THE SEISMIC PERFORMANCE OF AN 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 (Ambraseys and Menu, 1988; 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 and capable to calculate the dynamic response of the site as well as the movement of sliding block (Tropeano et al., 2011); 2D finite difference analyses performed with the FLAC code, reproducing the heterogeneity of soil and topographic effects.
These 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 the subsoil 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 the 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 the Newmark method. The average displacement of the sliding block by two-dimensional finite difference analysis, considering the stiffness variability with to depth, resulted comparable with the values obtained by the other methods.

Keywords: Slope, embankment, dam, seismic displacements

INTRODUCTION

The seismic performance of earth dams has proved to be good in general, but during past important earthquakes, dams have been frequently damaged; for this reason the problems related to seismic stability and permanent displacement of dams have given considerable attention.
In this paper a methodology is proposed to assess the safety condition of the dams, verifying the structure to the maximum expected earthquake to the site. The methodology, comparable to those suggested in the technical international recommendations and in the scientific literature, consists of the following steps:

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

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Usually the seismic performance of the 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.

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 an appropriate 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 very simple geotechnical model, accounting for the statistical uncertainty of the response.
Actually there are few specific procedures for the dynamic analysis of the dams. These methods, developed for analysis of slope stability, were adapted to the earth dams, considered as artificial isolated slopes.

In this paper, the analysis of seismic performance of the Dam on the Angitola river is assessed by the forementioned methods. According to the requirements of the Italian codes, the reference seismic motions were defined for different limit states, simulated for the numerical analyses by recorded accelerograms selected following the spectral compatibility procedure proposed by Bommer & Acevedo (2004). Preliminarily the permanent displacement was estimated by the empirical relationships proposed by Ambraseys and Menu (1988), Bray and Rathje (1998) and Tropeano et al. (2009), calibrated for the Italian seismicity. Afterwards the acceleration time histories were selected and the analyses were performed by different methods: rigid model block (Newmark, 1965) and non linear coupled 1D approach (Tropeano et al., 2011). Finally, the displacements predicted by above methods are compared with the results of the bi-dimensional analyses performed by finite difference method implemented in the FLAC code (Itasca, 2005).

SITE DESCRIPTION AND INPUT MOTION

The Monte Marello 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 through two zoned dams built from 1964 to 1968, the total storage volume is about 0.21 Mm^3. In Table 1 geometrical and hydraulic features are summarized of the main dam, analysed in this study. Figure 1 shows the plan view and the main cross section of the dam. The crest is 140.8 m long, 6 m wide, and about 29.8 m high above the foundation level. The upstream shell has 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 has 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, whereas the downstream slope is 1/3. A concrete diaphragm 21 m long was built under the core.
The soil profile and geotechnical characterization of the site, were deduced only from the results of Standard Penetration Test (SPT), made along four verticals, two of which are located in the core (S3 and S4) and two (S1 and S2) in the downstream shell.

Table 1. Geometrical and hydraulic features of the main dam.

Maximum storage level	28m
Crest length	140.8 m
Crest width	6.00 m
Height	29.80 m
Freeboard	1.90 m

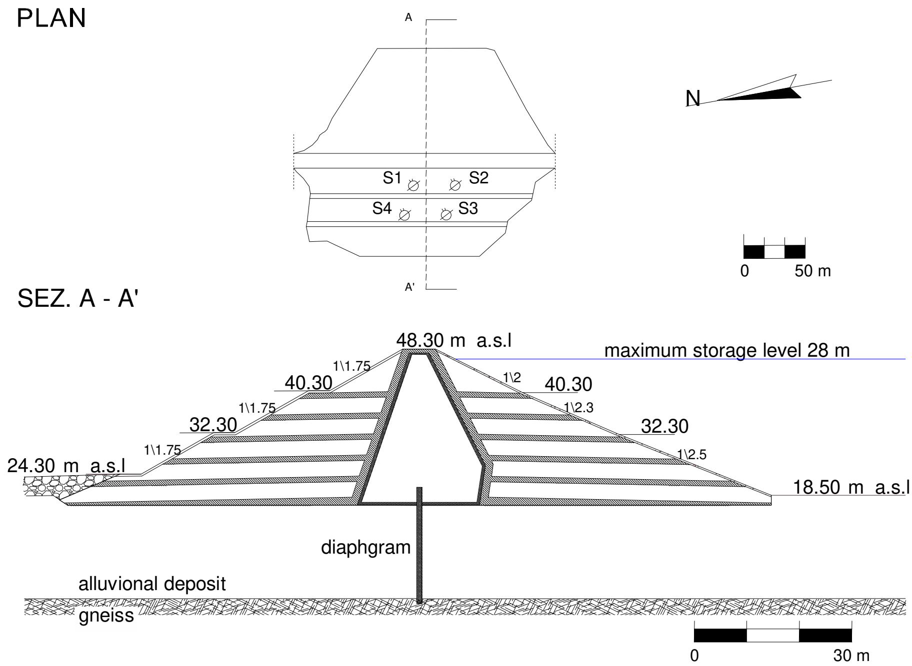


Figure 1. Plan view and main cross section of Monte Marello zoned dam.

The geology of the supporting soils of the 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; whereas, the shells consist on gneiss and alluvional deposit extracted from the Marello mountain, which have good mechanical properties, but low permeability. For this reason sub-horizontal drains were interposed. A concrete-face slab 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 the analyses (Sanzone, 2009). The evaluation of stability conditions was carried out using the pseudo-static approach with conservative value of $c' = 0$.
For supporting 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 supporting soil: $V_S = 251$ m/s.

Table 2. Soil parameters used for the numerical simulations.

Parameter		Supporting soil	Shells	Core
		alluvional deposit	alluvional deposit	silty sand
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
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)	$1.2 \cdot 10^5$	$1.3 \cdot 10^5$	*variable*
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', interpreted by a linear regression function (Sanzone, 2009).

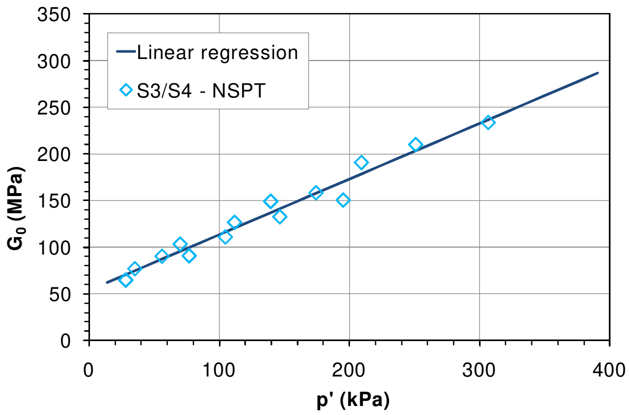


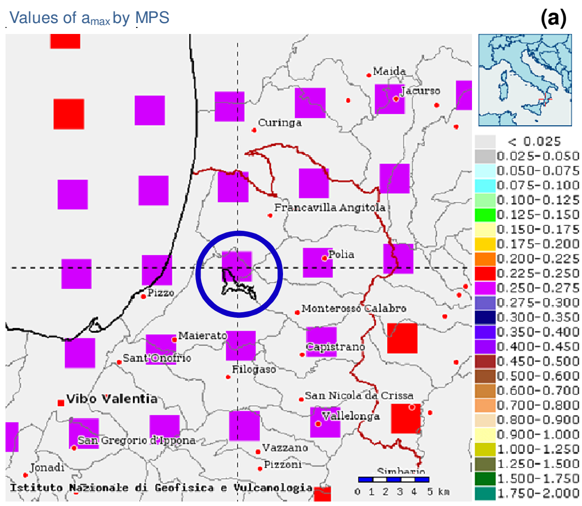
Figure 2. Relation G_0:p' for the core material.

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

Seismic input

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

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



Parameters (MPS) **(b)**

Exceding annual frequency λ	Return period, $T_R=1/\lambda$ (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 extrapolate the value of a_{max} corresponding to the return periods referred to the serviceability (SLD) and ultimate limit state (SLC) (Table 3), the hazard curve was interpolated with a linear regression function (Sanzone, 2009).

Table 3. Values of peak ground acceleration, a_{max}, used in the analysis.

Limit state	Return period (years)	a_{max} (g)
Serviceability	75	0.114
Ultimate	1462	0.408

The disaggregation parameters supplied by the hazard maps are used to compute the contribution of the different seismogenic sources to the definition of the seismic scenarios, corresponding to the different return periods (T_R of 75 and 1462 years). These seismic scenarios are characterized by the ranges of magnitude $5 \leq M \leq 6$ and $5 \leq \mathrm{M} \leq 7$ for SLC and SLD respectively, and distance of Joyner & Boore (1981) 5km $\leq d_{jb} \leq$ 15km and 5km $\leq d_{jb} \leq$ 20km. These ranges were used to select 16 acceleration time histories from the seismic web-site database (SISMA by Scasserra et al., 2008; PEER).

The procedure used for selecting seismic ground motions is that proposed by Bommer and Acevedo (2004). The main characteristics of the selected accelerograms referred to serviceability limit state, SLD, and ultimate limit state, SLC, are summarized in Table 4, where the values of the mean period, T_m, and the significant duration, $D_{5\text{-}95}$, are also reported.

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

	Earthquake	Record	a_{max} (g)	M	d_{jb} (km)	T_m (s)	$D_{5\text{-}95}$ (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)	$D_{5\text{-}95}$ (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 spectrum of the selected records and the reference elastic response spectrum furnished by NTC (2008) for rock site. In correspondence of the range of natural frequencies, estimated for the dam by the relationship of Dakoulas and Gazetas (1985), a good agreement among these spectra was observed.

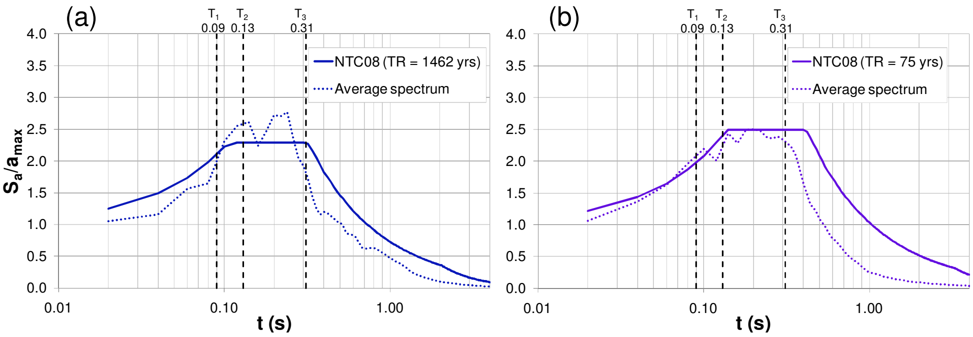


Figure 4. Comparison between the normalized reference response spectra, supplied by NTC (2008), and the average of the selected records for SLC (a) and SLD (b).

ANALYSIS METHODS

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

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

All the analyses were performed for conditions of maximum reservoir and empty tank, and for both the 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 by the limit equilibrium method proposed by Sarma (1973).

The calculated values of k_c are always higher than the peak acceleration coefficient $k_{\max}=a_{\max}/g$ (where g is the gravity acceleration) corresponding to the serviceability limit state. For the analysis to the ultimate limit state a reduction of $k_{\max}$ was applied, in order to consider the ‘flexibility of the earth structure’, i.e. its capability to sustain deformations and displacements. For simplified and 1D dynamic analyses, without taking into account the geometrical effects, the topographic amplification factor, S_T=1.2, was applied.

Simplified relationships

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

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

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

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 method proposed by Bray and Rathje (1998) can be considered as a prototype of this approach and consists into two stages:

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

In the procedure proposed by Bray and Rathje (1998) the

$$k_{eq,\max} = k_{\max} \cdot NRF \cdot \alpha_F \quad \text{with} \quad \alpha_F = f\left(T_m\right) \tag{3}$$

In the equation (3), NRF is the 'nonlinear response factor' and α_F the frequency factor tacking into to account the effect of ground-motion asynchronism.
The displacement can be computed as follows:

$$\log\left(\frac{u}{k_{eq,\max} \cdot D_{5-95}}\right) = -1.87 - 3.477 \cdot \frac{k_c}{k_{eq,\max}} + 0.35 \cdot \varepsilon \tag{4}$$

This procedure is reviewed with particular reference to Italian seismicity by Tropeano et al. (2009), computing the permanent displacement by the equation (2), where $k_{\max}$ has to be assumed as $k_{eq,\max}$.
In this procedure, significant duration, $D_{5\text{-}95}$, and mean period, T_m, were directly estimated from the selected time history of acceleration used as input motion in the analyses (Tropeano et al., 2009).

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, the non linear behaviour of the soil was assumed. 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 FLAC 5.0 code (Itasca 2005), which performs seismic ground response analysis in the time domain. In the code is implemented a finite difference method by explicit algorithm for the numerical solution of the dynamic equilibrium equations. The analyses were performed to reproduce the permanent displacements and to assess the influence of the bi-dimensional geometry and the sliding mechanism.

2D model

The mesh grid representative the main section of the dam (Figure 5) was modelled by 7500 quadrilateral elements. According to the well-known rule of the thumb by Lysmer & Kuhlemeyer (1969), the size of the elements was setted to reproduce a maximum frequency of about 10Hz. The seismic input motions were preliminarily low-pass filtered to a frequency of 15 Hz.

To avoid undesired wave reflections in correspondence to the model boundaries, a 'quiet boundary' condition was adopted for the bedrock (Lysmer & Kuhlemeyer, 1969), consisting of viscous dampers acting along normal and tangential directions, whereas 'free-field boundary' conditions were used for the lateral contours. These latter consist of one-dimensional columns simulating the behaviour of a lateral semi-infinite medium, linked to the mesh grid through viscous dashpots.

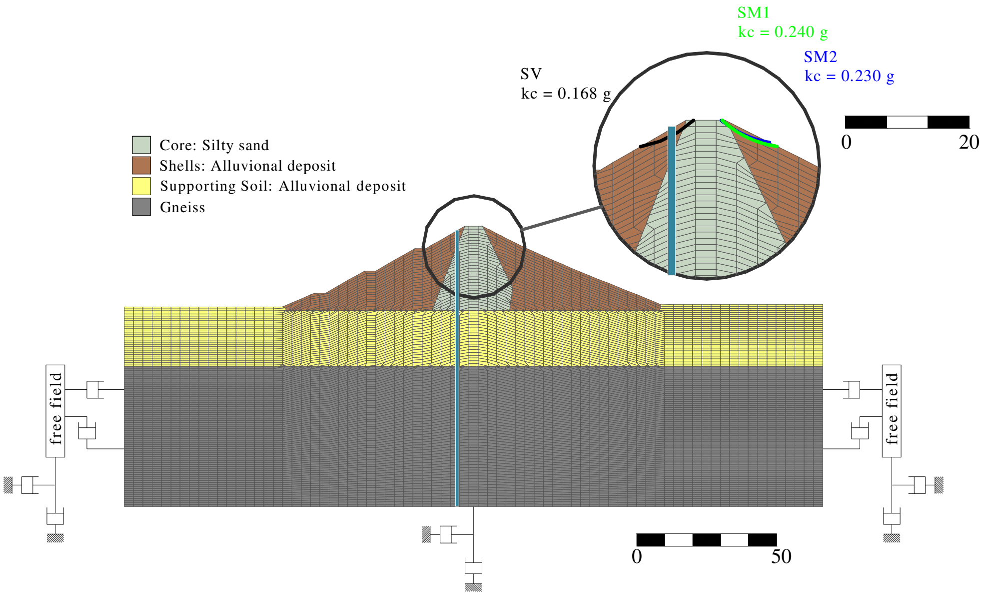


Figure 5. Mesh, sliding surface considered in the FDM analyses, and profile used in the ACST code.

In the analyses, for the core and shells soils, the pre-failure behaviour was assumed as linear equivalent visco-elastic, characterized by the small strain parameters and the hyperbolic laws; the damping was included in the FDM algorithm according to the well-known Rayleigh formulation, i.e. assuming that the damping tensor is a linear combination of the mass and the stiffness tensors. The damping-frequency function was referred to the values of the small strain damping ratio, D_0, reported in Table 2. In these effective stress analyses, soil behaviour at failure was represented by a Mohr-Coulomb plastic envelope, with an non-associated flow rule and hysteresis controlled by the hyperbolic model.
The behaviour of the supporting soils was imposed linear visco-elastic and the seismic loading was applied to the base of the mesh.
The numerical modelling was performed assuming perfect efficiency of the concrete-face slab.

RESULTS AND COMPARISONS

The analyses implemented for evaluating the seismic performance of the Monte Marello earth dam were carried out under both 1D and 2D conditions. All accelerograms were scaled to the expected peak surface acceleration a_{max} = 0.114g and a_{max} = 0.408g, for SLD and SLC respectively. In the computation with empirical relationships and stick-slip model, the acceleration time history was amplified by the factor S_T = 1.2 to consider the effect of the topographic amplification, according to NTC (2008).
The results of the conditions of the empty and full tank are similar; in fact, the sliding surfaces occur in the uppermost part of the dam.

The displacements, with reference to the serviceability limit state, are negligible in all the examined cases.

For the ultimate limit state, the maximum displacements, relating to the more critical sliding surface (SV - k_c = 0.168), are summarized in Table 5 and, also, shown in Figure 6.

Table 5. The displacements computed by the different approaches

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	42.7	12.0	1.0	3.4	7.8
Loma Prieta '89 (090)			24.1	9.8	5.8	4.8	6.6
Umbria Marche *2nd* '97 (018)			27.1	9.1	3.2	7.9	5.6
Umbria '84 (090)			43.5	12.0	2.4	2.8	13.5

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)

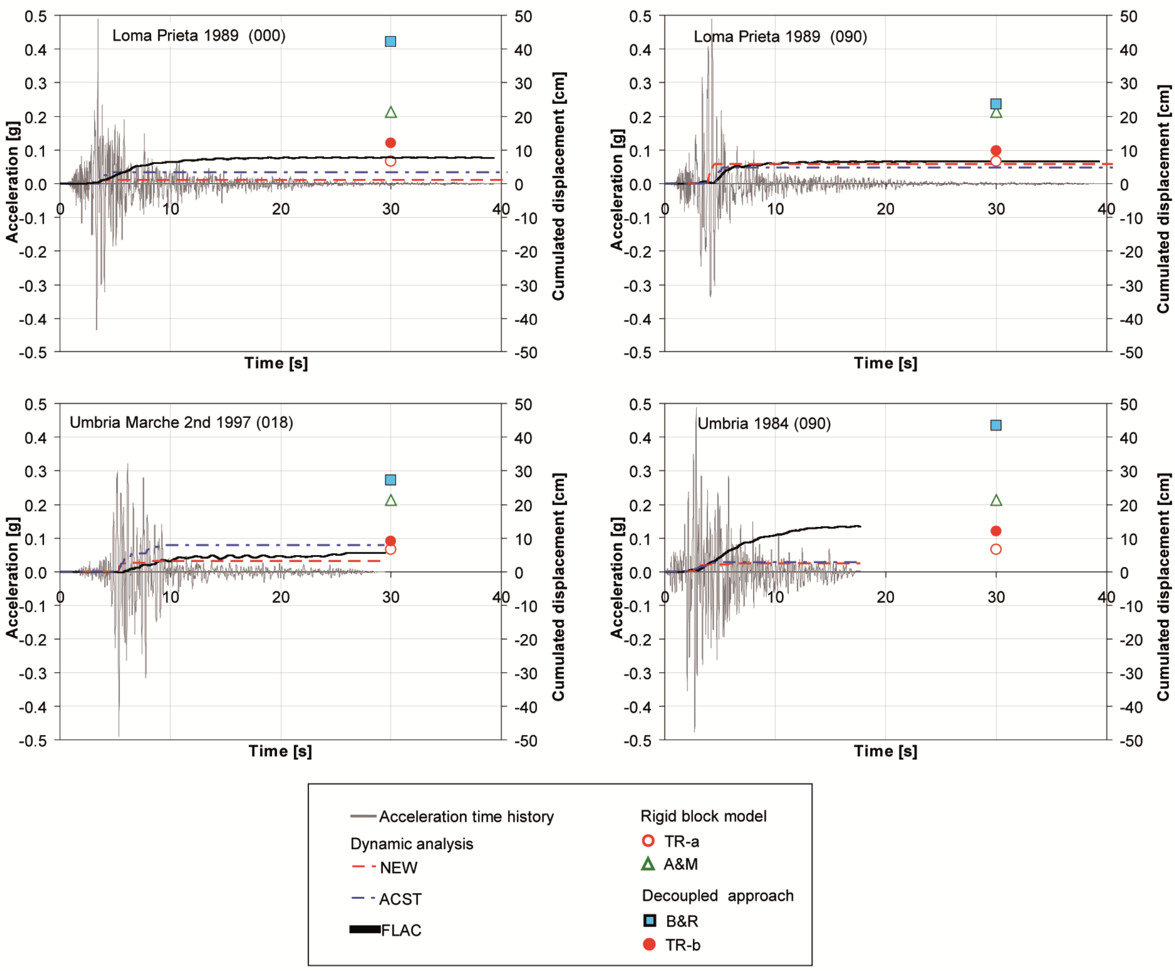


Figure 6. The displacements provided by the simplified methods (symbols) and computed by dynamic analyses (lines).

The simplified relationship A&M (Ambraseys and Menu, 1988) provides a cumulated displacement of about 22 cm; whereas, the relationship TR-a (Tropeano et al., 2009) suggests a displacement less than 7 cm.
The displacements obtained by the decoupled simplified approach show a maximum value of 43.5cm, using the procedures B&R (Bray and Rathje, 1998); whereas, the same approach TR-b (Tropeano et al., 2009), based on the Italian seismicity, estimates a value around 10cm, for all accelerograms.
The displacements calculated by the Newmark rigid sliding block model (NEW) and the coupled approach (ACST) are significantly lower than those estimated by the simplified methods. In particular, considering the analyses on the stick-slip model is possible to estimate a maximum displacement of about 8cm, therefore, comparable to those from simplified methods relating to the Italian seismicity (TR-a and TR-b).
The results of the two-dimensional FDM analyses (FLAC) permitted to assess the performance of the entire body of the dam. In this paper, to compare the results of the different methods, was reported the time histories displacement of the node located in correspondence of the basis of the SV sliding surface. The permanent displacement shows a variability between 5.6cm and 13.5cm, for all accelerograms used as input; the peak value is obtained for the 090 component of the Umbria 1984 earthquake, this record has the higher energy content.

CONCLUSIONS

The aim of this work was to verify the seismic performance of the Monte Marello earth dam subjected to the expected earthquake, with reference to different limit states. The seismic induced displacements are recognised as the most efficient performance parameter. In this work the attention was focused on the methodological aspects of the seismic analysis, starting from the characterization of the input motion, to the evaluation of displacement through different approaches.

A detailed seismic hazard study allowed to figure out the main parameters and response spectrum of the expected earthquake for different return periods; on the basis of such parameters, was selected recorded accelerograms checking the spectral compatibility with the reference input motion.

The simplified relationship and decoupled approach proposed by the international literature predict systematically more conservative values with respect to the analogue methods only based on the Italian seismicity.
The comparison among all methods proposed by the Authors and the two-dimensional advanced dynamic analyses, performed by FLAC 5.0, allowed to verify a good agreement of the results, notwithstanding the likely effects of topographic irregularity.

The safety of Monte Marello dam is assured in the occurrence of the reference input motion; in fact, for all considered seismic scenarios and for each adopted method of analysis, the forecasted seismic performance of the dam was satisfactory. In particular, for ultimate limit state, the vertical displacement of the crest, computed by the 2D analyses, is equal to few centimeters, considerably lower than the freeboard (1.90 m). The seismic stability of the shells, also, was verified; in fact, the more critical sliding surfaces cut the shallow layers of the dam and the computed displacement are negligible, for the serviceability limit state, and lower than 13.5cm, for the ultimate limit state, ensuring the safety with regarding to the collapse.

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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