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SEISMIC CAPACITY ASSESSMENT OF KOURIS EARTH DAM

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ABSTRACT

We investigate the seismic capacity of Kouris dam in Cyprus. Kouris dam was built in the mid-eighties and is the largest and the most important dam of the island. Due to the significance of the dam, its seismic capacity should be re-assessed according to current analysis methods in order to determine whether it meets a set of contemporary safety criteria using updated seismological data. We first discuss the properties of the dam and we critically assess all available data. An appropriate finite element model of the embankment dam is created with the aid of PLAXIS 2D software, while the seismic capacity of the dam is assessed using the pseudo-static method. Residual deformations are also estimated using Newmark's sliding block method. A series of parametric investigations were carried out in order to determine the dam safety factor against various seismic loading scenarios. The dam is found to satisfy the criteria set by ICOLD (International Commission for Large Dams).

INTRODUCTION

Dams are structures of paramount importance since their possible failure would have severe consequences. Many of the existing dams have been designed in past decades using the state-of-the-art knowledge and methods at the time. Following recent experience from dam failures and according to new concepts that have emerged in earthquake engineering, re-evaluation of the seismic capacity of existing dams has been performed, or is under way, in several countries. In this study, we discuss the assessment of the seismic capacity of Kouris dam, the largest earth dam in the island of Cyprus. Kouris dam is located 15 km northwest of the city of Limassol. The construction of the dam lasted four years and was completed in 1989. Figure 1 shows the aerial view of Kouris lake and the embankment. The height of the crest is 110 meters and its width is 550m. The reservoir covers an area equal to 3.6 sq.km. with a maximum capacity of 115,000,000 m^3 of water. Due to its size, possible failure of the dam will have detrimental effect on the economy of Cyprus.

GEOMETRY AND MATERIAL PROPERTIES

The central cross-section of Kouris dam is shown in Figure 2. It is an earthdam with a clay core. The crest of the dam is at 253.5 above sea level (a.s.l.) and the maximum water level is at 247 a.s.l. The maximum water level is controlled with the aid of the spillway, which is placed at the right abutment (Figure 3). The project includes also a drainage tunnel and two intake towers. This study is limited to the seismic assessment of the embankment and hence the auxiliary structures are not discussed any further.

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The materials of the embankment are also shown in Figure 2. The upstream shell has two parts. The outer part comprises of river gravels which are free draining and the inner part comprises of terrace gravels which are also free draining but less permeable. Both materials have practically the same shear strength. The downstream shell is constructed with terrace gravels mixed with talus deposits. The shell materials are very coarse, with combined gravel and boulder content exceedeing 75% in all cases. In the upstream shell in particular, the boulder content is at least 45%. Thus soil liquefaction is deemed impossible regardless of the intensity of seismic shaking. Both shells were covered with rip-rap protection that is either crushed reef limestone or calcarenite. A cofferdam was first built (now part of the foot of the upstream shell) in order to divert the flow of the Kouris river. The riverbed deposits comprised of a layer of gravel 5m thick. For the core foundation, the in-situ river gravels were removed down to the bedrock. So, in the central cross-section of the dam, the clay core rests directly on the bedrock. At the abutments, all talus formations and top soil were removed. The maximum depth of the grout curtain is approximately 100m.

Figure 1: Aerial view of Kouris dam (Google Earth)

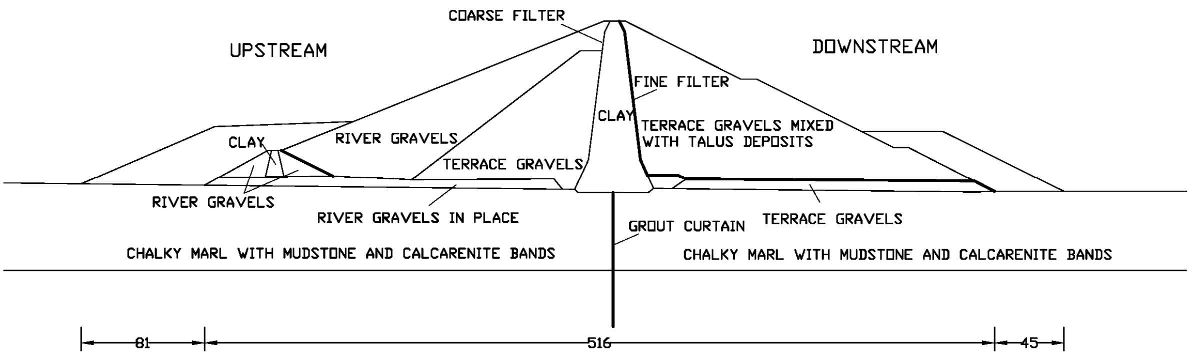


Figure 2: Central cross-section and materials of the embankment.

Kouris dam is founded on a weak rock mass, namely marly chalk containing mudstone and calcarenite bands. The berms, shown in Figure 2, were not considered in the initial design of the dam It was decided at a later stage to add berms at both the upstream and the downstream shells, probably due to stability concerns related to the presence of laminated mudstone bands inside the upper part of the Lophos beds geological formation. In absence of available data on the berms, it is reasonable to assume that the berm material is similar to that of their adjacent shell.

The dam was designed with the requirement that no irreversible deformations should occur for the operational earthquake. For the most credible earthquake (MCE), the dam should be able of retaining the reservoir, while deformations should be considered acceptable provided that no loss of life or excessive damage to downstream property occurs.

Figure 3: The downstream shell and right abutment.

Table 1 shows the parameters of the materials of the embankment. There are two sets of values for the strength parameters c and ϕ shown in Table 1. The values outside parentheses are those used in our finite element analyses, while those inside parentheses were used in the dam design in 1983 (SOGREAH 1983).

Table 1: Unit weight and strength parameters for embankment dam materials.

		Unit weight		Shear strength			
		moist	saturated	drained		undrained	
		kN/m^3	kN/m^3	c' (kPa)	ϕ'(°)	c_u (kPa)	ϕ_u(°)$_u$
River gravels	G	19.1	21.6	370 (0)	45 (43)	-	-
Terrace gravels	GT	19.1	21.6	340 (0)	42.5 (40)	-	-
Terrace gravels mixed with talus deposits	GM	22	22.8	340 (0)	42.5 (40)	-	-
Clay core material	C	19.7	20.3	0	29	170 (130)	5 (4)

For the river gravels (G) and the terrace gravels / talus deposits mixtures (GM), the strength parameter values used herein were measured in large shear box tests (SOGREAH 1983). The large shear box tests were performed at a range of normal effective stress consistent with the height and unit weights of the dam (0 to 4.5MPa). The ϕ and c were established based on a straight line approximation to the actual curved failure envelope. No test data was available for the terrace gravels (GT). Hence, it was assumed that the GT gravels have the same strength parameters as the GM gravels (the same assumption was adopted during the design of the dam). The strength parameter values inside the parentheses, which were used in the original design, were estimated based on triaxial tests. However, the triaxial tests had the limitation that the maximum particle size allowed in the specimens was 17mm. leaving out of the specimen approximately 60 to 80% of the coarser fraction. As a consequence, the significant apparent cohesion caused by interlocking, which is characteristic of very coarse grained materials, such as those used for the Kouris dam shells, was neglected. Hence, the strength parameters determined through triaxial testing are smaller than those from the large shear box tests and, thus, overconservative. The results of the large shear box tests are representative of the actual material strength of the coarse grained shell materials.

The conservative use of the triaxial test data was favoured during the design of the dam over the large direct shear box data to account for the problems related to the proper mixing of the materials. We judge that such a consideration is not necessary because any local segregation during the laying and compaction of the shells, creating pockets of less coarse grained material, has minimal effect at the large scale of the potential sliding mass of a dam. At that scale, the shell material can be considered fairly uniform, having the average grain size distribution, despite any small scale local segregation.

For the clay core, the strength parameters used in the present analysis were the average values established from three drained and three undrained triaxial tests on specimens prepared using compaction at water content equal to the optimum water content +2% (i.e., wet of optimum). This was the water content value chosen for the actual construction of the core. In the original dam design, the c_u and ϕ_u were slightly more conservative than those used herein.

SEISMIC PERFORMANCE CRITERIA FOR EARTH DAMS

According to the International Commission for Large Dams (ICOLD) the design earthquakes for the seismic assessment of a large dam are: (a) the Safety Evaluation Earthquake (SEE), (b) the Design Basis Earthquake (DBE) and (c) the Operating Basis Earthquake (OBE). SEE is the earthquake ground motion that a dam must be able to resist without uncontrolled release of the reservoir. The Safety Evaluation Earthquake can be considered equal to the Maximum Credible Earthquake (MCE), the event that produces the largest ground motion expected at the dam site on the basis of the seismic history and the seismotectonic setup in the region. The Design Basis Earthquake, which corresponds to an earthquake with a return period of 475 years, is the reference design earthquake for the appurtenant structures. The Operating Basis Earthquake (OBE) is the earthquake that is expected to occur during the lifetime of the dam and has a probability of occurrence of 50% during a service life of 100 years. Therefore, the return period of OBE is considered equal to 145 years (ICOLD 2010), but return periods of the order of 200 or 500 years are also used occasionally (Wieland and Fan, 2004).

ICOLD (2010) recommends that for the dam embankment, the performance criteria for the Operational Basis Earthquake (OBE) are that no structural damage (cracks, deformations, leakage etc) which affect the operation of the dam and the reservoir is permitted. For the Maximum Credible Earthquake (MCE) structural damage (cracks, deformations, leakage, etc) is accepted as long as the stability of the dam is ensured and the uncontrolled release of large quantities of water from the reservoir is prevented. Thus, the performance requirements linked to MCE are (ICOLD 2010): (a) sliding stability safety factors of slopes should be greater than 1.0, (b) no loss of freeboard (i.e. after the earthquake the reservoir level shall be below the top of the impervious core of the dam), and (c) no internal erosion is allowed (i.e. after the earthquake at least 50% of the initial thickness of the filter and transition zones must be available) (Wieland 2012).

Based on the above, the parameters that need to be estimated for the seismic assessment of earth dams are: (a) the available safety factor (dam stability), and (b) the expected deformations developed in the dam as a result of seismic loading. According to Eurocode 8 (2004), the seismic performance of ground slopes to the design earthquake can be assessed through established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods. Pseudo-static analysis is generally considered to be a conservative approach and is preferred in engineering practice. The expected deformations can be obtained either with nonlinear response history analysis or with sliding block analysis (Kramer 1996).

SITE SEIMSMICITY

The design basis earthquake (DBE) refers to the earthquake with 475 years return period (10% probability of exceedance in 50 years). Therefore, the corresponding peak ground acceleration (PGA) can be obtained from the seismic zonation map of Cyprus. The site where the dam is built belongs to the zone of the highest seismicity and the DBE peak ground acceleration is equal to 0.25g.

In order to estimate the PGA of the most credible earthquake (MCE), either probabilistic or deterministic seismic hazard assessment (PSHA and DSHA) should be carried out. A deterministic hazard assessment can be performed assuming that the two major faults that may generate a strong earthquake are the faults of Yerasa and Trachoni (Algermissen and Rogers 2004), with lengths 26km and 23km, respectively. The distance of Kouris dam from these faults is 12 and 8.5 kilometers, respectively. Considering that strike-slip faults of this length can produce seismic events of magnitude approximately equal to M_w=6.5 and using an appropriate ground motion prediction equation (see Algermissen and Rogers 2004), we estimate a mean PGA value approximately equal to 0.40g. According to ICOLD, the PGA value of the SEE earthquake in DSHA should be equal to the mean plus one standard deviation. Assuming a dispersion of the logarithms (approximately equal to the COV) equal to 0.3, the final PGA of the MCE earthquake would be:

$$\overline{PGA} = e^{\log(0.40\text{g})+0.30} = 0.57\text{g} \quad (1)$$

The above estimation is in good agreement with the PGA value of 0.55g of the MCE earthquake considered in the dam design. Moreover, Algermissen and Rogers (2004) produced seismic hazard maps for the island of Cyprus based on PSHA. According to these maps, the PGA with 10% probability of exceedance in 250 years (2370 years return period) is approximately equal to 0.58g. This value refers to firm rock sites, while for soft rock sites the report gives a value equal to 0.45g. As our site has mean shear wave velocities that exceed $V_{S,30}$=800m/s, the firm rock value should be adopted.

Based on the discussion above, the MCE ground motion is considered to have PGA equal to 0.57g. The peak ground acceleration of the OBE earthquake is conservatively considered equal to 0.25g, a value that is close to that of the DBE (475 years return period, substantially larger than the 145yrs return period normally considered for OBE). However, return periods of the order of 200 or 500 years have been occasionally used in practice, as reported by Wieland and Fan (2004).

PSEUDO-STATIC ANALYSIS OF EARTH DAMS

Large concrete and embankment dams have been designed against earthquakes since the 1930s. For about 50 years the pseudo-static analysis method has been used in conjunction with a seismic coefficient of 0.1. According to Eurocode 8, Part 5 (2004), the pseudo-static method for slope stability analysis should not be applied to cases of soils capable of developing high pore water pressures and significant stiffness degradation under cyclic loading. Both requirements are met in the case of Kouris dam and therefore this method is here adopted, since it gives a conservative estimate of the safety factor and is preferred in engineering practice.

According to Eurocode 8the design seismic inertia forces are applied in the form of horizontal (a_H) and vertical (a_V) accelerations obtained with the aid of the following equation:

$$a_H = 0.5\left(PGA/g\right) S\, S_T\, g \quad (2)$$

$$a_V = \pm 0.5 a_H \quad (3)$$

where PGA is the peak ground acceleration at the free field, S the soil amplification factor and S_T is the topographic amplification factor.

For Kouris dam, S was considered equal to 1.0 (firm rock foundation) and the topographic amplification factor was assumed equal to 1.2, since the average slope angle is approximately 20^o (less than 30^o). Since the topographic amplification factor refers to the crest of dam, we assume S_T=0.5×(1.2+1.0)=1.10. Hence, in the case of Kouris dam, the horizontal pseudo-static seismic coefficient was equal to 0.55×PGA.

Equations (2) and (3) were derived generally for slopes and note specifically for dams. Andrianopoulos *et al.* (2014) performed a large number of nonlinear response history analyses for several earthdams and derived improved relationships for the pseudo-static seismic coefficient. For dam size similar to the Kouris dam and deep sited failure mechanism (as is the case for the Kouris dam based on the present analyses), the seismic coefficient according to Andrianopoulos *et al.* (2014) is on average around 0.5×PGA, practically coinciding with the value assumed in the present study.

FINITE ELEMENT MODELLING

A two-dimensional finite element model of the most critical cross-section of the dam (the central one) was created in the program PLAXIS 2D (Brinkgreve et al. 2002). 15-node plain strain triangular elements were used to mesh the dam and the underlying bedrock. The FE model (Figure 4) extends 500m from the axis of the dam in both the upstream and the downstream directions, while its depth is 125m below the surface. The size of the model was selected on a trial-and-error basis. The materials used in the FE model belong to three groups, namely, (i) gravels used for the upstream and downstream shells, (ii) the clay core, and (iii) the bedrock. All embankment materials are modelled as elastic-perfectly plastic materials following the Mohr-Coulomb failure criterion

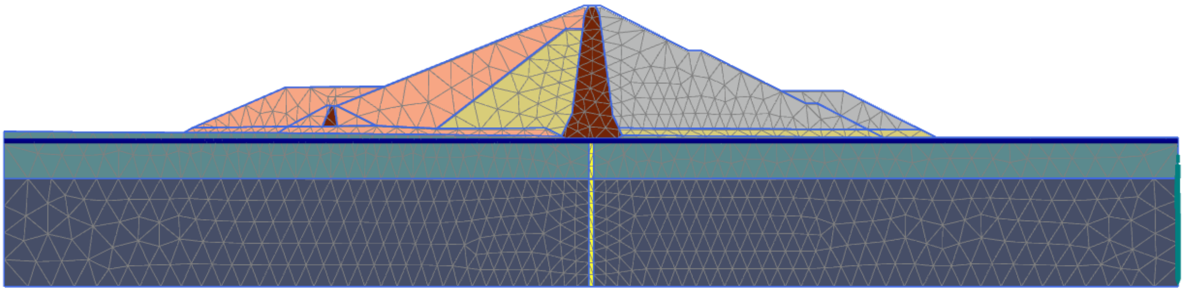


Figure 4: Finite element mesh of Kouris dam.

The mechanical properties of the shell gravels are approximately calculated assuming a shear wave velocity value equal to 350m/s, corresponding to small-strain (maximum) shear modulus G_{max} =245MPa. For plastic stability analysis using the Mohr-Coulomb model, we assume a reduced, secant value equal to 1/5 of G_{max}.

The bedrock consists of thin to medium bedded Miocene chalky marl of blocky structure. The uniaxial compressive strength of the intact rock is equal to 10MPa. The upper part is 40m thick and is weaker than the bedrock mass below since the upper part contains a substantial number of layers of laminated mudstone/siltstone bands. Nonetheless, these layers seem to not exhibit large scale continuity (persistence), as observed in geological/geotechnical investigation trenches. Hence, the bedrock rock mass was modelled in the finite element analyses as an equivalent continuum following Hoek and Brown (1980). The material properties of the bedrock were calculated based on the GSI (Geological Strength Index) rock mass classification system. For the upper part of the bedrock, the GSI index (Marinos and Hoek 2001) is equal to 35, while for the deeper and healthier bedrock, its value is equal to 65. The representative m_i parameter for marls is equal to 7. With the aid of RocLab software (Rocscience 2007), we obtained equivalent cohesion and friction angle pairs equal to 285kPa and 30^o for the upper part of the bedrock and 576kPa and 38.1^o for the lower part. An interesting observation is that the upper part of the rock mass turns to be weaker than the shell materials. This fact results in the critical failure mechanisms being deep seated (i.e. penetrating deep in to the foundation bedrock), as shown in subsequent sections. Finally, the Young modulus was found equal to 1.3GPa and 7.4GPa for the upper and lower parts of the bedrock, respectively.

The permeabilities were considered equal to 55, 138 and 138 m/d, for the river gravels, the terrace gravels and the terrace gravels mixed with talus deposits, respectively. For the clay core the permeability was considered 0.86×10^{-3} m/d and for the upper layer equal to 0.354 m/d.

NUMERICAL RESULTS

The finite element calculations were performed in PLAXIS 2D in several sequential calculation phases that correspond to the construction and the operation phases of the dam. After the construction and reservoir filling phases, the earthquake loading (both horizontal and vertical) is applied pseudo-statically in an elastic-plastic deformation analysis is carried out. Safety analysis follows in order to compute the global safety factor. This is achieved by incrementally reducing the strength parameters tanφ and c of the soil until a failure mechanism develops (strength reduction approach). More specifically, the analysis is performed in the following six stages:

Stage 0: The problem is initialized considering the soil profile before the construction of the dam and considering the water level before the construction of the dam (flush with the river bed). This step calculates the initial stresses of the soil strata due to gravity loading and the initial pore pressures.

Stage 1: In this step the dam is introduced in the model and is subjected to gravity loading. In this step, the embankment is modelled as linear elastic. Stage 2: Linear elastic materials are replaced by nonlinear Mohr-Coulomb materials and calculations are performed in order to make sure that the model is in equilibrium.

Stage 3: The reservoir water is raised to its highest level and the downstream filter is activated. In this stage, the phreatic level is only approximately specified.

Stage 4: Transient flow analysis is performed in order to calculate the exact phreatic level using the material permeabilities specified. This step is followed by a nil analysis step.

Stage 5: Seismic accelerations are applied incrementally.

Stage 6: The safety factor (SF) is obtained by incrementally reducing the strength parameters tanφ and c of the materials until a failure mechanism develops.

Pseudo-static analysis

Pseudo-static analysis has been performed for two seismic levels, the Maximum Credible Earthquake (MCE) and the Operational Basis Earthquake (OBE), considering two loading directions for the horizontal acceleration. For MCE, a uniform horizontal acceleration equal to $a_H = 0.5\times1.1\times0.57g = 0.32g$ was applied, while for OBE the corresponding uniform acceleration was $a_H = 0.5\times1.1\times0.26g = 0.14g$. A uniform vertical acceleration $a_V = \pm0.5a_H = \pm0.16g$, was also applied downwards for MCE and $a_V = \pm0.07g$ for the OBE. EC8 requires the vertical acceleration to be applied both upwards and downwards. We show results only for a_V being applied downwards since this was found to be the most conservative assumption.

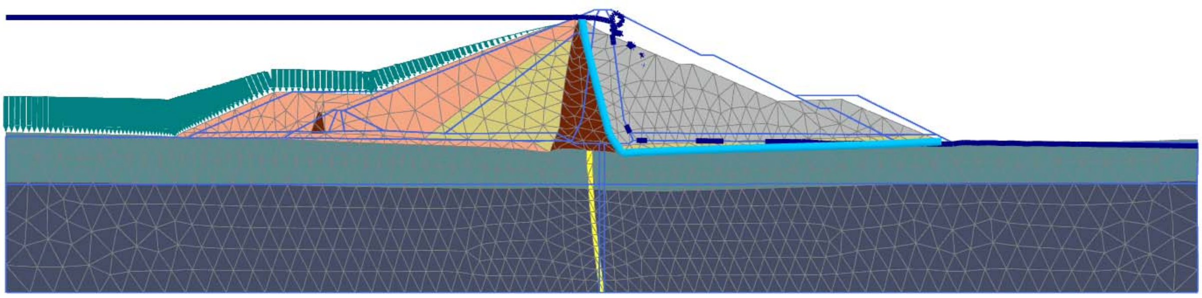


Figure 5: Deformed mesh after the application of the pseudostatic acceleration (MCE upstream).

Figure 5 shows the results when the MCE is applied towards the positive direction, i.e. from the upstream to the downstream direction.

The safety factors obtained for both OBE and MCE, are listed in Table 2.Figure 6 and Figure 7 show the failure mechanism of the two loading directions, while Table 2 shows the available safety factors. It is shown that, in all cases, the failure mode mobilizes a major portion of the embankment that includes part of the clay core and goes deep into the bedrock. Nonetheless the safety factor is always above 1.0. Hence, given that the pseudo-static method is conservative, the capacity of the dam is considered adequate. The safety factor for the downstream failure is the smallest, indicating that downstream slope failure is more critical than the upstream one. For the OBE earthquake scenario, the factor of safety is well above 1.5, which is the safety factor value that would have been required under static conditions (in the absence of earthquake).

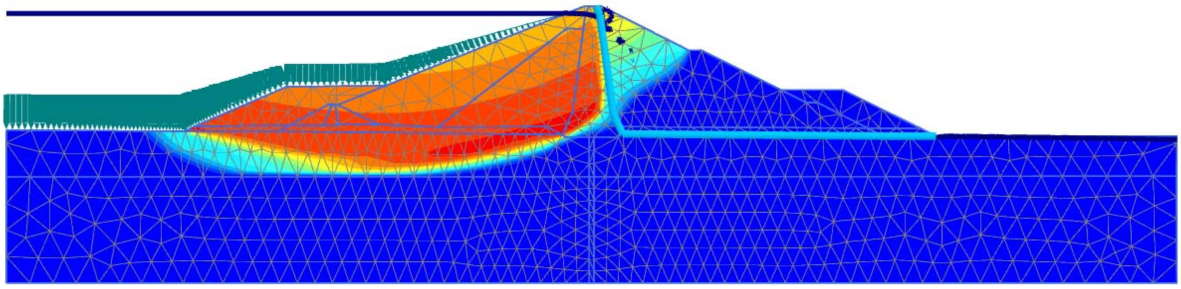


Figure 6: Failure mechanism (upstream).

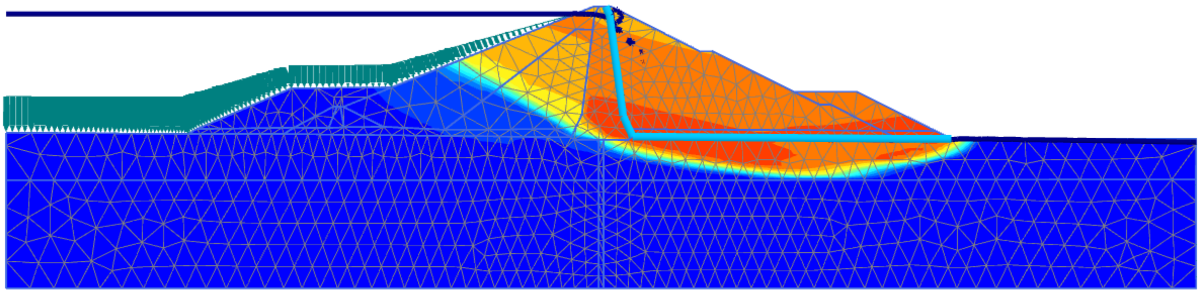


Figure 7: Failure mechanism (downstream).

Table 2 Safety factors of Kouris dam.

Direction of loading	Safety factor	
	Operational Basis Earthquake	Maximum Credible Earthquake
Upstream direction	2.39	1.26
Downstream direction	1.66	1.05

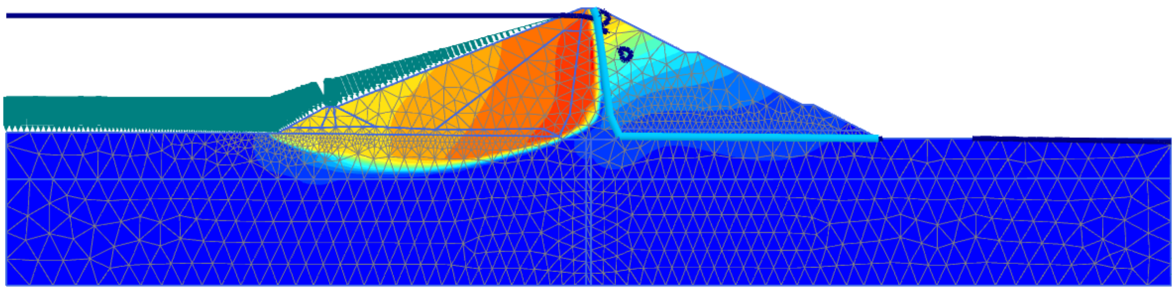


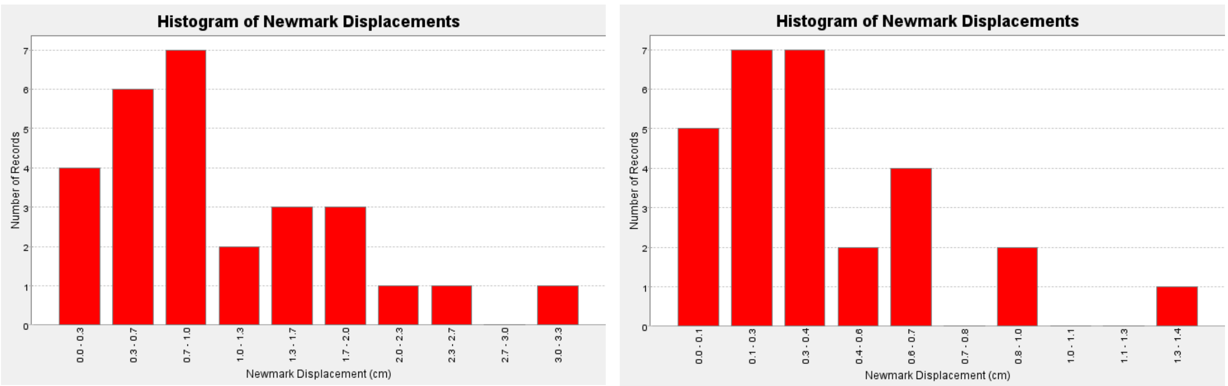
Figure 8: Failure mechanism without the berms, SF=1.17 (MCE upstream).

In order to investigate the effect of the berms, we also performed safety analysis without them. Figure 8 shows the failure mechanism when the MCE is applied towards the upstream direction. Our analyses indicate that the contribution of the berms is slightly detrimental, since the safety factors obtained were found equal to 1.17 and 1.01 for the upstream and downstream slopes, respectively, for the MCE scenario. These values, when compared to those of Table 2, imply that the dam is slightly safer if the berms are removed. This admittedly counterintuitive observation can be explained as follows. The failure mechanisms in the case of the Kouris dam are deep seated due to the presence of weak rock mass at the dam base (weaker than the shell material). As such, the contribution of the berms to the stability of the dam is limited to extending the length of the deep seated slip surface, as observed in the figures above. However, at the same time, the berms contribute to the failure mechanism their one destabilizing inertial forces. In the case of the Kouris dam, these additional inertial forces negate contribution of the berms. Moroever, Andrianopoulos et al. (2014) show results from dynamic analyses suggesting that the local amplification of the ground motion at the top of the berms may be equal or larger than that at the crest of the dam. Hence, the detrimental effect of the berms may be more intense in reality than what is implied by the present pseudo-static analyses.

PERMANENT DEFORMATIONS

The pseudo-static method provides the safety factor, an index of the stability of the dam, but gives no information regarding the permanent deformations after a ground motion. Such information can be obtained with the Newmark sliding block analysis method (Kramer 1996). The method considers that a slope will develop permanent deformations if the inertia forces are strong enough to cause sliding of the slope. The displacement of a rigid block can be calculated for any base excitation time history if the acceleration that causes the initiation of slip is known. This acceleration is known as the yield or critical acceleration, and is denoted a_y and the corresponding seismic coefficient is referred as $k_y = a_y/g$. In the sliding block analysis, the vertical component of the ground motion is usually neglected. The yield acceleration was found after trial and error iterations using PLAXIS 2D as the horizontal acceleration that renders the factor of safety to become equal to 1.0. The vertical seismic coefficient was set equal to zero in these analyses. The yield seismic coefficients were found equal to 0.43g and 0.36g for the upstream and the downstream shell, respectively.

Twenty eight ground motion records were selected in order to calculate the permanent deformations. The records had magnitudes in the range between 6 and 6.5 and rupture distances between 8.5 and 15 km. No filter was applied on the focal mechanism, while the records used correspond to soft rock and stiff soil sites (assumed representative of the dam-bedrock system). The ground motions are scaled to the PGA of the earthquake considered. In order to avoid excessive scale factors, only records with PGA values in the range of 0.30g-0.60g were considered.



(a) (b)

Figure 9: Histogram of maximum residual displacements (a) downstream shell (a_y = 0.36g), (b) upstream shell (a_y = 0.43g).

Sliding block analysis was performed with the aid of the program *Newmark* developed by Jibson and Jibson (2013) of the United States Geological Survey (USGS). In this paper, two performance objectives are considered, the first corresponding to the OBE and second to the MCE. The PGA is equal to 1.10×0.26g=0.29g for the OBE and 1.10×0.57g=0.63g for MCE, 1.10 is the topographic amplification factor. For the OBE case, the ratio a_y/a_{max} is above 1.0 for both the upstream and the downstream shells. More specifically, the ratio was found equal to a_y/a_{max} = 0.36/0.29= 1.24 for the downstream shell and 0.46/0.29 = 1.77 for the upstream shell. Thus, no permanent deformation develops in the OBE scenario. These values are considerably above 1.0, indicating that the dam meets the requirements of the operational basis earthquake (OBE), where no permanent or irreversible deformations are allowed. The permanent displacements for the MCE scenario are grouped in the histograms of Figure 9. The mean value and the corresponding standard deviation is 1.1 and 0.8 (cm) for the downstream shell and 0.4 and 0.3 (cm) for the upstream shell. These values are too small for the dam to develop large cracks that would lead to significant losses of reservoir water and deterioration of the structural integrity of the core. Damage of such small extend can be easily accommodated by the presence of the upstream filters.

CONCLUSIONS

Earth dams have been designed many years ago and their seismic capacity should be reassessed on the basis of the most recent standards and seismological data. For this purpose, we combine the pseudo-static analysis and the Newmark's sliding block analysis method for the seismic assessment of Kouris dam. More specifically, the pseudo-static analysis method was used in order to calculate the available safety factor when seismic loading is applied either in the upstream or in the downstream direction. Kouris dam was found to meet the criteria set by ICOLD (International Commission for Large Dams), since, for the OBE scenario, no permanent displacements develop and, for the MCE scenario, the factor of safety is always larger than 1.0 and the permanent displacements are of the order of millimetres. It was observed that the possible failure modes mobilise large part of the dam cross-section and sink deep into the underlying bedrock, implying that the overall stability is controlled by the properties of the bedrock. Under such conditions, the addition of berms may be detrimental to the seismic safety of a dam.

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