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Seismic Hazard and Structural Analysis of the Concrete Arch Dam (Rules Dam on Guadalfeo River)

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Abstract

The aim of this paper is to describe the seismic hazard performance on the site of Rules Dam, in Granada province (southern Spain), and the seismic influence on this body's dam, as well as on its critical elements, the reservoir and the interaction fluid-structure. The seismic hazard defines the Deterministic Seismic Hazard Assessment (DSHA) and the Probabilistic Seismic Hazard Assessment (PSHA), which are important to calculate the Safety Evaluation Earthquake (SEE) and the Operating Basis Earthquake (OBE), respectively. This recent seismogenic zone provides important data to do the analysis, such as regional geologic setting, seismic history and seismology. In the Spanish code the Peak Ground Acceleration (PGA) for this area is 0.17 g, however in the current analysis the greatest soil acceleration registered is 0.35 g, which is about twice the value. Three accelerograms (controlling earthquakes), by using the Engineering Strong-Motion database, have been chosen to identify the seism's main characteristics. The dam analysis using different software needs to be done to calculate the vibration periods, the hydrodynamic pressure and the maximal vertical stresses. Time-history analyses have been made to analyze the consequences of a dam failure and to estimate minor damage acceptance. The analyses show that the stresses exceed the tensile maximum allowed creating plastic hinges. There are other factors which can affect the dam's behavior such as the vertical component of the earthquake and the silt in the reservoir bottom. The concrete arch gravity dam needs to be modeled in two- and three-dimensions, in accordance to classic theoretical method and current codes, considering its big dimensions (length of the crest: 620 m; radius: 500 m; area of the reservoir whit a operating level: 308 Ha). A dam is an extremely strategic work which needs to be carefully designed to avoid environmental damage to water reservoirs and nearby facilities and for human security. Given that the recent sources of hazard in Spain are from 2015, it would be advisable to reassess the seismic hazard particularly related to existing dams of category A (Spanish code) in areas of high seismicity.

Keywords: Seismic hazard; concrete arch gravity dam; dynamic analysis.

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1. Introduction

The focus of this paper is to describe the seismic risk on Rules dam site and its influence in relation to the interpretation of the fluid-structure's dynamical problems. The seismic hazard of a new seismogenic zone is useful to recalculate the actions on the dam. In particular, deterministic and probabilistic analysis will be made to define the SEE and OBE, respectively, using four different attenuation equations. In the Engineering Strong-Motion database three accelerograms have been chosen to do a time-history analysis and to identify the main characteristics of the controlling earthquakes. The dam is modeled by two- and three-dimensions to account for the interaction of the rock foundation and water. The dam analysis in three dimensions defines the modal analysis and the fundamental dam period (0.284 s). In two dimensions, considering a triangular dam shape, the vertical compressive stresses of the element in the bottom of the upstream face have been calculated. It is useful to do the dynamic analysis to know the cyclic behavior of the material subjected to stresses. During the seism, the water in front of the structure exerts a cyclic dynamic load on the wall and the critical mode occurs during the phase when pressure goes in direction to the wall. This phenomenon added to the inertia dam can reach intense stresses. The study of tensional states is necessary to analyze the consequences of a dam failure and to estimate minor damage acceptance.

2. Seismic hazard of a new seismogenic zone

The seismic hazard assessment has been made on basic criteria, as the Cornell method [10], which is based on (i) earthquake recurrence time following a Poisson process and on (ii) event magnitude that is exponentially distributed by Gutenberg and Richter. The model used includes a total of 11 zones and considers a radius of 150 km from the dam site. The coordinates of the dam are: 36.51° (latitude), - 3.29° (longitude). The mean annual rate of exceedance and the b-values have been taken from the new Spanish seismogenic source model but they have been opportunely recalculated taking in to account the many uncertainties of the procedure [13]. The dam is situated in a rocky stratigraphic profile with an average shear wave velocity over 750 m/s.

2.1. SEE and OBE definition

From the disaggregation analysis – which is made to separate the magnitude and distance contribution that has generated acceleration –, for a dam fundamental period (T_d), the following numbers are obtained: Mw = 5.9 (magnitude moment) and $R_{epi.}$ = 7.5 km (epicentral distance). In Fig. 1 (right) the Pseudo-Spectra Accelerations (PSA) with these values are shown (T is the structural period). The four attenuation relations do not use the same parameters, therefore the values have been adapted (see [6,7,8,9] for the attenuation relations). In Fig. 1 (left) annual probability of exceedance is shown – expressed in terms of return period – in function of PGA. The differences between the curves depend mainly on attenuation equations used and on their standard deviations: when standard deviation decreases the return period increases. The standard deviation values used range from ± 0.19 to ± 0.29 (for the analytic analysis zero has been used). The green curve values, in Fig. 1 (left), are higher because the equation has not been well-constrained for low magnitudes; therefore the curve overestimates higher T_r. This analysis, through a probabilistic approach, has been made only for the seismogenic zone where the dam is: ZS35 [11].

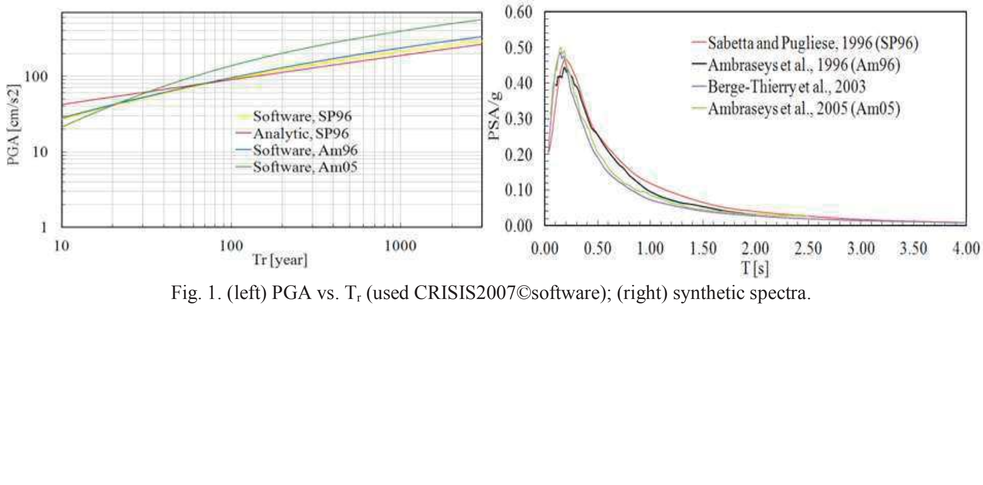

Fig. 1. (left) PGA vs. T_r (used CRISIS2007©software); (right) synthetic spectra.

In Table 1 there are three earthquakes chosen according to disaggregation analysis and the following parameters: M_s (surface-wave magnitude), D_{Ia} (significant duration), R_{jb} (distance from the surface projection of the fault) and the style of fault ruptures that generate the seism. The PGA used in the analysis is 0.25 g. This value is in accordance to literature [16]. The events recorded have been made according to three instrumental orientations (east-west, north-south and up-down) but in Table 1 only the heaviest ones are shown.

Table 1. Characteristic of controlling earthquakes [12] for time-history analyses.

Event, date, time	M_w	M_s	Depth [km]	PGA [cm/s^2]	D_{Ia} [s]	$R_{epi.}$ [km]	R_{jb} [km]	Fault
Greece, 13/09/1986, 17:24:34	5.9	5.7	27.6	234.04	5.17	6.6	-	Normal
Italy, 26/09/1997, 09:40:24	6.0	5.9	5.70	201.39	11.77	4.8	1.63	Normal
Italy, 29/05/2012, 07:00:02	6.0	5.9	8.07	232.12	11.96	9.9	-	Thrust

To design the dam is necessary to calculate the SEE and OBE levels. In the former the damage can be accepted but uncontrolled release of water should not. In the latter these should be none or insignificant damage to the dam. To calculate the SEE the deterministic approach is used; instead, to calculate the OBE the probabilistic approach is used. There are some differences between the two procedures, on important one is that the DSHA does not account for the frequency of earthquake occurrence. In Fig. 2 (left) the Uniform Hazard Spectra (UHS) and controlling earthquakes considering all zones are shown. The UHS provides the accelerations in function of structure period for a fixed return period (in this case: T_r = 1000 years). In Fig. 2 (right) the deterministic spectra obtained for four historical earthquakes by two different attenuation relations are displayed.

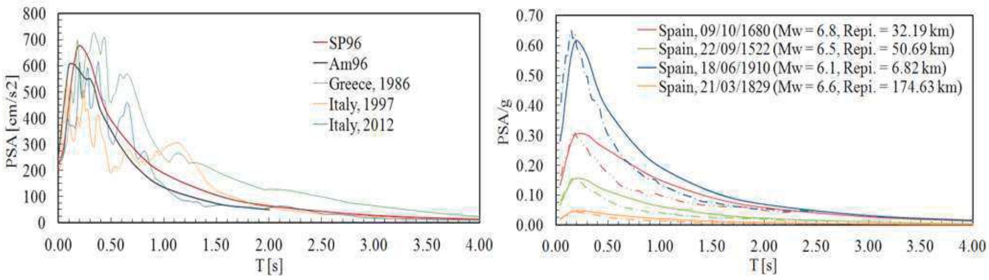

Fig. 2. (left) OBE and controlling earthquakes; (right) SEE by SP96 (in dashed line by Am05).

In the Spanish code [15] the PGA for this area is 0.17 g, however, in the full analysis the soil greatest acceleration registered is 341.72 cm/s^2 which is about twice.

3. Case of study: Rules dam

The Rules dam is situated in southern Spain on the Guadalfeo River in the Granada province. It is an important concrete arch-gravity dam with single curvature in plan, with 620 m of crown length and with radius of 500 m. The maximum height of the vertical cantilever is 130 m and the downstream slope face is 1:0.60. The dam is formed by 32 blocks in total (see Fig. 3). The capacity reservoir for on operating level is 117.07 Hm^3 for a depth (H) of 113 m.

3.1. Direct modeling and analysis

It is necessary to model the arch-dam in two- and three-dimensions because there is an interaction between arch and cantilever units: the load actions create movements that generally consist of three translational and three rotational components. Vertical movements and rotations in vertical tangential planes are considerate negligible [1]. Another important consideration is the interaction between the dam and the rock, in particular, in the abutments where the rock creates a force concentration against the dam. This effect will be not shown in this paper.

Fig. 3. (left) Rules dam; (right) dam 3D model and some data – SAP2000©software.

The assumptions adopted to model the dam and to calculate the pressures are in the references [4,5]. Regarding them, it is possible to do some considerations: the upstream face of the dam is not vertical but has inclination of 10,20°, which reduce the horizontal hydrodynamic forces. When water is assumed as compressible, the pressures depend on frequency; in this case, when the natural frequency of the reservoir is close to the natural frequency of the dam-reservoir system, the $P_{hyd,\ compressible}$ (see Table 2) is 80.72 times greater. Considering the damping, the result is similar, as the range of the damped dam, reservoir and foundation is not large: 2.0 % - 8.5 %. In Table 2 the $P_{hyd,\ incompressible}$ are also shown (C_w tends to infinity). The numerical simulation has been made by Finite Element Method (FEM) and the gravity method. The FEM has been used for two- and three-dimensions; instead, the gravity method has been used only for 2D analysis. The last method is based on rigid body equilibrium and on beam theory. In the FEM analysis the solid elements have a dimension of 3.00 m per 2.50 m to account for the monolith joints in accordance to literature [2]. The solid element has eight nodes; each one has three translation degrees. The presence of the monolith joints is fundamental to study the nonlinear effects, which are: concrete cracking (the crack opens and closes during the earthquake), water cavitation, temperature, horizontal and vertical construction joints opening during earthquake shaking, formation of plastic hinges, geometrical nonlinearity and sliding on the base [3]. In this example, the input data used are: E_c = 44.40 GPa (modulus of elasticity of the concrete), γ_c = 24 kN/m^3 (mass density of the concrete[1]), ν_c = 0.20 (Poisson's ratio of the concrete), ξ_d = 5.0 % (dam damping), E_f = 41.55 GPa (modulus of elasticity of the foundation rock), γ_f = 27.47 kN/m^3 (mass density of the foundation rock), ν_f = 0.33 (Poisson's ratio of foundation rock), ξ_s = 8.5 % (system damping), T_s = 0.393 s (system fundamental period), γ_w = 10 kN/m^3 (mass density of water), C_w = 1438 m/s (velocity of pressure waves).

3.2. Results: fluid-structure interaction

In Table 2 the hydrodynamic and hydrostatic (P_{hys}) analytical pressures along the height of the dam are shown (the y axis is zero in the bottom of the reservoir). In the fourth column (Relative Error) there are the differences between the hydrodynamic pressures calculated analytically and by CADAM2000©software ($P_{hyd,\ software}$). The mainly difference depends on the fact that the first pressure is obtained considering a parabolic distribution for a rigid dam while the second pressure take into account the dam deformation, i.e. the acceleration of the dam in the form of vibrations (in this case, the first three modes have been considered). The analytical pressures have been calculated idealizing the dam as a triangular shape because the transversal behavior is similar to a thick gravity dam with a large thickness of the base. The pressures P_{hyd} have been calculated using the wave reflection coefficient α_w = 0.41. It is the ratio of the amplitude of the reflected hydrodynamic pressure wave and the amplitude of a vertical propagating pressure wave incident on the reservoir bottom. This coefficient indicates that the waves are partially reflected in the reservoir and partially transmitted into the substrate. It depends on the impedance, i.e. the dynamic stiffness between the layer of the reservoir and the layer of the rocks. This coefficient identifies the complex interface forces in the frequency domain between both layers. When this coefficient is one (rigid foundation) the pressure has the same values of the analytical $P_{hyd,\ compressible}$. Another part of the energy is lost due to radiation of pressure waves in the upstream direction. The hydrostatic pressure is affected by the increase or reduction

[1]In the 3D analysis, to consider the presence of the galleries and drains in the body's dam, the mass density of the concrete is less than 14 %.

(depending on the acceleration direction) of the effective volumetric weigh of water. According to d'Alembert principle, when the acceleration is directed upwards the hydrostatic pressure decreases. There is a positive effect due to hydrodynamic pressure: in general, arch dams are stiffened by the pre-stress, therefore it is more advantageous to build them instead of constructing gravity dams [17]. The equations used to calculate the hydrodynamic pressure with compressible and incompressible water are in the references [5].

Table 2. Results of the analysis. The upstream pressures are expressed in kN/m^2.

y/H	$P_{hyd,\ software}$	$P_{hyd,\ compressible}$	Relative Error	P_{hys}	$P_{hyd,\ incompressible}$	[2]P_{hyd}	[2]$P_{hyd,\ vertical}$	P_{hyd}/P_{hys}
0.95	17.15	22.21	1.30	58.36	11.88	25.20	25.45	0.43
0.79	53.50	76.73	1.44	233.38	46.72	99.10	100.54	0.42
0.58	100.91	118.46	1.17	408.41	88.37	187.47	193.18	0.46
0.42	149.07	167.50	1.12	641.79	113.54	240.86	253.07	0.38
0.21	242.10	282.48	1.17	875.16	136.08	288.68	315.11	0.33
0.00	337.79	350.55	1.04	1108.53	143.88	305.22	352.42	0.28

Besides, to determine the sliding, overturning and uplifting of the stability analysis, it is also necessary to calculate the cantilever and the arch stresses. Figure 4 shows the time-history of the stresses (σ_y) of the upstream bottom element of the vertical cantilever of the higher block of the dam (see Fig. 3). Figure 4 shows the case of empty and full reservoirs and both dynamic actions: seismic deadweight (including self-weight) and hydrodynamics. For the empty reservoir the accelerations of the first event (see Table 1) has been used. The "unfavorable" case, obtained through the acceleration of the third event, means that all forces have the same sign. It is possible to see that there is little difference between the analytical and computational analysis.

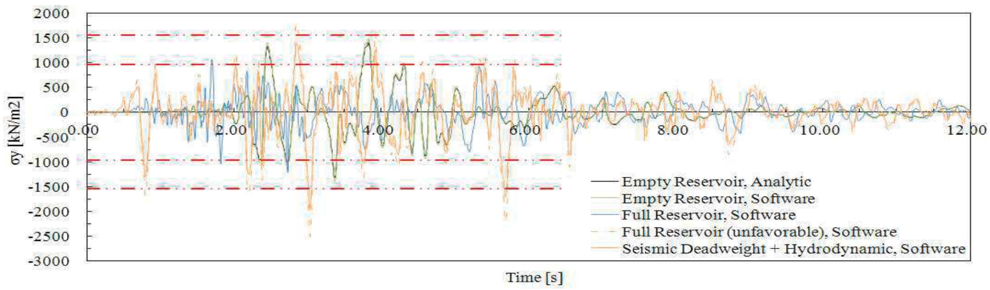

Fig. 4. Time-history of the analytic and computational analysis (Seismosignal©software).

The full reservoir analysis of the second event, by using software, demonstrates that the stresses are lower due to the hydrostatic and uplift negative stresses, but during the seism, uplift pressures within the crack can be assumed to be zero, therefore the σ_y total increases. The stress analysis determines the potential crack and the plastic hinge formation. For each vibration it accumulates plastic deformations producing a hysteretic behavior that depends on dissipated energy. From Eurocode 2 [14] the tensile maximum value adopted that generates significant nonlinear plastic deformations is f_{ctd} = 1547 kN/m^2 (dashed red line in Fig. 4). In Fig. 4 other tensile strength of the concrete with lower characteristics (f_{ctd} = 960 kN/m^2) is shown – this type of the concrete can be used for gravity dams. When the stress exceeds this value it is necessary the nonlinear analysis to complete the seismic evaluation and to know the capacity of the structure to cumulate the inelastic deformations. This consideration, in the original project of the dam, may have not been made accurately since that the structure was built earlier (in 2003) than the modern seismogenic zone.

[2]Hydrodynamic pressure (horizontal and vertical) with effects of reservoir bottom absorption [4].

4. Conclusions and future research

In this paper a complete seismic hazard of the dam site and a dynamic analysis to define the stresses in the body's dam has been made. The first consideration is that, in the Spanish code, the PGA that should be used is 0.17 g; however, for deterministic analysis the value is 0.25 g, i.e. 1.47 greater. Considering all the analyses the value is 2.05 greater. Using these acceleration values, the dynamic analysis has been made to determine the potential crack and the formation of plastic hinges. It is possible to see that for full reservoir (unfavorable) the stress values exceed the tensile maximum value that generates nonlinear deformations. Since that the dam was built in 2003 while the new seismogenic zones were made in 2015, these considerations may not have been taken into account in the original project. Therefore it would be advisable to reassess the seismic hazard, particularly in relation to existing dams of category A (Spanish code) in areas of high seismicity. This paper also aimed to encourage the reduction of the gap between theory research and practical engineering. A dam is a strategic structure which needs to be carefully designed to avoid environmental damage to water reservoirs and to maintain human security, for this the authors will develop other papers about these issues: stochastic dynamic analysis and Bayesian probabilistic method.

Acknowledgements

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