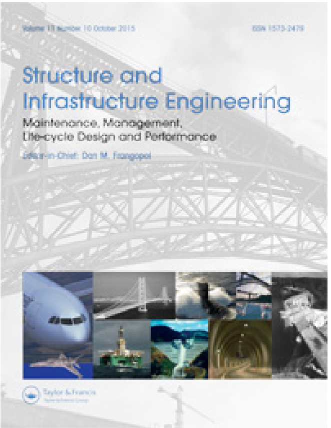

Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance
Publication details, including instructions for authors and subscription information:
http://www.tandfonline.com/loi/nsie20

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Maria Luísa Braga Farinha[a], Laura Caldeira[b] & Emanuel Maranha das Neves[c]

[a] Concrete Dams Department, National Laboratory for Civil Engineering, Av. do Brasil 101, 1700-066 Lisbon, Portugal

[b] Geotechnical Department, National Laboratory for Civil Engineering, Av. do Brasil 101, 1700-066 Lisbon, Portugal

[c] Instituto Superior Técnico, Technical University of Lisbon, Av. Rovisco Pais, 1049-001 Lisbon, Portugal

Published online: 03 Oct 2014.

To cite this article: Maria Luísa Braga Farinha, Laura Caldeira & Emanuel Maranha das Neves (2015) Limit state design approach for the safety evaluation of the foundations of concrete gravity dams, Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance, 11:10, 1306-1322, DOI: 10.1080/15732479.2014.964265

To link to this article: http://dx.doi.org/10.1080/15732479.2014.964265

Structure and Infrastructure Engineering, 2015
Vol. 11, No. 10, 1306–1322, http://dx.doi.org/10.1080/15732479.2014.964265

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Maria Luísa Braga Farinha[a*], Laura Caldeira[b1] and Emanuel Maranha das Neves[c2]

[a]*Concrete Dams Department, National Laboratory for Civil Engineering, Av. do Brasil 101, 1700-066 Lisbon, Portugal;* [b]*Geotechnical Department, National Laboratory for Civil Engineering, Av. do Brasil 101, 1700-066 Lisbon, Portugal;* [c]*Instituto Superior Técnico, Technical University of Lisbon, Av. Rovisco Pais, 1049-001 Lisbon, Portugal*

(*Received 29 January 2014; final version received 26 June 2014; accepted 12 July 2014; published online 3 October 2014*)

The application of the limit state design (LSD) in the geotechnical area has increased over the last two decades, but this approach is not yet widely used in dam safety evaluation. This study aims to widen the use of the LSD application for large dams, in particular concrete gravity dam foundations. This paper starts with a brief reference to the LSD approach in recently published guidelines for dam design, followed by a detailed description of the LSD formulation when applied to the foundation of concrete dams. The relevance of the joint application of the concepts of ultimate limit states and of numerical methods is highlighted. Comments are made regarding the criteria adopted in order to determine the characteristic values of the material mechanical properties, with an emphasis on discontinuities, taking into account the spatial variability. The sliding safety assessment of the foundation of a concrete gravity dam using the LSD and a discrete element model, both in persistent and in an accidental design situation, is presented. Results led to the conclusion that the LSD methodology may be followed for dam foundation design with the partial factor values prescribed in Eurocode 7.

Keywords: civil and structural engineering; geotechnical engineering; codes of practice and standards; dam safety; foundations; failure modes; mathematical modelling

1. Introduction

The majority of concrete gravity dam failures are due to problems in the foundation rock mass (Deroo & Boris, 2011; International Commission on Large Dams [ICOLD], 1995), thus great care has to be taken at the design stage. The design of the rock foundation requires stress–strain, seepage and sliding stability analyses. The stresses on the foundations of gravity dams are relatively low, so bearing capacity failure of the rock mass is unlikely and its safety is controlled by sliding either along the dam–concrete interface or along rock mass discontinuities or sub-horizontal weak layers in the foundation, close to the base of the dam. The conventional approach to assess the sliding stability of concrete gravity dams is based on principles of limit equilibrium (Londe, 1973), and on an overall safety factor. Both the dam and its foundation are assumed to be rigid bodies and the method consists in assuming a failure mode and calculating the driving and resisting effective forces acting on the sliding surface, with the ratio of these two forces being the overall safety factor.

Regarding the stability evaluation of gravity dams, different criteria have been adopted by different bodies, and great effort has been made in order to analyse the differences in the results (CFBR, 2006; European Club of ICOLD, 2004a, 2004b; FERC, 2002; USACE, 1983, 2000, 2005). These differences are mainly related with different ways of simulating the distribution of uplift pressures along the base of the dam, the possibility of crack formation and propagation along the dam–foundation interface and stability criteria, expressed in terms of minimum values for the overall safety factor.

Concerning the overall safety factor, several definitions have been proposed (Alonso, Carol, Delahaye, Gens, & Prat, 1996; Asadollahi & Tonon, 2010; Kovari & Fritz, 1989, 1993). In all cases, it provides a measure of the distance from the limit equilibrium. Nevertheless, generally, uncertainty about water pressure distribution and shear strength parameters (cohesion and friction angle in effective stresses) along the slip surfaces requires parametric studies to be carried out. The above-mentioned conventional deterministic approach is followed by many design codes, among which is the Portuguese regulation for dam design (NPB, 1993). The ICOLD is aware of the shortcomings of the overall safety factor concept and is moving towards probability approaches (ICOLD, 1988, 1993).

Actually, two alternative approaches may be employed: semi-probabilistic and probabilistic procedures. The former allows the consideration of the different levels of uncertainty regarding the various actions and material properties, through the use of characteristic values of actions, material properties and geometry data (called basic

*Corresponding author. Email: lbraga@lnec.pt

variables) and partial factors. This is the method prescribed in Eurocode 7 (EC7) for geotechnical design (CEN, 2004). The latter would allow a probability of failure to be estimated, taking explicitly into account the uncertainties in the basic variables (ICOLD, 1993). However, the calculation of probabilities of failure in rock engineering presents serious difficulties, mainly due to the usual lack of knowledge concerning the probability distribution functions and the spatial variability of the material properties, and to the current inability to reproduce statistical descriptions of the joint patterns in numerical models. Despite these difficulties, a few studies were presented in the last decade using probabilistic approaches for the seismic analysis of dams (e.g. Lupoi & Callari, 2012). Approximate solutions of the probability of failure were obtained by Bernstone, Westberg, and Jeppsson (2009) to assess the safety of an operating concrete dam, taking into account recorded uplift data.

The European standard EC7 (CEN, 2004) is based on the limit state design (LSD), namely ultimate limit state (ULS) and serviceability limit state (SLS) design (CEN, 2004). Its implementation has resulted in many changes to geotechnical design (Orr, 2012). Nowadays, an LSD approach is not routinely used in dam safety evaluation, contrary to current practice in both geotechnical and structural design. However, EC7 may be extended to special structures, such as dam foundations and tunnels, or to the design of foundations of nuclear power plants and offshore structures. In these cases, though, there may be additional requirements.

This study aims to widen the use of the LSD application for large dams, in particular concrete gravity dam foundations. Gravity dams resist the thrust of the reservoir water with their own weight. The seepage flow through the foundation, in the upstream–downstream direction, gives rise to both seepage and uplift forces, which, in turn, reduce the stabilising effect of the structure's 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 as accurately predicted as possible at the design stage using numerical models. The hydraulic gradients also deserve particular attention, due to their role in the internal erosion processes.

However, these particular aspects only started to be tackled in a reliable way in the 1980s, with the development of numerical models which simulate the hydromechanical (HM) interaction, which is particularly important in this type of structure, using the finite element method (e.g. Noorishad, Ayatollahi, & Witherspoon, 1982). In more recent years, several studies have been carried out using models of flow in discontinuous media, with discrete element models, mainly for gravity dams, taking into account the water pressures resulting from the seepage conditions (Barla, Bonini, & Cammarata, 2004; Farinha, 2010; Gimenes & Fernández, 2006; Lemos, 1999; Mostyn, Helgstedt, & Douglas, 1997). However, the uncertainties in these flow models still need to be evaluated.

This paper presents a study about the joint application of the concept of ULS with partial factors and of numerical methods in the safety assessment of the foundation of a concrete gravity dam, in static conditions, for two different design situations: the persistent situation and the accidental situation related to the total clogging of the drainage curtain. According to EN 1991-1-7 (CEN, 2006), a risk analysis which includes an estimate of the likelihood of the clogging of the drainage system is required for a CC3 type of structure, like a large dam. In the present case, as high levels of quality control are required not only during dam construction but also during operation, this event is not likely to occur, thus, it is considered as an accidental situation.

Based on prescribed values of EC7 for the partial factors, a safety verification of the foundation design of Pedrógão concrete gravity dam, in Portugal, will be presented. This example only envisages the ULS corresponding to the overall stability concerning sliding. Comments are made regarding the criteria adopted in order to determine the characteristic values of the rock mass strength properties, with an emphasis on discontinuities, taking into account the spatial variability. The potential failure modes regarding sliding which are analysed involve the dam foundation rock mass, taking into account the orientation of the main sets of discontinuities within the dam foundation. Analysis was carried out for the critical failure mechanism, which was adequately identified. Dam safety assessment of sliding follows EC7 (CEN, 2004).

2. LSD approach in dam design

Taking into account the dominant role of the Eurocodes concerning matters of structural safety all around Europe, it is not surprising that the subject of LSD has been raised by the European Club of the ICOLD. However, until now, the official publications of this organisation have made little mention of it. Nevertheless, the final report on sliding safety of existing gravity dams of the European Club (European Club of ICOLD, 2004a) refers, in its Appendix dedicated to Regulatory Rules, Guidelines and Normal Practice in different countries, to the Chinese Technical Standards related to the design of hydraulic engineering structures, namely the Design Specification for concrete gravity dams, which clearly states that hydraulic structures must be designed for the ULS and the SLS, the latter called normal operation limit states. In addition to these Chinese standards, guidelines on design criteria for concrete gravity dams based on the LSD were published in

Australia, in 1991, and some documents and recommendations were published in France, concerning the safety and serviceability verification for both gravity dams (CFBR, 2006, 2012) and embankment dams (CFBR, 2010).

It is relevant to make a brief reference to the above-mentioned Chinese standard, according to which the safety evaluation against sliding involving the dam foundation is identified as an ULS. The different partial factors are combined for the following design situations: sustained (corresponding to persistent situations according to Eurocodes), transient and occasional (corresponding to accidental situations) status. In the Chinese standard, ULS verification is based on the following expression:

$$\gamma_0 \Psi E\left(\gamma_G G_k, \gamma_Q Q_k, A_k\right) \leq \frac{1}{\gamma_d} R\left(X_k/\gamma_m\right), \qquad (1)$$

where γ_0 is the importance factor of the structure ($\gamma_0 = 0.9$, 1.0, 1.1), Ψ is the factor of design situation ($\Psi = 1.0$, 0.95, 0.85) and γ_d is the structure coefficient, equal to 1.2 for the sliding limit state.

The remaining symbols are those used in Eurocodes: $E()$ is the calculation model of the effects of the actions; $R()$ is the calculation model of the resistances; γ_G, γ_Q and γ_m are the partial factors for permanent actions, variable actions and ground parameters (material properties), respectively; and G_k, Q_k and A_k are the characteristic values of permanent, variable and accidental actions, respectively. For the assessment of the ULS of sliding, a rigid body limit equilibrium approach is used. The overall shear resistance is evaluated by means of the Mohr–Coulomb criterion based on cohesion and internal friction angle in terms of effective stresses. The procedure for calculating the characteristic values is not included in the above-mentioned Appendix of the European Club report, but will be given particular attention in this paper.

It is also interesting to mention that the LSD approach proposed in the previously mentioned guidelines on design criteria for concrete gravity dams published by the Australian National Committee on Large Dams (ANCOLD) in 1991 was recently replaced by a factor of safety or working stress approach, in revised guidelines published in September 2013 (ANCOLD, 2013). This move away from the LSD approach was justified by the need to reflect the preferred design approach (DA) amongst Australian dam engineers.

Regarding French publications in this area, the documents adopt the limit state format similar to Eurocodes. The recommendations mark an important departure from conventional deterministic practices, requiring the designer to set at source the safety factors on the various basic variables involved in the computations. Concerning gravity dams, this standardised method is now being used routinely in France for new projects, safety reviews and design of remedial works (Peyras et al., 2008; Royet & Peyras, 2013).

3. Formulation

3.1. General aspects

In this paper, the use of LSD for dam safety assessment is based on EC7 (CEN, 2004), as mentioned in the introduction. This European standard describes the general principles and requirements for geotechnical design, primarily in order to ensure safety (resistance and stability), serviceability and durability of geotechnical structures. As such, it should be used in conjunction with EN 1990 Eurocode: basis of structural design (CEN, 2002b), EN 1991 Eurocode 1: actions on structures (CEN, 2002a) and EN 1998: design of structures for earthquake resistance (CEN, 2005). EC7 consists of two parts: Part 1, General Rules (CEN, 2004), which presents the general rules of geotechnical design; and Part 2, Ground Investigation and Testing (CEN, 2007), which discusses the use of field investigations and laboratory testing for geotechnical design.

EC7 serves as a reference document for the geotechnical design of foundations for special foundation works [clause 2.1(21) of EC7 (CEN, 2004) and clause 1.1(2) of EN 1990 (CEN, 2002b)], such as the foundations of concrete dams. This study focuses on large dams, particularly on concrete gravity dams and the safety analysis of their foundations.

3.2. DAs and design situations

EN 1997-1 (CEN, 2004) and EN 1990 (CEN, 2002b) include, as options, the following three different DAs for ULS verifications in persistent and transient design situations (Frank et al., 2004). DA1 requires, in principle, two calculations involving two sets of partial factors (Combinations 1 and 2). Where it is obvious that one of these sets governs the design it will not be necessary to carry out calculations for the other (Annex B.1 (2), CEN, 2004). This DA may be termed an *action and material factor approach* with partial factors applied at the source, that is to actions, rather than to the effects of actions, and to shear strength parameters, rather than to resistances.

DA2 requires a single calculation to either actions or effects of actions and to resistances. It may be termed an *action effect and resistance factor approach*. It must be noted that, in this case, the application of partial factors to the effects of actions does not deviate significantly from the conventional overall factor safety approach. DA3 requires a single calculation where partial factors are applied to actions or effects of actions from the structure and to ground strength parameters and may be termed an *action effect and material factor approach*.

A National Annex allows each European country to set, within certain limitations, the safety levels for civil engineering works through the partial factors values, called Nationally Determined Parameters. The National Annex may also specify the procedure to be used when alternative procedures are given in the Eurocode, namely the DAs. In this study, the Portuguese National Annex will be followed, which requires DA1 to be adopted for the ULS design in both the persistent and transient situations. For the accidental situations, the partial factors included in this Annex are also assumed.

The actions as well as the shear strength of the materials will be represented by their characteristic values. The design values result from the application of partial factors to the characteristic values. Obviously if the partial factor is 1, the characteristic and design values are equal. A particular and important aspect of the actions acting on a concrete gravity dam, when compared with the structures covered by EC7, is the large values of the water pressure on the upstream face of the dam, as well as the significant weight of the dam body itself. Two design situations are considered, as shown in Figure 1: the persistent situation considering the retention water level (RWL) at the reservoir (Figure 1(a)), and the accidental situation involving the complete clogging of the drainage system of the dam foundation (Figure 1(b)).

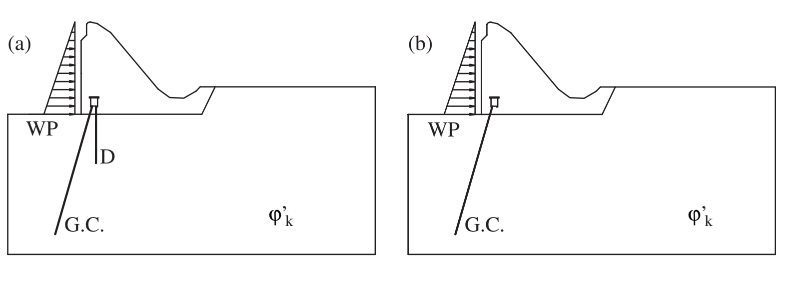
According to the Eurocode system, for the persistent design situation, the following partial factors must be used: (i) Combination 1 – partial factors for permanent actions greater or equal to 1 ($\gamma_G \geq 1$, namely 1.0 or 1.35) and for the shear strength equal to 1 ($\gamma_M = 1$); (ii) Combination 2 – partial factors for permanent actions equal to 1 ($\gamma_G = 1$) and for the material properties greater than 1 ($\gamma_M > 1$, namely 1.25). For the accidental design situation, the partial factors for permanent actions are equal to 1 ($\gamma_G = 1$) and for the shear strength properties greater than 1 ($\gamma_M > 1$, namely 1.1).

In the present case, for Combination 1, two permanent actions (water pressure on the upstream face of the dam and dam weight) must be combined with different partial factors, as one is unfavourable and the other favourable (Combination 1 with load case 1 – C1.1). The recommended value of γ_G is 1.35, for unfavourable permanent actions, and 1.0, for favourable permanent actions. Only one strength parameter is used in the following – the friction angle in effective stresses at the rock mass discontinuities. The corresponding partial factors are 1 and 1.25, respectively, for Combinations 1 and 2 (C2) of the persistent design situation and 1.1 for the accidental design situation (A). Figure 2 shows the load cases and corresponding partial factors. An additional load case considering both permanent actions as unfavourable (Combination 1 with load case 2 – C1.2) is also shown, for comparison with conventional procedure in dam design. However, in gravity dams it is obvious that the self-weight always has a stabilising effect in the ULS verification of sliding.

3.3. Application of LSD in a numerical modelling framework

The current practice of ULS verification assumes a rigid perfectly plastic behaviour associated with a previously selected failure mechanism, usually requiring the study of a large number of mechanisms in order to identify the critical one. The constitutive models based on stress–strain relations are only used in SLS verifications. Nevertheless, models of the latter type can and shall be used, with the adequate design values, in the ULS verifications, since they enable the prompt and unequivocal identification of the critical failure mechanism, and allow consideration of the interaction between the effects of different actions. Therefore, they provide more adequate knowledge of the forces in place.

The above-mentioned procedures imply the use of numerical analysis to determine the effects of actions. However, it is very important to stress the singular nature of the geotechnical actions when compared with the structural ones. The latter are independent of the mechanical characteristics of the materials, while the former, if originated at the ground, either soil or rock mass, are dependent on the mechanical characteristics of the ground. However, more importantly, the stress transmitted to the ground by the external action can generate, in an elemental area of the sliding surface and simultaneously, both a shear stress (having a destabilising effect) and a



NOTE: G.C., grout curtain; D., drainage curtain; WP, water pressurewith the reservoir at the retention water level; φ'$_k$, characteristic friction anglein effective stresses.

Figure 1. Design situations: (a) persistent situation considering the reservoir at the RWL and (b) accidental situation involving the complete clogging of the drainage system of the dam foundation.

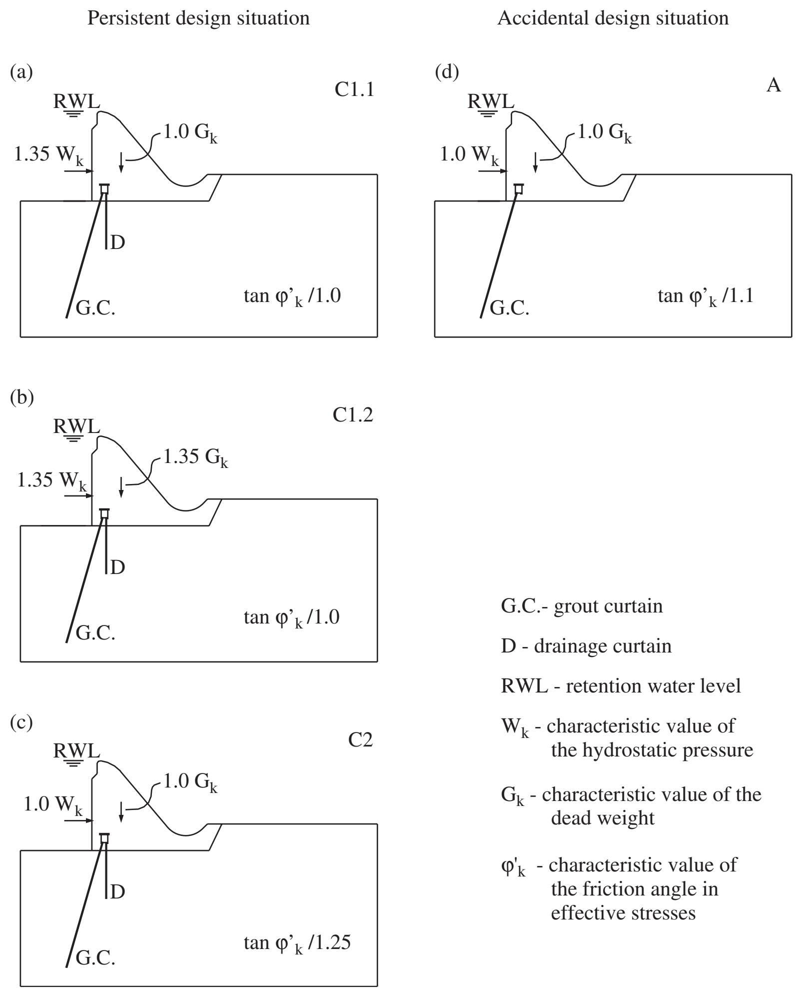


Figure 2. Combinations, load cases and corresponding partial factors for the different design situations: (a) C1.1; (b) C1.2; (c) C2; (d) A.

normal stress (having a stabilising effect), the latter is due to the frictional nature of the mobilised resistance on the assumed surface.

Therefore, due to the difficulties in distinguishing the effects induced by the different actions, these actions, when introduced in the model, should not be affected by any partial factor, the failure mechanism and the results of all numerical analyses always being induced or calculated by a *strength reduction procedure*. The critical failure mechanism may be determined through the analysis of the foundation displacements, considering design values of strength affected by an increasing partial factor. This is done until a failure surface is determined, which is assumed as the critical failure mechanism for the overall sliding. To ensure that such ULS will not occur, or, rather, to check its sufficiently low probability of occurrence, analysis is carried out for the identified mechanism.

In numerical analysis, when there is more than one partial factor for actions, a reference value, $\gamma_{F,1}$, must be selected (in general associated with the dominant action) and applied to the whole effect of the combination of actions; and the partial factors of other actions, $F_{k,i}$, must be referenced to this value, so the following expression must be used (Gulvanessian, Calgaro, & Holichy, 2002):

$$E_{\mathrm{d}} = \gamma_{F,1} E\left\{F_{k,1}; \frac{\gamma_{f,i}}{\gamma_{f,1}} F_{\mathrm{rep},i}\right\}, \quad i > 1, \tag{2}$$

where E_{d} is the design effect of the actions, $E\{\,\}$ is the result of the calculation model, $F_{k,1}$ is the dominant action for the considered limit state associated with $\gamma_{F,1}$, $F_{\mathrm{rep},i}$ are the remaining actions, $\gamma_{f,i}$ are the corresponding partial factors to $F_{\mathrm{rep},i}$ and i is the number of remaining actions in addition to the dominant one. The numerical analysis is performed with the characteristic values of the dominant action and the characteristic values of the remaining actions affected by the relation $\gamma_{f,i}/\gamma_{f,1}$.

The critical failure mechanism is reached by a progressive reduction of the strength from its design value, by performing a sequence of analysis, until failure occurs. The final value of the reduction factor corresponding to the last stable situation is designated by strength reduction factor (SRF), which is the well-known *overall safety factor*. An auxiliary factor, the over-design factor (ODF), is then introduced as

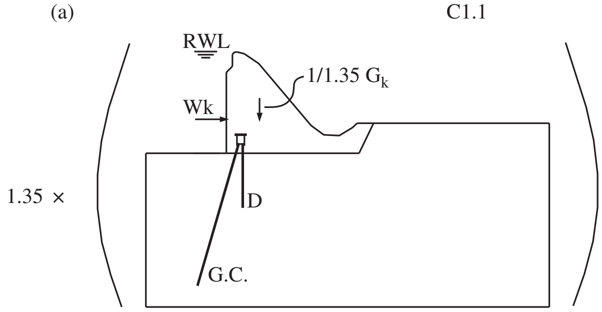
$$\mathrm{ODF} = \frac{\mathrm{SRF}}{\gamma_{F,1}}. \quad (3)$$

This ODF gives an additional margin of safety beyond that required by Eurocode for the studied limit state. The safety verification implies that this factor be larger than or equal to 1.

In the application of this procedure to the example shown in Figures 1 and 2, the dominant action is the water pressure on the upstream face of the dam, therefore according to Equation (2), its partial factor was not initially applied in the numerical model. Figure 3 presents a schematic representation of the use of actions' partial factors when numerical analysis is carried out.

For C1.1, the numerical analysis was performed with the characteristic value of the water pressures at the upstream face of the dam, 0.74 (1/1.35) of the characteristic value of the weight of the dam body and the design value (equal to the characteristic one) of the shear strength of the rock mass discontinuities. The design value of φ' ($\tan\varphi'_d = \tan\varphi'_k/\gamma_M = \tan\varphi'_k$) is reduced by applying a strength reduction procedure, SRF $>$ 1 ($\tan\varphi' = \tan\varphi'_d/\mathrm{SRF}$) and the resulting horizontal displacement at the crest of the dam (failure indicator) is registered. The value of the ODF is obtained from the final value of the SRF divided by 1.35 ($\gamma_{F,1}$). It should be noted that the procedure described above for C1.1 has been developed for the analysis of a combination of favourable and unfavourable loads. In this case, however, the dam weight has an important role in terms of resistance, and thus the procedure presented will result most probably on the conservative side.

(a) C1.1



(b) C1.2

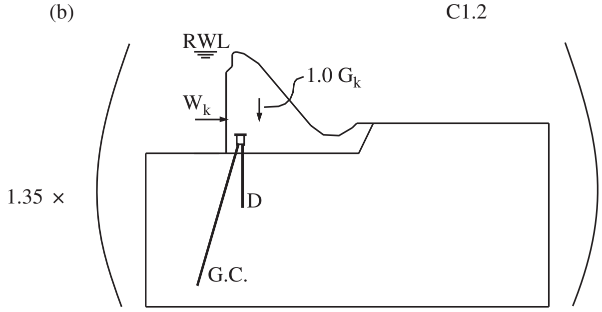


Figure 3. Use of the actions' partial factors of Combination 1 when numerical analysis is carried out: (a) C1.1; (b) C1.2.

For C1.2, the numerical analysis was carried out with the characteristic value of the water pressures at the upstream face of the dam, the characteristic value of the weight of the dam body and the design value (equal to the characteristic one) of the shear strength of the rock mass discontinuities. The value of the ODF is also obtained from the final value of the SRF divided by 1.35 ($\gamma_{F,1}$). For Combination 2 (C2), the calculation was performed using the design value of the shear strength, applying the partial factor, γ_M, to its characteristic value. In this case, only $\gamma_{\tan\varphi'}$ is required and, according to EN 1997-1, its value is 1.25. In a procedure similar to the one described for Combination 1, an SRF value corresponding to the last stable situation was obtained. For this combination, the ODF coincides with the SRF.

For the accidental design situation (A), the calculation was made using the design values of the friction angle in effective stresses at the rock mass discontinuities, applying the partial factor $\gamma_{\tan\varphi'}$ with a value of 1.1. In a similar procedure, an SRF value was obtained. For this combination, the ODF also coincides with the SRF. A value of ODF equal to 1 indicates that both sliding safety and the available margin of safety are exactly what is required by EC7 (CEN, 2004). If ODF $>$ 1, the margin of safety is more than adequate and ODF $<$ 1 means inadequate safety (though not necessarily meaning failure).

3.4. *Characteristic values*

The basic variables involved in the overall sliding stability of a gravity dam are the reservoir water level, the unit weights of both the dam body and the foundation, the shear strength parameters along the sliding surface and the geometry of the dam, and its foundation discontinuities. For the persistent and accidental design situations, the reservoir water level was assumed to be at the RWL. Thus, for this variable, as well as for the unit weights of the structure and of the foundation, whose variability is small [coefficient of variation (COV) less than 5%], the corresponding characteristic values are taken as the mean values. Thus, the uncertainties are associated with the shear strength parameters at the failure surface and the geometry of the foundation discontinuities.

Due to the lack of data, the uncertainties associated with the geometry of the rock mass discontinuities were not explicitly taken into consideration. Instead, an unfavourable configuration was assumed in the numerical modelling (Rocha, 1978). It is important to highlight that, being an ULS verification, the peak values of the shear

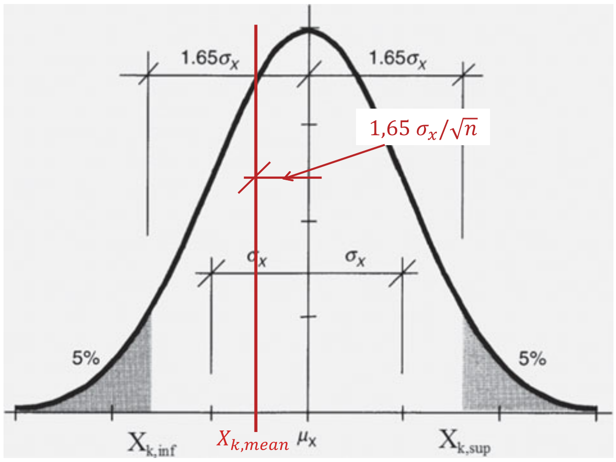


Figure 4. Evaluation of the characteristic values with a statistical basis using a normally distributed variable *X*.

strength or any contribution of the dilatancy must be replaced by the critical state values, that is the strength for large relative displacement between discontinuities' surfaces, with a constant shear stress and no volumetric variation (the distance between the discontinuities' surfaces is constant, not dependent on the distortion).

The evaluation of the characteristic value of the shear strength with a statistical basis is performed (Figure 4), in accordance with EC7 (CEN, 2004), calculating the lower value of the critical state friction angle in effective stresses associated with the 0.05 fractile, $\phi'_{k,\text{inf}}$, if the extent of the failure surface involved in the sliding mechanism is small, using the following Equation (a normal distribution for the shear strength variable being assumed):

$$\tan \phi'_{k,\text{inf}} = \overline{\tan \phi'} - k_{5\%}\sigma_{\tan \phi'} \tag{4}$$

or of the 95% lower confidence limit for the population mean of the same parameter, $\phi'_{k,\text{mean}}$, if a larger rock mass volume is involved in the failure mechanism, as in

$$\tan \phi'_{k,\text{mean}} = \langle \mu_{\phi'} \rangle_{95\%} = \overline{\tan \phi'} - k_{5\%}\frac{\sigma_{\tan \phi'}}{\sqrt{n}}, \tag{5}$$

$$\tan \phi'_{k,\text{mean}} = \langle \mu_{\phi'} \rangle_{95\%} = \overline{\tan \phi'} - t_{5\%}\frac{s_{\tan \phi'}}{\sqrt{n}}, \tag{6}$$

where $\sigma_{\tan \phi'}$ is a known standard deviation of the variable $\tan \phi'$, given in the bibliography or derived from test results (then represented by $s_{\tan \phi'}$), $k_{5\%}$ is the value of the standard normal variable associated with a probability of 5%, $\langle \mu_{\phi'} \rangle_{95\%}$ is the 95% lower confidence limit for the mean, $\mu_{\phi'}$, and $t_{5\%}$ is the variable value of the *t*-student distribution, with a number of degrees of freedom equal to the number of available tests (*n*) minus 1, associated with a probability of 5%. Equation (5) can be used when the variance of the distribution is known *a priori* and Equation (6) can be used when this variance is unknown, being replaced by the variance of the test results.

Regarding failure mechanism dimension, no criterion is indicated in EC7. For the establishment of this criterion, it is necessary to take into account the spatial variability and averaging of the parameter value along the failure surface, as explained below. Neglecting the other components of the uncertainty associated with the determination of the geotechnical parameters, namely, measurement and interpretation errors and statistical uncertainty, and considering only the spatial variability, for a normal distribution, the characteristic value $\phi'_{k,\text{var}}$ can be estimated based on the following equation:

$$\tan \phi'_{k,\text{var}} = \overline{\tan \phi'}(1 - k_{5\%}\,\text{COV}_{\text{inherent}}\Gamma), \tag{7}$$

where $\text{COV}_{\text{inherent}}$ represents the coefficient of variation induced by the spatial variability and Γ is the variance reduction function resulting from the averaging process of the parameter along the spatial extent (average length *L*, area *A* or volume *V*) of the governing failure mechanism.

Several functions are available for this variance reduction function, the linear, the exponential and the quadratic exponential being the most common. However, in the current cases, these all give very similar results. In the following, the simplest function, the linear, is adopted:

$$\Gamma^2 = \begin{cases} \frac{\delta}{L}\left(1 - \frac{\delta}{3L}\right) & \text{if} \quad \frac{L}{\delta} > 1, \\ 1 - \frac{L}{3\delta}, & \text{if} \quad \frac{L}{\delta} \leq 1, \end{cases} \tag{8}$$

where δ represents the fluctuation scale (related to the effective autocorrelation distance) and *L* is the length of the sliding surface in the considered direction. The square of total variance reduction function can be obtained by multiplying the square of the variance function in each direction.

The comparison of the values obtained with Equations (4) to (7) allows the assessment of the extent of the failure surface, that is whether the extent of the failure surface is considered either small or large. Thus, if the characteristic value calculated with Equation (7) is larger than that obtained with Equations (5) or (6), the critical sliding surface can be considered large, otherwise, Equation (4) must be used. Despite not being foreseen in EC7, the spatial variability is explicitly considered in the study presented here, based on an assumed scale of fluctuation, in order to analyse its influence on ULS sliding verification.

4. Application to Pedrógão dam

4.1. General characteristics

Pedrógão dam (Figure 5) is a straight gravity dam located on the River Guadiana, in the south-east of Portugal, with a

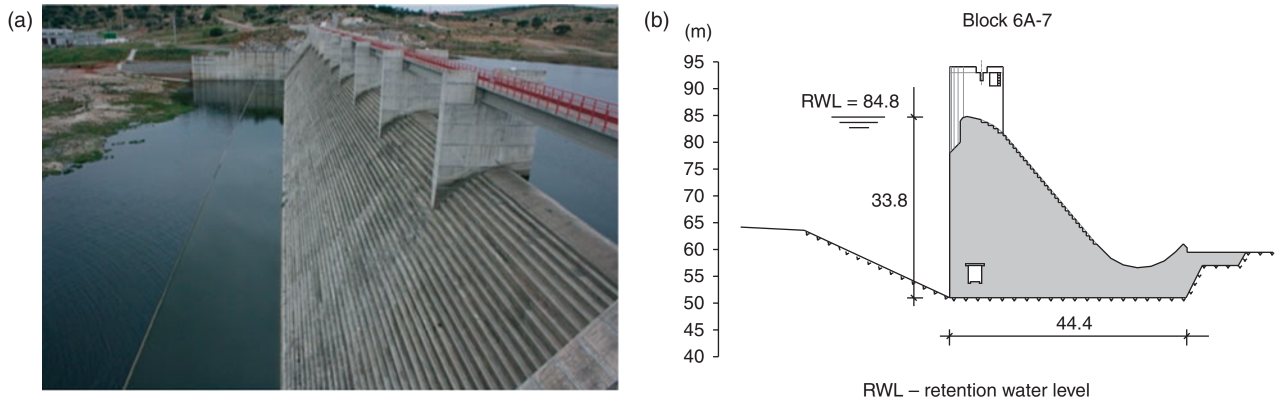


Figure 5. Pedrógão dam: (a) downstream view from the left side of the uncontrolled spillway and (b) cross-section of the central area of the dam.

maximum height of 43 m and a total length of 448 m, of which 125 m is of conventional concrete and 323 m of roller-compacted concrete. The dam is part of a multi-purpose development designed for irrigation, energy production and water supply (Miranda & Maia, 2004) and creates a reservoir which allows the turbines of Alqueva dam, the main structure of the development, located about 23 km upstream from Pedrógão, to pump water from the downstream to the upstream reservoir in a reverse motion. 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 RWL. Figure 5(b) shows a cross-section of the central area of the dam, in which the base length in the upstream–downstream direction is 44.4 m and the dam height is 33.8 m. It should be noted that the cross section of this dam is unusual, as in the majority of gravity dams the base length is around 20% lower than their height.

Since the beginning of the first filling of the reservoir, in November 2005, Pedrógão dam has shown a peculiar seepage pattern, with some local high flow rates and uplift pressures. A discontinuous HM model of the dam foundation was developed, in which the main seepage paths, identified with *in situ* tests, were represented, which allowed recorded discharges and water pressures during normal operation to be accurately interpreted (Farinha, 2010). In the study presented here, however, a design stage is assumed, and thus the current performance of the dam is not taken into account.

4.2. Characteristics of the foundation rock mass

The foundation of Pedrógão dam 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 poor. The most relevant geotechnical aspects of the foundation rock mass in the area of the river bed below the central area of the dam, which is going to be analysed, are described in detail in Caldeira, Farinha, Maranha das Neves, & Lemos (2013).

During excavation it was possible to identify the main joint sets in the different foundation areas (EDP, 2004). Figure 6 depicts the average position of the main sets of rock joints, in the bottom of the valley, in relation to the dam. The sub-horizontal joint set may be relevant regarding strength and/or deformability of the dam foundation and must be pointed out due to its substantial extension at the valley bottom, near the riverbed and downstream from the dam. The hydro-geological studies led to the conclusion that the ground-water table was controlled by the River Guadiana. The results of the Lugeon tests revealed a low permeability rock mass at depth, except in localised zones or along discontinuities. Figure 7 illustrates the areas of poor geomechanical properties and of highest permeability in the foundation of Pedrógão dam.

From samples collected in boreholes PD5, PD7 and PD12 at given depths (see Figure 7 and Table 1), five

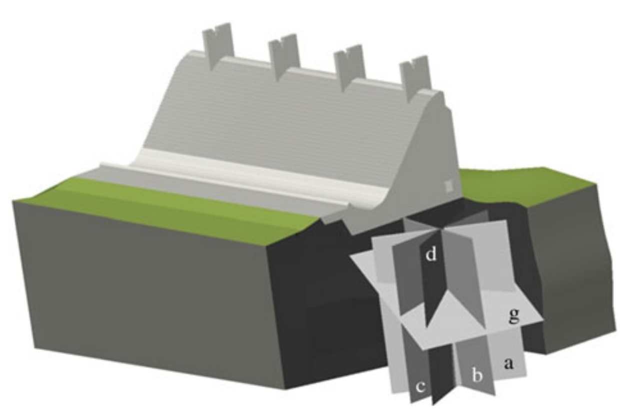


Figure 6. Average position of the main sets of rock joints (a, b, c, d and g) in relation to the dam.

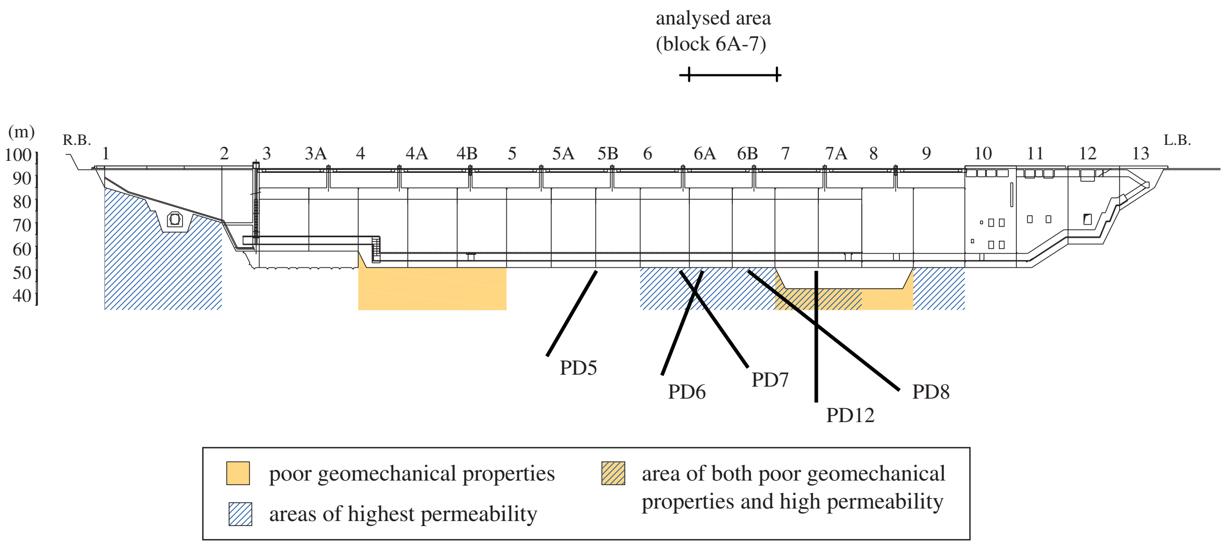


Figure 7. Areas of poor geomechanical properties and of highest permeability in the foundation of Pedrógão dam. Location of the boreholes drilled for geological and geotechnical investigation in the central area of the dam.

compression and shear tests of the discontinuities' planes were carried out at different normal effective stresses at the Portuguese National Laboratory for Civil Engineering (LNEC, 2004). The results presented in Table 1 are the normal stiffness at a normal stress of 1 MPa, shear stiffness corresponding to a shear displacement of 0.2 mm at the same normal stress, ultimate friction and ultimate dilatancy angles. The friction angle at the critical state (without dilatancy) is presented in the same table and used in the calculation of the characteristic values. This variable presents a low variation when compared with the other strength parameters, so its use constitutes a great advantage in the probabilistic context.

The mean value and the COV of the friction angle sample are equal to 34.3° and 7.4%, respectively. Following the recommendation of Schneider (1999), a value of 10% was adopted for COV. Two different characteristic values were calculated using Equations (4) and (5) proposed in EC7: $\phi'_{k,\text{inf}}$, corresponding to the lower limit of the population, is 29.7°, and $\phi'_{k,\text{mean}}$, corresponding to the lower limit of the mean value, is 32.3°.

Figure 8 shows the influence of spatial variability on the characteristic values, as a function of the scale of fluctuation for $L = 94$ m (Equations (7) and (8)). Conservatively, the length in the direction perpendicular to the cross section of the dam was considered small and the corresponding variance reduction function was assumed to be equal to 1. Figure analysis leads to the conclusion that the characteristic value considering the spatial variability, $\phi'_{k,\text{var}}$, decreases gradually with an increase in the scale of fluctuation, and that it is higher than $\phi'_{k,\text{mean}}$ for values of the scale of fluctuation lower than 20 m. So, according to EC7, if the scale of fluctuation is lower than 20 m, the failure surface must be considered large and the characteristic value equal to $\phi'_{k,\text{mean}}$. Otherwise, the failure surface extent is small and the characteristic value is equal to $\phi'_{k,\text{inf}}$.

In order to assess the influence of spatial variability in terms of dam sliding stability, a scale of fluctuation of 0.60 m (equal to the discontinuities' average spacing) is assumed in this paper. Given that the critical sliding surface has a dimension of 94 m, a characteristic value considering the spatial variability, $\phi'_{k,\text{var}}$, of 33.9° is obtained.

Table 1. Compression and shear test results in effective stresses of the discontinuities' planes.

Borehole identification	Sample depth (m)	Normal stiffness[a] (MPa/mm)	Shear stiffness[b] (MPa/mm)	Ultimate friction angle (°)	Ultimate dilatancy angle (°)	Friction angle at the critical state (°)
PD5	20.15	15.2	2.9	41.3	5.5	35.5
	22.70	25.4	1.9	37.5	6.5	32.9
PD7	14.20	39.4	2.4	38.7	4.2	36.2
PD12	37.55	23.6	2.5	41.8	6.3	35.3
	38.20	13.2	3.0	39.8	7.8	31.4

[a] Normal stiffness at normal stress of 1 MPa.
[b] Shear stiffness corresponding to a shear displacement of 0.2 mm at a normal stress of 1 MPa.

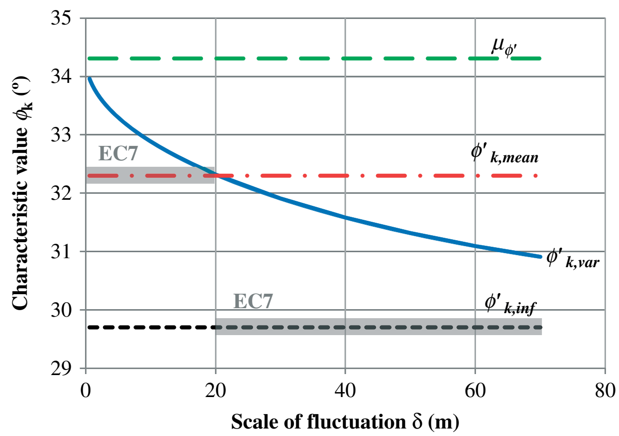


Figure 8. Influence of spatial variability on the characteristic values proposed by EC7, as a function of the scale of fluctuation, for $L = 94$ m.

4.3. Two-dimensional HM discontinuum model

The sliding verification of Pedrógão dam is carried out with a 2D HM discontinuum model, using the discrete element method (DEM), involving the cross section of the central area of the dam. The discontinuous model developed to study the ULS concerning sliding is shown in Figure 9. The model presented here is based on a previously developed model, which had been calibrated taking into account the quantity of water collected at the dam's drainage curtain (Farinha, 2010). In this model, two sets of discontinuities 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 (for the sliding safety assessment it is not necessary to consider the actual joint spacing; a coarse mesh with an average zone size of 5.0 m × 5.0 m proved to be sufficient).

Although there is no evidence from site investigations, an additional rock mass joint was assumed, dipping towards upstream (see α angle in Figure 9), in order to allow a failure mechanism of sliding along foundation discontinuities to develop. This hypothetical situation may simulate a combined mode of failure, where the failure path occurs both along the dam–foundation interface and through the rock mass, in geology where the rock is horizontally or near horizontally bedded and the intact rock is weak (USACE, 1994).

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, as shown in Figure 5(b). The analysis was carried out utilising software UDEC (Itasca, 2004), which 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 time step, as described in Lemos (2008). It is assumed that rock blocks are impervious and that flow takes place only through the set of interconnecting discontinuities. Effective normal stresses are obtained at the mechanical contacts. Flow is simulated by means of the parallel plate model, and the flow rate per model unit width is thus expressed by the cubic law. The medium is assumed to be deformable and the flow is dependent on the state of stress within the foundation.

Both dam concrete and rock mass blocks are assumed to follow a linear elastic behaviour, with the properties shown in Figure 9. The Mohr–Coulomb failure criterion is assigned to discontinuities. As previously mentioned, in ULS verifications the most important ground properties are those related to strength, deformability generally being

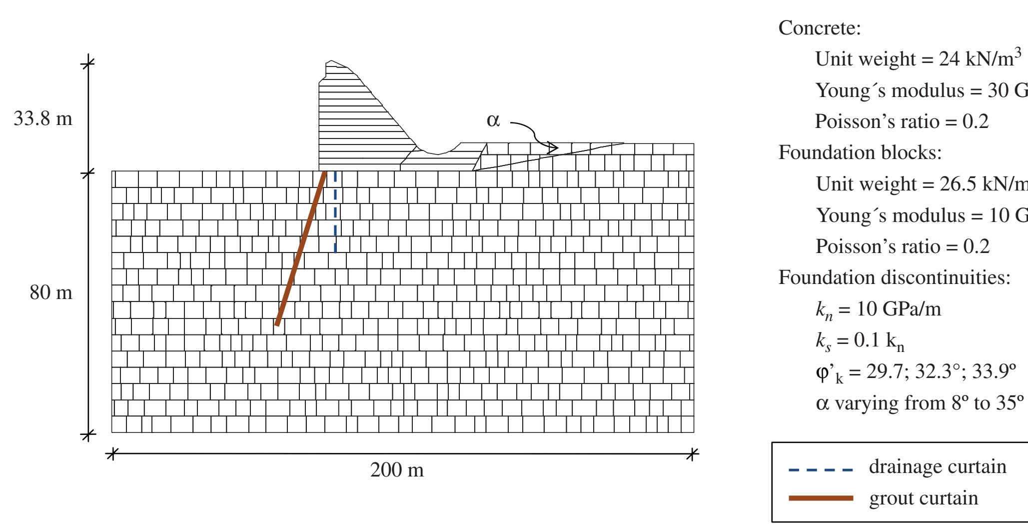


Figure 9. Discontinuum model of Pedrógão dam foundation and material properties.

second in importance. For the present case, a sensitivity numerical analysis showed that the stiffness variation has a very small influence on water pressure build up. Therefore, values of 10 GPa/m for the normal stiffness (k_n) and 1 GPa/m for the shear stiffness (k_s) were assumed at the foundation discontinuities and at the dam–foundation interface. In these discontinuities, as mentioned in Section 4.2, three different values of the characteristic friction angle in effective stresses ($\phi'_{k,\mathrm{mean}} = 32.3°$, $\phi'_{k,\mathrm{inf}} = 27.7°$ and $\phi'_{k,\mathrm{var}} = 33.9°$) were considered. In rock joints, cohesion in effective stresses was assumed to be zero, while at the dam lift joints this parameter was assigned 2.0 MPa. As the aim of this study is the ULS foundation analysis, the strength reduction method was only applied to the foundation rock mass properties. This procedure is adequate in discrete element analysis and has the advantage of enabling the evaluation of displacement indicators during the process of strength reduction. In rock joints, dilatancy may be relevant for small displacements, but it should not be considered in failure analysis.

Analysis was carried out in two loading stages. First, the mechanical effect of gravity loads with the reservoir empty was assessed. In the UDEC model, an *in situ* state of effective stress with $K_0 = 0.5$ 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. Second, the hydrostatic loading corresponding to the full reservoir (RWL) 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 HM analysis. In both stages, vertical and horizontal displacements at the base of the model and horizontal displacements perpendicular to the lateral model boundaries were prevented. Regarding the hydraulic boundary conditions, joint contacts along the bottom and sides of the model were assumed to have zero permeability. On the rock mass surface, the head was 33.8 m upstream from the dam, and 0.5 m downstream. The water head of 33.8 m upstream from the dam simulates the water in the reservoir at its RWL. The drainage system was simulated by assigning domain water pressures along the drain axis, assuming the atmospheric pressure at the drains' head.

In UDEC, the hydraulic aperture of the discontinuities is given by

$$a = a_0 + \Delta a, \tag{9}$$

where a_0 is the aperture at nominal zero normal stress and Δa is the 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. In the study presented here it was assumed that $a_0 = 0.1$ mm, and that $a_{\mathrm{res}} = 0.02$ mm. The value of a_0 was defined taking into account both field data and the results of numerous tests performed at US dam sites in the depth range 0–60 m, which indicated that most conducting apertures were in the range of 50–150 μm at this shallow depth (Barton, Bandis, & Bakhtar, 1985). The maximum aperture was limited to $20 \times a_{\mathrm{res}}$. It was assumed that the grout curtain was 10 times less pervious than the surrounding rock mass.

4.4. Identification of the foundation failure mechanism

In order to identify the foundation failure mechanism, numerical analysis was carried out using the DEM and several different geometries, each one of them with the rock mass joint downstream from the dam dipping towards upstream at a different angle α (as shown in Figure 9). Values of α varying from 8° to 35° were considered. For each calculation, the strength of the foundation discontinuities was gradually reduced, in a sequence of analysis. In this study, the reduction factor was applied to the tangent of the friction angle in effective stresses ($\tan \varphi'$) only. The critical failure mechanism is that for which instability is achieved for the lowest value of the SRF. It was concluded that a constant value of SRF was obtained for an inclined rock mass joint dipping from 10° to 15° towards upstream. In the study presented here, a rock mass joint dipping 11° towards upstream was considered.

4.5. Numerical results and verification of the ULS of sliding

Analysis was carried out with the reservoir at the RWL, both with constant joint hydraulic aperture and taking into account the HM interaction. Figure 10 shows a detail of a dam and foundation deformation due to the simultaneous effect of dam weight, hydrostatic loading and flow, in one of the studied load cases of the persistent design situation. In this figure, in which block deformation is magnified 3000 times, both horizontal and vertical crest displacements are represented.

Figure 11 shows the total head contours in the foundation, in both the persistent and accidental (clogging of the drainage system) design situations. The main aim of drainage is to reduce the hydraulic head, therefore, clogging of the drainage system is a serious problem, as this leads to an increase in water pressures within the dam foundation, and thus along the base of the dam.

The variation in water pressures along the dam–foundation joint and along the rock mass joint downstream from the dam dipping 11° towards upstream is shown in Figure 12 (the failure surface is highlighted in black). Figure 12 also shows a comparison of water pressures along the base of the dam with both bi-linear and linear

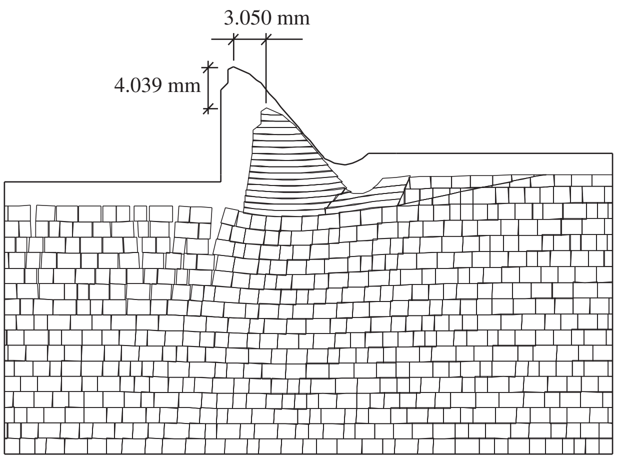


Figure 10. Block deformation (magnified 3000 times) due to dam weight, hydrostatic loading and flow [Combination 2 (C2); characteristic value of the friction angle in effective stresses of 32.3°].

uplift distribution, usually assumed in stability analysis with and without drainage systems, respectively. Figure 12(a) illustrates that variations in water pressures are highly dependent on the pressure at the drainage line. Water pressures calculated in the hydraulic analysis in which the HM effect is not taken into account (constant joint aperture) are very close to those given by the bilinear distribution of water pressures along the base of the dam, and are the same regardless of a_0, because the joint hydraulic aperture remains constant. Previous studies in which it was assumed that a_0 could vary between 0.05 and 0.2 mm showed that, upstream from the drainage line, water pressures increase as a_0 increases, and are lower than those obtained with constant joint aperture (Caldeira et al., 2013).

Downstream from the drainage line, water pressures obtained when the HM interaction is taken into account are higher than those obtained with constant joint aperture, and are thus, in this case, slightly higher than those given by the bilinear distribution of water pressures, which highlights the relevance of carrying out numerical analysis. Figure 12(b) shows the water pressures along the base of the dam with the drainage system clogged. Water pressures along the inclined rock mass joint downstream from the dam vary almost linearly between the pressure at the toe of the dam and that corresponding to the water height downstream from the dam.

Figure 13 depicts the variation in dam crest horizontal displacements during the process of reduction of the

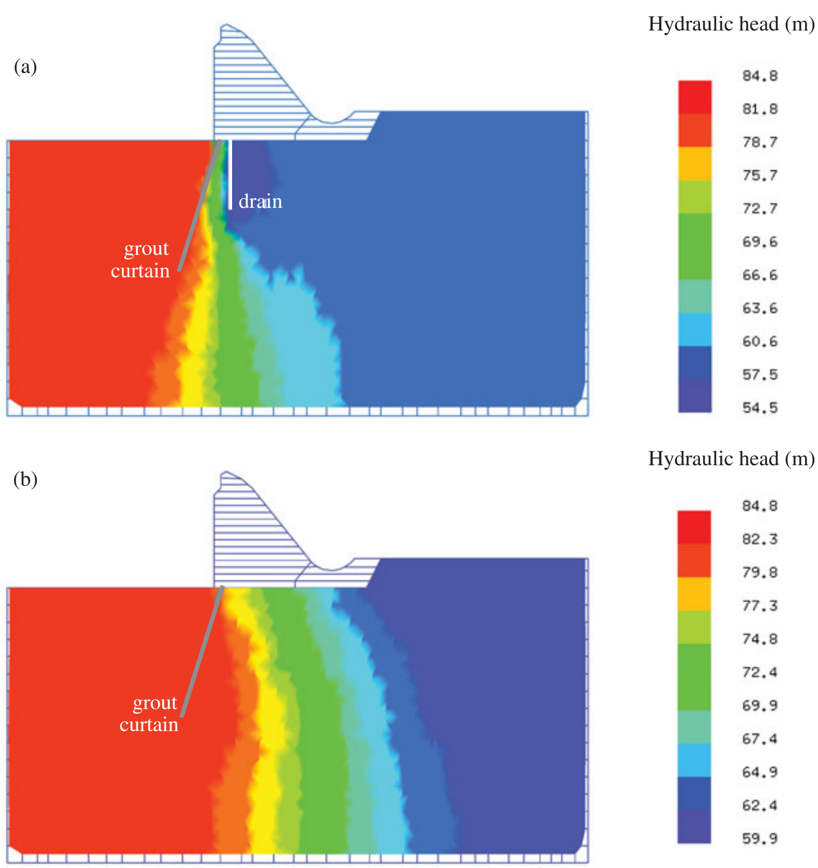


Figure 11. Total head contours for full reservoir in both design situations: (a) with the drainage system operating properly and (b) with the drainage system clogged.

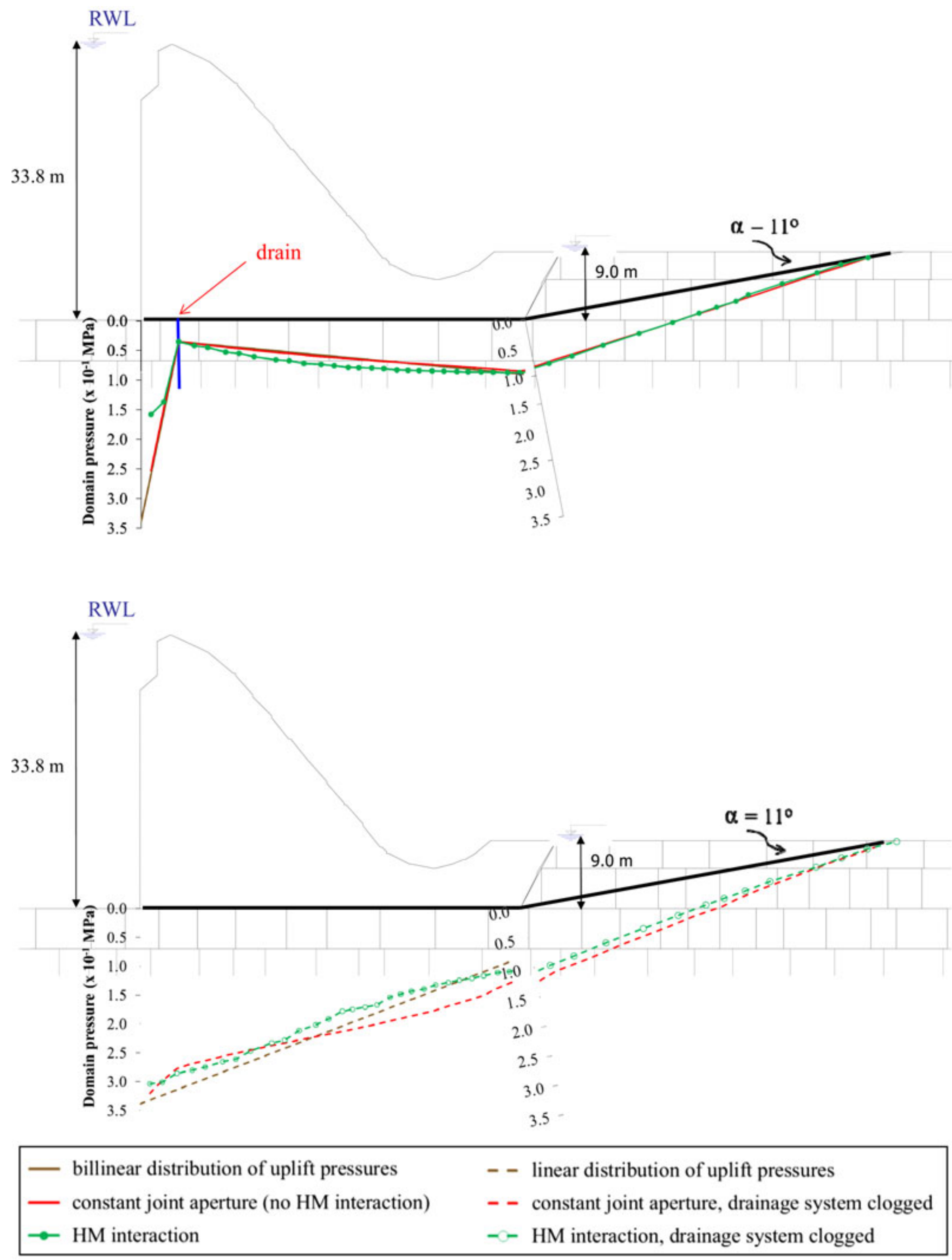


Figure 12. Water pressure along the dam–foundation joint and along the rock mass joint downstream from the dam: (a) dam with both grout and drainage curtains and (b) dam with the drainage system clogged.

tangent of the friction angle in effective stresses ($\phi'_{k,\mathrm{mean}} = 32.3°$), in which, for ease of analysis, friction angles in the x-axis are shown in reverse order. Figure analysis leads to the conclusion that, in this particular case, the effect of dam weight reduction (C1.1) on dam crest horizontal displacements is very close to that due to the increase in water pressures owing to complete clogging of the drainage system of the dam foundation (A), although in the latter accidental situation, failure is reached for a lower friction angle. Results obtained considering both permanent actions (dam weight and hydrostatic pressure on the upstream face of the dam) as unfavourable (C1.2) are very close to those obtained in Combination 2 of the persistent design situation (C2). It should be noted that load Combination C1.2 does not need to be taken into account in sliding stability analysis, as the dam weight always has a

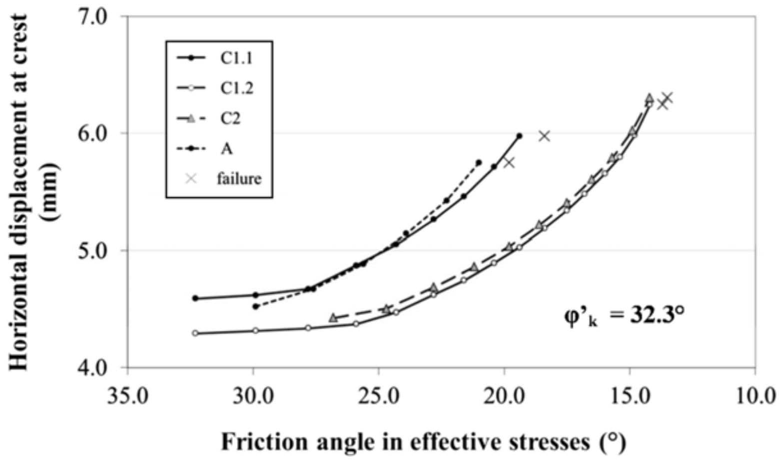


Figure 13. Variations in dam crest horizontal displacements during the SRF process, for the different load cases and design situations (characteristic value of the friction angle in effective stresses of 32.3°).

stabilising effect. Figure analysis shows evidence that a dam's SLS is not reached before the sliding ULS is attained. This conclusion is based on the very small horizontal displacement calculated for the failure indicator at the crest of the dam, when the foundation ULS is reached.

Table 2 presents the results of the LSD according to the DA1 of EC7, in both design situations (see Figure 1), for the different load cases (Figure 2) and for the different characteristic values of the friction angle in effective stresses. Figure 14 shows the variation of the ODF with the friction angle in effective stresses. In all the analysed cases, the ODF is greater than 1, indicating a safe design (the base sliding stability is ensured). In the example of Pedrógão dam, sliding stability is governed by the persistent design situation and load case C1.1. In this case, an increase in the ODF of 11.8% is obtained when $\phi'_{k,\mathrm{mean}}$ is used instead of $\phi'_{k,\mathrm{inf}}$. When spatial variability is taken into account this increase is 24.4%. However, if the scale of fluctuation was higher than 20 m, the increase in the ODF due to spatial variability would be lower than 11.8% as a result of the decrease of $\phi'_{k,\mathrm{var}}$ (see Figure 8).

It should be noted that in a dam without an operating drainage system, the ODF values would be lower than 1, indicating an unsafe design, as shown in Caldeira et al. (2013), because it must be considered to be a persistent design situation.

It can be observed that ODF absolute values obtained for this particular dam are higher than that prescribed in EC7. The conclusion could be drawn that the safety levels associated with dam design are larger than those used in other structures. However, this dam presents an unusual cross section as mentioned in Section 4.1. A large dam is a special structure, the failure of which may have severe consequences [class CC3 according to EN 1990 (CEN, 2002b)]. For this type of structure, a special reliability level is required (class RC3), for which a penalising factor must be applied to actions or resistances. A value of 1.1 is suggested in EN 1990 only for actions. Adopting this procedure, the minimum ODF value associated with $\phi'_{k,\mathrm{inf}}$ decreases from 1.19 to 1.08.

However, instead of applying factor 1.1 to γ_F for unfavourable actions, EN 1990 states that it is normally preferable to require high levels of quality control during construction, maintenance and operation. This is the usual procedure in large concrete dams and thus the ranges presented in Table 2 may be considered as final values. In EC7 DA1, the ULS is usually governed by Combi-

Table 2. Results of the LSD according to DA1of EC7.

Design situation	Permanent design situation					Accidental design situation
	Combination 1				Combination 2	Combination 2
Characteristic value of the friction angle in effective stresses	C1.1		C1.2		C2	A
	SRF	ODF	SRF	ODF	ODF	ODF
32.3°	1.80	1.33	2.50	1.85	2.00	1.50
29.7°	1.60	1.19	2.30	1.70	1.80	1.40
33.9°	2.00	1.48	2.70	2.00	2.20	1.60

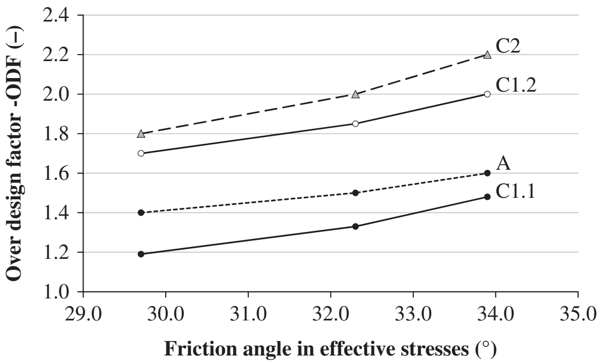


Figure 14. Variation of the ODF with the friction angle in effective stresses.

nation 2. Nonetheless, results analysis showed that in this case, despite the geotechnical character of the problem, the safety is controlled by Combination 1, considering the gravity load as favourable. This can be explained by the high hydrostatic pressures on the upstream face of the dam and by the high uplift pressures when compared with the geostructure's self-weight. This result can be considered relevant for further applications of the procedure for other gravity dams.

5. Conclusions

LSD is the current trend in both structural and geotechnical engineering, not only in Europe, but also in Australia, Canada, Japan and the USA. It is true that the modus operandi for justifying both the deterministic and LSD approaches is basically the same, that is the examination of loads, strength parameters and the safety verifications. However, in accordance with the LSD analysis, partial factors are used to weight actions, strengths and model factors, which take into account the uncertainties involved in the analysis.

The main advantage of this approach is that it provides a unique safety concept which may be applied to any kind of structure. It is significantly different from the traditional allowable stress and overall safety factor DAs, as it allows different limit states to be defined regarding safety and serviceability. In addition, LSD imposes a rigorous and systematic formalism on calculations, which will hopefully lead to standardising analytical procedure and unequivocal vocabulary. This formal structure will ensure that all analyses are conducted in the same pattern, leading to a standard practice that will allow the expected introduction of the main requirements for dam safety in the Eurocodes.

Regarding the foundations of concrete gravity dams, ULSs include overall and sliding instabilities; however, it is well known that the latter limit state controls safety in rock mass foundations. This study presents in detail the LSD procedures, proposed in Eurocodes, for concrete gravity dam sliding stability assessment. The application of these procedures to Pedrógão dam is presented, taking into account information regarding the characteristics of the foundation rock mass provided from tests carried out both in *in situ* and in the laboratory. The DEM and a two-dimensional nonlinear HM discontinuum model were used to analyse the sliding safety. The critical failure mechanism was identified using the SRF method. The same method was used to verify the ULS for both persistent and an accidental design situation involving the complete clogging of the drainage system of the dam foundation. Results led to the conclusion that the LSD methodology may be followed for dam foundation design with the partial factor values prescribed in EC7.

LSD requires the use of characteristic values of both loads and material strength parameters, which have to be carefully set. In the geotechnical context, the definition of the strength parameters is not unique and depends on the failure surface extent and on the spatial variability of the analysed property. Two different values are proposed in EC7, and the rational choice between them involves the evaluation of the scale of fluctuation of the property variation based on both *in situ* and laboratory data. Spatial variability is introduced in the study presented here as a means of obtaining an estimate of the characteristic value of the strength parameter. In fact, this value of the strength parameters should be averaged over the extent of the potential failure surface. The determination of the scale of fluctuation is often complex; however, as shown in the case of Pedrógão dam, the consideration of spatial variability could allow optimisation of the design as well as reduction of costs. This issue is an innovative approach in the rock mechanics field, which merits further research.

Dam stability analysis is accurately carried out with HM models in which the rock mass discontinuities are explicitly represented. The results of these models, however, depend mainly on the fracture pattern and on HM characterisation data, and therefore, detailed field data are required. The fracture pattern in the foundation of Pedrógão dam was assumed as a deterministic parameter and an unfavourable situation was considered, after a detailed study regarding the identification of the foundation failure mechanism. However, the application of this methodology to other dams may require carrying out of numerical analysis, taking into account different foundation fracture patterns.

The numerical analysis raises problems regarding the direct application of load partial factors due to the difficulty in distinguishing the favourable and unfavourable effects caused by different actions. The procedure to overcome this difficulty is presented in this paper. This study is part of an ongoing project regarding the application of a semi-probabilistic approach to the safety evaluation of large concrete dams, in accordance with the safety philosophy on which Eurocodes are based.

Acknowledgements

Thanks are due to the dam owner, EDIA, Empresa de Desenvolvimento e Infra-Estruturas do Alqueva, SA for permission to publish data related to Pedrógão dam.

Notes

1. Email: laurac@lnec.pt
2. Email: emanuelmaranhadasneves@gmail.com

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