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THE SEISMIC PERFORMANCE OF A EARTH DAM BY DIFFERENT DISPLACEMENT-BASED METHODS

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ABSTRACT

The performance-based design of earth dams and the rehabilitation of existing ones require the evaluation of seismic performance based on permanent displacements caused by expected the earthquake. The paper reports a comparison between different methods with increasing complexity for estimating seismic displacements: simplified rigid block method, based on empirical relationships (Bray and Rathje, 1998; Tropeano et al., 2009); simplified uncoupled method, again based on the sliding block analysis, but accounting for soil deformability; coupled 'stick-slip' approach, based on a 1D lumped mass model to calculate together dynamic response of the site and movement of sliding block (Tropeano et al., 2011); 2D finite differences analyses by the FLAC code, reproducing the heterogeneity of soil and topographic effects.
The methods were applied to the case of the dam of Marello mountain across the Angitola river (Southern Italy). The parameters for static and dynamic geotechnical characterization of subsoil model have been taken from the results of the site investigation published in technical reports.
The spectral shape and peak ground acceleration specified by the Italian Seismic Hazard Map, representative of input motion on outcropping bedrock, allowed to choose a set of spectrum-compatible acceleration time histories to simulate the seismic input.
The sliding displacements predicted using simplified method resulted strongly dependent on topographic coefficient. Both uncoupled and coupled approaches have shown conservative permanent displacements compared to Newmark method. The average displacement of the sliding block by two-dimensional finite difference analysis, considering the stiffness variability related to depth, results comparable with values obtained by other methods.

Keywords: Slopes, embankments, dams, seismic displacements

INTRODUCTION

The seismic performance of earth dams has proved to be good in general, but during past major earthquakes, dams have been frequently damaged; for this reason the problems related to seismic stability and permanent displacement of dams have given considerable attention.
A methodology was proposed to assess the safety condition of dams, verifying the structure to the maximum credible earthquake, MCE, for a site. The methodology, comparable to those suggested in technical international recommendations and in state-of-the-art procedures, consists of the following steps:

1) definition of the performance level required;
2) definition of a seismic action;
3) evaluation of the performance of the dam, considering the behaviour of construction materials and foundation soil.

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Usually the seismic performance of earth dams is based on permanent displacements induced by the dynamic action. The procedures to evaluate the displacements are:

a) empirical relationships (e.g.: Ambraseys and Menu, 1988; Bray and Rathje, 1998; Tropeano et al., 2009);
b) displacement methods (e.g.: Newmark, 1965; Tropeano et al., 2011);
c) advanced dynamic methods (e.g.: Itasca, 2005).

The forementioned methods require a geotechnical model and a seismological analysis with increasing complexity. For example, the response of a structure subjected to extreme actions, which bring the material behaviour over the linear field, needs a right knowledge of unconventional geotechnical parameters if the real physical phenomena must be correctly modelled.
For this reason, the simplified methods can be used because they require a simpler definition of seismic geotechnical behaviour model, accounting for the statistical uncertainty of the response.

Actually there are few specific procedures for the dynamic analysis of dams. These methods, developed for analysis of slope stability, were adjusted for the case of dams, considered as artificial isolated slopes.

In this paper, the analysis of seismic performance of Angitola Dam is carried out through the forementioned methods. Displacement-based analyses were carried out with different complexity degrees. The empirical statistical relationships proposed by Ambraseys and Menu (1988) and Bray and Rathje (1998) and those proposed by Tropeano et al. (2009) calibrated for Italian seismicity were applied. Starting from an accurate seismic hazard site analysis, the acceleration time histories was selected for the dynamic analyses that was carried out with: rigid model block analyses (Newmark, 1965) and non linear coupled 1D approach (Tropeano et al., 2011). Finally, the prediction of the above methods are compared with the results of 2D finite differences (FDM) equivalent linear analyses (FLAC - Itasca, 2005).

SITE DESCRIPTION AND INPUT MOTION

The Angitola dam is located in Southern Italy and it dikes the course of the Angitola river, in the southern part of the S. Eufemia bay. The actual water reserve was obtained with two zoned dams built through 1964 and 1968. The total storage volume is about 0.21 Mm^3 and the dams retain about 21 Mm^3 of water.
In this paper the left dam (main dam) is analysed. In Table 1 geometrical and hydraulic features are summarized. Figure 1 shows the plan view and the main cross section of the left dam. The crest is 140.8 m long, 6 m wide, and about 29.8 m high above the foundation level. The upstream shell have three different slopes: 1/2, 1/2.3, 1/2.6, respectively, at altitude 40.30, 32.20 and 19.50 m a.s.l. The downstream shell have constant slope of 1/1.75 with intermediate three quays 4 meters long. The core of the dam has upstream slopes 1/0.5 and counterslope of 1/0.33, downstream slopes 1/3. A concrete diaphragm 21 m long was built under the core.
The soil profile and geotechnical characterization of the site, was deduced only from the results of some Standard Penetration Test (SPT), made along four vertical, two of which are at the core (S3 and S4) and two (S1 and S2) at the downstream.

Table 1. Geometrical and hydraulic features of dam.

Maximum storage level	26.90 m
Crest lenght	140.8 m
Crest width	6.00 m
Height	29.80 m
Freeboard	1.90 m

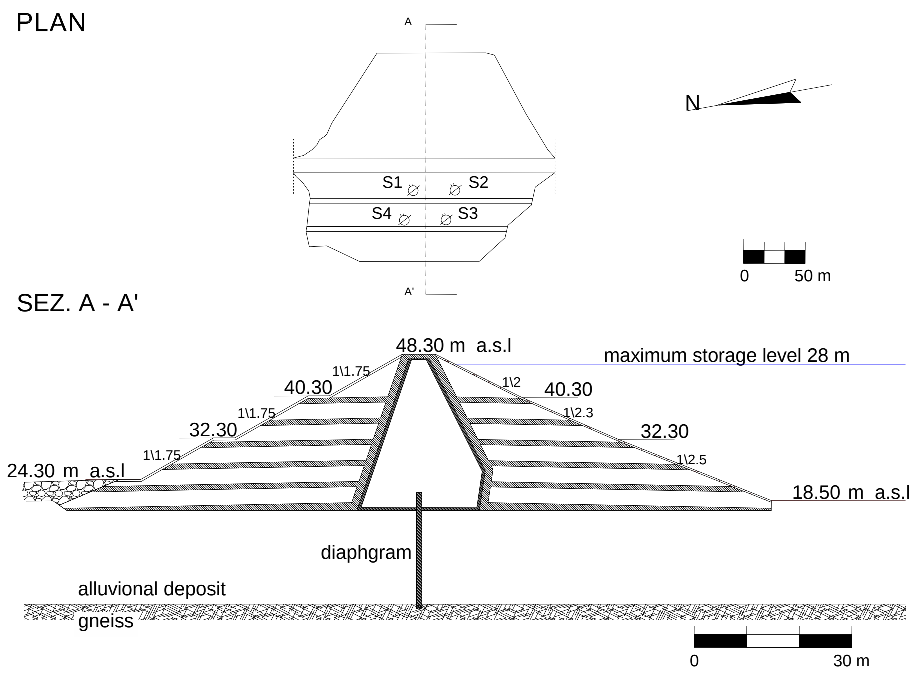


Figure 1. Plan view and main cross section of Angitola zoned dam.

In this paper has been only referred to the parameters reported in the technical report SND/RT/96/1. The geology of the foundation soils of the left dam is characterized by two sedimentary sequences: alluvial terraces (Quaternario and late Quaternario) about 20 m thick and fractured gneiss schist filled by clay material (Paleozoico).
The dam core was built with silty sand (I_P = 20%). The shells consist on gneiss and alluvia deposit from Marello mountain, they have good mechanical properties, but less permeability. For this reason sub-horizontal drains were interposed. A rockfill cover protects the upstream slope against the erosion due to the changes in the water level.
Table 2 reports the average properties of the soils and the shear strength parameters used in analysis. The evaluation of stability conditions was carried out using the pseudo-static approach with conservative value of $c' = 0$.
For foundation soils and shells, the shear wave velocity was estimated with empirical correlations, the average value assumed as representative of the respective formations are:

- for shells: V_S = 259 m/s;
- for foundation soils: V_S = 251 m/s.

Table 2. Soil parameters used for the numerical simulations.

			Foundations	Shells	Core
Material			alluvional deposit	alluvional deposit	silty sand
Physical properties	Bulk unit weight:	γ (kN/m^3)	20	20	19
	Fine fraction:	*CF* (%)	10 - 20	10 - 20	70
	Plasticity index:	I_P (%)	-	-	20.2
	Poisson ratio:	ν	0.3	0.3	0.3
Shear strenght characteristics	Peak cohesion:	c' (kPa)	0	0	0
	Peak friction angle:	φ' (°)	32	38	27
	Bulk modulus:	K (kPa)	$2.6 \cdot 10^5$	$4.2 \cdot 10^5$	$4.05 \cdot 10^5$
	Initial stiffness:	G_0 (kPa)	$1.2 \cdot 10^5$	$1.3 \cdot 10^5$	*Linear regr.*
	Damping ratio:	D_0 (%)	2	2	*2*

For the core, the shear wave velocity and small-strain shear stiffness were considered variable with the depth.
In Figure 2 are reported the values of initial stiffness G_0, versus the average effective stress p′. The values of G_0 were interpreted with a linear regression function (Sanzone, 2009).

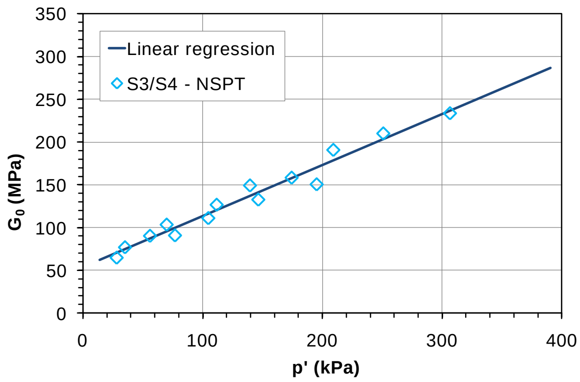


Figure 2. Relation G_0:p′ for the core material.

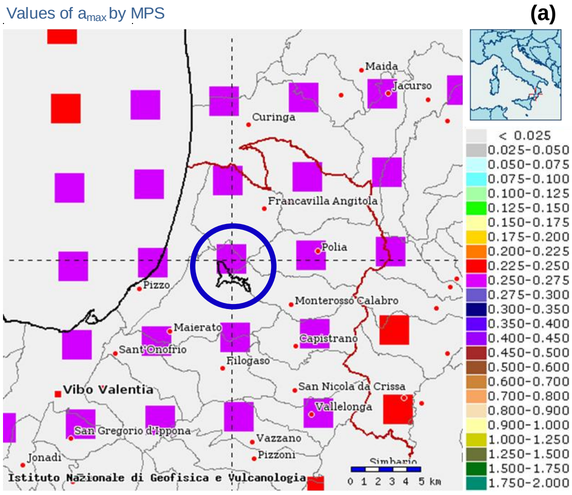
In all analyses (1D and 2D) the pre-failure behaviour of the soil was represented by small strain stiffness and damping corresponding to the values of G_0 and D_0 reported in Table 2.

Seismic input

In the seismic analyses of the Angitola dam were considered real accelerograms, selected to match the response spectrum provided by the seismic Italian Code (NTC, 2008). The ground motion parameters, referred to the study area, were obtained from Italian seismic hazard maps (Working Group MPS, 2004).

Figure 3a shows the peak ground acceleration, a_{max}, (referred to rock site) corresponding to a probability of exceedance 10% in 50 years (return period, T_R = 475 years); Figure 3b lists maximum acceleration as a function of exceeding annual frequency. Figure 3c lists the return periods of earthquake design for different limit states, suggested by the Italian guidelines for the seismic safety of operation dams.

Values of a_{max} by MPS **(a)**



Parameters (MPS) **(b)**

Exceding annual frequency λ	Return period, T_R=1/λ (years)	a_{max} (g) (Lat: 38.7504, Lon: 16.2428, ID: 42334)		
		16*th* percentile	50*th*	84*th* percentile
0.0004	2500	0.4451	0.5116	0.6022
0.0010	1000	0.3173	0.3619	0.4111
0.0021	476	0.2311	0.2702	0.2959
0.0050	200	0.1513	0.1852	0.202
0.0071	141	0.1246	0.1563	0.167
0.0099	101	0.1038	0.1332	0.1402
0.0139	72	0.0852	0.112	0.1174
0.0200	50	0.0686	0.0929	0.0961
0.0333	30	0.0499	0.0694	0.0708

Parameters (NTC08) **(c)**

Categoy	Construction	Utility coefficient C_U	Wildlife V_N (years)	Reference life V_R (years)	Return period, T_R			
					SLO	SLD	SLV	SLC
No Strategic	Existent	1.5	50	75	45	$\boxed{75}$	711	$\boxed{1462}$

Figure 3. The seismic parameters expected in the study area.

To obtain the value of seismic intensity measure (a_{max}), for the seismic scenarios (Figure 3c) suggested by the guideline, the hazard curve was interpolate with different regression functions (Sanzone, 2009). Table 3 report the values of a_{max} used in the analyses.

Table 3. Values of a_{max} used in the analysis.

Return period (years)	a_{max} (g)
75	0.114
1462	0.408

Disaggregation Maps is used to compute the contributions to the mean annual rate of exceedance of peak ground acceleration (a_{max}) values corresponding to different mean return periods (T_R of 75 and 1462 years) from different scenarios. These scenarios are characterized by of magnitude, M, distance of Joyner & Boore (1981), d_{jb}, and ε, number of standard deviation from the median ground motion as predicted by an attenuation law. These values were used to select 16 acceleration time histories from on-line seismic database (SISMA by Scasserra et al., 2008; PEER).

The procedure used for selection of seismic ground motions is that proposed by Bommer and Acevedo (2004). The main characteristics of the accelerograms selected for analyses of damage limit state, SLD, and collapse limit state, SLC, are summarized in Table 4, where are, also, reported the value of median period, T_m, and significant duration, D_{5-95}.

Table 4. Characteristics of real earthquake records for SLD (a) and SLC (b).

	Earthquake	Record	a_{max} (g)	M	d_{jb} (km)	T_m (s)	D_{5-95} (s)
(a)	Coyote Lake '79	COYOTELK/1320	0.13	5.7	9.1	0.30	5.8
	Lazio-Abruzzo '84	ATI/WE	0.11	5.9	12.9	0.28	9.8
	Lazio-Abruzzo '84	ATI/NS	0.10	5.9	12.9	0.33	9.7
	San Francisco '57	SANFRAN/100	0.11	5.3	8.0	0.21	3.7
(b)	**Earthquake**	**Record**	**a_{max} (g)**	**M**	**d_{jb} (km)**	**T_m (s)**	**D_{5-95} (s)**
	Loma Prieta '89	LOMAP/000	0.13	6.9	10.5	0.30	6.5
	Loma Prieta '89	LOMAP/090	0.11	6.9	10.5	0.39	3.7
	Umbria '84	GBB/090	0.07	5.2	8.8	0.28	6.7
	Umbria-Marche *2nd* '97	AAL/018	0.19	5.8	14.7	0.33	4.1

Figure 4 shows the spectrum compatibility between the average response spectrum of selected records and the elastic response spectrum of NTC (2008) for soil type A. A good agreement between spectra in the range of natural frequencies estimated for the dam, using the relationship of Dakoulas and Gazetas (1985), was observed.

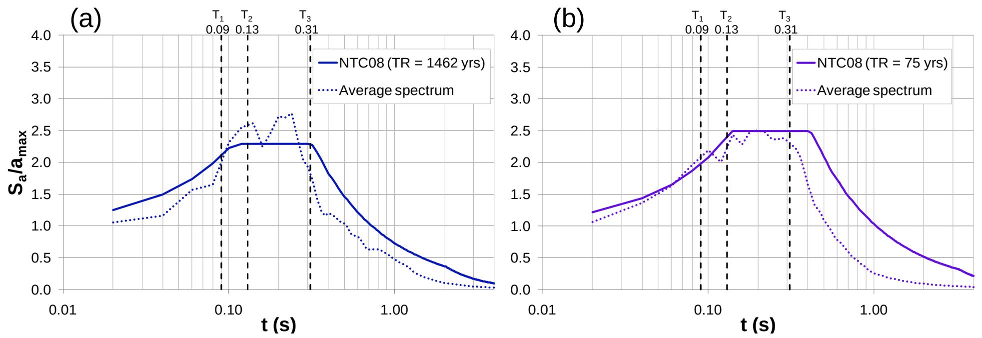


Figure 4. Comparison of NTC08 and computed average elastic response spectra for SLD (a) and SLC (b).

ANALYSIS PROCEDURE

The displacement-based analyses were adopted according to the following procedure:

- the most critical slip surfaces and the corresponding yield accelerations were determined through the pseudo-static approach;
- displacements induced by seismic actions were evaluated with empirical relations;
- seismic displacements were calculated with the simplified uncoupled method, based on the sliding block analysis, accounting for soil deformability also;
- seismic displacements were calculated with a non linear coupled 'stick-slip' approach (Tropeano et al., 2011);
- seismic displacements were calculated with 2D finite differences analyses by the FLAC code, reproducing the heterogeneity of soil and topographic effects.

All analyses are performed for conditions of maximum reservoir and empty tank and for the both limit states.

The pseudo-static approach was used to evaluate the critical acceleration coefficient, k_c, and the associated failure surface corresponding to a condition of incipient rupture for the upstream and the downstream slopes. The critical sliding surfaces are shown in Figure 5: three possible trigger areas were considered, corresponding to sliding circular surface along the downstream (SV, k_c = 0.168) and upstream (SM1 for full tank, k_c = 0.240; SM2 for empty tank, k_c = 0.230). The slip surfaces and the corresponding critical acceleration coefficients were calculated with the Sarma method (Sarma, 1973).

The calculated values of k_c are systematically higher than peak acceleration a_{max} corresponding to damage limit state, subsequently the pseudo-static stability tests for this condition are verified. For the collapse analysis a reduction of a_{max} it was applied, which considers the 'flexibility of the earth structure', i.e. the ability to sustain deformations and displacements. For simplified and 1D dynamic analyses that don't allow to take in account the geometrical effects, it was applied a topographic amplification, S_T. Considering S_T = 1.2 everywhere, the acceleration is always less than the equivalent minimum critical acceleration. The pseudo-static stability tests for this condition is again verified.

Simplified relationships

The relationships used in this study to compute earth dam displacements were those proposed by Ambraseys and Menu (1988) (eq. 1) and Tropeano et al. (2009) (eq. 2).

$$\log\ u = 0.90 + \log\left[\left(1 - \frac{k_c}{k_{\max}}\right)^{2.53} \cdot \left(\frac{k_c}{k_{\max}}\right)^{-1.09}\right] + 0.35 \cdot \varepsilon \qquad (1)$$

$$\log\left(\frac{u}{k_{\max} \cdot D_{5-95} \cdot T_m}\right) = -1.35 - 3.41 \cdot \frac{k_c}{k_{\max}} + 0.35 \cdot \varepsilon \qquad (2)$$

These relationships were derived by the sliding rigid block analysis (Newmark, 1965).

Decoupled simplified approach

The decoupled simplified approach is based on the assumption that the sliding block analysis can be decoupled from the ground response analysis of the earth structure. The decoupled procedure is divided into two phases:

1) evaluation of equivalent acceleration coefficient, k_{eq}: from 1D seismic response of the slope, related to the fundamental period of potentially unstable mass, T_S;
2) estimation of displacements through empirical relationships: based on the rigid block model (Newmark, 1965), using the equivalent acceleration value returned by the first step. The vulnerability of the slope is expressed by the value of critical acceleration.

This procedures consider a nonlinear response of soil through the coefficient S_{NL} and the effect of ground-motion asynchronism, through the frequency factor, α_F. The equivalent acceleration is, therefore, expressed by the following relationship:

$$k_{eq,\max} = k_{\max} \cdot S_{NL} \cdot \alpha_F \cdot S_T \tag{3}$$

where the coefficient S_T expresses the topographic effects.
The permanent displacements can be estimated using the relationship proposed by Tropeano et al., 2009:

$$\log\left(\frac{u}{k_{eq,\max} \cdot D_{5-95} \cdot T_m}\right) = -1.35 - 3.41 \cdot \frac{k_c}{k_{eq,\max}} + 0.35 \cdot \varepsilon \tag{4}$$

In this procedure, significant duration, $D_{5\text{-}95}$, and median period, T_m, were estimated through attenuation relationships based on hazard parameters of MPS (Tropeano et al., 2009).

Among the procedures available in literature, was also used the method proposed by Bray and Rathje (1998). In this case $k_{eq,max}$ is equal to:

$$k_{eq,\max} = k_{\max} \cdot NRF \cdot \alpha_F \tag{5}$$

where NRF is the 'nonlinear response factor' that can be found in Bray and Rathje, 1998. The displacement are computed with the relationship:

$$\log\left(\frac{u}{k_{eq,\max} \cdot D_{5-95}}\right) = -1.87 - 3.477 \cdot \frac{k_c}{k_{eq,\max}} + 0.35 \cdot \varepsilon \tag{6}$$

The coupled approach
A lumped-mass stick-slip model was implemented in a computer code (ACST) by Tropeano et al. (2011). In this model the dynamic site response and the sliding block displacements are computed simultaneously; and the soil is considered with non linear behaviour. This computer code was used to calculate the permanent displacements of the dam. The profile used for the analysis with ACST is indicated in Figure 5.

2D finite differences analysis
The 2D response analyses of the dam were carried out using the FDM code FLAC5.0 (Itasca 2005) which performs seismic ground response analysis in the time domain. In this code was implemented a FDM explicit algorithm for the numerical solution of the dynamic equilibrium equations. The 2D analyses were performed to reproduce the permanent displacements and to assess the influence of the dam geometry and the sliding mechanism.

2D model
The mesh grid used to model the main section of the dam (Figure 5) is composed by 7500 quadrilateral and triangular elements. The mesh is extended 125 m to each side of the dam centre line and vertically down to a depth of 75 m. Thickness of the mesh elements was set to reproduce a frequency content up to 10 Hz, according to the well-known rule of the thumb by Lysmer and Kuhlemeyer (1969).

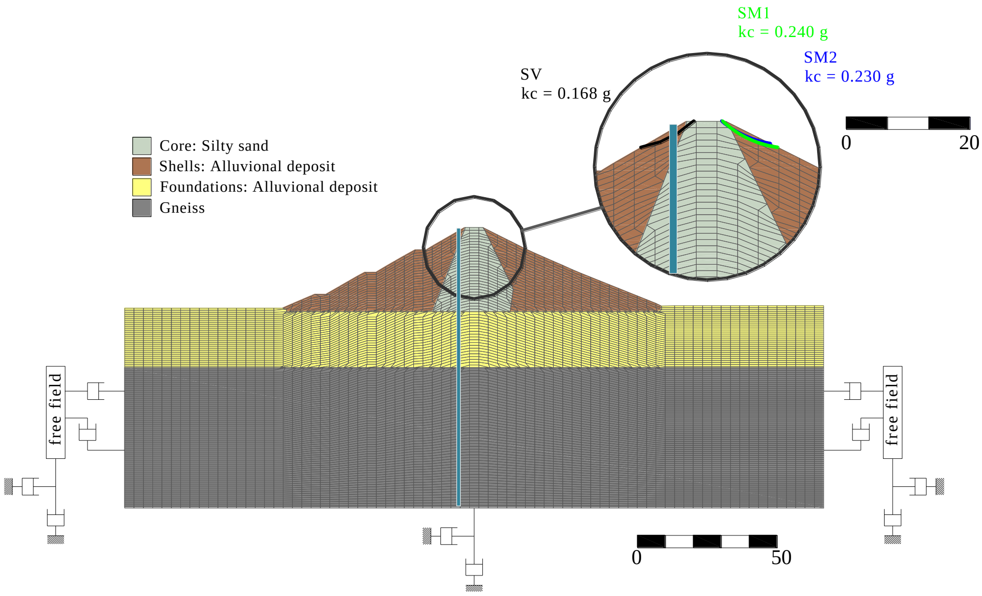


Figure 5. Mesh, sliding surface considered in the FDM analyses, and profile used in the ACST code.

'Free-field boudary' conditions were used for the lateral contours; these consist of one-dimensional columns simulating the behaviour of a lateral semi-finite medium, linked to the mesh grid through viscous dashpots.

Numerical modelling was performed assuming perfect efficiency of the upstream rockfill cover. The prefailure behaviour of the soil core was represented by a Mohr-Coulomb elasto-plastic model, the foundation soils was modelled as linear elastic. Physical and mechanical parameters used in the analysis are reported in Table 2. Seismic loading was applied imposing the select input accelerograms at the base of the mesh.

RESULTS AND COMPARISONS

The analyses implemented for evaluation of seismic performance of the Angitola earth dam were carried out under both 1D and 2D conditions. The displacement for exercise limit states (T_R= 45, 75 and 711 years) are negligible in all examined cases. The conditions of empty and full tank show that the results are similar, because the sliding surfaces occur in the uppermost layers. All accelerograms were scaled to the same peak surface acceleration a_{max} = 0.408 g. In the computation with empirical relationships and with 1D analyses, the acceleration time history was, further, amplified by the factor $S_T = 1.2$, to consider the effect of topographic amplification, according to Eurocode 8 (EC8).
Table 5 resumes the maximum displacements for downstream sliding surface (k_c = 0.168) and for collapse limit state, obtained with the different methods. These displacements are, also, shown in the Figure 6.
Simplified relationship proposed by Ambraseys and Menu (1988) provides cumulated displacement of about 22 cm. The other simplified relationship, based to Italian seismicity (Tropeano et al., 2009), provides displacements less than 7 cm. Similar results, with maximum displacements less about 50 cm, was obtained using the decoupled simplified procedure (Bray and Rathje, 1998; Tropeano et al., 2009). The different displacement values obtained using the relationships proposed by Bray and Rathje (1998) dependent on duration and frequency content of the selected accelerogram, instead displacement evaluated with the relationships of Tropeano et al. (2009), is dependent only on magnitude and distance. Mean period and significant duration, in fact, were evaluated by the empirical attenuation relationships proposed by the same Authors.

Table 5. Maximum displacement computed from 1D and 2D response analyses

Accelerogram	Rigid block model simplified relationships		Decoupled simplified approach		Dynamic methods		
	A&M [1]	TR-a [2]	B&R [3]	TR-b [4]	NEW [5]	ACST [6]	FLAC [7]
	u_{max} (cm)						
Loma Prieta '89 (000)	21.3	6.6	38.3	36.6	1.0	3.4	10.6
Loma Prieta '89 (090)			48.5		5.8	4.8	17
Umbria Marche *2nd* '97 (018)			36.5		3.2	7.9	9.4
Umbria '84 (090)			37.2		2.4	2.8	16

Notes:
[1] Ambraseys & Menu (1988) - 90*th* percentile
[2] Tropeano et al. (2009) - 90th percentile (displacement relationship only)
[3] Bray & Rathje (1998) - 90*th* percentile
[4] Tropeano et al. (2009) - 90*th* percentile
[5] Rigid block method (Newmark, 1965)
[6] Coupled approach, ACST code (Tropeano et al., 2011)
[7] Finite difference analysis, FLAC 5.0 code (Itasca, 2005)

The displacements calculated by Newmark rigid sliding block model and coupled approach (ACST code) are significantly lower than those estimated by the simplified analysis.

In Figure 6 are shown the cumulated displacements obtained by 1D and 2D analyses, for the SLC selected accelerograms (cf. Table 4b). The two-dimensional FDM analysis produces displacements about 10 cm, for the 000 component of Loma Prieta record and for 018 component of Umbria Marche record selected in this study. The maximum displacement values were computed with the accelerograms indicated as "Loma Prieta (090)" and "Umbria (090)" because these records have the higher energy content among the accelerograms selected in this study.

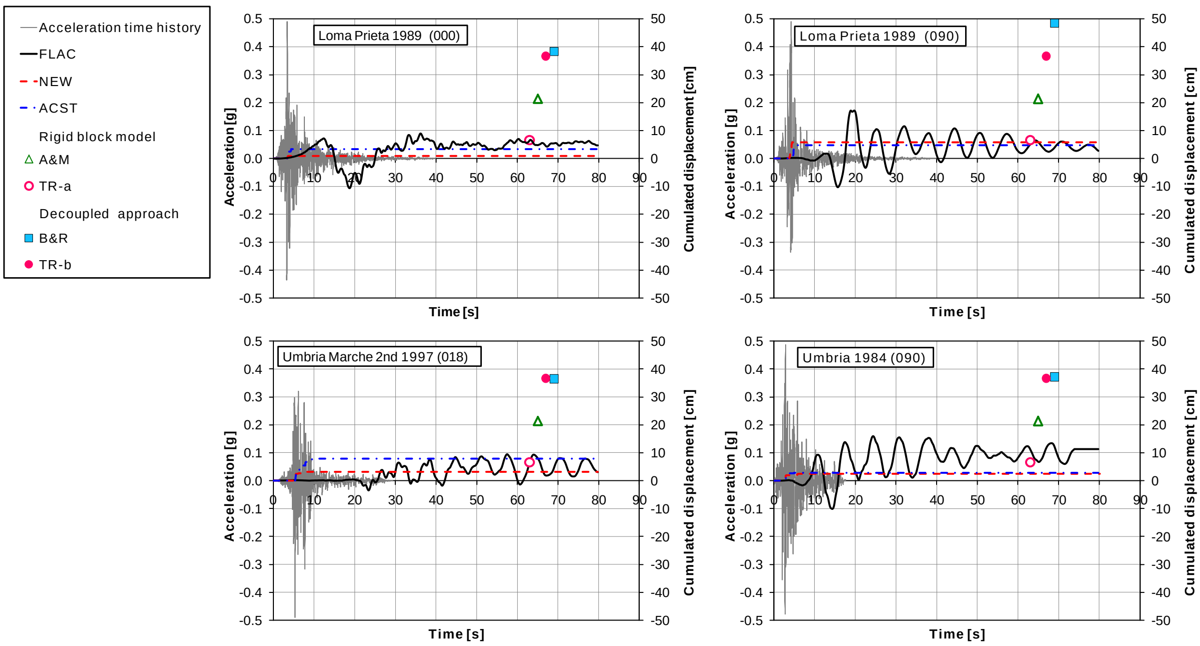


Figure 6. Displacements computed by simplified relationships (symbols) versus the time histories predicted by dynamic analyses (lines).

CONCLUSIONS

The aim of this work was to verify the seismic performance of the Angitola earth dam subjected to a series of maximum credible earthquake. The seismic induced displacements are recognised as the most efficient performance parameter. In this work the attention was focused to the methodological aspects of analysis, starting from the characterization of seismic input, until the evaluation of displacement through different approaches (from the simplified to more complex).

The uncompleted geotechnical model of the dam analysed in this study didn't allow to use more advanced dynamic analyses. There isn't, in fact, sufficient knowledge about the mechanical proprieties of foundation soils and about the hydraulic condition. For this reason the analyses was made considering some conservative assumptions.

For the Angitola dam, the comparison between the simplified and advanced methods show a good agreement of results. In particular the empirical laws give a conservative prediction of displacements related to the confidence level of the relationships.

For all seismic scenarios considered and for each of the methods of analysis adopted, the estimated seismic performance of the dam was satisfactory: the maximum displacement was considerably lower than the freeboard (1.90 m).

The reliability of the results obtained from the different methods is theoretically proportional to the complexity of model. Nevertheless the comparison with more simplified procedures is necessary because these latter, even if they use a less detailed degree of geotechnical model, are less dependent on the basic hypothesis not always fully satisfied.

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