In the concrete-rock interface, these pressures reduce the effective stresses on the surface and, consequently, the shear resistance.

Concrete dam foundation rock mass usually needs treatments to satisfy the requirements of stability, deformation and permeability[6]. These treatments are: i) consolidating grouting, which increases the shear strength and deformability of the uppermost strata of the foundation rock mass, known as consolidation; ii) the grout curtain close to the upstream face of the dam, which creates a solid barrier, reducing the seepage under the dam; and iii) the drainage curtain drilled downstream from the grout curtain, which controls the seepage under the dam and reduce the uplift. Hydraulic piezometers, located downstream of the drainage curtain, are used to evaluated the efficiency of the grout curtain and the uplift mitigation measures. The Portuguese code[2], as well as other international codes[7 8], establish that the drainage curtain have to reduce the difference between the hydraulic head of upstream and downstream (Figure 2) to one third ($\alpha = 1/3$).

Through the continuous observation of Portuguese dams, it has been verified that the average pressure values downstream of the drainage curtain, present significant scatter. Though these pressures are in general smaller than those prescribed, in few cases the observed pressures, globally or punctually in foundation, are greater than the target value.

Using the gathered data from the Portuguese gravity, arch or arch-gravity large dams, the uncertainty in the efficiency of drainage curtains was modeled. This model considers four random variables, which intend to represent different phenomenon that influence the uplift pressures. The uplift quantifier factor is given by,

$$k_w(t, \Delta H) = k_{w0} \cdot e^{p \cdot t} + m \cdot \Delta H \qquad (3)$$

where k_{w0} is an initial value of the uplift quantifier factor under the drainage line, related to the upstream headwater at the crest level ($\Delta H=0$) at the beginning of dam exploitation ($t=0$), which depends on foundation rock mass properties and treatment measures undertaken; p characterizes the time dependent changes and depends on the deformations of foundation discontinuities and possible drainage's efficiency loss caused by calcareous deposits or filling with grout; and *m* represents the influence that the reservoir headwater level variation, related to the crest level (ΔH), has on kw. This influence was assumed to be linear.

Expression (3) was adjusted to pressure data of piezometers from sixteen dams. Due to the variation of the adjust quality, it was admitted that other factors might influence the drainage system efficiency. That uncertainty, hard to quantify, was considered through a fourth random variable which represents the standard deviation of data in relation to the mean value given by expression (3).

The observations were divided into two groups, representing wide discontinuity spacing or slightly fractured rock mass and close discontinuity spacing or highly fractured rock mass. A probability distribution function was adjusted to each random variable.

Figure 2 shows the general uplift diagram on the concrete-rock interface. The uplift quantifier factor, k_w, varies between 0 and 1. In the worst case scenario, the diagram would be linear, corresponding to zero efficiency of the drainage system. To take that into account, α parameter was admitted to quantify uplift pressures.

The α parameter might be related with k_w by,

$$k_w = \frac{\alpha}{\alpha_{max}} \Leftrightarrow \alpha = k_w \times \alpha_{max} \tag{4}$$

where α_{max} depends on the drainage curtain localization,

$$\alpha_{max} = 1 - \frac{L_{dr}}{L} \tag{5}$$

The value of α_{max} tends to one when the profile base length (L) is much greater than the distance of the upstream face to the drainage curtain (L_{dr}). Generally, for gravity dams, α_{max} varies between 0.8 and 0.9.

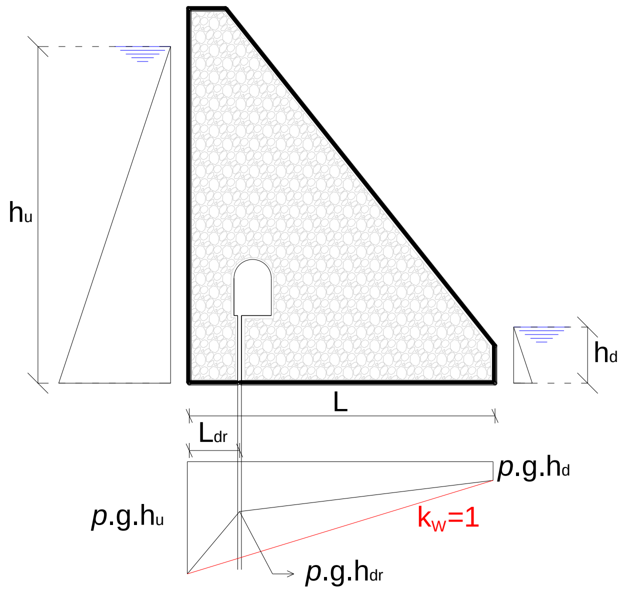


Figure 2. General uplift diagram on the concrete-rock interface.

2.3 Shear strength

According to the Portuguese codes[2], in the eminence of failure, the contribution of cohesion is null and the shear strength coincides with the frictional resistance. In those conditions, the discontinuity shear resistance is given by the Mohr-Coulomb criteria[9 10],

$$\tau_n = -\sigma_n' \times tan\,\varphi \tag{6}$$

where τ_n is the shear stress, σ_n' is the effective normal stress and φ is the interface friction angle.

The uncertainty of rock mass proprieties is, generally, substantial. For a good quality rock mass, as those usually present in the concrete dams foundation, coefficients of friction around one $(\varphi = 45°)$ are usually admitted. According to the results of tests made in the National Laboratory of Civil Engineering (LNEC), a coefficient of variation of 15% of the coefficient of friction should be considered[9]. Admitting that a unitary coefficient of friction corresponds to a characteristic value (95% of probability of being achieved) and that the Eurocode 0[3] establishes a lognormal distribution for the materials resistance parameters, the distribution of the coefficient of friction is characterized by a mean value of 1.28 ($\mu_\varphi = 52°$) and a standard deviation of 0.192 ($\sigma_\varphi = 10.87°$).

3. SLIDING SAFETY ANALYSIS

3.1 General considerations

According to the failure scenario evaluation criteria established in the Portuguese code[2], the global safety factor is given by,

$$FS = \frac{(P - U - F_{sv}) \times tan\,\varphi / \gamma_\varphi + I_j + I_{pd}}{(I_w + F_{sh} + I_{ws})} \tag{7}$$

where the coefficient of friction $tan\,\varphi$ is affected by a partial safety coefficient, γ_φ, of 1,2. The passive resistance is usually reduced by a factor of safety of about 3 ($I_{pd} = I_p / 3$).

3.2 Safety probabilistic approach

For the failure scenarios, the limit state expression that represents the sliding safety evaluation is given by,

$$g = (P - U - F_{sv}) \times tan\,\varphi + I_j + I_{pd} - (I_w + F_{sh} + I_{ws}) \tag{8}$$

Though seismic action effects have large uncertainty and the Eurocode 8[11] has sufficient information to consider it as a probabilistic variable, it was considered herein as a deterministic action. Thus, this probabilistic approach represents a conditional analysis subject to the occurrence of accidental events and allows the evaluation of the contribution of the uplift and concrete-rock interface resistance uncertainty on safety. According to the Portuguese codes[1 2], a high intensity seismic action corresponds to a thousand-year return period's acceleration, defined in the national appendix of Eurocode 8[11], should be considering in designing and assessing of medium-height gravity dams.

Using the Bayes theorem, the annual probability of failure is given by,

$$p_f = p(G \le 0 \mid S) \times p(S) \tag{9}$$

where $p(G\leq 0 \mid S)$ is the probability of failure subjected to the occurrence of a seismic action for the reference period (100 years) and $p(S)$ is the annual probability of seismic action occurrence given by the inverse of the return period[11]. For a thousand-year return period, the probability of a seismic action occurrence is 10^{-3}.

3.3 Monte Carlo method

Following a probabilistic approach, the probability of failure described in the expression (8) is evaluated by the integral of the joint probability density function of the random variables in the unsafe region:

$$p_f = \int_{g(X)\leq 0} f_X(x)\,\partial x \tag{10}$$

where $f_X(x)$ represents the joint probability density function.

In the most cases, an analytical solution of the expression (10) is impossible. Simulation methods are simple procedures to compute this probability, providing the exact solution when the number of simulations tends to infinite[12 13].

All simulation techniques have origin in the Monte Carlo method[14]. In practical problems, crude Monte Carlo method requires a large number of simulations. In this paper, a conditional probability of failure is performed. Thus, the number of simulations required made it possible to use this method.

The Monte Carlo method consists in re-writing the probability integral in expression (10) as,

$$p_f = \int_{g(X)\leq 0} f_X(x)\,\partial x = \int_X I\cdot f_X(x)\,\partial x \tag{11}$$

where I is an indicator function equal to 1 if $g(x)\leq 0$ and equal to zero otherwise.

Therefore, generating a sufficient number of random samples and performing the expression (8), the probability of failure is given by,

$$p_f = \frac{\sum I}{N} \Leftrightarrow p_f = \frac{n_f}{N} \tag{12}$$

where from all N simulations performed, n_f is the number of simulations which resulted in a negative value of the limit state expression. According to Lemaire[15], to evaluate a probability of failure of 10^{-n}, it is necessary to carry out between 10^{n+2} and 10^{n+3} simulations. For a failure mode in which a probability of failure of 10^{-6} is expected, this method requires an enormous computational effort. In this case, to evaluate the conditional probability of failure, the number of simulations required was much smaller.

4. CASES OF STUDY: SLIDING SAFETY EVALUATION FOR SIX DAMS

As case studies, the central cross-section of six Portuguese concrete gravity dams were chosen. Monte Novo, Pedrógão, Penha Garcia, Pretarouca, Ranhados and Rebordelo dams (Table 1) were chosen because they are founded on different rock mass types and located in different seismic zones. For each profile, the expression (8) was performed 25000 times, varying the coefficient of friction and the uplift quantifier factor according to its probability distribution functions.

The uplift quantifier factor, given by expression (3), was evaluated for $t=5$ *years*. It is considered that in an interval of five years, measures to reduce possible high uplift pressures could be taken and the maximum value for a five-year period could be representative during the reference period.

Based on the expression (12), the probability of failure, conditioned to the occurrence of a high intensity seismic action, was evaluated to each profile.

Figures 3 to 8 show the values of generated random variables. They also show the limit state expression, in red, which is the boundary between the safety and the failure regions, above and below the red line, respectively.

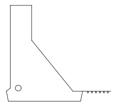
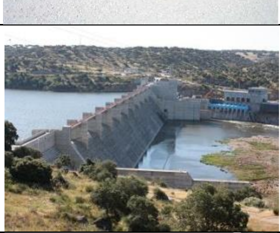
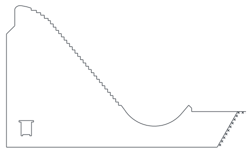
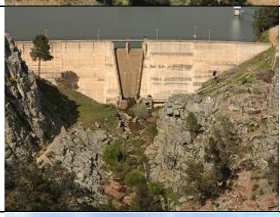
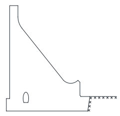
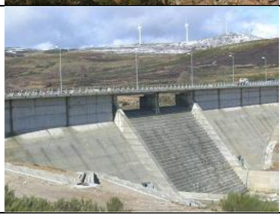
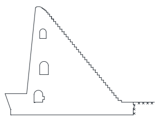
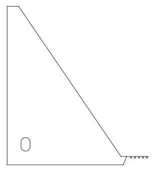
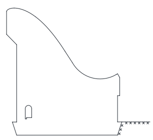
Name	Downstream view	Profile	Characteristics
Monte Novo dam			h=24.45 m A=251.48 m^2 h_u=21.45 m h_d =4.60 m a_s=1.644 m/s^2
Pedrógão dam			h=33.80 m A=846.52 m^2 h_u=33.80 m h_d =10.00 m a_s=1.644 m/s^2
Penha Garcia dam			h=25.00 m A=243.17 m^2 h_u=22.00 m h_d =4.00 m a_s=1.482 m/s^2
Pretarouca dam			h=25.50 m A=344.10 m^2 h_u=25.50 m h_d =5.20 m a_s=1.077 m/s^2
Ranhados dam			h=40.45 m A=667.71 m^2 h_u=37.45 m h_d =2.15 m a_s=1.077 m/s^2
Rebordelo dam			h=29.00 m A=477.54 m^2 h_u=29.00 m h_d =6.00 m a_s=1.077 m/s^2

Table 1. Downstream view, cross-section and characteristics of evaluated dams (Height (h), area (A), upstream water level (h_u), downstream water level (h_d) and seismic acceleration (a_s)).

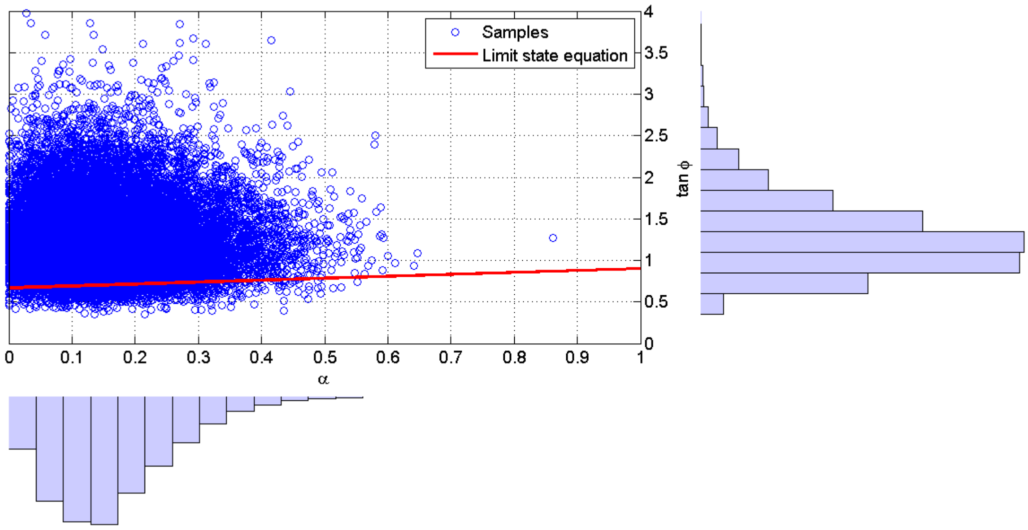


Figure 3. Random variables' values generated and safety criteria evaluation to Monte Novo dam profile.

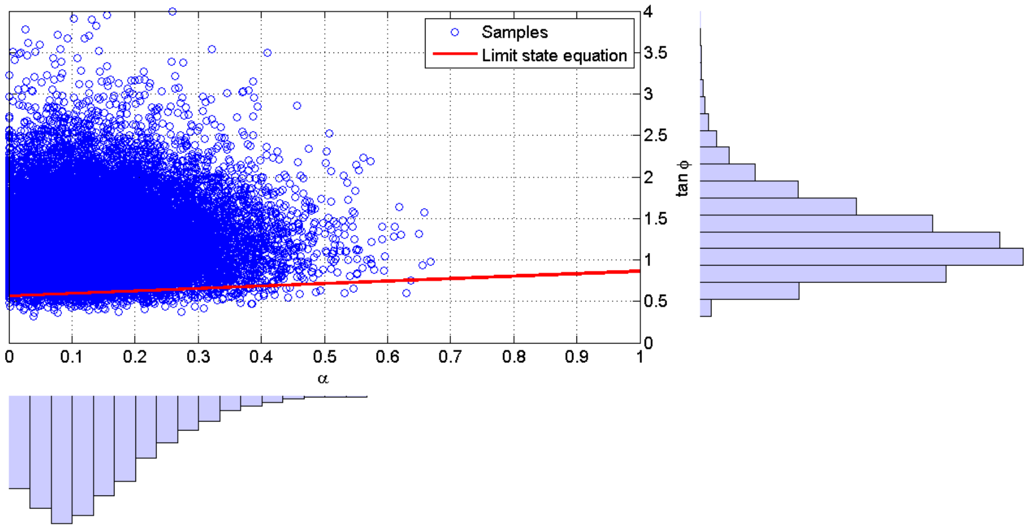


Figure 4. Random variables' values generated and safety criteria evaluation to Pedrógão dam profile.

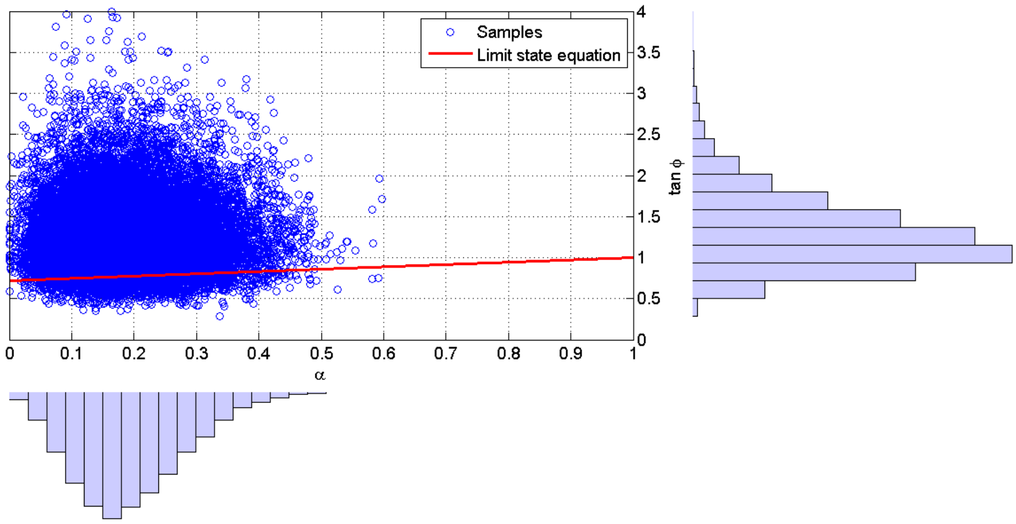


Figure 5. Random variables' values generated and safety criteria evaluation to Penha Garcia dam profile.

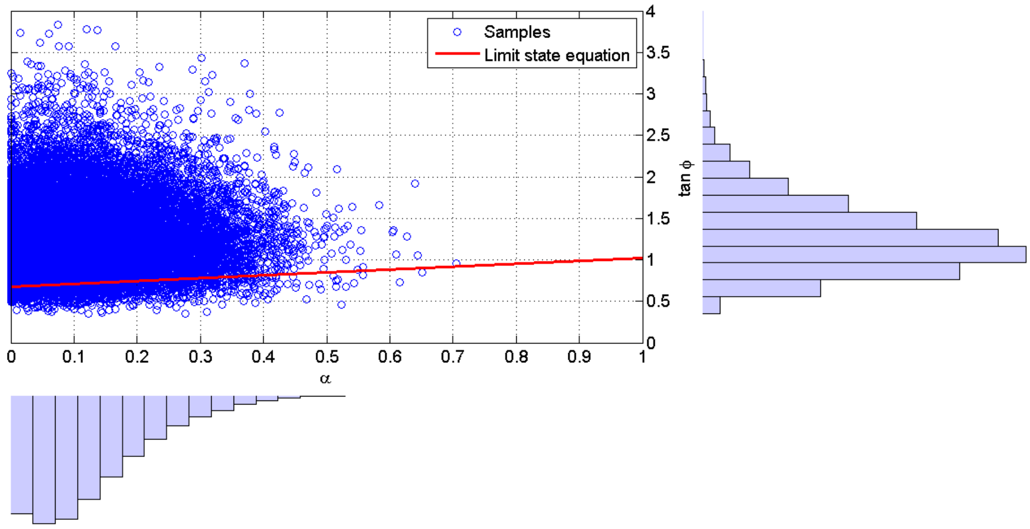


Figure 6. Random variables' values generated and safety criteria evaluation to Pretarouca dam profile.

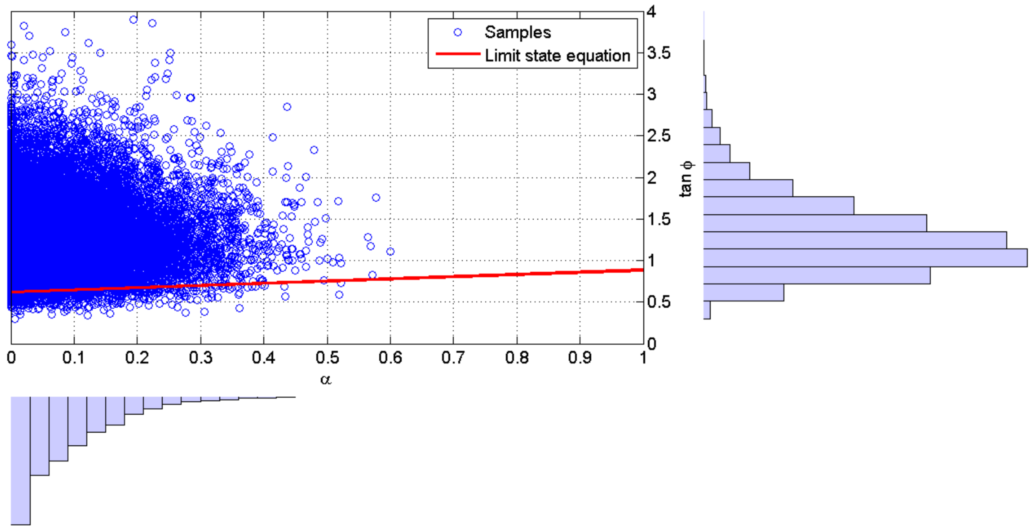


Figure 7. Random variables' values generated and safety criteria evaluation to Ranhados dam profile.

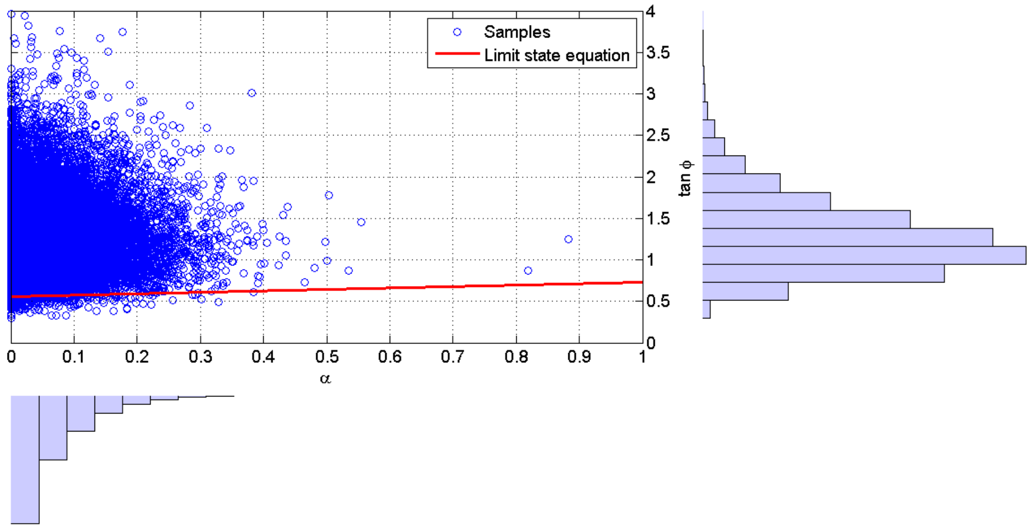


Figure 8. Random variables' values generated and safety criteria evaluation to Rebordelo dam profile.

Table 2 shows the type of rock mass foundation, the global safety factor, described in expression (7), the probability of failure conditioned to the occurrence of a high intensity seismic action for the reference period and the annual probability of failure conditioned to the occurrence of a high intensity seismic action evaluated to each analysed dam.

Dam name	**Rock mass foundation**	**Global safety factor (FS)**	$\mathbf{P_f^{100}}$ **(G≤0|SMP)**	$\mathbf{P_f^{1}}$ **(G≤0|SMP)**
Monte Novo	Schists and greywackes	1.13	5.0%	5.0×10^{-5}
Pedrógão	Highly fractured granites	1.31	1.9%	1.9×10^{-5}
Penha Garcia	Schists	1.03	8.4%	8.4×10^{-5}
Pretarouca	Slightly fractured granites	1.05	6.3%	6.3×10^{-5}
Ranhados	Slightly fractured granites	1.19	2.8%	2.8×10^{-5}
Rebordelo	Slightly fractured granites	1.37	1.0%	1.0×10^{-5}

Table 2. Type of rock mass foundation, global safety factor, conditional probability of failure for 100 years and annual conditional probability of failure of each dam.

The inclination of the red lines, corresponding to the limit state, shows the effects of the design geometrical options on the safety margin. Though the high intensity seismic action, Pedrógão dam has one of the biggest global safety factor (FS=1.31) and smaller annual probability of failure. That happens because of the generous spillway dimensions and the significant downstream passive pressure.

A comprehensive relation between the annual probability of failure and the global safety factor was confirmed (Figure 9).

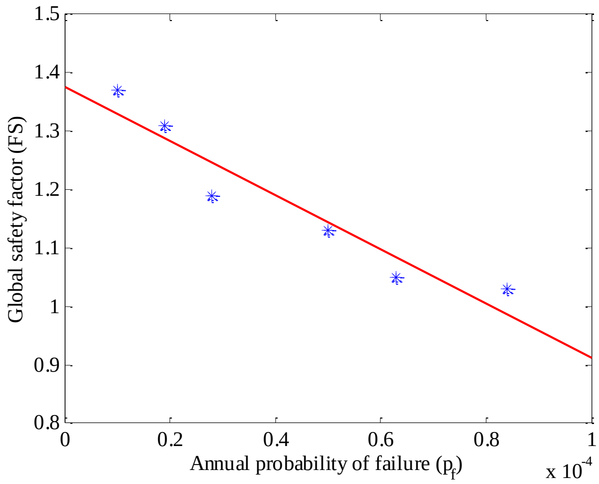


Figure 9. Relation between the probability of failure and the global safety factor.

5. CONCLUSIONS

The Portuguese codes consider a global safety factor, regarding to concrete dam safety evaluation. Adopting a probabilistic safety approach, the structural Eurocodes are based on the principles of limit states design in conjunction with the partial safety factor method.

Using as case studies the Monte Novo, Pedrógão, Penha Garcia, Pretarouca, Ranhados and Rebordelo dams, this paper shows the contribution of the uncertainty of the hydraulic and mechanical rock mass properties regarding to the sliding safety evaluation for concrete gravity dams for the design loads, including high intensity seismic actions. The geometrical data, as well as the dead-weight action, hydrostatic pressure and maximum considered earthquake effects, were considered as deterministic variables. The coefficient of friction and the uplift, quantified by means of a factor under drainage line, were considered as probabilistic variables.

The uncertainty model of the coefficient of friction was related to the tests performed in LNEC, whereas the uncertainty model of uplift pressures was related to the piezometric data from 16 Portuguese concrete dams.

Using the Monte Carlo method, the frequency of violating the safety criteria for the occurrence of extreme seismic actions and the probability of failure were evaluated.

A probability of failure of 10^{-5} was obtained for a high intensity seismic occurrence, defined in the national appendix of Eurocode 8 for 1000 years of return period. According to the importance of this type of structures[3], this value can be considered a little bit high.

Statistically, most dam accidents or incidents are due to foundation failure. Thus, it is important to retain the importance of a good hydraulic and mechanical rock mass properties characterization in safety evaluation.

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