Discussion and suggestions on design code provisions for seismic stability analyses of artificial slopes

Discussion et suggestions sur les dispositions des normes sismiques pour l'analyse de la stabilité des pentes artificielles

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ABSTRACT

The provisions of European (EC7 and EC8) and Italian (NTC-08) building codes for stability analyses of man-made slopes (i.e. embankments and cut slopes) are discussed in the paper and the major criticalities and uncertainties are identified. These relate to: 1. The values of the reduction coefficient of the expected horizontal peak ground acceleration for the pseudo-static analyses; 2. The use of the partial coefficients for the ultimate limit state (ULS) and the serviceability limit state (SLS) analyses; 3. The selection of recorded and/or artificial acceleration time histories in the Newmark sliding block procedures; 4. Reference requirements for a judgment on the stability of slopes given that the permanent displacement increases exponentially as the critical seismic coefficient decreases. For each of the four points listed above is proposed an interpretation of the examined Codes, where the rules are unclear, or an integration, where the rules are absent. The safety verifications of an actual artificial slope, the embankments of a reservoir basin for flood reduction of the Parma stream, a tributary on the right side of the Po River (Italy), are considered to exemplify concretely the proposed suggestions. The aforementioned basin has recently been the object of a detailed study performed in order to evaluate its safety conditions following the new national seismic classification. An in-depth survey was carried out within the study and the necessary information needed to define a reliable geotechnical modelling of the embankment and soil foundation was obtained. Seismic stability analyses of the embankment of the reservoir basin were performed on a number of representative sections using both pseudo-static and Newmark displacement methods. The peak ground acceleration values and the acceleration time histories at the base of the embankment were assessed by means of one-dimensional ground response analyses. Reference input motions for ground response analyses were obtained from the recent Italian seismic hazard estimates in accordance with NTC-08 requirements. Return periods of the seismic event of 50, 475, and 975 years were considered. Stability analyses discussed in the present paper were performed referring to the 475-year return period of the earthquake and relate to the internal side of the embankment of the most critical section in the reservoir conditions of rapid water drawdown.

RÉSUMÉ

Cette communication examine les dispositions des normes européennes (EC7 et EC8) et des normes italiennes (NTC-08) pour l'analyse de la stabilité des pentes artificielles et identifie les critiques et les incertitudes les plus importantes . Celles-ci concernent: 1. Les valeurs du coefficient de réduction de la valeur maximale de l'accélération attendue au site pour les analyses pseudo-statiques; 2. L'utilisation des coefficients partiels pour l'analyse de l'état limite ultime (ULS) et de service (SLS); 3. Le choix des accélérogrammes naturels et/ou artificiels pour l'application des méthodes basées sur l'analyse de Newmark; 4. Les indications de référence pour une évaluation de la stabilité des pentes, compte tenu que le déplacement permanent augmente de façon exponentielle par rapport à la diminution du coefficient sismique critique. Pour chacun des quatre points mentionnés ci-dessus, nous proposons une interprétation des codes examinés, au cas où les règles ne seraient pas claires, ou une intégration, au cas où les règles seraient absentes. Pour illustrer les suggestions proposées, nous avons considéré les contrôles de sécurité d'une pente artificielle (les levées d'un bassin pour la réduction des crues de la rivière Parma - Italie) qui ont été récemment l'objet d'une

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étude détaillée, dans le but d'évaluer la situation de sécurité selon la nouvelle classification sismique nationale. Des analyses de la stabilité des remblais dans des conditions sismiques ont été effectuées sur un certain nombre de sections représentatives, en utilisant la méthode pseudo-statique et la méthode basée sur l'analyse de Newmark. Les valeurs de l'accélération maximale et les accélérogrammes à la base du remblai ont été évalués par des analyses numériques 1-D de la réponse sismique. Les données sismiques pour l'analyse de la réponse locale, ont été obtenues à partir des estimations récentes de l'aléa sismique conformément aux normes NTC-08, en faisant référence à des analyses des périodes de retour des séismes de 50, 475 et 975 ans. Les analyses de stabilité discutées dans cette étude ont été effectuées pour une période de récurrence de séisme de 475 ans et se concentrent sur le côté interne de la section la plus critique du remblai en cas de vidange rapide du bassin.

Keywords: Seismic stability analyses for artificial slopes, European and Italian Seismic Codes, design seismic action, time-domain analyses, ultimate and serviceability limit state conditions.

1 INTRODUCTION

In Italy and in Europe there are regolatory gaps and many uncertainties about safety verification of earth dams. In fact the Italian Code NTC-08 (D.M. 14/01/2008) prescribes that the dams are the object of a specific code, but the latest Italian code on such topic (D.M. 24/03/1983) is unapplicable because it refers to a previous and out-dated seismic zonation. The European Code for geotechnical design of embankments applies also to small dams (EC7 §12.1) but no definition is given for the word "small".
Lacking of specific guidelines we can use the provisions of European (EC7 and EC8) and Italian (NTC-08) building codes for stability analyses of artificial slopes (i.e. embankments and cut slopes).

For stability analyses in non-seismic conditions the provisions are clear but incomplete: both EC7 and NTC-08 codes prescribe to carry out the ultimate limit state (ULS) analyses in accordance with the design approach DA1-C2 (A2+M2+R2). Furthermore the two Codes have the same values of the partial coefficients and so in non seismic conditions they lead to the same results. In both Codes the values of the strength geotechnical parameters that must be used for the serviceability limit state (SLS) analyses are not specified.

The European Seismic Code EC8 is mainly focused on the design of structures. As a matter of fact the paragraph on slope stability (§4.1.3) prescribes that "*a verification of ground stability shall be carried out for* <u>*structures*</u> *to be erected on or near natural or artificial slopes, in order to ensure that the safety and/or serviceability of the* <u>*structures*</u> *is preserved under design earthquake*." The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods (§ 4.1.3.3), but it is not specified if ULS, SLS or both analyses are considered and which design values of the geotechnical properties should be used. All in all the European Code EC8 has a regulatory omissions concerning natural and artificial slopes not interacting with structures, such as embankments and earth dams, and a serious gap about natural and artificial slopes interacting with structures.

The Italian Code NTC-08 prescribes that the verification of the geotechnical works and systems under earthquake loading conditions in general, including man-made slopes and embankments, must be carried out assuming equal to one the values of the partial factors for the actions ($\gamma_F = 1$) and equal to those for non-seismic conditions the values of the partial factors on the geotechnical parameters (γ_M) and for the strength (γ_R). Thereinafter, the NTC-08 specifies that the seismic stability of the excavation slopes and embankments can be verified with the same methods than for natural slopes (pseudo-static methods, sliding block methods and dynamic analysis). For these the limit state condition must be evaluated using the characteristic values of geotechnical parameters ($\gamma_M = 1$) (§ 7.11.3.5.2), but it specifies also that safety verifications (which verifications? ULS, SLS, or both?) must be carried out using the partial factors (§ 7.11.4). The Circular of the NTC-08 Code recommend to calculate the permanent displacements of the excavation slopes and embankments under design earthquake referring to the characteristic values of geotechnical parameters ($\gamma_M = 1$) (§ C7.11.4). Both European and Italian Codes allow to analyse slope stability in seismic conditions by means of pseudo-static methods, displacement methods and dynamic analysis methods. The latter are infrequent and difficult to apply and cannot be considered alternate to the other ones (§ C7.11.3.5), therefore only a critical review of the first two methods shall be carried out.

Pseudo-static method: the NTC-08 and the EC8 Codes differ in evaluating the static equivalent seismic action by means of the β_s coefficient of reduction of the maximum acceleration expected at site. For the EC8 Code the β_s coefficient is equal to 0.5 and for the NTC-08 it depends on the ground type and on the design ground acceleration on type A ground, and in any case assumes a value in the range 0.20 to 0.30. Furthermore for the EC8 Code the k_v/k_h ratio is equal to ±0.5 or to ±0.33 depending on the ratio between the design ground acceleration in the vertical direction and the design ground acceleration on type A ground, a_{vg}/a_g, practically depending on the hypocentral distance, whereas for the NTC-08 Code the k_v/k_h ratio is always equal to ±0.5. Thus, in the same situation, these different requirements lead to very different results on the slope stability conditions.

Sliding block method: the two Codes differ on the selection of the design earthquake. According to the Italian Code NTC-08, at least 5 real strong motion recordings must be used in seismic slope stability analyses performed by means of Newmark procedure and the use of artificial accelerograms is not allowed. According to the European Code EC8, artificial accelerograms are not excluded although the accelerograms recorded on soil sites in real earthquakes are preferred, and no prescription about the number of accelerograms is given. Neglecting these little differences, the design seismic action to calculate the permanent displacements by means of the sliding block methods is the same for both Codes, unlike the pseudo-static methods. For both Codes there are no prescriptions about the threshold or the limit value of the displacement, which must be chosen and justified by the designer. The Instructions of the NTC-08 Code precise that the calculated displacements must be considered only as an estimate of the order of magnitude of the real displacement (§ C7.11.3.5).

As said above, the responsibility for the admissibility or not of the results, obtained from stability analyses of embankments or earth dams, carried out by means of the sliding block methods, rests entirely and only with the designer. Therefore any ambiguities and all uncertainties in the use of the partial factors are not so important. Both Codes seem to state: "The designer acts as he thinks best, motivating his choices, and gives a consequent judgment".

On the basis of such considerations, remarks and suggestions useful in performing pseudo-static and Newmark displacement analyses are provided in the paper and the results obtained from the study of an actual case are presented.

2 COEFFICIENTS β_s FOR THE PSEUDO-STATIC ANALYSIS OF THE SLOPES

The values of the coefficient β_s reported by the Italian Code NTC-08 ($0.20 \leq \beta_s \leq 0.30$) for pseudo-static analyses of slopes correspond to an expected permanent displacement for a safety factor FS = 1 of about 15-20 cm, whereas the value $\beta_s = 0.5$ reported by the European Code EC8 corresponds to an expected permanent displacement lower than 5 cm (Rampello et al., 2010). Therefore it seems reasonable to conclude that the NTC-08 Code refers to the ULS analyses and the EC8 to the SLS analyses.

3 PARTIAL FACTORS: YES OR NO

The up-to-date geotechnical Codes, including the European EC7 and EC8 Codes and the Italian NTC-08 Code (§2.3), require to evaluate safety by means of semi-probabilistic methods. In semi-probabilistic methods for SLS analyses the characteristic, that is cautious, values of the geotechnical strength parameters are used and values further reduced by means of the partial coefficients are adopted for the ULS analyses. The designer is not required to have knowledge of statistics and probability to apply semi-probabilistic methods, because statistic and probabilistic aspects of the problem have been (or should be) before considered in calibrating the method, i.e. in choosing characteristic values, partial safety factors, etc..

In our opinion this method can be applied also to slope stability analyses. By analogy with embankment safety verifications in non seismic conditions, we believe that SLS analyses must be carried out assuming the characteristic values of strength soil parameters and that the ULS analyses must be carried out reducing characteristic values of the strength parameters by the partial factors $\gamma_M > 1$ ($\gamma_M = 1.25$ for effective stress analyses).

The updated geotechnical Codes, including the European EC7 and EC8 Codes and the Italian NTC-08 Code, require to evaluate and to compare the design value of the resistance (R_d) and the design value of the effect of actions (E_d): the verification is satisfied if $E_d \leq R_d$. In seismic slope stability analyses by means of the block sliding procedure, E_d and R_d should correspond to the permanent displacement and to the threshold displacement respectively. The partial factors for the geotechnical parameters (γ_M) are used to calculate E_d, that is the permanent displacement, while the partial factor γ_R is used to calculate R_d, that is the threshold displacement.

As a consequence the SLS analyses by means of the pseudo-static method for FS = 1 should be carried out using $\beta_s = 0.5$ and the characteristic values of soil strength parameters ($\gamma_M = 1$), whereas the USL analyses should be carried out using $0.2 \leq \beta_s \leq 0.3$, depending on the ground type and on the design ground acceleration on type A ground, and the design values of the soil strength parameters ($\gamma_M > 1$). Similarly two different allowable displacements should be used for SLS and USL analyses and characteristic values ($\gamma_M = 1$) or design values ($\gamma_M > 1$) of the soil strength parameters respectively.

4 HOW MANY AND WHICH ACCELEROGRAMS?

Following the semi-probabilistic methods, the EC7 and NTC-08 Codes provide that characteristic values of the resistance of some geotechnical system as piles and anchorages are determined by correlation factors related to the number and to the variability of loading tests or of profiles of tests. Similarly the characteristic values of the calculated permanent displacement of slopes should be determined by correlation factors related to the number of accelerograms used in the analysis and related to the variability of the results.

The EC8 Code does not specify prescriptions about the minimum number of accelerograms to be used for calculating the seismic slope displacements, while the Italian Code NTC-08 requires to select at least 5 accelerograms. The characteristic value of the calculated displacement that must be compared with the allowable displacement is presumably the higher of the five obtained values.

We suggest to compare the allowable displacement with the higher value between the two values obtained by using the correlation factors related to the number of selected accelerograms and to the variability of calculated displacements:

$$s_k = \text{Max}\ \{\xi_A \cdot (s_{calc})_{mean}\ ;\ \xi_B \cdot (s_{calc})_{max}\}$$

The values of the correlation factors, ξ_A and ξ_B, should be resulting from a statistical and probabilistic analysis. A pure example is indicated in Table 1.

Table 1. Correlation factors ξ to derive characteristic values of seismic slope displacements

Number of signals	5	7	≥ 10
ξ_A	1,2	1,1	1
ξ_B	1	0,9	0,8

As already mentioned in the introduction, NTC-08 do not allow the use of artificial acceleration time histories in the dynamic analyses of geotechnical works and systems, including stability analyses of slopes, natural or artificial, in seismic conditions. The use of artificial signals instead is allowed, although discouraged, by EC8.

The aforesaid suggestions of the two codes are based on the observation that, being calibrated on response spectra obtained from a large number of actual events, the artificial spectrum-compatible signals are characterized by a too large, and thus unrealistic, frequency band.

In fact, as documented by the example proposed in paragraph 6, results in good agreement with those determined from recordings of actual earthquakes can be obtained using appropriately generated artificial acceleration time histories.

As demonstrated (Mucciarelli et al., 2004), artificial accelerograms simulated by means of adequate generation techniques are characterized by a very realistic distribution of the main seismic phases, with a *strong motion* phase actually corresponding to that of real earthquakes. At present the most reliable generation procedures are based on the simulation technique for synthetic non-stationary signals proposed by Sabetta and Pugliese (1996), in which the time-dependent coefficients of the Fourier series are obtained from a physical spectrum appropriately defined as a function of magnitude, distance and site conditions. The signal obtained by this technique is then processed by using an iterative correction in the frequency domain to fit a definite spectral form and a correction in the time domain to eliminate anomalous trends (eg deflection of baseline). The artificial acceleration time histories simulated by this procedure show a very good adaptation to recorded signals, in terms of both specific time-domain parameters (peak acceleration, peak velocity) and frequency content (Fourier spectrum, elastic response spectrum).

5 THE SEISMIC PERMANENT DISPLACEMENT OF THE SLOPE AS THE ONE AND ONLY REFERENCE REQUIREMENT OF STABILITY?

The calculated permanent displacement of the slope must be compared with a threshold or limit value to assess slope safety under earthquake loading conditions by means of the displacement method. Choosing the threshold value, a very important and involving responsibility, we must consider the presence or absence of structures and/or infrastructures involved in the landslide, their stiffness and resistance, the level of risk for people and/or for the environment associated with a total and/or a partial collapse of the geotechnical system, etc.. All these elements are specific and must be evaluated case by case, hence the value to be taken must be chosen by the designer and not by the Code author. Ideally, allowable displacements for analyses would be established from a database in which observed slope displacements from earthquakes are correlated to measures of damage in structures associated with the slope displacements. Unfortunately, however, such data do not exist in sufficient quantity to be useful, and hence there is no rational basis for selecting allowable displacements. Accordingly, allowable displacement levels are established from engineering judgement. As a general guide it is possible to give some quantitative indications from technical literature. The levels of displacement considered tolerable for earth dam by Seed (1979) and Hynes-Griffin and Franklin (1984) were on the order of 100 cm, the limiting displacements used by Bray et al. (1998) for landfills were from 15 to 30 cm, which is similar to an earlier 15 cm value recommended by Seed and Bonaparte (1992). According to ASCE Guidelines for analyzing and mitigating landslide hazards in California (2002) "for slip surfaces intersecting stiff improvements (such as buildings, pool, etc.), computed median displacements should be maintained at < 5 cm, for slip surfaces occurring in ductile (i.e., non strain softening) soil that do not intersect engineered improvements (e.g., landscaped areas and patios), computed median displacements should be maintained at < 15 cm, for slip surface occurring in soil with significant strain softening (i.e. sensitivity > 2), if k_c was calculated from peak strength, displacements as large as 15 cm could trigger strength reductions, which in turn could result in significant slope de-stabilization. For such cases, the design should either be performed using residual strengths (and maintaining displacements < 15 cm), or using peak strengths with displacements < 5 cm."

The value of the calculated permanent displacement of a slope, s, for an assigned accelerogram depends on the value of the critical seismic coefficient, k_c, which is a random variable. The calculated permanent displacement grows exponentially as the seismic coefficient decreases. Therefore we think that for the ultimate limit state verifications of artificial slopes by means of the sliding block method must be checked not only the permanent calculated displacement but also the derivative of the displacement with respect to the critical seismic coefficient.

For example the verification of two conditions could be to required to improve the Codes:

1. calculated displacement for $k_c = k_{c,d}$ less than the allowable displacement ($s_{calc} < s_{adm}$), and
2. calculated displacement for k_c= 0.95 $k_{c,d}$ less than s_{adm} or less than 1.2 s_{calc}.

6 AN EXAMPLE: THE EMBANKMENTS OF AN OPERATING RESERVOIR BASIN FOR FLOOD REDUCTION

To concretely exemplify the previous suggestions we considered the safety verifications of a real and existing man-made slope: the embankments of a reservoir basin for flood reduction of the Parma stream, a tributary on the right side of the Po River (Italy).

The maximum volume of the reservoir in the highest flood condition is about $14 \cdot 10^6$ m^3; the embankments have a total length of about 4000 m and have recently been the subject of an interdisciplinary study in order to verify its safety under seismic conditions (Compagnoni et al., 2010).

The definition of the geology of the area and the stratigraphic and geotechnical characterization of the foundation soils and embankment materials were performed by means of geophysical tests (refraction and tomography, DH tests) and geotechnical in-field (boreholes, SPT) and laboratory tests (ordinary static and resonant column tests). Based on the characterization of the foundation soils four different profiles have been identified that are representative of many different stratigraphic and geotechnical conditions in the studied area: two downstream, close to the structure of the dam, one in the central area of the reservoir basin and the last upstream.

Seismic response analyses were performed for each vertical considering three different return periods of the seismic event (TR = 50, 475, 975 years, the first corresponding to serviceability limit state and the other two to ultimate states) in order to derive the input motion at the base of the embankment for the subsequent seismic slope stability analyses. For each return period, the local seismic response analyses were performed by assuming 14 different seismic signals, 7 artificial acceleration time histories and 7 recordings of actual earthquakes, obtained according to the site reference spectrum suggested by the NTC-08 (Pergalani and Compagnoni, 2008).

The artificial accelerograms were simulated by means of Sabetta and Pugliese (1996) procedure, using different values of the magnitude-distance couple and progressively adjusting signals to improve the fitting to the target spectrum.

The 7 recorded acceleration time histories were selected from the database ITACA 7 (Luzi and Sabetta, 2006) so that the difference between average spectrum and target spectrum was less than 20% in the range 0.15 to 2.00s, and the ratio between the maximum acceleration of the signal and the expected peak acceleration at the site was between 0.5 and 2.

The elastic spectra of artificially generated and recorded acceleration time histories, selected with the previous described criteria, are shown in Figure 1, compared with the target spectrum.

Three typical sections were identified for stability analyses along the embankments of the reservoir basin, representing respectively the downstream area (close to the dam), the central area and the upstream area.

In the downstream area, the embankments reach a maximum height of about 18m; they are composed mainly of granular material and have an impervious earth core.

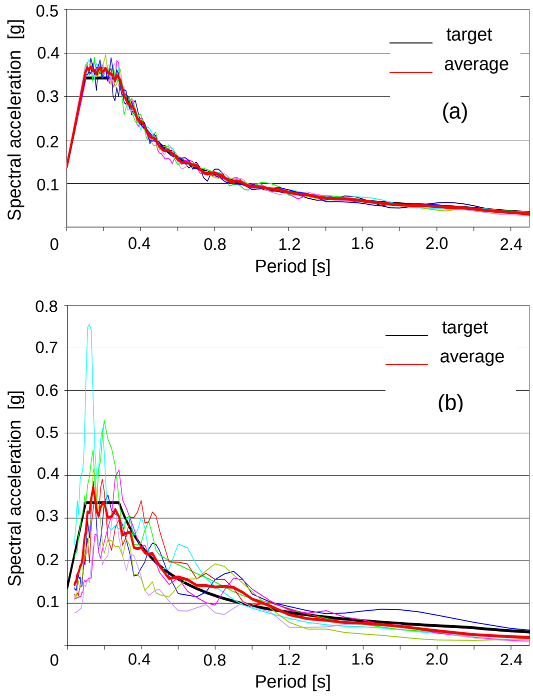

Figure 1. Elastic spectra of artificial (a) and recorded (b) acceleration time histories compared with the target spectrum (after Pergalani e Compagnoni, 2008).

In the remaining portions, where the height is lower, the embankments have no earth core and consist of material with a particle size similar to that of the underlying foundation layer (well graded gravel in silty-sandy matrix). Below the gravelly foundation layer, which has a thickness between 10 and 17m, different from area to area, a layer of clayey silt was found with a thickness between about 2 and 10m. The unit weight and the mechanical properties of the embankment and foundation soils are summarized in Table 2. Everywhere the slope of the embankments are about 1:3 on the inside of the reservoir basin and 1:2 to the outside where a berm is also present.

Seismic slope stability analyses were carried out, for the three considered sections and the four corresponding representative geotechnical profiles, by means of both pseudostatic and pseudodynamic (sliding block) approaches.

According to the NTC-08 Italian seismic zonation, the peak acceleration on rock expected at the site for an earthquake with a return period of 475 years is equal to 0.139g. Consequently, the value of β_s coefficient for the pseudostatic analyses is equal to 0.24 and the horizontal and vertical seismic coefficients are respectively equal to $k_h = 0.0293$ and $k_v = 0.0146$.

Table 2. Unit weight and mechanical parameters of the embankment and foundation soils

	γ [kN/m^3]	ϕ' [°]	c' [kPa]	c_u [kPa]
Embankment	21	39	-	-
gravelly soil	20	39	-	-
clayey silty soil	19	27	10	120

The obtained results showed that the most critical conditions occur on the water side of the embankment, in the reservoir condition of rapid water drawdown, for the typical section shown in Figure 2, located at the central area of the studied reservoir basin.

The value of the peak ground acceleration corresponding to the 50th percentile, obtained at this site by means of the local seismic response analyses, for the free-field conditions with reference to the 475-year return period of the earthquake, was equal to 0.122 g.

The critical seismic coefficient and the global safety factors (defined as the ratio between the shear strength, τ_f, and the shear stress, τ, and assumed constant along the potential sliding surface) from static and pseudostatic analyses (for TR = 475 years) are respectively: k_c= 0.0883; FS_{st} =1.348; FS_{TR475} =1.205.

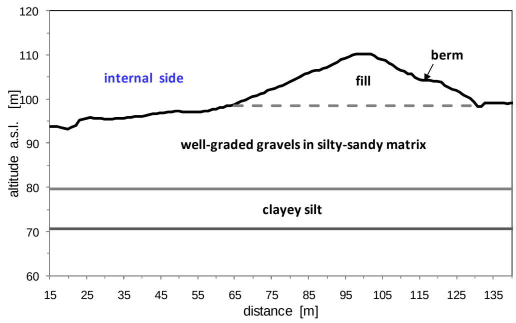

Figure 2. Geometric scheme of the analysed section.

Figure 3 compares the average curves of the elastic acceleration response spectra at the base of the embankment for the set of the artificial and recorded signals with the spectrum at the site for a 475 year return period of the earthquake, obtained by means of the simplified procedure suggested by the NTC-08 based on the categorization of the subsoil. From Figure 3 we can observe that by applying the simplified procedure provided by the Italian Code, the spectral ordinates are overestimated for periods lower than 0.3 s and underestimated for periods higher than 0.3 s and up to 1.5 s. It can be also noted that differences between the average spectra for the artificial signals and the recorded acceleration time histories are not relevant.

The Fourier spectra of the 14 accelerograms at the base of the embankment, assessed by means of the ground response analyses, are shown in Figure 4, together with the corresponding input signals.

Table 3 shows the values of displacement obtained by numerical integration of the relative equation of motion for the Newmark rigid block model and the values of the peak acceleration of the signals used in the analyses.

The results in time domain (peak acceleration of the signals and Newmark displacements) (Table 3), and the frequency domain (response spectra and Fourier spectra (Figures 3 and 4) do not show specific abnormalities resulting from the application of artificial acceleration time histories.

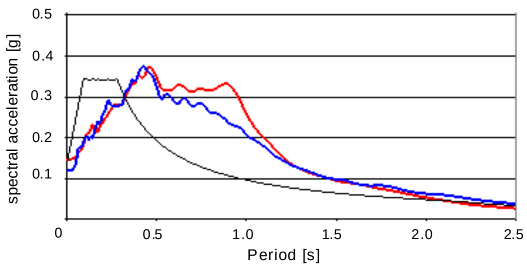

Figure 3. Average response spectra for the set of artificial (blue line) and recorded (red line) signals and reference site spectrum suggested by the NTC-08.

Table 3. Displacements obtained from numerical integration by means of the Newmark procedure for the analyzed section

	Signal	PGA[g]	Newmark displacement
artificial	Acc1	0,1146	0,171
	Acc2	0,1083	0,046
	Acc3	0,1244	0,378
	Acc4	-0,1071	0,054
	Acc5	-0,1180	0,072
	Acc6	0,1365	0,435
	Acc7	-0,1214	0,192
recorded	SRC0	-0,1580	0,615
	MTL	0,1231	0,110
	CAT	-0,1861	1,380
	CSN0	-0,1401	0,853
	NVL	-0,1686	2,129
	CSA_NS	0,1112	0,151
	CSA_WE	0,0848	-

7 CHARACTERISTIC AND DESIGN VALUES OF THE CRITICAL SEISMIC COEFFICIENT

Figure 5 shows the calculated values of the critical seismic coefficient k_c of the slope as a function of the partial factor γ_M for geotechnical strength parameters c'_k and $\tan\phi'_k$.

We observe that k_c decreases almost linearly with γ_M in the considered range. The values of k_c for $\gamma_M = 1$ (characteristic values of the geotechnical strength parameters) and for $\gamma_M = 1.25$ (design values M2 of the geotechnical strength parameters) are respectively $k_{c,k} = 0.0883$ and $k_{c,d} = 0.0216$, then the ratio $\dfrac{k_{c,k}}{k_{c,d}}$ is 4.09.

In other words a reduction of 20% of the geotechnical parameters corresponds to a reduction of 76% of the seismic critical coefficient.

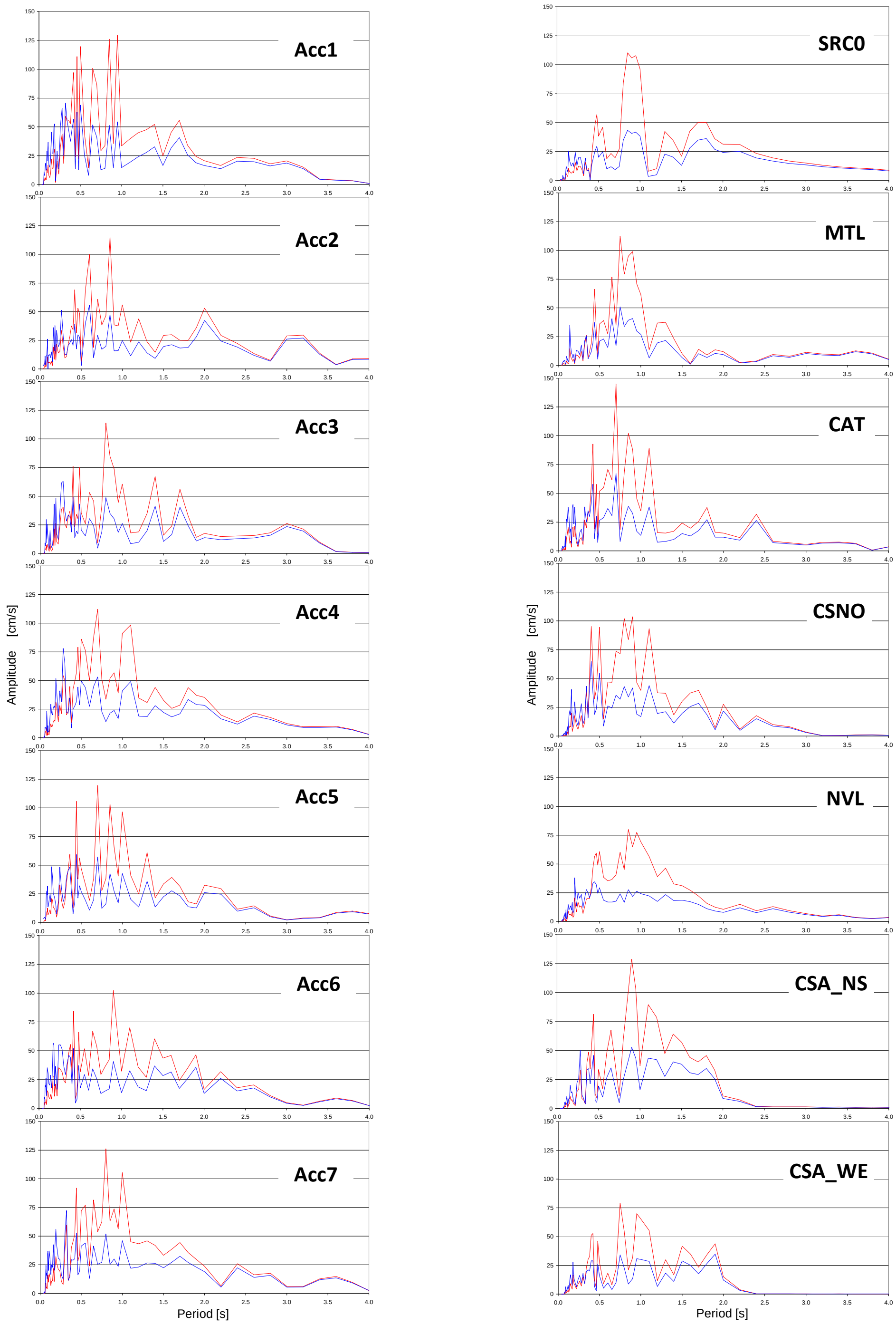

Figure 4. Fourier spectra of the acceleration time histories obtained by means of local seismic response analyses (red lines) adopting artificial (left column) and recorded (right column) signals (blue lines) (after Pergalani e Compagnoni, 2008).

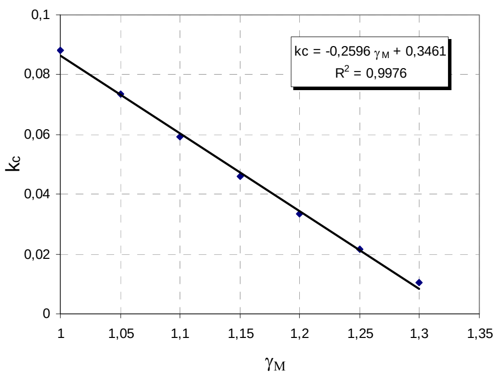

Figure 5. Calculated values of critical seismic coefficient versus partial factor γ_M.

The calculated values of the permanent displacement of the slope (in cm) for a given accelerogram and seismic critical coefficient values between 0.01 and 0.04 are well reproduced by the equation:

$$s = a \exp(-b\ k_c)$$

The values of the coefficients a and b, and of the coefficients of determination R^2 for the 7 recorded accelerograms and for the 7 artificial accelerograms are shown in Table 4.

Table 4. Coefficients of the exponential regressions $s{=}a{\cdot}\exp(-b\ k_c)$ for the 14 design accelerograms

	Signal	a	b	R^2
recorded	CAT	57.044	59.954	0.9771
	CSA_NS	89.820	72.873	0.9992
	CSA_WE	65.161	100.89	0.9960
	CSN0	65.573	58.274	0.9967
	MTL	50.321	55.225	0.9992
	NVL	22.780	29.606	0.9923
	SRC0	45.816	54.282	0.9991
artificial	Acc1	100.310	65.614	0.9993
	Acc2	109.920	83.819	0.9987
	Acc3	76.883	65.372	0.9990
	Acc4	88.689	70.585	0.9997
	Acc5	99.276	66.827	0.9994
	Acc6	81.400	73.316	0.9952
	Acc6	92.094	68.712	0.9983

The corresponding curves are shown in Figures 6 and 7. We observe that the curves from the recorded accelerograms are more scattered than those from the artificial accelerograms, especially for lower values of the critical seismic coefficient.

The statistical parameters of the permanent slope displacement are shown in Table 5 for the values of the critical seismic coefficient respectively corresponding to the design value $k_{c,d}$ = 0.0216 and to the 95% of the design value $k_{c,95\%d}$ = 0.02052.

The permanent displacement values calculated by the artificial accelerograms are greater and less scattered than the values calculated by the recorded accelerograms.

Table 5. Main statistical parameters of the calculated slope displacement distribution for k_c values corresponding respectively to the design value (k_c, d = 0.0216) and 95% of the design value ($k_{c,95\%d}$ = 0.02052)

Signal	**recorded**		**artificial**	
kc =	0.0216	0.0205	0.0216	0.0205
s (cm)				
min	7.63	8.22	15.92	18.08
max	18.31	20.13	23.77	26.10
mean	14.20	15.50	20.02	21.78
Std dev.	3.70	4.18	2.81	2.96
COV (%)	26.0	27.00	14.0	13.6

Applying the previously suggested verification rule of the stability conditions and with reference to the recorded accelerograms we have:

USL Verification

Design critical seismic coefficient: $k_{c,d}$ = 0.0216
Critical seismic coefficient equal to 95% of the design value: $k_{c,95\%d}$ = 0.02052
Number of used accelerograms:
N = 7 wherefrom: $\xi_A = 1.1$ $\xi_B = 0.9$
Calculated permanent displacement for $k_c = k_{c,d}$:
$s_{calc,d}$ = Max(1.1 x 14.20; 0.9 x 18.31) =
Max(15.63; 16.48) = 16.48 cm
1.2 $s_{calc,d}$ = 1.2 x 16.48 = 19.78 cm
Calculated permanent displacement for
$k_c = k_{c,95\%d}$:
$s_{calc,95\%}$ = Max(1.1 x 15.50; 0.9 x 20.13) =
Max(17.05; 18.12) = 18.12 cm

The first condition is satisfied if:
$s_{calc,d} = 16.48$ cm < s_{adm}
The second condition is satisfied if:
$s_{calc,95\%} = 18.12$ cm < s_{adm}
or even if:
$s_{calc,95\%} = 18.12$ cm < 19.78 cm = 1,2 $s_{calc,d}$

Therefore in this case the derivative of the function s = f(k_c) in around of the $k_{c,d}$ value is admissible ($s_{calc,95\%}$ < 1,2 $s_{calc,d}$) and the verification outcome depends only on the assumed allowable permanent displacement s_{adm}.

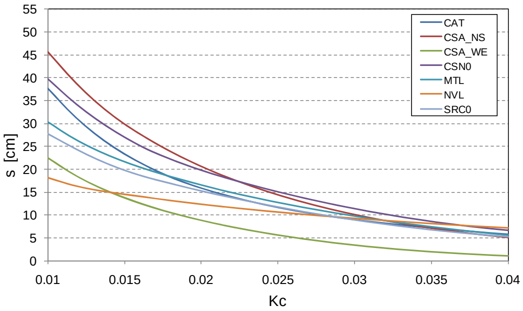

Figure 6. Calculated slope displacements versus critical seismic coefficient for the 7 recorded acceleration time histories.

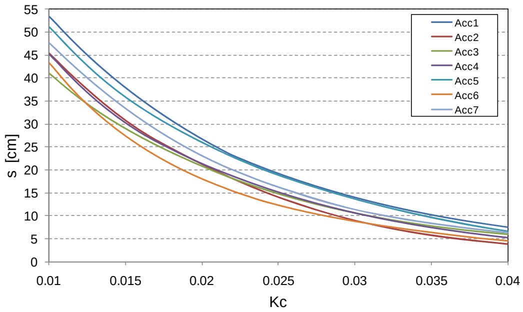

Figure 7. Calculated slope displacements versus critical seismic coefficient for the 7 artificial signals.

SLS Verification

Design critical seismic coefficient: $k_{c,d} = 0.0883$
Critical seismic coefficient equal to 95% of the design value: $k_{c,95\%d} = 0.0839$

Since both values are greater than the maximum acceleration value of the design accelerograms assumed in the ultimate state condition, they will be all the more so greater than the maximum acceleration expected in the conditions of serviceability limit state. Therefore, the calculated displacement is zero, less than the threshold value for the conditions of serviceability limit state, whatever it is.

CONCLUSIONS

In this paper a critical analysis on the safety evaluation of made-man slopes according to the European and to the Italian Codes has been carried out. Uncertainties, gaps and differences between the two Codes are highlighted. In particular we observed that for slope stability analyses carried out by the pseudo-static method the two Codes require very different values of the coefficient of reduction β_s of the maximum acceleration expected at site. The value $\beta_s = 0.5$ suggested by the Eurocode 8 corresponds for FS=1 to permanent displacements less than 5 cm, reasonable for a SLS verification, while values $0.2 \leq \beta_s \leq 0.3$ suggested by the Italian Code correspond for FS=1 to permanent displacements of about 15 or 20 cm, reasonable for a USL verification.

Both Codes allow the use of the displacement method, but they do not contain specific provisions and leave a wide discretion to the designer. The opportunity to choose the allowable value of the permanent slope displacement, evaluating the actual conditions to the exposure of the geotechnical system can be confirmed. However, since the seismic critical coefficient is very sensible to the variation of the geotechnical parameters and the calculated permanent displacement to the value of the seismic critical coefficient, we suggest to carry out safety verifications considering not only the allowable displacement but also the derivative of the displacement with respect to the critical seismic coefficient.

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REFERENCES

ASCE Committee (2002) – Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazard in California.

Bray, J.D., Rathje, E.M., Augello, A.J., and Merry, S.M. (1998) - Simplified seismic design procedure for geosynthetic-lined, solid-waste landfills. Geosynthetics International 5, 203-235.

CIRCOLARE 2 febbraio 2009, n. 617 - Istruzioni per l'applicazione delle 'Nuove norme tecniche per le costruzioni' di cui al D.M. 14 gennaio 2008. G.U. n. 47 del 26/2/2009. Suppl. ord. n. 27.

Compagnoni, M., Pergalani, F., Madiai C., Simoni G., Crespellani T., Vannucchi G., Martelli L. (2010) - Verifiche sismiche di stabilità degli argini di alcune casse di espansione del fiume Po, 29° Convegno Nazionale GNGTS, Prato, 26-28 ottobre 2010, 233-236.

D.M. 14/01/2008 (NTC-08) - Nuove Norme Tecniche per le Costruzioni. G.U. del 4/2/2008. Suppl. ord. n. 30.

EN 1997-1 (2004) - Eurocode 7: Geotechnical design. Part 1: General rules. November 2004.

EN 1998-1 (2004). Eurocode 8 - Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. December 2004.

EN 1998-5(2004). Eurocode 8 - Design of structures for earthquake resistance. Part 5: Foundations, retaining structures and geotechnical aspects. November 2004.

Hynes-Griffin, M.E., and Franklin A.G. (1984) - Rationalizing the seismic Coefficient Method, Miscellaneous Paper GL-84-13, Department of the Army, Waterways Experiment station, Vicksburg, MS.

Luzi, L., Sabetta, F. (2006) - Data base dei dati accelerometrici italiani relative al periodo 1972-2004, Istituto Nazionale di Geofisica e Vulcanologia, 2006.

Mucciarelli, M., Spinelli, A., Pacor, F. (2004) - Un programma per la generazione di accelerogrammi sintetici “fisici” adeguati alla nuova normativa, XI Convegno Nazionale “L’ingegneria Sismica in Italia, Genova, 25-29 gennaio 2004.

Pergalani, F., Compagnoni, M. (2008) - Verifiche tecniche delle vasche di laminazione sugli affluenti del Fiume Po – Attività 4: Analisi sismica. Autorità di Bacino del Fiume Po.

Rampello, S., Callisto, L. and Fargnoli, P. (2010) - Evaluation of slope performance under earthquake loading conditions. Italian Geotechnical Journal, XLIV, No 4, 29-41.

Sabetta, F., Pugliese, A. (1996) - Estimation of response spectra and simulation of nonstationary earthquake ground motions. Bulletin of the Seismological Society of America, vol 86, No 2, 337-352.

Seed, H.B. (1979) - Considerations in the earthquake-resistant design of earth and rockfill dams. Geotechnique, 29, 215-263.

Seed, R.B., and Bonaparte, R. (1992) - Seismic analysis and design of lined waste fills: current practice. In Stability and performance of Slopes and Embankments, ASCE, Geotechnical Special Publication No 31, R.B. Seed and R.W. Boulanger (eds), 1521-1545.