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Safety evaluation of arch dams under seismic actions

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ABSTRACT: The safety evaluation of arch dams for seismic actions is usually made by a method of safety coefficients. However the random nature of the seismic actions and of the characteristics of the concrete suggests the use of a statistical method for the safety evaluation of dams under seismic vibrations.
The main purpose of this work is to present the different methods of safety evaluation of arch dams under seismic actions, deterministic or statistic. The connection between safety coefficients and failure probability is also presented.

1 INTRODUCTION

In the safety evaluation of arch dams, current and failure scenarios must be considered. The current scenarios correspond to normal conditions for use of the structures and the failure scenarios are associated with an exceptional occurrence and correspond to a high potential risk.

Three levels are recognized for safety evaluation of structures [1].

Level 3 is the "reference" level, where the actions and strengths are statistically defined and the failure probability is computed. It implies obtaining several samples from the structure (according to the probability distribution of strengths and deformability) and several samples from the earthquakes (according to a seismic risk model) and computing the failure probability (number of failures divided by number of samples). The safety evaluation at level 3 may be simplified by using several one-dimensional problems instead of a multidimensional one. The failure is thus considered to depend on a control variable (for instance the maximum strain on a surface), the action is characterized by one variable (for instance the peak acceleration, for the seismic action) and the failure probability is computed from the integral of the product of the strength density function by the difference from unity of the distribution function of the action effect.

At level 2 several one dimensional problems are also used instead of a multidimensional one, but the probability of failure is computed from reliability indices assuming a normal distribution for the safety margin (difference between the value of strength and the value of the action effect).

At level 1 the strengths and the action effects (represented for instance by the characteristic values) are compared by schemes depending on partial safety coefficients.

2 SAFETY COEFFICIENTS

At level 1 the strengths and the action effects are compared by schemes such as the following

$$S_{dE} \leq R_d \qquad (1)$$

where

$$S_{dE} = \gamma_{gi} S(G_i) + \gamma_{qE} S(Q_E) + \psi_{Ej} S(Q_j) \qquad (2)$$

$$(i=1,2,...)\ ;\ j \neq E$$

S_{dE} means the value of the control variable when the earthquake is the main action, R_d the value of the control variable when the strength is minimized, γ_{gi} is the partial safety coefficient of the constant action i, γ_{qE} the partial safety coefficient of the earthquake action, ψ_{Ej} the coefficient owing to the combination of the earthquake action with the variable action j, Q_j the value of the action variable j, G_i the value of the constant action i, Q_E the value of the earthquake action and S( ) the action effect.

In the safety evaluation of concrete dams for earthquake actions two levels for the action are usually considered: i) at the first level (which corresponds to current scenarios) is considered the Operating Basis Earthquake (OBE). It is an earthquake whose peak acceleration has a probability of 0.5 of being exceeded during the lifetime of the dam. For this earthquake the dam must remain operational; ii) at the second level (that corresponds to failure scenarios) is considered the Maximum Design Earthquake (MDE). It is the earthquake with maximum peak acceleration expected at the dam site (or, at least, with a very low probability of to be exceeded during the life time of the dam). For this earthquake the dam must suffer no failure.

3 STATISTIC SAFETY EVALUATION (LEVEL 3 SIMPLIFIED)

In order for failure of the dam to occur, it is necessary that after loss of cohesion of the concrete in a significative volume of the dam the static actions (dead load and hydrostatic pressure) can no longer be supported.

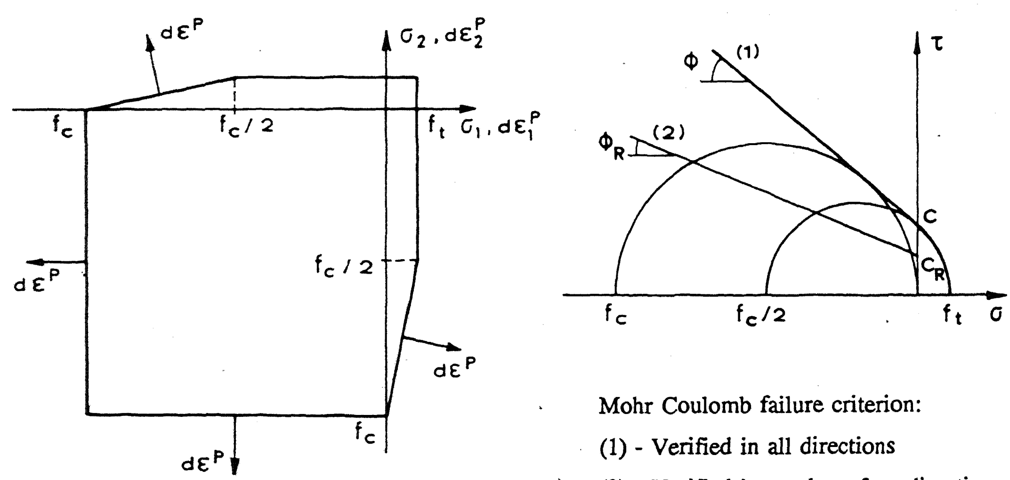
That situation may occur when through the whole thickness of a section there is complete loss of cohesion (owing to the fact of the limit strain of the concrete to being reached). Since the peak strains due to an earthquake are first reached at the upstream and downstream faces of an arch dam, loss of cohesion through the whole thickness of a section occurs when the peak strain reaches the limit strain at the middle surface. It therefore seems that the maximum peak strain at the middle surface is an efficient control variable for failure of arch dams.

The vulnerability function gives the control variable defined above, function of the peak acceleration of the earthquake. Computation of the vulnerability functions depends on the model assumptions considered. A thin shell model for the structure (taken to be elastically supported) was assumed coupled to a 3D reservoir and taking into account the compressibility of the water [2].

An elasto-plastic behavior is assumed for the concrete [3] with a plastic associated flow rule for shear ruptures, where the yielding surface is given by the Mohr Coulomb criterion, Fig. 1 (for plane stress, as occurs in each layer of a thin shell). Brittle tension ruptures originate cracks. Stresses vanish on the faces of opened cracks and comply with the Mohr Coulomb criterion (with residual strengths) on the faces of closed cracks.

It was assumed that the earthquake action was characterized by one variable - the peak ground acceleration. Ii is assumed that the peak ground accelerations have a type I distribution function

$$F_I(Y) = e^{-e^{-\alpha(Y-\mu)}} \qquad (3)$$



Mohr Coulomb failure criterion:

(1) - Verified in all directions

(2) - Verified in crack surface directions

$d\varepsilon^P$ - Plastic strain increment

C - Cohesion

ϕ - Internal friction angle

f_c - Compressive strength

C_R - Crack residual cohesion

ϕ_R - Crack residual internal friction angle

f_t - Tensile strength

Fig. 1 - Yielding surface. Mohr Coulomb criterion.

where α and μ are parameters computed from a seismic risk model [4]. The distribution of maximum peak strains at the middle surface (action effect) may be computed from the distribution function of peak ground accelerations.

$$F_S(\varepsilon) = F_A(\nu^{-1}(\varepsilon)) \tag{4}$$

where

$$\varepsilon = \nu(a) \; ; \; a = \nu^{-1}(\varepsilon) \tag{5}$$

v(a) is the vulnerability function

It was assumed that the simple compression strength has a log-normal probability density function

$$f(R) = \frac{1}{\sqrt{2\pi}\delta R} e^{-\frac{\ln^2(R/\beta)}{2\delta^2}} \tag{6}$$

where the mean value is

$$\bar{R} = \beta e^{\delta^2/2} \tag{7}$$

and the standard deviation is

$$\sigma = \beta[\, e^{\delta^2}(e^{\delta^2}-1)\,]^{1/2} \tag{8}$$

It was assumed that the limit strain has a log-normal probability density function $f_R(\varepsilon)$ with a lower limit at the strain that corresponds to the yield stress. We thus have an one-dimensional problem, the failure probability being computed from (Fig.2):

$$P_f = P(S>R) = \int_0^\infty (1 - F_S(\varepsilon)) f_R(\varepsilon) d\varepsilon \tag{9}$$

For each simple compression strength there is corresponding failure probability. Assuming:

$$P_f = g(R) = e^{a+bR} \tag{10}$$

where a and b are constants, the density probability function of failure probability is computed from

$$f(P_f) = f(R) / \left| \frac{d\, g(R)}{dR} \right| \tag{11}$$

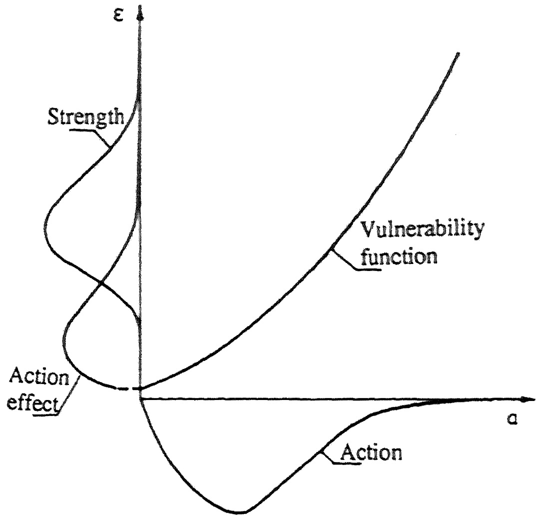


Fig.2 - One-dimensional model for the computation of failure probability.

4 EXAMPLE

4.1 The method developed was used in the analysis of Ciria 5 dam [5] considered to be located at three different sites of high seismicity: i) Site 1 is Faro in South Portugal; ii) Site 2 is Kalaritikos in Greece; ii) Site 3 is California in USA.

The finite element mesh used in the discretization of the dam and the reservoir is presented in Fig.3.

The probability density function assumed for the simple compression strength of the concrete has a mean value of 45 MPa and a coefficient of variation of 0.2. The value f_c=45 MPa corresponds to the strength of the concrete for short duration loads with a static strength of f_c=30 MPa. On the other hand f_c=30 MPa corresponds approximately to the fractile 0,05. This means that f_c=45 MPa and f_c=30 MPa are the two values of strength chosen in definition of the function that establishes the link between the simple compression strength and the probability of failure.

The probability density function assumed for the limit strain has a mean value of 0.3%. Two values were assumed for the coefficient of variation: 0.2 and 0.4.

The elasticity modulus and the Poisson coefficient were assumed to be deterministic and respectively equal to 30 GPa and 0.2. The simple tension strength was assumed to be f_t=0.1f_c. The damping was assumed to be equal to 5% of critical damping, for all vibration modes.

The assumed shape of the density power spectrum, presented in Fig.3, is suggested in the Portuguese code, RSA [6], for near earthquake, in rock. From the density power spectrum were generated stationary

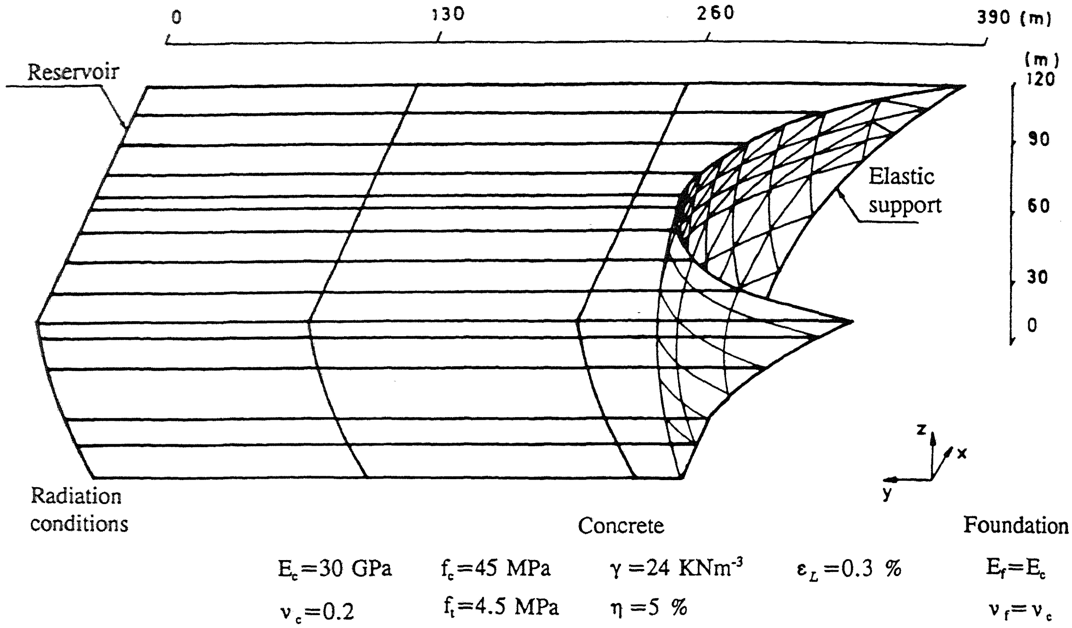


Fig.3 - Finite element model of the structure and reservoir.

accelerograms of 10 s duration. The accelerograms were scaled for different peak accelerations.

From the risk studies [4] were computed the peak accelerations for return periods of 1000 and 10000 years, the fractile 0.95 and the parameters α and μ that characterize the type I distribution functions of the peak accelerations at the three sites. These values are presented in Tab.1.

4.2 The assumed finite element model was used in the computation of the vulnerability functions for the simple compression strengths f_c=45 MPa and f_c=30 MPa. The limit strain of 0.3% is reached at the middle surface (failure of the dam) for a peak ground acceleration of 2.4g when f_c=30 MPa and of 2.8g when f_c=45 MPa.

The density probability functions of: i) the peak ground accelerations during 100 years (f_A); ii) the maximum peak strains at the middle surface generated by the earthquakes (f_S); iii) the limit strains (f_R), are presented in Fig.5. The vulnerability function, for f_c=30 MPa, is also presented in Fig.5.

From this one dimensional problem were computed the failure probabilities (P_f) at the three sites assuming two different variation coefficients (c_ε) of the limit strain (Tab.2).

Tab.3 shows the median and the 0.95 fractile of the probability density function of the failure probability. These values are compared with the safety coefficient defined as the ratio between the peak ground acceleration that originates failure of the dam (assuming characteristic strengths) and the peak ground acceleration with a return period of 10000 years.

Since no failure can occur with the MDE, the safety

Tab.1 - Maximum peak accelerations for return periods of 10000 and 100 years. α and μ parameters. Fractile 0.95 of the maximum peak acceleration in 100 years

SITE	T=10000years a (cm/s^2)	T=1000years a (cm/s^2)	α (s^2/cm)	μ (cm/s^2)	$a_{0,95}$ (cm/s^2)
1	400	230	0,01363	60,82	279
2	545	320	0,01032	94.82	383
3	710	350	0,006213	-33,96	444

MODEL FOR THE COMPUTATION OF THE FAILURE PROBABILITY

f_c = 30 MPa

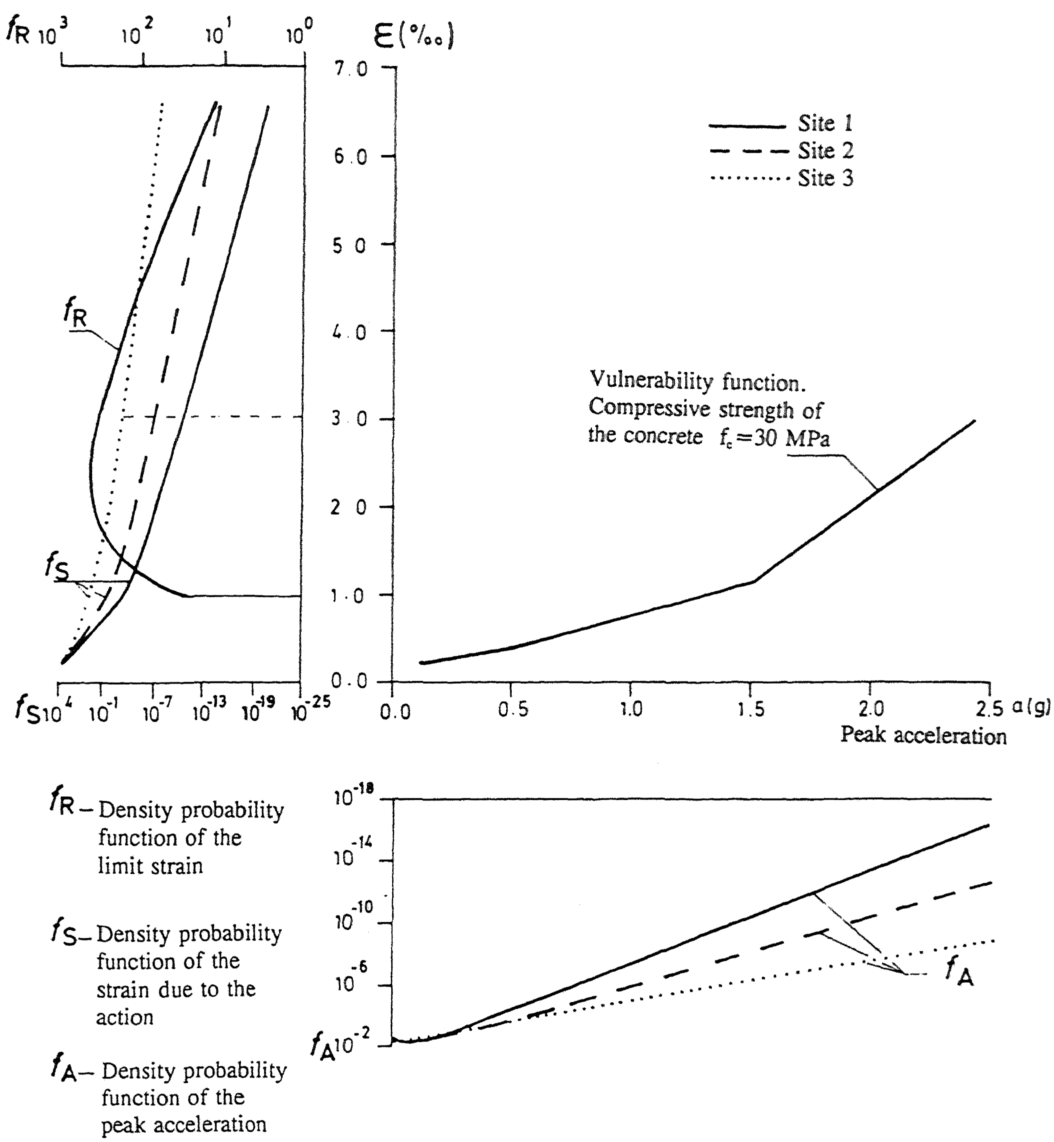


Fig.4 - One-dimensional model for the computation of the failure probability. Dam located at the three sites.

coefficient must be greater than 1. Owing to the uncertainties associated with the computation of the MDE (for instance the earthquake with peak acceleration that has a probability of 10^{-6} of not being exceeded during the life time of the dam, 100 years, and thus with a return period of 10^8 years) and to the uncertainties associated with the computation of the peak ground acceleration that originates failure, the use of a safety coefficient (defined as above) not lower than 3, is suggested.

5 CONCLUSION

Arch dams present a very good performance under seismic actions. Indeed, we computed that the ground

Tab.2 - Failure probability of the dam for fc=30 MPa and fc=45 MPa. Limit strains with variation coefficients of 0.2 and 0.4

SITE	P_f $c_\epsilon=0,4$ $f_c=30$ MPa	P_f $c_\epsilon=0,4$ $f_c=45$ MPa	P_f $c_\epsilon=0,2$ $f_c=30$ MPa	P_f $c_\epsilon=0,2$ $f_c=45$ MPa
1	$9,9 \times 10^{-11}$	$1,8 \times 10^{-14}$	$1,2 \times 10^{-12}$	$9,8 \times 10^{-16}$
2	$1,8 \times 10^{-8}$	$3,0 \times 10^{-11}$	$7,3 \times 10^{-10}$	$2,0 \times 10^{-12}$
3	$4,5 \times 10^{-6}$	$8,3 \times 10^{-8}$	$7,9 \times 10^{-7}$	$8,1 \times 10^{-9}$

Tab.3 - Median and 0.95 fractile of the probability density funtion of the failure probability. Limit strains with variation coefficients of 0.4 and 0.2. Safety coefficients.

SITE	$F_{0,5}$ $C_\epsilon=0,4$	$F_{0,95}$ $C_\epsilon=0,4$	$F_{0,5}$ $C_\epsilon=0,2$	$F_{0,95}$ $C_\epsilon=0,2$	s
1	$3,0\times10^{-14}$	$3,4\times10^{-11}$	$1,5\times10^{-15}$	$5,0\times10^{-13}$	6,0
2	$4,2\times10^{-11}$	$8,3\times10^{-9}$	$2,8\times10^{-12}$	$3,5\times10^{-10}$	4,4
3	$1,1\times10^{-7}$	$2,8\times10^{-6}$	$1,1\times10^{-8}$	$4,5\times10^{-7}$	3,4

acceleration that originates failure of the studied dam is greater than 2. This is in accordance with experience, as may be seem from Pacoima dam, which suffered an earthquake with a 1.2g peak ground acceleration with minor damage.

The 0.95 fractile of the probability density function of the failure probability in site 3, which has one of the highest seismicity indices in the world (the peak ground acceleration that corresponds to a 10^4 years return period is 0.7g), is about 10^{-6} to 10^{-7} and the respective safety coefficient is about 3, which confirms the very good performance of arch dams under seismic actions.

The methodology presented may be used in computing the probability of unsatisfactory performance under seismic actions for current scenarios. Indeed, for the control variable, we may use the maximum peak tension in the middle surface for taking into account the cracking through the whole thickness.

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