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SEISMIC DIPLACEMENT ANALYSIS OF HOMOGENEOUS SLOPES: A REVIEW OF EXISTING SIMPLIFIED METHODS WITH REFERENCE TO ITALIAN SEISMICITY

Ernesto AUSILIO[1], Francesco SILVESTRI[2], Antonello TRONCONE[3], Giuseppe TROPEANO[4]

ABSTRACT

The simplified displacement-based procedures for seismic slope stability represent a good-working balance between simplicity and reliability, since both slope ductility (i.e. the capacity of sustain permanent displacements) and deformability (basically affecting the asynchronous slope motion) are accounted for. In this paper the procedure proposed by Bray & Rathje (1998) is reviewed with particular reference to Italian seismicity on a set of subsoil models, representative of the different soil classes specified by Italian and European codes. The relationship expressing the decrease of the equivalent acceleration with increasing earthquake/soil frequency ratio is then obtained by means of dynamic 1D site response analyses. Statistical correlations between calculated Newmark displacements, significant ground motion parameters and the ratio of seismic load resistance to peak demand are then derived and compared to similar relationships proposed in literature.

Keywords: Slope stability, Simplified analysis, Seismic coefficient, Displacements

INTRODUCTION

The usual design approaches to analyse the seismic slope stability typically refer to two classes of methods:

- pseudo-static, in which the seismic action is represented by an "equivalent acceleration" used in a conventional limit equilibrium slope stability analysis;
- displacement-based analysis, in which the permanent displacements induced by earthquake acceleration-time history can be calculated by Newmark's rigid sliding block model.

In both cases, soil deformability and coupling between dynamic response of the system and the frequency content of the seismic motion are not considered. Such coupling can produce resonance phenomena and asynchronous motion, with consequent reduction of the inertial effects with respect to those calculated under the hypothesis of rigid behaviour of the slope (e.g. Makdisi & Seed, 1978).

These effects can be correctly modelled through dynamic stress-strain analyses including elasto-plastic soil models. Such rigorous approaches need the knowledge of a number of constitutive parameters which are often difficult to be estimate. Moreover they are strongly affected by the choice of design accelerograms. From this point of view, the development of displacement-based methods, based on dynamic analyses accounting for soil deformability is mandatory. For up-to-date seismic design codes, such methods require to be reliable, yet not too much conservative, and provide suitable equivalent seismic coefficients to be used in the pseudo-static analysis. Such coefficients can be evaluated by few synthetic parameters representative of both ground motion and slope geotechnical model.

[1] Assistant Professor, DDS, University of Calabria, Italy, Email: ausilio@dds.unical.it
[2] Professor, DDS, University of Calabria, Italy Email: f.silvestri@unical.it
[3] Research Assistant, DDS, University of Calabria, Italy, Email: troncone@dds.unical.it
[4] Ph.D. Student, DDS, University of Calabria, Italy, Email: tropeano@dds.unical.it

In this context, among the procedures available in literature, the method proposed by Bray & Rathje (1998) for municipal solid-waste (MSW) landfills appears particularly interesting for its potential introduction in the Italian standards of practice. The objective of this study was, therefore, to assess an analogous procedure applicable to natural and/or man-made slopes, and compatible with the Italian seismicity.

METHOD

Analysis procedure

In the displacement-based simplified procedures for seismic slope stability, two levels of analysis can be individuated (Blake et al., 2002):

- level I: preliminary screening;
- level II: prediction of permanent deformation.

The preliminary screening analysis allows an immediate detection of the slopes for which the expected ground deformations are smaller than a specified threshold value, u_{amm}, or if a more accurate displacement analyses is required.

Screening procedures have been proposed in literature by Seed (1979) and Hynes-Griffin & Franklin (1984) for application to earth dams, Bray et al. (1998) for MSW landfills and Stewart et al. (2003) for hillside residential and commercial buildings. The main characteristics of the above procedures of screening are listed in Table 1. All procedures define a “seismic equivalent coefficient”, k_{eq}, to use in the pseudo-static analysis as follows:

$$k_{eq} = f_{eq} \cdot \left(a_{r,\max} / g\right) \tag{1}$$

where: $a_{r,\max}$ is the maximum horizontal acceleration for a rock site condition, g is gravity, f_{eq} is a reduction factor related to the local seismicity.

Table 1. Features of screening procedures for seismic slope stability (adapted after Kramer & Stewart, 2004)

	Seed (1979)	H-G & F (1984)[a]	Bray et. al. (1998)	Stewart et al. (2003)
Application	Dams	Dams	Landfills	Urbanized slopes
u_{amm} (cm)	100	100	15 – 30	5 – 15
Magnitude	6.5, 8.25	3.8 – 7.7 (most 6.6)	8	user-selected
Seismic coefficient	0.1, 0.15	$0.5 \cdot a_r/g$	$0.75 \cdot a_r / g$	$f_{eq} \cdot a_r/g$
f_{eq}	-	0.5	0.75	f_{eq} ($a_{r,max}$, M_w, r_{jb}, u_{amm})
Safety factor	1.15	1.0	1.0	1.0
[a] H-G & F (1984) = Hynes-Griffin & Franklin (1984)				

Figure 1 represents the flow-chart of the screening procedure, in the most up-to-date version by Stewart et al. (2003). Therein, the reduction factor f_{eq} is evaluated on the basis of seismic motion parameters ($a_{r,max}$ plus the moment magnitude, M_W, and the distance of Joyner & Boore (1981), r_{jb}) and of the threshold Newmark displacement, u_{amm}, specified according to the tolerable level of damage. Its value usually varies between 0.4 and 0.8.

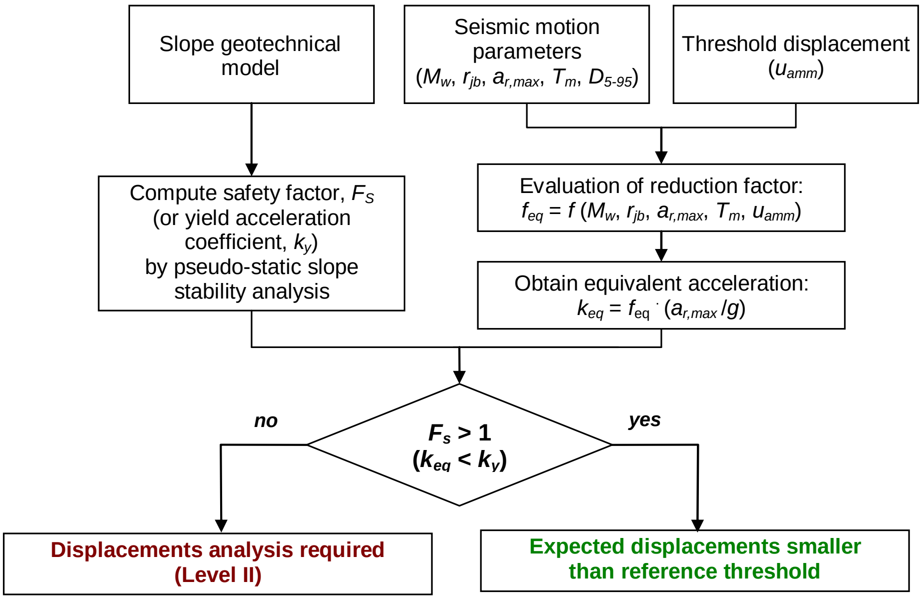

Figure 1. Flow chart of a typical screening analysis procedure.

The second level of analysis aims to the prediction of the slope displacements through two stages:

1. estimate of an equivalent acceleration, used again to represent the seismic loading, after statistical processing of dynamic site response analyses, with linear equivalent or non linear soil modelling;
2. probabilistic evaluation of the displacement by means of simplified dynamic (Newmark) analysis.

The method by Bray & Rathje (1998), illustrated in Figure 2, can be considered as a prototype of a second level approach.

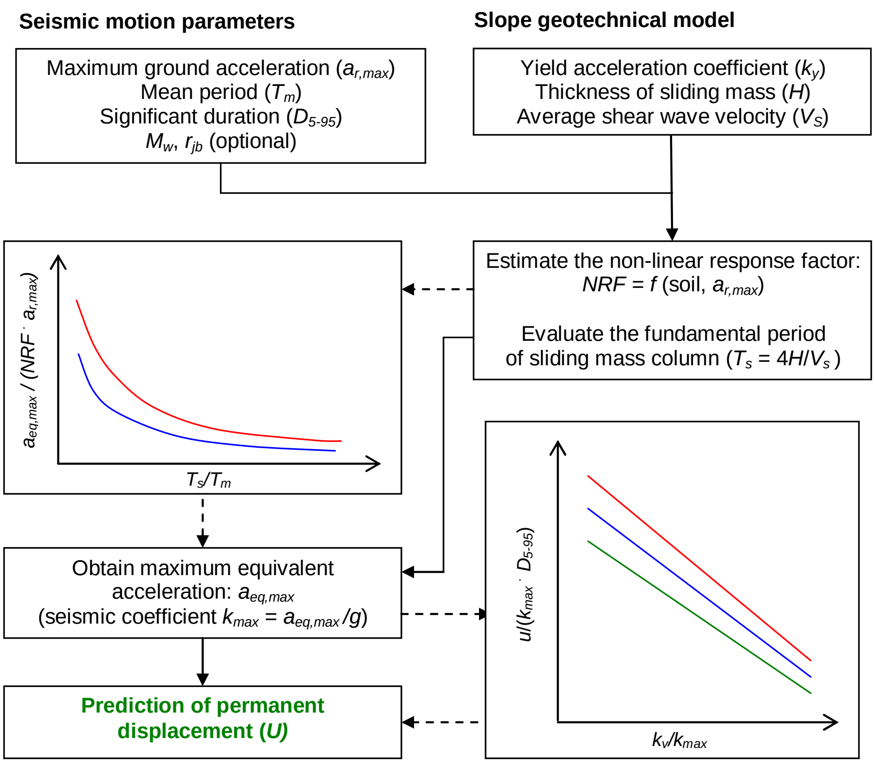

Figure 2. Flow chart of Bray & Rathje (1998) procedure.

The application needs the preliminary definition of the seismic action, not only in terms of peak acceleration ($a_{r,max}$), but also frequency content (mean period T_m) and significant duration of shaking ($D_{5\text{-}95}$, defined between 5%-95% normalized Arias intensity). Such parameters can be evaluated by site-specific seismic hazard analyses or empirical attenuation relationships (e.g. Abrahamson & Silva, 1996, and Rathje et al., 1998).

The slope geotechnical model is characterised by the thickness (H) and the fundamental period (T_S) of the potentially unstable soil mass, and by the yield seismic coefficient (k_y) corresponding to the likely collapse mechanism. Nonlinear behaviour of the soil is taken into account, in synthetic way, by means of a “nonlinear response factor”, *NRF*, which is an amplification factor decreasing with the peak acceleration $a_{r,\,max}$.

It should be pointed out that the above procedure has been developed for municipal soil-waste landfills and with reference to a regional seismicity quite different from Italy. Therefore, the method has been adapted to the seismic Italian context and extended to different kinds of geomaterials; the procedures adopted will be illustrated in the following.

Seismic action

The input acceleration time histories have been selected from a database of records of Italian seismic events (Scasserra et al., 2006; Lanzo, 2006). The database consists of 107 earthquake three-component records with moment magnitude between 4 and 7. The records have been first classified according to the lithology of registration sites (‘rock’, ‘stiff soil’ or ‘soft soil’); thereafter they have been grouped into intervals of 0.5 degrees of moment magnitude, and finally sorted according to the compatibility with the mean values of the horizontal acceleration predicted by the attenuation law by Abrahamson & Silva (1997), plus or minus a half standard deviation (Figure 3).

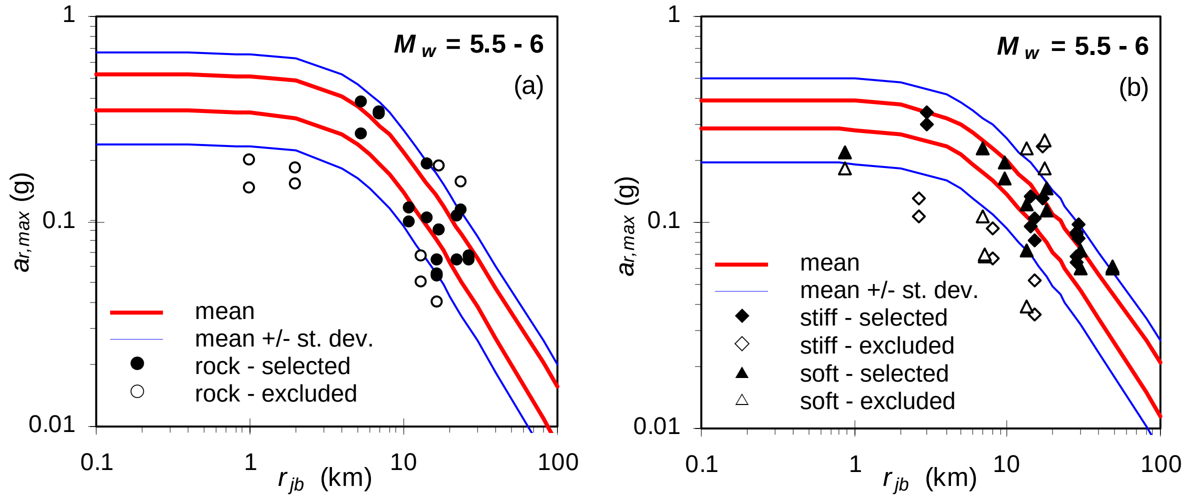

Figure 3. Examples of selection of compatible time records for (a) rock and (b) soil sites.

The final set consisted of 47 ‘rock’, 52 ‘stiff’ and 25 ‘soft’ time records; such data have been utilized for dynamic analyses, and to compute the ground motion parameters which have been used in the method. Figures 4a,b, respectively, show the significant duration, $D_{5\text{-}95}$, and the mean period, T_m, calculated for the set of compatible time records selected. These synthetic parameters have been grouped according to magnitude intervals and subsoil types, and compared to the corresponding values as estimated through the empirical laws by Abrahamson & Silva (1996) and Rathje et al.(2004). The plots show that the empirical attenuation relationships, overall, underestimate and overestimate, the actual duration and period, respectively.

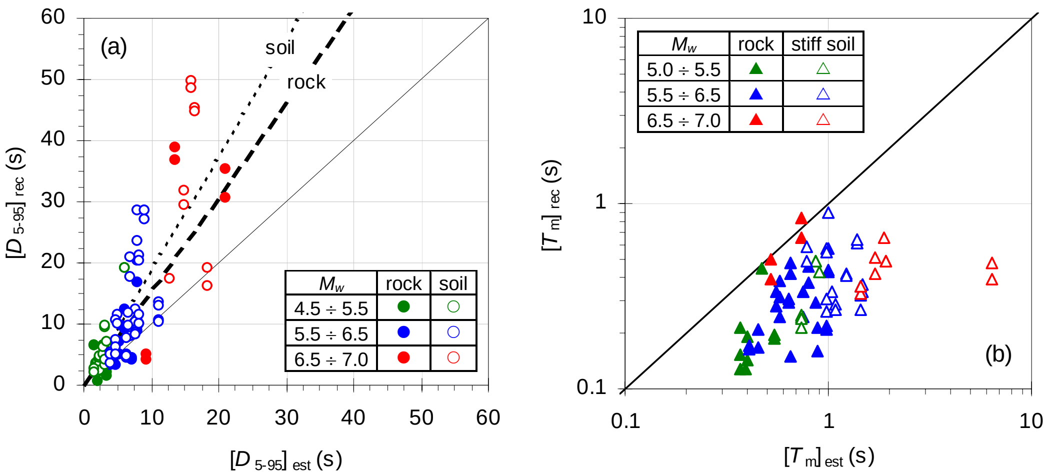

Figure 4. Significant duration (a) and mean period (b) of the records compared to the values estimated with the empirical relationships by Abrahamson & Silva (1996) (a) and Rathje et al. (2004) (b).

Subsoil models

Figure 5 reports the subsoil classes as defined by Seismic Eurocode EC8 as a function of the 'equivalent velocity' $V_{S,30}$ (EN 1998-1, 2003) and of the soil depth *H*, compared to the alternative chart proposed by Bouckovalas et al. (2006) with the aim to fill the existing gap between ground types defined by EC8.

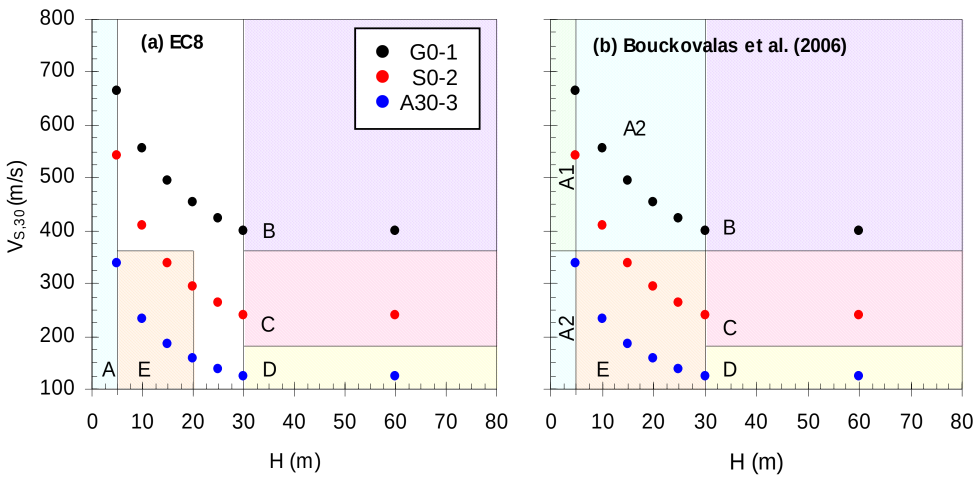

Figure 5. Definition of ground types according to (a) EC-8 and (b) Bouckovalas et al. (2006) (adapted from ETC12, 2006).

In this study, a series of typical soil profiles is used, referring to the more complete classification proposal (Figure 5b), and still keeping the equivalent velocity as reference parameter. Three different lithologies are considered, with variable thickness, to represent all the defined subsoil classes: medium density gravel and sand, and soft clay, characterized by thickness values varying from 5 to 60 m, and by typical index properties (Table 2).

Table 2. Index properties of the soil types.

ID	soil	I_P (%)	γ (kN/m^3)	φ' (°)	V_{S30} (m/s)
G0-1	gravel	-	21	44	399
S0-2	sand	-	20	35	238
A30-3	clay	30	18	25	124

The corresponding shear wave velocity profiles were deduced (Figure 6) using empirical literature correlations (Tropeano, 2006). The 21 soil profiles so obtained have been classified into the 6 classes suggested by Bouckovalas et al. (2006), according to the combination between depth and equivalent shear wave velocity (Figure 5). Hence, the input data for the analyses were defined as follows:

- Class A1: rock or rock-like geological formation, including 5 m of gravel or sand at the surface; acceleration-time records on rock as deep seismic input (*cf.* Fig. 6);
- Class A2: gravel with bedrock depth from 10 to 30 m, sand with bedrock depth of 10 m, or clay with bedrock depth of 5 m; acceleration-time records on rock as deep seismic input (*cf.* Fig. 6);
- Class B: gravel with bedrock depth more than 30 m; acceleration-time records on rock as deep seismic input (*cf.* Fig. 6);
- Class C: sand with bedrock depth more than 30 m; acceleration-time records on stiff soil as superficial seismic input (*cf.* Fig. 6);
- Class D: clay with bedrock depth more than 30 m; acceleration-time records on soft soil as superficial seismic input (*cf.* Fig. 6);
- Class E: sand with bedrock depth from 15 to 30 m, or clay with bedrock depth from 10 to 30 m; acceleration time records on rock as deep seismic input (*cf.* Fig. 6).

The acceleration time records used as an input in the dynamic analyses have typically been those recorded at rock sites, except for deep sand or clay profiles (classes C and D). In such cases, stiff and soft records were respectively chosen, after deconvolution from surface to bedrock.

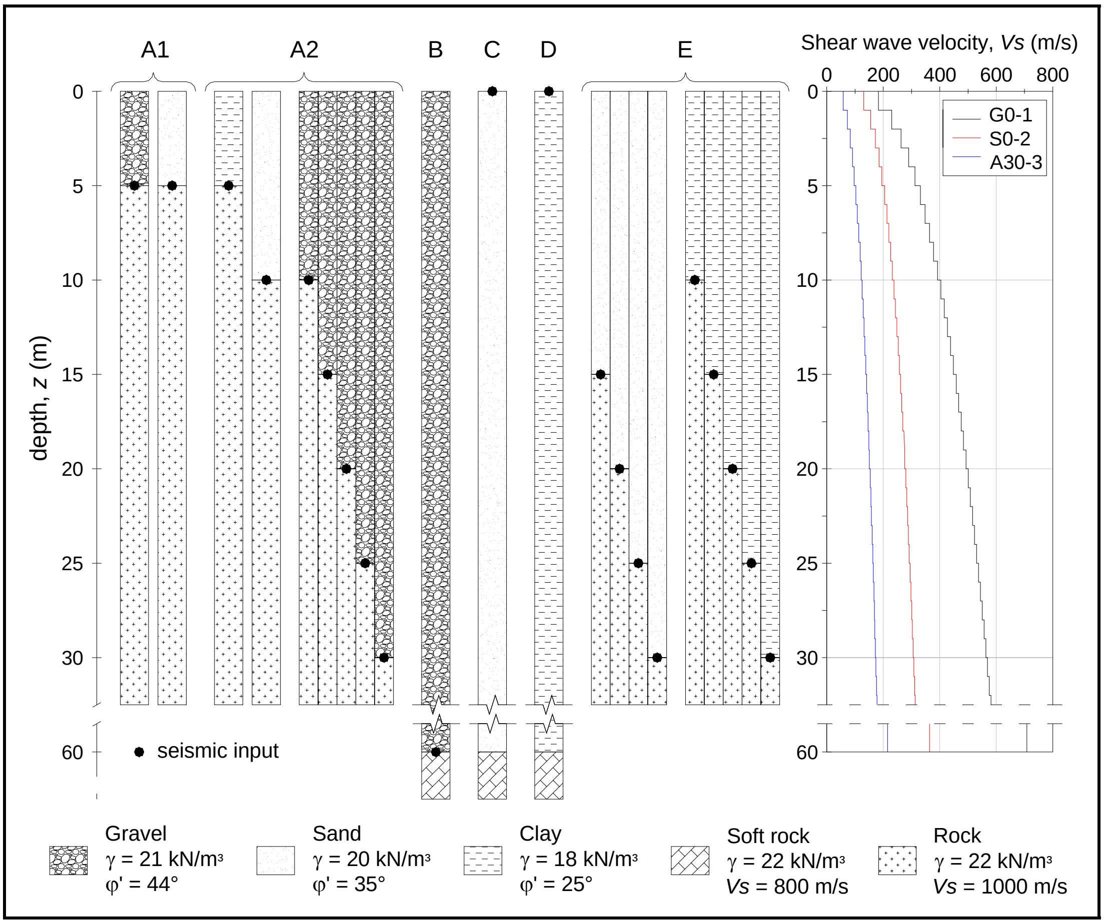

Figure 6. Subsoil profiles assumed in this study.

The nonlinear and dissipative soil behaviour for linear equivalent analysis was defined through the literature curves, expressing the variation of the normalized shear modulus (G/G_0) and the damping ratio (D) with shear strain (γ), reported in Figure 7.

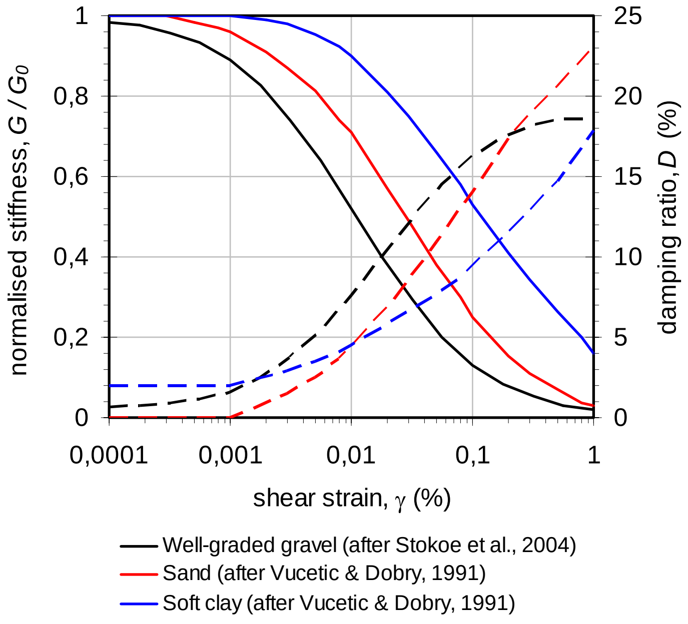

Figure 7. Variation of normalised stiffness and damping with shear strain for the soil types.

EVALUATION OF MAXIMUM EQUIVALENT ACCELERATION

Analysis

In principle, the time-dependent seismic loading for a slope corresponds to a horizontal equivalent acceleration, $a_{eq}(t)$, i.e. to a seismic coefficient, $a_{eq}(t)/g$, which represents the horizontal resultant inertia force acting on the potentially sliding mass, divided by the weight of the mass itself. In conventional pseudo-static stability analyses, the maximum value of equivalent acceleration, $a_{eq,max}$, is considered as a representative parameter of seismic motion, better than the peak acceleration at the bedrock, $a_{r,max}$, or at surface, $a_{s,max}$. For a soil column, the value of $a_{eq,max}$ can be estimated as the resultant of single maximum values of inertial forces in time through the expression:

$$a_{eq,\max} = \frac{1}{\sigma_v(H)} \int_0^H \gamma \cdot a_{\max}(z) dz \qquad (2)$$

that, nevertheless, is a conservative evaluation because it doesn't consider the asynchronous motion. Therefore, referring to the dynamic equilibrium of a soil column, the value of $a_{eq,max}$ was calculated from the shear stress time history, $\tau(t)$, and the total vertical stress, σ_v, evaluated to the depth H of a possible sliding surface:

$$a_{eq,\max} = \max\left[\frac{\tau(H,t)}{\sigma_v(H)} \cdot g\right] = \frac{\tau_{\max}(H)}{\sigma_v(H)} \cdot g \qquad (3)$$

For more complex geometries (i.e., not one-dimensional), a rigorous analysis of $a_{eq,max}$ requires the use of two-dimensional finite element analyses (e.g., QUAD4M; Hudson et al., 1994). Rathje & Bray (1999) demonstrate that 1-D analyses generally provide a conservative approximation of $a_{eq,max}$ for deep sliding surfaces and a slightly underestimate for shallow surfaces near slope crests.

Following eq.(3) the actual $a_{eq.max}$ values have been achieved from shear stress time history $\tau(t)$, calculated for different possible depths of sliding surfaces of the 21 soil profiles reported in Fig. 6. One-dimensional seismic site response (SSR) analyses were carried out by using the software EERA (Bardet et al., 2000). Table 3 synthetically reports the amount of simulations and data values resulting for each subsoil class from the different acceleration time histories.

Table 3. Synthesis of SSR simulations

Subsoil class	# soil profiles n_s	# depths n_h	Subsoil type of time records	# acceleration time histories n_a	# simulations $(n_a \cdot n_s)$	# data values $(n_a \cdot n_s \cdot n_h)$
A1	2	1	*Rock*	74	148	148
A2	7	1	*Rock*	74	518	518
B	1	6	*Rock*	74	74	444
C	1	6	*Stiff Soil*	98	98	588
D	1	6	*Soft Soil*	42	42	252
E	9	1	*Rock*	74	666	666

Results

Following the procedure sketched in Fig.2, for each subsoil class the value of $a_{eq,max}$ needs to be referred by the expected maximum acceleration at ground surface ($a_{s,max}$). Such value can be expressed as the product of $a_{r,max}$ (maximum horizontal acceleration for outcropping rock) and an amplification factor, which in this study was evaluated following four different approaches:

a) constant *S* coefficients, as specified by EC8 (*cf.* Table 4);
b) constant *S* coefficients, modified as suggested by Bouckovalas et al., 2006 (*cf.* Table 4);
c) literature correlations expressing a nonlinear response factor, *NRF*, decreasing with $a_{r,max}$ depending on soil classes (*cf.* Table 4 and Figs. 8a-9a);
d) specific relations obtained after the SSR analyses carried out in this study, expressing a non-linear response factor, *FRN*, variable with $a_{r,max}$ depending on soil classes (*cf.* Table 4 and Figs. 8b-9a).

Table 4. Constant (*S*) and nonlinear (*NRF, FRN*) response factors used in the analyses.

Subsoil classes	Constant response factor (*S*)		Non-linear response factor	
	(a)	(b)	(c)	(d)
	EC8	Bouckovalas et al. (2006)	literature curves (*NRF*)	this study (*FRN*)
A1	1.00	1.00	1	1
A2	1.00	1.25	1	$1.468 \cdot a_{r,max}^{-0.1251}$
B	1.20	1.30	Seed et al. (1978)	$1.0177 \cdot a_{r,max}^{-0.2017}$
C	1.15	1.15	Seed et al. (1978)	$1.0624 \cdot a_{r,max}^{-0.2362}$
D	1.35	1.10	Idriss (1990)	$0.539 \cdot a_{r,max}^{-0.4171}$
E	1.40	1.35	Bray & Rathje (1998)	$1.2274 \cdot a_{r,max}^{-0.2052}$

Figure 8 highlights that the values of *FRN*, computed as $a_{s,max}/a_{r,max}$ resulting from SSR analyses, are always higher than unity, and also greater than those from literature (*NRF*), except for class D, which evidences de-amplification at $a_{r,max} > 0.23g$. Figure 9a illustrates in detail the single data sets resulting for the different subsoil classes compared with the four different evaluations of the response factor: note that both EC8 indications and proposed modifications yield a systematic underestimate of the computed *NRF*, again except for class D.

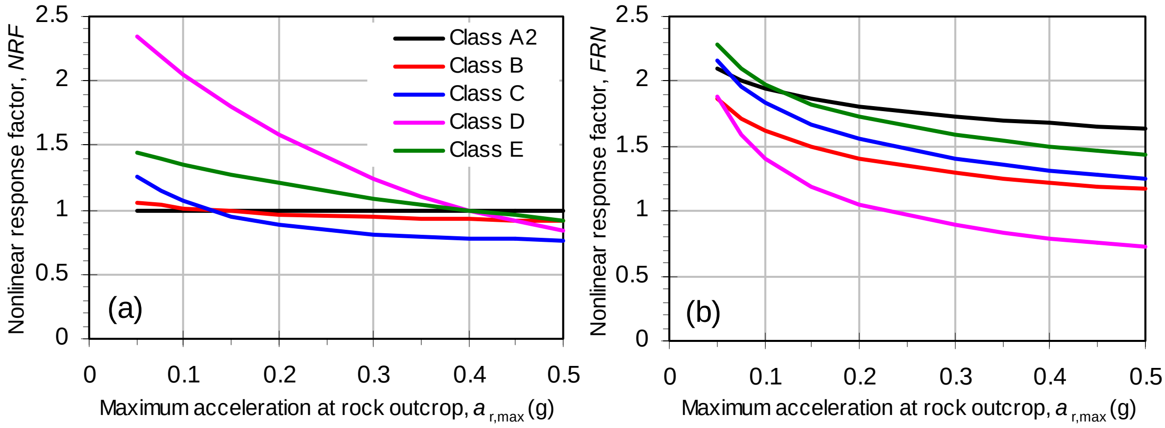

Figure 8. Nonlinear response factors (cf. table 4): a) literature curves b) this study.

In Figure 9b the values of $a_{eq,max}$, computed applying eq. (3) to SSR analyses and normalised by the expected maximum surface acceleration ($a_{s,max}=NRF \cdot a_{r,max}$), are plotted against the ratio (T_s/T_m) between the fundamental subsoil period and the mean value of acceleration time history. Note that the range of variability of T_s/T_m increases with soil deformability. For stiffer soils (classes A1 and A2), a larger number of data points is characterised by T_s/T_m ratio falling in the range 0÷1, i.e. the main earthquake frequency content is expected to fall below the first mode resonance condition. The opposite happens as the subsoil becomes more deformable, because (especially for classes D, E) T_s can often assume higher values than T_m. In such cases the soil column responds to shaking with the excitation of higher modal forms, which implies higher energy dissipation and asynchronous motion, resulting in a decrement of $a_{eq,max}$ value.
All the data sets for stiff to deformable soils in Figure 9b have been statistically processed to obtain the median curves and those relevant to probability of exceedance equal to 16% and 84%. For the subsoil class E, the median curve by Bray & Rathje (1998) is also reported for comparison.

Figures 10a÷d show all the median curves computed for the various soil classes through the four approaches (a, b, c and d) above described. It is worth noting that, under the assumption of linear amplification, with respect to the use of the *S* values at present recommended by EC8 (Fig. 10a), the modifications proposed by Bouckovalas et al. (2006) can lead to a minor dispersion of the mean curves (Fig. 10b), and to less restrictive equivalent seismic coefficients.

Furthermore, the use of the non-linear response factors *FRN*, proposed in this study for the various subsoil classes, leads to a much lower dispersion of the median curves (Fig. 10d), compared to those obtained using the literature *NRF* values (Fig. 10c).

It is then possible to refer to a single median curve of all the data obtained (Fig. 10e), showing a reduction factor of the peak surface acceleration decreasing below 0.4 for the deposits with a fundamental period larger than the predominant value of the seismic motion. Such values are, therefore, visibly lower than the value of 0.5 specified by both EC8 (EN1998-1, 2003) and seismic Italian code (OPCM 3274, 2003). The same graph shows the curves relevant to 16% and 84% probability of exceedance.

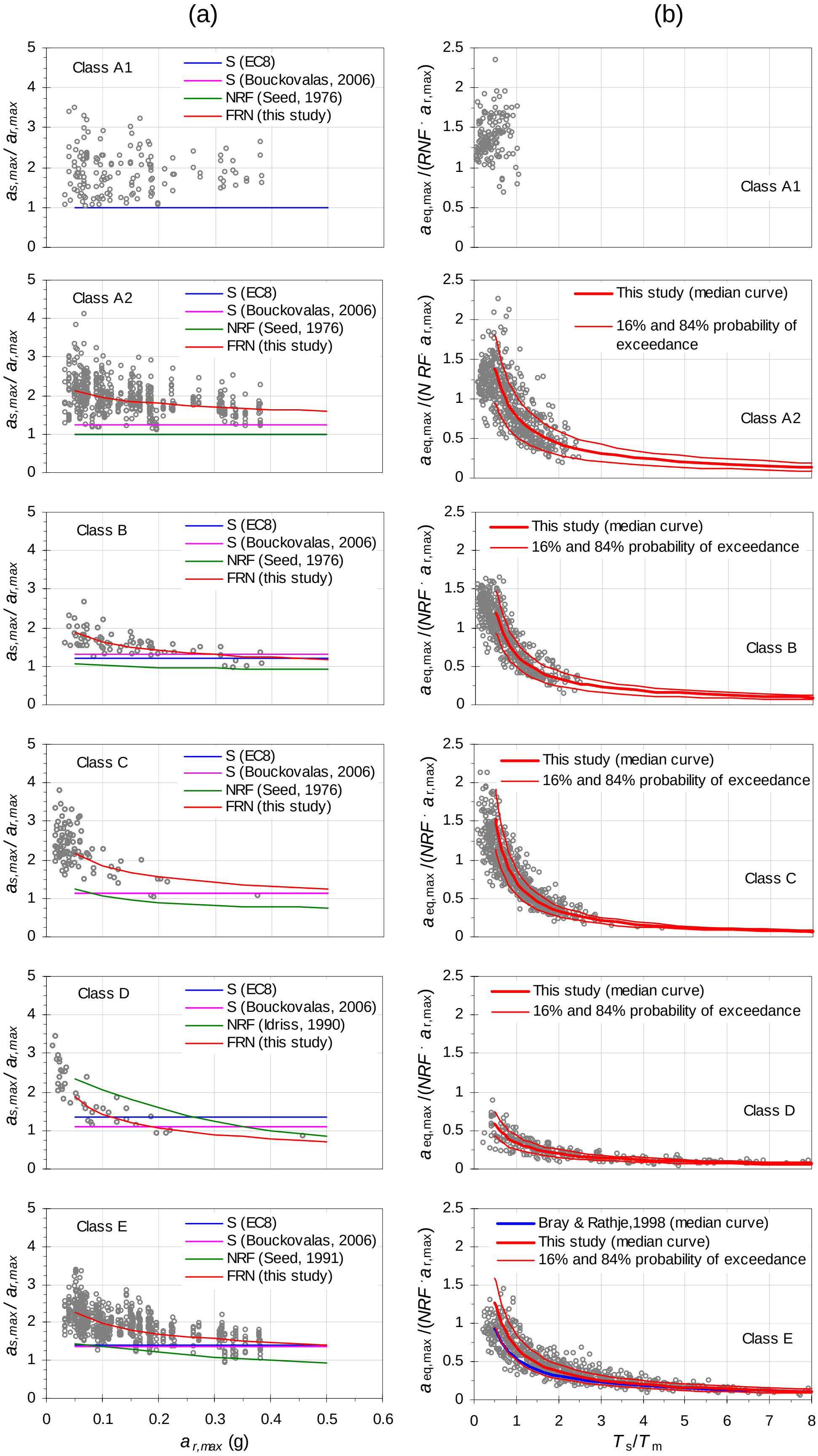

Figure 9. Data sets and median curves of nonlinear response coefficients (a) and normalised $a_{eq,max}$ (b), for each subsoil class.

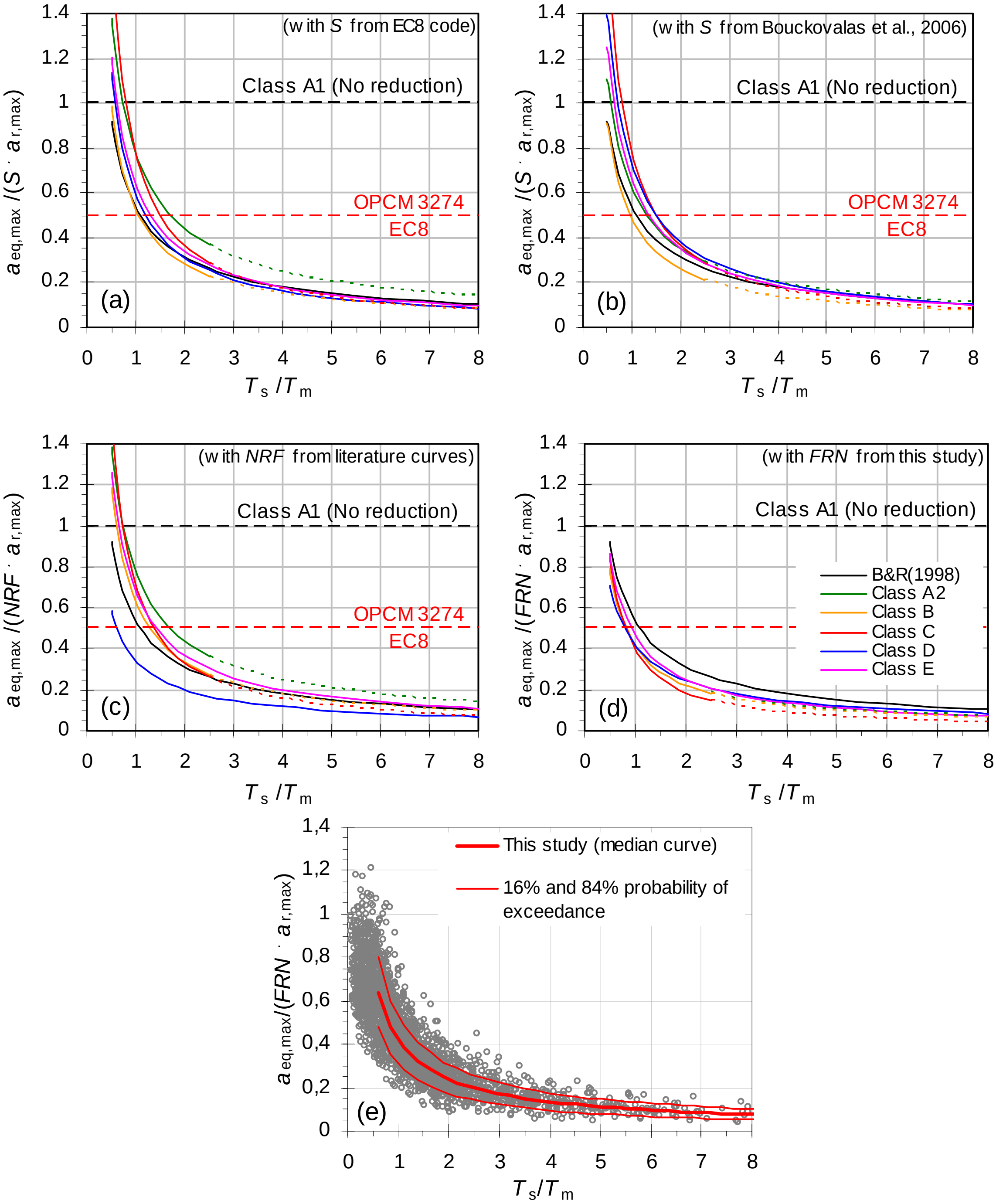

Figure 10. Reduction curves of the equivalent acceleration obtained for the four different approaches (a÷d) and overall median curves (e).

PREDICTION OF DISPLACEMENTS

Analysis

The dynamic analyses for the prediction of permanent displacements required a preliminary selection of suitable accelerograms, to exclude the seismic records expected to be less likely to trigger a sliding mechanism. In fact, the database was restricted to only those records picked at seismic stations having epicentral distances lower than the upper bound value predicted by the well-known relationship by Keefer & Wilson (1989), as a function of surface wave magnitude, MS. Summarising, 15 seismic

records were sorted out of 37 available for rock subsoil, 14 out of 49 for stiff subsoil, and 12 out of 21 for soft subsoil (Tropeano, 2006).
For each seismic record, the dynamic Newmark's analysis was used to obtain four displacement values, since for both horizontal motion components (usually EW and NS), the sliding displacement was computed both uphill and downhill. The calculations were performed through a computer code that directly returns the displacement values related to acceleration ratios, k_y/k_{max}, varying in a range between 0.1 and 0.9 with a prefixed step equal to 0.1 (Tropeano, 2006).
The results were processed in terms of absolute displacements, U, as well as of their values divided by the product between peak acceleration (k_{max}) and significant duration ($D_{5\text{-}95}$), as recommended by Rathje & Bray (1998). For each data set, regressions and statistic tests were carried out (Tropeano, 2006), using log-normal and Pearson's Beta probabilistic distributions, as suggested by Conte & Rizzo (1996).

Results
The absolute and normalised displacements are plotted against the acceleration ratio in Fig. 11a and Fig. 11b, respectively; the median and 90% probability of exceedance regression lines are shown with different thickness. At comparable conditions, lower regression curves are obtained for accelerograms recorded on rock (blu lines); since not affected by site effects, they generally present lower number of peaks and duration than those logged on stiff and soft soils (yellow and green lines). By referring to the normalised displacement (Fig. 11b), on the other hand, the scatter between the different accelerogram data sets can be considered negligible for the practical purpose.

Figure 11c shows the comparison among the median curves computed referring to various magnitude ranges (red lines), those proposed by Bray et al. (1998) and the confidence belt reported by Makdisi & Seed (1978). Figure 11d illustrates the overall median predictions and the 16% probability of exceedance, compared to those proposed by Bray & Rathje (1998). In both cases it may be observed that, on the average, the literature indications provide more conservative predictions than those obtained in this study. This could be expected given the different duration and frequency content of the set records used in this study, as shown by Fig. 4.
Finally, Figure 11e shows the comparison between the upper bound curves of absolute displacements and other similar correlations proposed in literature. The results of the present study substantially coincide with those obtained by Simonelli & Fortunato (1996), which were limited to the set of the accelerograms recorded during Irpinia earthquake in 1980 (M_S = 6.9). Such records are apparently characterised by the highest values of energy released, within the whole database considered in this study.

CONCLUSIONS

The simplified methods to evaluate seismic slope displacements generally make use of limit curves, which empirically relate computed and/or observed displacements to the ratio between k_y (i.e. the yield seismic acceleration) and k_{max} (i.e. peak acceleration). Such procedures do not require dynamic analyses and the determination of "design accelerograms", but can lead to an over-conservative estimate of displacements.

The innovation initially proposed by Makdisi & Seed (1978), and subsequently introduced in practice by Bray & Rathje (1998), is based on the elaboration of a method for the estimate of the equivalent acceleration, through the introduction of a set of synthetic parameters, representative of both seismic action and dynamic ground response. However, the original formulation of the method was verified to present some applicability limits, mainly due to the regional characteristics of seismic motions. Therefore, the aim of this study was the calibration of a similar procedure, with specific reference to Italian seismicity and to a wider spectrum of subsoil models. These latter were referred to the EC8 criterions for ground classification, recently acknowledged by the Italian standards.

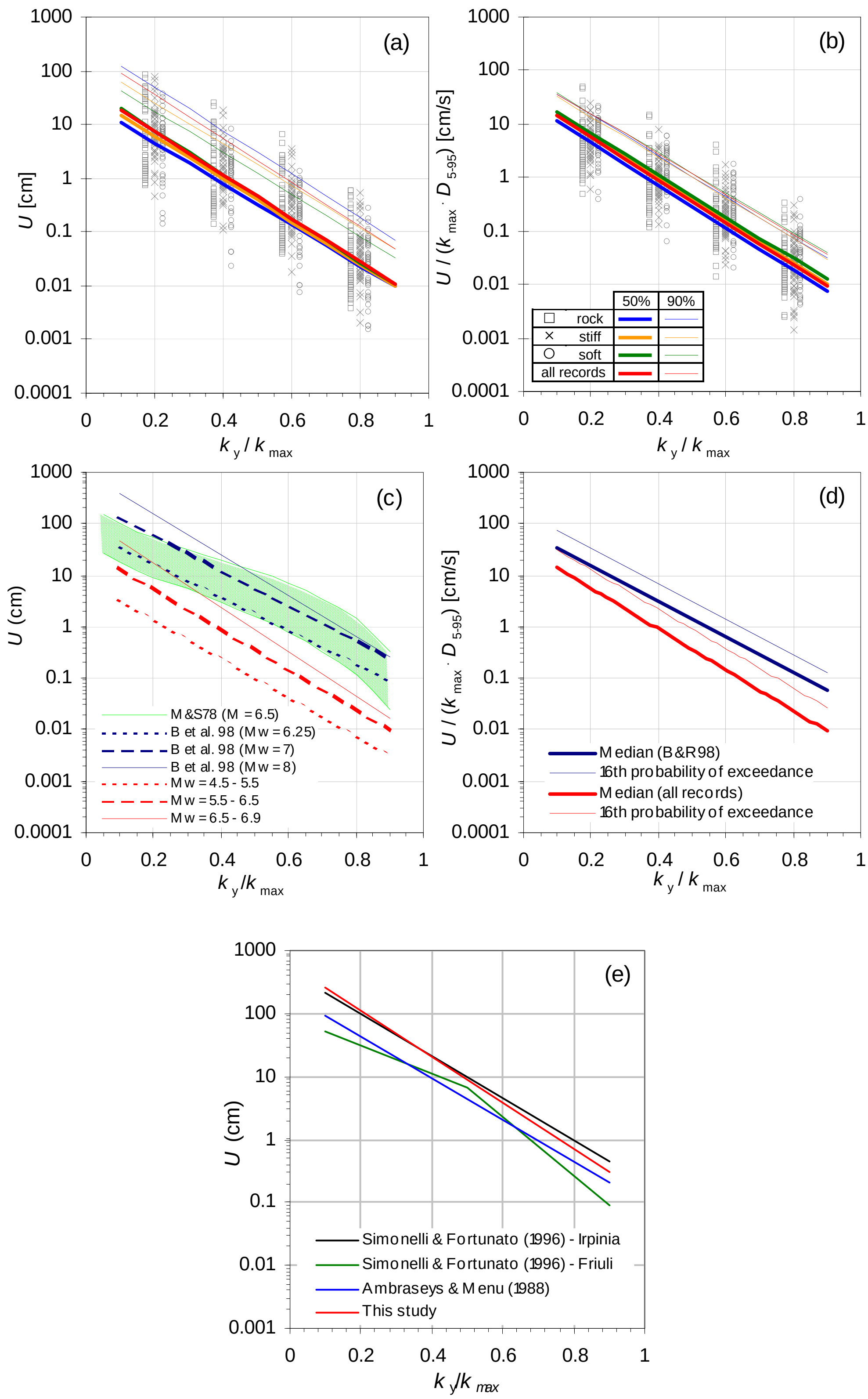

Figure 11. Statistical processing of displacements (a, b) and comparisons with literature (c, d, e).

It is worth noting that the site seismic response analyses carried out in this work yielded amplification coefficients even higher than those at present suggested by national and European standards, and decreasing with the reference acceleration. The use of such coefficients should not further penalize the seismic design actions for a slope, which can be conveniently reduced accounting for the effects of both ductility (i.e. the possibility to sustain displacements) and deformability, which tends to reduce the resultant of inertia forces due to the asynchronous motion. This method substantially considers such effects through one single parameter, the fundamental period of deposit, T_s, which, therefore, proves to be more significant than the combination of values of 'equivalent S-wave velocity' and thickness, as addressed by the recent EC8 workshop (ETC-12, 2006).

The analyses shown herein proved that the reduction factor for the equivalent acceleration can be expressed as an unique decreasing function of the period ratio, notwithstanding the soil type. The possibility of applying a significant reduction factor to the pseudo-static actions increases with the deformability, and thus the slope vulnerability. However, such possibility is only available if the frequency content of the motion is reliably estimated. The relations obtained can be used not only for a straightforward prediction of permanent slope displacements, but also for a rational evaluation of the equivalent seismic coefficient for the pseudo-static analyses, for instance if a threshold displacement and a particular subsoil typology are specified.

The statistical processing of the dynamic sliding block analyses first of all confirmed the validity of the normalisation of permanent displacement with respect to earthquake amplitude and duration, to reduce the scatter of their dependency on the acceleration ratio. The results also indicated less conservative median and upper bound regression lines, compared to those suggested by different Authors.

Nevertheless, some limits still exist in the application of this formulation to Italian seismicity, namely the need of specific attenuation relationships for mean period and significant duration. With the help of more sophisticated analytical tools than those used so far, an up-to-date validation would need to compare simplified dynamic analyses with more sophisticated ground modeling. This might take into account, for instance, soil non-linearity, which should affect the reduction components due to deformability and ductility at the same time. Also, geometrical crest and base effects might be expressed through methods with a degree of complexity intermediate between over-simplified topographic amplification factors, such as those currently specified by the standards, and full dynamic 2D analyses.

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