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The multiple facets of probabilistic seismic hazard analysis: a review of probabilistic approaches to the assessment of the different hazards caused by earthquakes

S. Barani[1,2], D. Spallarossa[1], P. Bazzurro[3] and F. Pelli[4]

[1] *DISTAV, Università degli Studi di Genova, Italy*
[2] *GEAmb S.r.l., Genova, Italy*
[3] *Ist. Univ. di Studi Superiori, Pavia, Italy (formerly at AIR Worldwide, San Francisco, USA)*
[4] *Geodeco S.r.l., Bogliasco (GE), Italy*

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ABSTRACT A seismic hazard assessment is often thought of as a process that calculates the mean annual rates of exceeding given ground-motion values on rock at a specific location. However, in many applications this view is reductive. A more complete definition should account for the hazard due to other effects induced by seismic activity at a site, such as the effect of soil deposits (including cyclic mobility and liquefaction) and topography on surface ground-motion, and for sites on slopes and for those straddle by fault lines, the effects of slope instability and of possible tectonic movements along faults. Moreover, for some coastal sites the effects of tsunami waves, not discussed in this paper, should be taken into account. The aim of this article is to present an overview of different probabilistic methods for advanced site-specific seismic hazard applications based on an extensive literature review and authors' experience. Application to real cases is also presented and discussed.

Key words: seismic hazard, local effects.

1. Introduction

The design of critical facilities like nuclear power plants (NPPs), nuclear waste repositories, pipelines (e.g., oil and gas pipelines), offshore platforms, major bridges, and hydraulic dams requires detailed geo-hazard studies in order to quantify the level of seismic risk to which such facilities are exposed. For instance, estimation of potential fault displacement may be fundamental for the design of pipelines, bridges and dams to prevent their failure due to damage from tectonic movement that may occur during their operational life [e.g., Alyeska pipeline (Alaska) that survived the 2002 Denali earthquake of magnitude 7.9 (e.g., Cluff *et al.*, 2003; Hall *et al.*, 2003; Sorensen and Meyer, 2003)]. Similar considerations apply to landslide movements and shaking levels induced by earthquake activity.

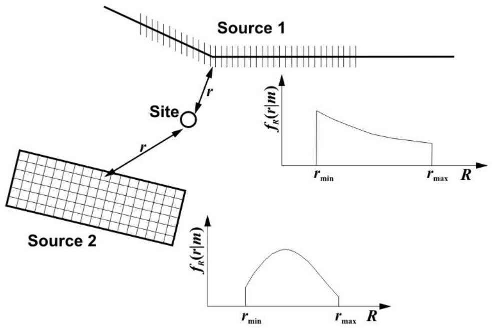
The term "Probabilistic Seismic Hazard Analysis" (PSHA) is commonly used to indicate a method to assess the ground-motion level expected with different likelihood at a rock site during a given period of time. This definition, however, is sufficient only in a relatively minor number of applications. A more general definition of PSHA should take into account a variety of earthquake-induced effects, such as fault displacement, slope displacement, liquefaction settlement, tsunami wave's maximum amplitude and velocity, and the ground shaking at the surface, both on flat ground conditions, which in most cases are represented by soil deposits, and on steep hills and ridges. In many cases site-induced phenomena are indeed responsible for a significant portion of the total damage and losses after earthquakes. The Indian Ocean tsunami following the December 2004 Sumatra earthquake, which killed more than 200,000 people [e.g., EERI, 2005], the tsunami generated by the 2011 Tohoku (Japan) earthquake, which caused more than 15,000 fatalities and was responsible for the Fukushima NPP accident (e.g., Fujii *et al.*, 2011), and the extensive liquefaction caused by the February 2011 Christchurch earthquake (e.g., Cubrinovski *et al.*, 2011; Mucciarelli, 2011), albeit rare, are perfectly fitting examples. Other noteworthy examples are the destructive Huascaran avalanche associated with the 1970 Peru earthquake (Ericksen *et al.*, 1970) and the numerous land and rock slides and debris flows triggered by the more recent Wenchuan (China) 2008 earthquake, causing about 20,000 fatalities at over 15,000 sites (Yueping *et al.*, 2009). Therefore, a PSHA should be rather viewed as a process that calculates the rate of occurrence of an earthquake-induced effect (e.g., level of shaking, fault displacement, slope displacement) at a site during a given period of time (e.g., McGuire, 2004). Independently of the induced hazard, the temporal occurrence of earthquakes can be modeled either as time-independent (e.g., Frankel *et al.*, 1996, 2002; Gruppo di Lavoro MPS, 2004; Petersen *et al.*, 2008) or a time-dependent process (e.g., Cornell and Winterstein, 1988; Working Group on California Earthquake Probabilities, 1995; Cramer *et al.*, 2000; Matthews *et al.*, 2002; Pace *et al.*, 2006; Petersen *et al.*, 2007; Akinci *et al.*, 2009). A schematic diagram illustrating the procedure for conducting a PSHA at a site for a generic earthquake effect C is shown in Fig. 1. Mathematically, it can be summarized as follows (McGuire, 2004):

$$\lambda_j (C > c) = \nu_j \iint P_j [C > c \mid \bar{s} \; at \; l] \; P [\bar{s} \; at \; l] \; d\bar{s} \; dl \qquad (1)$$

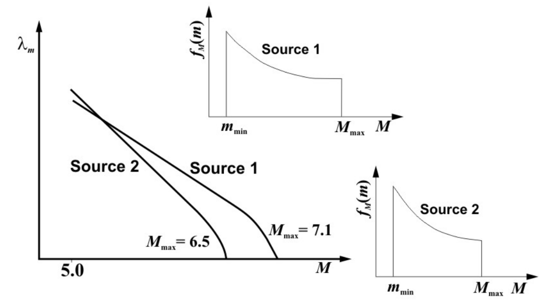
where $\lambda_j (C > c)$ is the annual rate that an earthquake-induced effect C exceeds a value c at a given site from an event caused by source j, ν_j indicates the mean rate of occurrence of earthquakes above a minimum magnitude of interest caused by source j, $P_j [C > c \mid \bar{s} \; at \; l]$ is the probability that c is exceeded at the site conditional on an earthquake with properties $\bar{s}$ at location l on source j, and $P [\bar{s} \; at \; l]$ indicates the annual probability that an earthquake with source properties $\bar{s}$ occurs at location l.

The aim of this work is to review probabilistic approaches for a comprehensive evaluation of the seismic hazard for the design and assessment of critical facilities. To this end, the following earthquake-induced hazards should be taken into account and evaluated: ground-motion hazard at site surface, slope displacement hazard, fault displacement hazard, and, for coastal sites, tsunami wave hazard. Concerning ground-motion hazard, site effects cannot be neglected, as the severity and frequency content of the ground shaking at a site are significantly dependent on the soil characteristics and local geomorphological features. The quantification of these effects is of primary importance for an accurate site-specific ground-motion hazard assessment [e.g., Costantino *et al.*, 1993; EPRI, 1993; Field and Petersen, 2000; Field and the SCEC Phase III Working Group, 2000; McGuire *et al.*, 2001, 2002; Baturay and Stewart, 2003; Cramer, 2003; Bazzurro and Cornell, 2004a, 2004b) that includes the linear/nonlinear response of soil and the

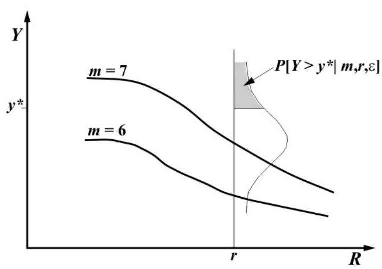
1) This step divides earthquake threat into sources (faults or areal sources) that generate earthquakes whose uncertain locations in space lead to distributions of distances. The uncertainty in source-to-site distance can be described by a probability density function $f_R(r|m)$ that is conditional on magnitude.



2) This step characterizes the seismicity of each source by specifying the distribution of earthquake magnitudes in a given period of time (i.e., number of earthquakes, λ_m, of magnitude $m_{min} <= m <= M_{max}$ per unit of time). The probability distribution of magnitude can be expressed in terms of a probability density function $f_M(m)$.



3) This step calculates the conditional probability of exceeding a specified value, y^*, of the interested parameter (e.g., ground acceleration, fault displacement, slope displacement), Y, for a given magnitude, m, distance, r, and ε standard deviations away from the median value predicted by an attenuation relationship for that parameter.



4) This step calculates the probability of exceeding y^* in a given period of time, t, by integrating over m, r, and ε. A Poisson process is generally assumed to model the occurrence of earthquake effects. Using alternative inputs leads to alternative hazard curves that can be reduced to a mean curve and a set of percentile curves.

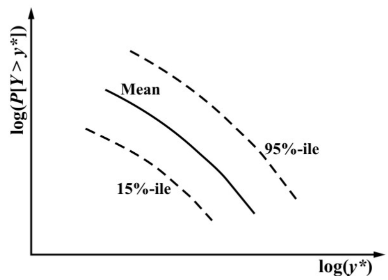


Fig. 1 - Four steps of a generic PSHA (after McGuire, 2004).

influence of local geomorphology (e.g., topographic effects, basin effects). Note that, to some extent, all hazards mentioned above have been considered in studies for critical facilities as far back as the 1990s. However, some of them have been often addressed in a deterministic manner rather than using fully probabilistic approaches. It should be noted, however, that this article does not intend to discuss the merits and demerits of probabilistic and deterministic approaches to seismic hazard assessment. The purpose, here, is simply to present a critical review of existing probabilistic methodologies that may not always be required or even appropriate for some hazard assessment applications.

In this article, we explore the multiple facets and criticalities of ground-motion hazard, focusing on methodologies that allow for the effects produced by soil deposits and topography on ground shaking. Slope displacement hazard will be also discussed in depth and a few words will also be spent on fault rupture hazard. Due to space limitations, liquefaction hazard and tsunami hazard will not be addressed here. Concerning liquefaction hazard, the interested reader is referred to studies such as Atkinson *et al.* (1984), Martin *et al.* (1999), Juang *et al.* (2001, 2002, 2008), Kramer and Elgamal (2001), Hwang *et al.* (2005), Kramer and Mayfield (2005, 2007), Finn and Wightman (2007), Goda *et al.* (2011). Regarding tsunami hazard, readers are referred to the studies of Lin and Tung (1982), Rikitake and Aida (1988), Downes and Stirling (2001), Geist and Parsons (2006), Annaka *et al.* (2007), Pacific Gas and Electric Co. (2010). Besides methodologies, the article will also present various case studies, showing applications of different approaches to handle specific induced effects within the wider PSHA framework. Criticalities related to PSHA will be also addressed, highlighting the importance of acknowledging and documenting sensitivity and uncertainty for a correct understanding and use of PSHA results.

2. Ground-motion hazard

The fundamental concepts of modern Probabilistic Ground-Motion Hazard Analysis (PGMHA) date back to 1968 when Cornell published his seminal work, introducing a method for evaluating the likelihood of exceedance (or occurrence) of a given earthquake ground shaking level at a site during a given period of time (Cornell, 1968). Although the original approach conceptually still holds, substantial progress has been made to refine the methodology. Recent advances in PGMHA have concerned the introduction of the logic tree formalism (Power *et al.*, 1981) to account for the epistemic uncertainty affecting input models and parameters as well as the introduction of sensitivity and uncertainty analyses aimed at the identification of the models and parameter values that have the highest influence on the hazard and its uncertainty (e.g., Rabinowitz and Steinberg, 1991; SSHAC, 1997; Grünthal and Wahlström, 2001; Barani *et al.*, 2007). Another important contribution to the progress and refinement of the original methodology is related to the hazard disaggregation process (e.g., Chapman, 1995; McGuire, 1995; Bazzurro and Cornell, 1999; Barani *et al.*, 2009) which provides insights into the earthquake scenarios driving the site hazard at given ground-motion levels.

PGMHA has since become a diffuse practice both in the scientific and engineering communities. Nowadays, many countries in the world have their national hazard map and many more which currently lack one will have a seismic hazard map to adopt, should they desire to do so, when the Global Earthquake Model (GEM: www.globalquakemodel.org/) will release its results in 2014. Moreover, many national and international building codes [e.g., Eurocode 8 (EC8): CEN, (2003); National Earthquake Hazard Reduction Program (NEHRP): BSSC, (2003); Italian building code: Ministero delle Infrastrutture e dei Trasporti, (2008)] use probabilistic ground-motion values (i.e., ground-motions with a certain probability of exceedance in a given time period) to define the seismic forces and displacement values to be used during design. Similarly, seismic design procedures and criteria for petroleum and natural gas platforms and other facilities (International Standard Organization for Standardization, 2004) are based on ground-motion levels corresponding to a specific mean return period (MRP which is defined as the reciprocal of the mean annual rate of exceedance MRE). Hence, the demand for probabilistic ground-motion hazard evaluations has significantly increased in recent years. However, it is noteworthy that many ground-motion hazard studies are conducted for rock conditions (generally, the term "rock" is used to indicate sites with average shear wave velocity, $V_{S,30}$, in the top 30 m of a soil profile greater than 760 - 800 m/s) and almost the totality of such studies implicitly assumes flat topography, thus neglecting the influence of local geology and geomorphology. Although the assumptions of rock conditions and flat topography are acceptable in the case of large-scale hazard mapping, they may not be when a hazard analysis is used for the design of a critical facility. Indeed, neglecting site response may result in a severe underestimation of the local hazard.

As conventional methodologies for rock ground-motion hazard assessment have been largely described in the scientific literature (e.g., Kramer, 1996; McGuire, 2004, 2008), here we focus on site-specific approaches that account for the effects of local soil properties and geomorphology on the ground-motion.

2.1. Soil effects in PGMHA

The effect of local soil deposits on probabilistic hazard estimates of ground-motion at the site surface is sometimes neglected or treated with little rigor. Two simple approaches are often used. The first, which is typically applied in the case of noncritical facilities, consists of using one or more ground-motion predictive equations (GMPEs) with parameter values tuned for specific soils classes. In other words, this approach assumes that the soil conditions at the site resemble those at the seismic stations in the database considered for the development of the GMPEs used in the PGMHA. "*This approach ignores virtually all site-specific information and, therefore, produces only a broad, generic assessment of the hazard*" (Bazzurro and Cornell, 2004b) and likely results in an overestimation of the true but unknown site hazard. Another widely used approach consists of multiplying the probabilistically evaluated hazard on rock by deterministic oscillator-frequency-dependent (or, sometimes, independent) amplification functions (or factors) derived, for example, from numerical soil response analysis. Although a suite of recorded or synthetic seismograms is generally driven through a numerical model of the soil column to extract the single deterministic value used, this approach essentially neglects the amplification factor record-to-record variability and the negative correlation between the input ground-motion intensity measure (IM) at the bedrock and the amplification factor, which is caused by the effects of soil nonlinearity. Moreover, "*it produces surface ground-motion levels whose exceedance rates are unknown, non-uniform, inconsistent across frequency and generally nonconservative*" (Bazzurro and Cornell, 2004b). In many applications, deterministic amplification functions are replaced by average amplification factors extracted from national building codes which, however, may only be loosely representative of the actual local soil conditions and may embed a level of conservatism that is, in general, unknown.

To the extent that soil amplification analyses provide an adequate representation of the dynamic soil behavior under the impact of strong ground-motion, the method proposed by Bazzurro and Cornell (2004a, 2004b; hereinafter called BC method or BC approach) overcomes the previous limitations by accounting for local soil conditions and by incorporating the amplification function variability and the correlation between bedrock input and amplification function, $Y = AF\,(f)$, which is defined as the ratio of the spectral acceleration at the surface $Z = S_a^s\,(f)$ to the rock-level spectral acceleration $X = S_a^r\,(f)$. The BC method, which is based on convolution (e.g., Benjamin and Cornell, 1970), consists of combining in a probabilistically robust way the rock-hazard curve with the probability distribution of the amplification function obtained from numerical soil response analyses:

$$G_Z(z) = \int_0^{\infty} P\left[Y \geq \frac{z}{x}\middle| x\right] f_X(x)dx = \int_0^{\infty} G_{Y|X}\left(\frac{z}{x}\middle| x\right) f_X(x)dx \tag{2}$$

where $G_Z(z)$ is the complementary cumulative distribution function (CCDF) of the spectral acceleration at the soil surface $S_a^s(f)$ [in other words, $G_Z(z)$ is the sought hazard curve for $S_a^s(f)$], $P\left[Y \geq \frac{z}{x}\middle| x\right]$ is the probability of exceeding a ground-motion amplification value z/x conditional on a rock-level amplitude x, $f_X(x)$ is the probability density function of the spectral acceleration on rock $S_a^r(f)$ (it can be obtained by differentiating the rock-hazard curve), and $G_{Y|X}$ is the CCDF of the amplification function Y conditional on a rock-level amplitude x. More specifically, $G_{Y|X}$ is given by:

$$G_{Y|X}\left(\frac{z}{x}\middle| x\right) = 1 - \hat{\Phi}\left[\frac{\ln\left(z/x\right) - \ln(\hat{m}_{Y|X}(x))}{\sigma_{\ln Y/X}}\right] \tag{3}$$

where $\hat{\Phi}[\cdot]$ indicates the complementary standard Gaussian CDF. Estimates of the conditional median of Y, $\hat{m}_{Y|X}(x)$, and the conditional standard deviation of $\ln(Y)$, $\sigma_{\ln Y/X}$, can be found by driving a suite of appropriately selected rock ground-motion recordings through a numerical model of the soil column and then regressing, for each frequency f, the values of $\ln(Y)$ on $\ln(X)$. The earthquake recordings used in the ground response analysis should be, to the extent possible, consistent with the scenario events controlling the site hazard as identified via hazard disaggregation. Furthermore, the definition of the IM of the input ground-motion should be consistent with that used by the developers of the GMPE adopted for the rock PGMHA. More specifically, if one uses a standard GMPE developed for the geometric mean of a ground-motion parameter, then the same geometric mean of the two components should be used during the statistical regression performed for the estimation of $AF(f)$ (Baker and Cornell, 2006; Barani *et al.*, 2010). A careless selection of ground-motion records and the mismatch between the definition of the IM in the GMPE and the IM used during regression would introduce potentially significant inaccuracies in the estimate of the annual rates of exceeding given ground-motion values at the soil surface. Of course, the same consistency of IM definition has to be assured also when the GMPE used in the rock PGMHA is developed for the larger of the two horizontal components of the ground-motion.

Therefore, the BC method can be summarized in three major steps: 1) calculation of the rock-hazard curve at the study site; 2) determination of $AF(f)$ via regression analysis of data extracted from analytical soil responses; 3) convolution of the hazard curve on rock with the soil response function.

Concerning predictors in the regression model, $S_a^r(f)$ was found the single most helpful parameter for the prediction of $AF(f)$ at the same oscillator frequency. $S_a^r(f)$ was found more informative than peak ground acceleration (PGA) and/or the pair of magnitude and distance values of the event that generated the rock input motion (Bazzurro and Cornell, 2004a). Therefore, a single regression model in $S_a^r(f)$ is sufficient to provide accurate and effective predictions of $AF(f)$ at a specific frequency, f. As an example, Fig. 2 shows the relationship between $S_a^r(f)$

and $AF(f)$ at an oscillator frequency of 3.5 Hz for a site in western Liguria characterized by recent alluvial deposits (mainly composed of gravel, sand, and silty-sand) of approximately 70 m thickness [for details see Pelli *et al.*, (2006)]. For the same site, Fig. 3 compares the median and 85th percentile uniform hazard spectra (UHSs) on rock for an MRP of 475 years with those incorporating soil effects. The median UHS corrected by the EC8 type B-soil amplification factor (S = 1.35) is also displayed for comparison (hybrid spectrum). The figure clearly shows significant ground-motion amplification for oscillator frequencies between around 1 Hz and 4 Hz. In particular, at a frequency of 3.5 Hz, the median spectral acceleration hazard increases of approximately 33%, from around 0.43 g on rock to 0.57 g at surface. At the same frequency, the soil spectral acceleration resulting from the application (at each frequency) of the EC8 ground factor to the median rock UHS is compatible with that obtained using the fully probabilistic approach. However, for frequencies above 3.5 Hz, the hybrid spectral acceleration values are significantly larger than those from the probabilistic method. Conversely, they are lower for frequencies below 3.5 Hz. The lesson from this simple example is that the application of average soil factors extracted from building codes to a rock UHS may yield response spectra that may be either over-conservative or under-conservative (with respect to the soil UHS resulting from the fully probabilistic approach) dependently of the frequency range. However, neglecting site amplification may result in non-conservative hazard estimates, particularly in the frequency range around the fundamental frequency of the soil deposit. In other words, critical facilities designed neglecting local site effects within the framework of PGMHA may have lower and unknown margin of safety than expected. Note that if the application at hand suggests that more than one ground-motion parameter is deemed necessary for representing the ground-motion hazard at the soil surface, then the scalar PGMHA can be replaced by its vectorized version (Bazzurro and Cornell, 2002) and multiple regression of the analytical soil responses will be necessary to assess the joint hazard at the soil surface (Bazzurro and Cornell, 2004b).

Besides the approach discussed above, Bazzurro and Cornell (2004b) proposed a more straightforward (but approximate) method that consists of including $AF(f)$ directly into an existing rock GMPE for $S_a^r(f)$, thus transforming it into a site-specific GMPE. This simplified approach will be discussed in the next section when it will be applied for the surface ground-motion hazard assessment of a rock site at the top of a ridge. Note that a probabilistic, albeit less robust, methodology using site-amplification distributions to modify existing rock GMPEs into site-specific relations was also proposed by Cramer (2003, 2005) but it will not be discussed here.

2.1.1. Including soil property uncertainty into $AF(f)$

Careful readers may have noticed that the example presented in the previous section neglects the $AF(f)$ variability produced by the uncertainty in the soil characteristics. Incorporating the soil property uncertainty into a site-specific PGMHA can be achieved by Monte Carlo randomization of the soil properties used in the ground response modeling and then by running numerical simulations for all randomized soil columns (e.g., Bazzurro and Cornell, 2004a; Prieto and Ramos, 2006; Barani *et al.*, 2008). On this subject, Barani *et al.* (2012a) point out the importance of modeling the uncertainties affecting the local soil stratigraphy and the soil parameters. They found that, for the case studies presented in their paper, the largest contribution to the total uncertainty in the amplification function is due to the uncertainty in the soil characteristics rather than to the

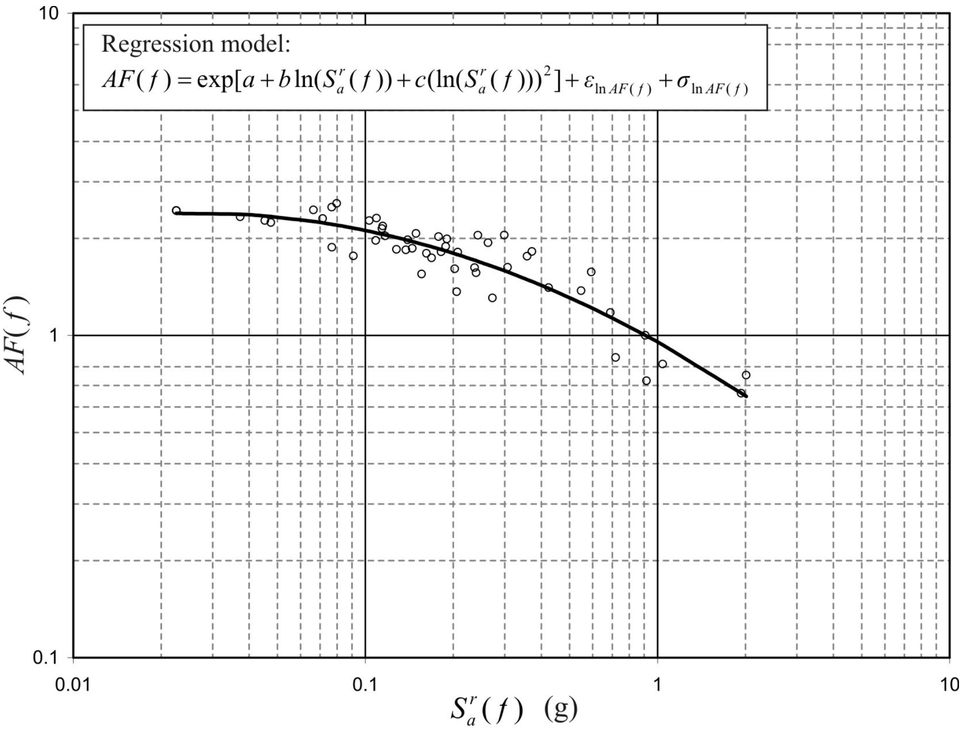


Fig. 2 - Regression of $AF(f)$ on $S_a^r(f)$ at an oscillator frequency of 3.5 Hz for a site in western Liguria (after Pelli *et al*., 2006).

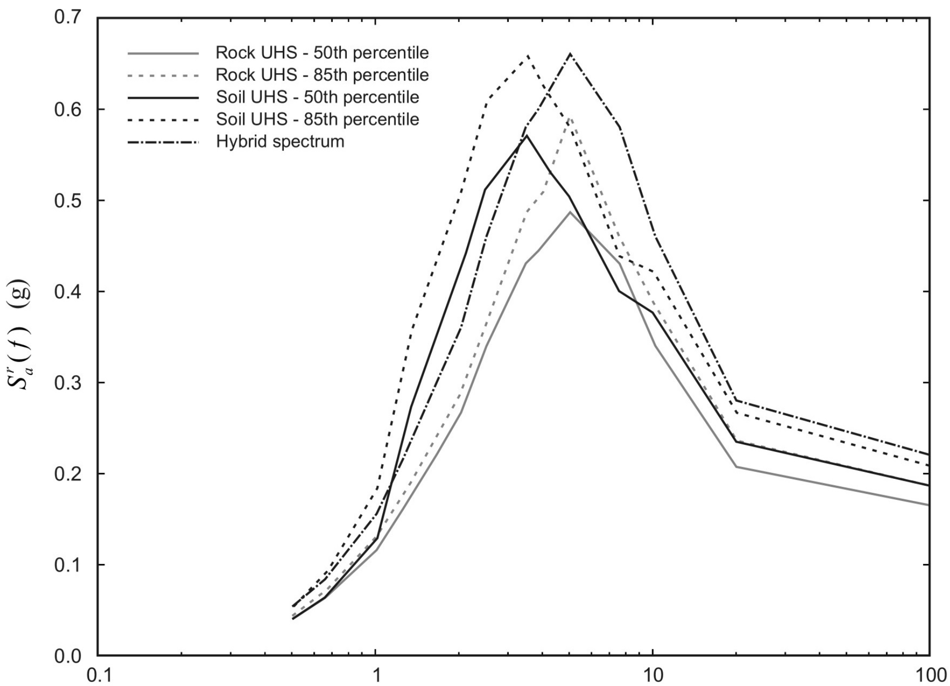


Fig. 3 - Comparison of rock and soil uniform hazard spectra for an MRP of 475 years with the rock UHS corrected by the EC8 type B-soil amplification factor (hybrid spectrum) for the same site considered in Fig. 2.

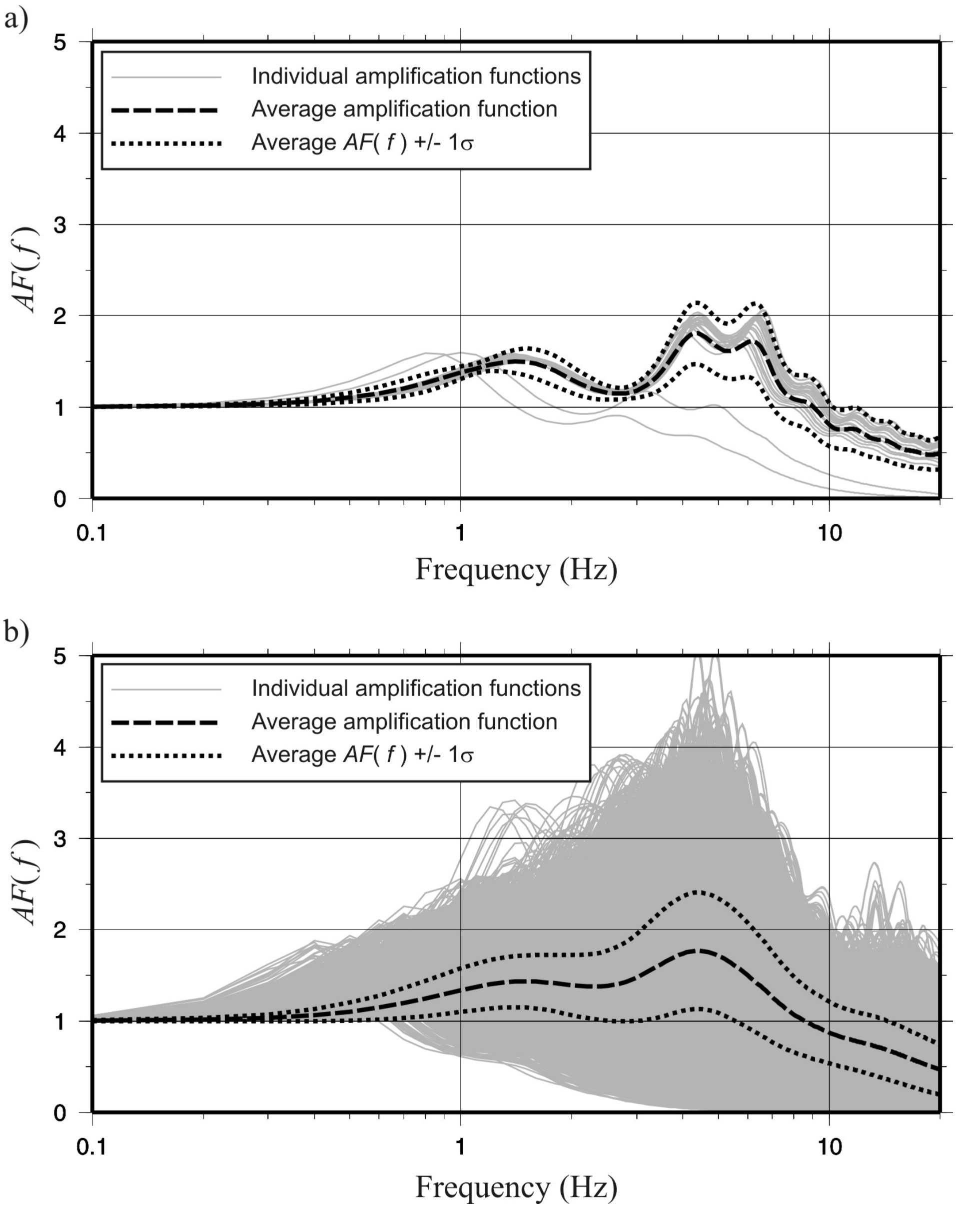


Fig. 4 - Families of amplification functions reflecting: a) the variability of the input motion alone and b) both the input motion variability and the soil property uncertainty for a target site in northern Tuscany.

record-to-record variability of the input bedrock motions. This is visible in Fig. 4 where, for the S2a site in northern Tuscany in the article of Barani *et al*. (2012a), a comparison is presented between the uncertainty in $AF(f)$ caused by the variability of the input motion alone (Fig. 4a) and the uncertainty in $AF(f)$ resulting from both the record-to-record variability and the soil property uncertainty (Fig. 4b). These findings differ from those in Bazzurro and Cornell (2004a), who analyzed two offshore sites characterized by different 100 m deep soil deposits (a saturated sandy site and a saturated soft clayey site) and found the bedrock ground-motion variability to have a larger impact on the $AF(f)$ than the uncertainty in the soil parameters. Although the reasons of the difference have not yet been fully investigated, it can be observed that Barani *et al*. (2012a) adopted a linear-equivalent soil response approach as opposed to the non-linear soil response method used by Bazzurro and Cornell (2004a). Moreover, the analyses by Barani *et al*. (2012a), unlike those by Bazzurro and Cornell (2004a), do not drive the soil columns to high nonlinear responses. The suite of seismograms employed by Barani *et al*. (2012a) covers a *PGA* range from 0.02 g to 0.54 g, inducing only minor nonlinear soil effects, while that used by Bazzurro and Cornell (2004a) includes very strong motion recordings with horizontal *PGA* up to 1.5 g. This larger range of input motions may result in a larger record-to-record variability than that found by Barani *et al*. (2012a). Perhaps more importantly, however, Barani *et al*. (2012a) modeled the uncertainty in more aspects of their analysis than Bazzurro and Cornell (2004a) did. They considered a more significant seismic impedance contrast between the bedrock and the overlaying alluvial deposits and considered the bedrock stiffness and the soil column depth as random variables (particularly important at one site where bedrock depth was unknown). An exhaustive sensitivity analysis has revealed that, for the case studies analyzed by Barani *et al.* (2012a), the shear wave velocity (V_S) is the dominant factor controlling the total uncertainty in $AF(f)$ and soil fundamental frequency. The uncertainty affecting the soil layer thickness (i.e., the uncertainty in the bedrock depth) may also contribute largely to the total $AF(f)$ uncertainty, especially in the case of profiles with very uncertain bedrock depth (Barani *et al*., 2012a). From these early findings it appears that the uncertainty in the soil characteristics should be taken into account in the assessment of surface hazard, especially in those cases where significant impedance contrasts are present.

2.2. Topographic effects in ground-motion hazard analysis

The influence of topographic irregularities and, more generally, the effects of geomorphology are virtually neglected in traditional PGMHA [however, they are considered in the simulation-based seismic hazard studies at the Southern California Earthquake Center (SCEC) (e.g., Field and the SCEC Phase III Working Group, 2000; Graves *et al*., 2010)]. However, it has long been recognized that geomorphology (e.g., topographic irregularities and alluvial valleys) may have a strong impact on the level and frequency content of the ground-motion induced by an earthquake at a site (e.g., Davis and West, 1973; Geli *et al*., 1988; Kramer, 1996; Bard and Riepl-Thomas, 1999; Paolucci, 2002; Bouckovalas and Papadimitriou, 2005; De Ferrari *et al*., 2010). Therefore, site-specific PGMHA should not neglect the amplification effects related to particular geomorphological features. To this end, Barani *et al.* (2012b) developed a method that allows the inclusion of rock topographic irregularities in the framework of PGMHA. The method is an extension of the original BC approach and, to the extent that the real response of the crest or ridge is accurately represented by the 2D (or 3D) response computed by the software at hand, it is applicable to any kind of rocky crests or ridges that may affect site response.

Compared to the original BC approach for 1D soil amplification, the method of Barani *et al*. (2012b) does not require the convolution between a site-specific response function and the reference hazard at the base of the topographic formation. Rocky ridges behave as perfect linear bodies and, consequently, the ground-motion amplification at the top of a ridge does not significantly vary with the input motion level and with its frequency content, which is controlled by the magnitude of the causative event and, to a lesser extent, by the source-to-site distance. In other words, if the ridge can be assumed to be a homogeneous rock system, then its resonance

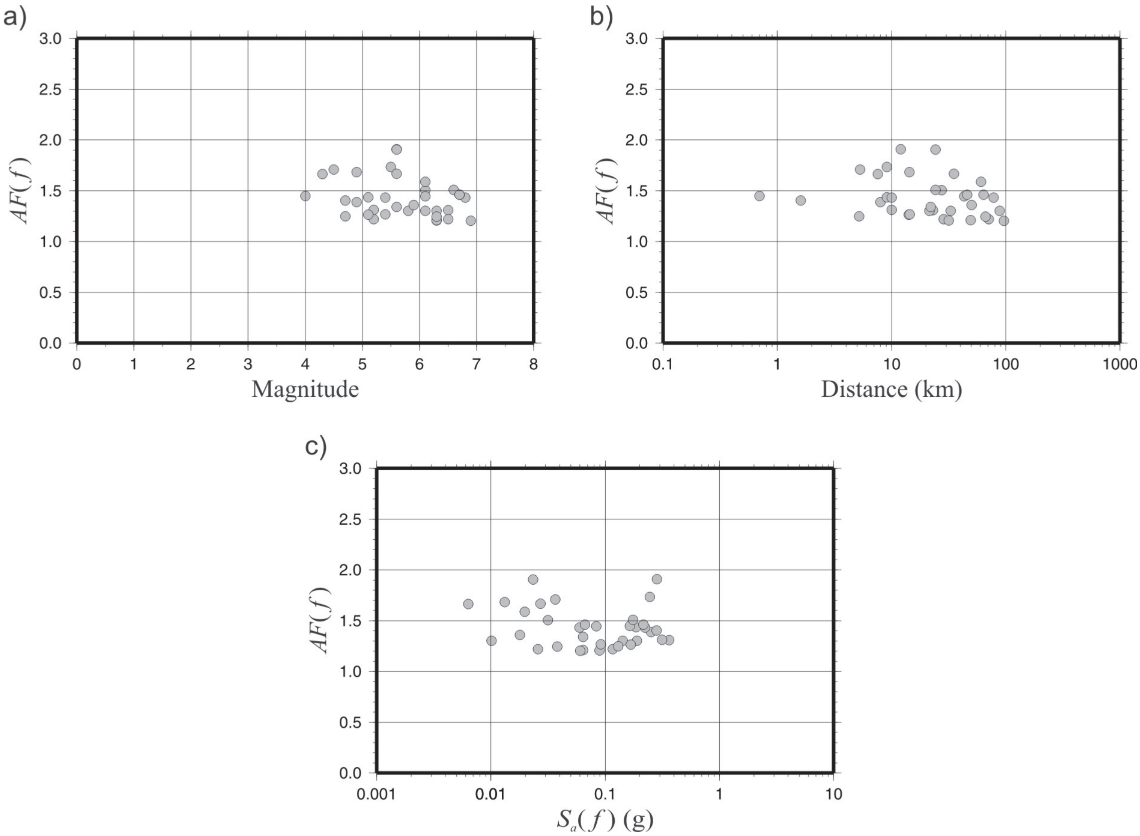


Fig. 5 - Variation of $AF(f = 100$ Hz) as a function of: a) magnitude, b) epicentral distance, and c) $S_a(f = 100$ Hz) at the top of the Narni ridge.

frequency is rather insensitive to the amplitude and frequency content of the input motion. As an example, Fig. 5 shows the variation of $AF(f)$ as a function of magnitude, epicentral distance, and spectral acceleration for $f = 100$ Hz ($\approx PGA$) at the top of the Narni ridge in the central Apennines [details on calculations can be found in the article of Barani *et al.* (2012b)]. Similar trends were obtained at other frequencies. Hence, the site amplification estimated via numerical simulation can be directly included into an existing rock GMPE for $S_a(f)$ [Barani *et al.*, (2012b), see also Bazzurro and Cornell, (2004b)]:

$$\ln S_a^{crest}(f) = \ln \overline{S_a(f)} + \varepsilon_{\ln S_a(f)}\sigma_{\ln S_a(f)} + \ln \overline{AF(f)} + \varepsilon_{\ln AF(f)}\sigma_{\ln AF(f)} \tag{4}$$

where $S_a^{crest}(f)$ is the spectral acceleration at the crest of the ridge, $\ln \overline{S_a(f)}$ is the median of $S_a(f)$ predicted by a GMPE given M and R (and, possibly, other characteristics such as the source mechanism), $\ln \overline{AF(f)}$ can be estimated (for each frequency) by averaging over the ln $AF(f)$ values resulting from 2D (or 3D) numerical simulations, $\sigma_{\ln S_a(f)}$ and $\sigma_{\ln AF(f)}$ are the standard errors of ln $S_a(f)$, conditional on M and R, and ln $AF(f)$, and $\varepsilon_{\ln S_a(f)}$ and $\varepsilon_{\ln AF(f)}$ are standard normal variables.

The dispersion measure for ln $S_a(f)$ at the crest of the ridge can be simply calculated as:

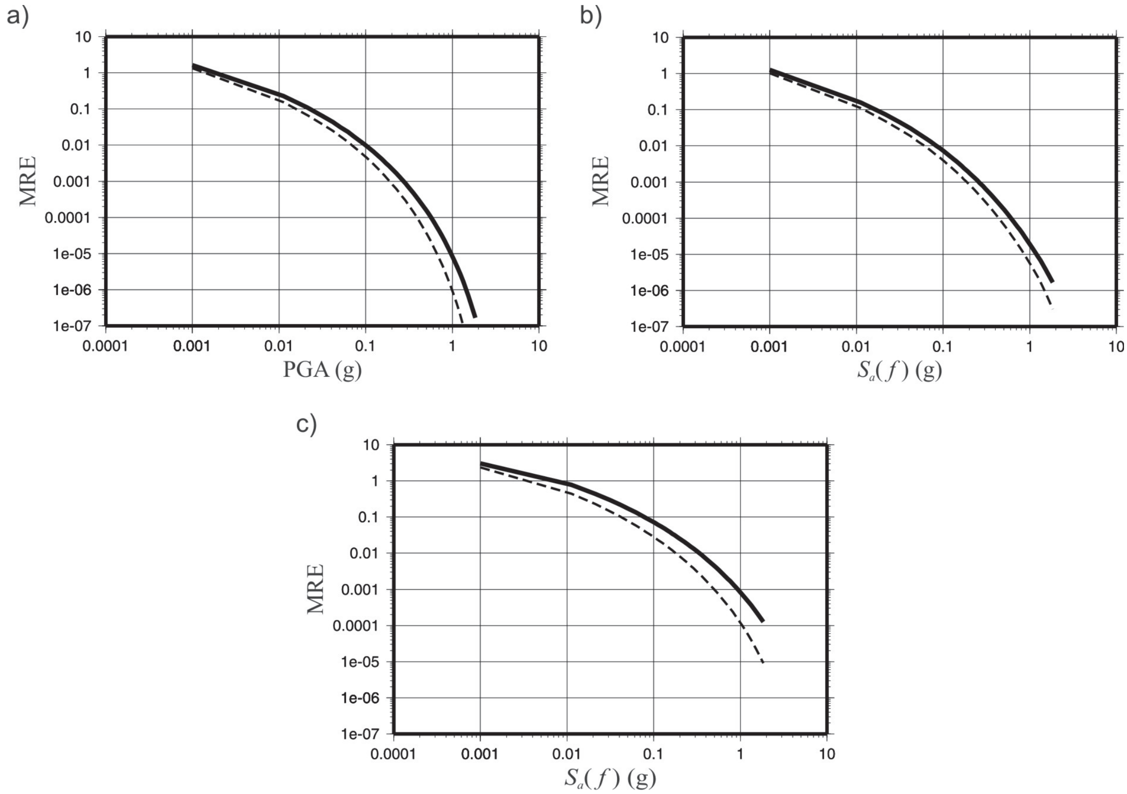


Fig. 6 - Ground-motion hazard curves for: a) *PGA*, b) 1.67 Hz (T = 0.6 s) spectral acceleration, and c) 6.67 Hz (T = 0.15 s) spectral acceleration at the base (dashed line) and top (solid line) of the Narni ridge.

$$\sigma_{\ln S_a^{crest}(f)} = \sqrt{\sigma^2_{\ln S_a(f)} + \sigma^2_{\ln AF(f)}} \qquad (5)$$

Note that, as the resonance of rocky crests and ridges displays only minor sensitivity to the characteristics of the incoming seismic motion, sufficiently accurate estimates of $\ln \overline{AF(f)}$ and $\sigma_{\ln AF(f)}$ can be obtained by using very few records in the numerical simulations.

Fig. 6 compares the *PGA*, 1.67 Hz and 6.67 Hz spectral acceleration hazard curves at the base (dashed line) and top (solid line) of the Narni ridge (Barani *et al.*, 2012b). Similarly to the example in Fig. 3 for 1D soil amplification assessment, it is apparent that neglecting site amplification produced by topographic irregularities may result in severe hazard underestimation, particularly in the frequency range where larger amplification is observed [in this example, at approximately 1.6 Hz and between 5 Hz and 8 Hz (Massa *et al.*, 2010)]. For instance, for an MRE of 0.0011/yr (MRP of 1000 years), the PGA hazard increases by approximately 42%, from 0.19 g to 0.27 g.

Presumably, the method described above can be extended with some changes to non-homogeneous ridges characterized by a soil cover at the surface or to alluvial valleys where basin geometry and filling sediments contribute to the definition of site response. The key aspect of this extension is whether the responses of these more complex irregularities can be considered realistic. This validation would require a wealth of real recordings at the base and at a top of ridges that, unfortunately, is currently unavailable. However, studies available from the literature (e.g., Pischiutta *et al.*, 2010; Lovati *et al.*, 2011; Barani *et al.*, 2012b) evidence that numerical analyses often yield an underestimation of the actual amplification measured via experimental methods. Possible causes for this may be related to oversimplification of numerical modeling or to analytical solutions, which could be unable to effectively reproduce the wave-field modified by topographic irregularities. Assuming for the sake of this discussion that the software adopted for the ridge response is reliable as well as the 2D or 3D model, a regression equation predicting the soil amplification as a function of one or, possibly, more rock ground-motion parameters is required. Again, the uncertainty affecting soil parameters could be taken into account through Monte Carlo randomization (see previous section) if enough computing power is available for performing many analyses using 2D or 3D models of the ridge. As in the case of flat soil sites, if a linear predictive model for $\ln AF(f)$ in terms of $\ln S_a(f)$ is appropriate, then Eq. (4) becomes [the complete mathematical formulation can be found in the article of Bazzurro and Cornell (2004b)]:

$$\ln S_a^{crest}(f) = c_0 + (c_1 + 1) \ln S_a(f) + (c_1 + 1)\, \varepsilon_{\ln S_a(f)} \sigma_{\ln S_a(f)} + \varepsilon_{\ln AF(f)} \sigma_{\ln AF(f)} \qquad (6)$$

where c_0 and c_1 are coefficients obtained from the regression of $\ln AF(f)$ on $\ln S_a(f)$.

As a consequence, the term $\sigma_{\ln S_a(f)}$ in Eq. (5) will be multiplied by $(c_1 + 1)^2$.

Alternatively to this simplified approach, one may prefer to apply the more rigorous method described in Section 2.1.

3. Slope displacement hazard

A further extension and refinement of the original probabilistic BC approach for 1D site amplification assessment is that of Barani *et al.* (2010) for the evaluation of permanent slope displacements induced by earthquake activity (the method is also applicable to embankments and earth/rockfill dams). Basically, the method, which conceptually originated from an early work of Bazzurro *et al.* (1994), uses a set of 2D (or 3D) numerical analyses to establish a probabilistic relationship (soil response function) between one or more ground-motion parameters and the permanent displacement at a specific location within the slope under study. Again, the soil response function can be coupled with the rock-hazard curve to establish the MRE of permanent slope deformations of different severity (Bazzurro *et al.*, 1994; Rathje and Saygili, 2008).

Contrary to the previous sections, we do not focus here on theoretical and methodological aspects of the procedure for probabilistic slope displacement hazard analysis (PSDHA), as the method is very similar to that presented in Section 2.1 and, moreover, it is exhaustively described in the article of Rathje and Saygili (2008). Rather, we turn the attention on advantages and disadvantages of methods used to estimate the seismic performance of slopes and on the effectiveness (i.e., predictive power) of different parameters in predicting soil displacement.

Seismic slope performance can be assessed in different ways, ranging from simple pseudo-static procedures, which consider the seismic shaking as an additional force, to advanced non-linear dynamic analyses. One of the most widely used procedures for evaluating earthquake-induced slope displacements is the Newmark (1965) sliding-block method (e.g., Miles and Ho, 1999; Barani *et al.*, 2007) that simplifies a potential failure mass as a rigid-block sliding on an inclined plane. The permanent displacement of the sliding mass is calculated by double-integrating the parts of the block acceleration time history that exceed a critical acceleration value (i.e., critical acceleration) as a function of time. The main advantage of Newmark's (1965) method is its theoretical and practical simplicity. However, it presents some limitations that are the result of several simplifying assumptions (Wartman *et al.*, 2003). Chief among these is the assumption that the landslide mass is hypothesized to behave in a rigid, perfectly plastic manner. While this assumption is reasonable for relatively thin landslides characterized by stiff or brittle materials, it may introduce significant bias as landslide masses become thicker and material becomes softer (Jibson and Jibson, 2003). To avoid such limitations, Makdisi and Seed (1978) proposed a decoupled procedure that, contrary to the Newmark (1965) rigid-block approach, accounts for the dynamic response of the sliding mass. The method consists of first running a dynamic analysis (generally 1D) of the slope assuming that no relative displacement occurs along the failure surface and then of using the resulting acceleration time history as the input for a rigid-block calculation. However, the decoupled analysis may not be very effective as it does not account for the effects of slope slip on the resulting ground-motion (Lin and Whitman, 1983). A more advanced method is the so-called "coupled procedure" in which the dynamic response of the sliding mass and the permanent displacement are modeled together. We applied these three procedures to the Salcito landslide [a detailed description of the analysis can be found in the article of Barani *et al*. (2010)] which was re-activated following the October 31, 2002 Molise earthquake, M_w = 5.8 (Bozzano *et al*., 2008). Specifically, the coupled analysis was performed by applying the finite-element computer program FLAC 5.0 (Itasca Consulting Group Inc., 2005) to a simple numerical model that simulates an infinite slope. Non-linear constitutive relationships for the soil formations were included in the slope model which also accounts for the pore pressure distribution through a preparatory stationary ground flow analysis. Concerning the coupled analysis, the ground response was evaluated through 1D numerical simulation performed using Shake91 (Idriss and Sun, 1993). The same suite of rock input motions was used in both the coupled and decoupled analysis as well as in the Newmark (1965) sliding block calculation. As a result of the comparison of the slope displacement values calculated using these three alternative approaches (Fig. 7), the following observations can be drawn:

1. double integrating acceleration time histories recorded on rock, without keeping into account site amplification, may severely under-predict slope displacements (up to 50% or more) and, consequently, should be avoided in the case of unstable masses characterized by thick and/or soft soils;
2. the decoupled approach can provide displacements similar to those achieved by the coupled procedure provided that the excess pore pressure (in case of saturated soil mass) is properly estimated and taken into account.

Another important issue that deserves special attention within the framework of PSDHA regards the predictors used in regressing soil response models. Here more than in other hazard applications, distinction should be made between frequency-dependent and -independent ground-motion IMs. Many studies (e.g., Jibson, 1993; Harp and Wilson, 1995) pointed out that ground-motion IMs that capture the intensity of the motion across a range of frequencies, such as the Arias intensity, I_a (Arias, 1970), and Housner intensity, *SI* (Housner, 1952), correlate better with landslide displacement than frequency-dependent IMs, such as *PGA* or spectral acceleration. In particular, I_a was found to be the most efficient IM for stiff slopes while *SI* is preferable for flexible slopes with initial fundamental period between 0.6 and 2.0 s (Makdisi and Seed, 1978;

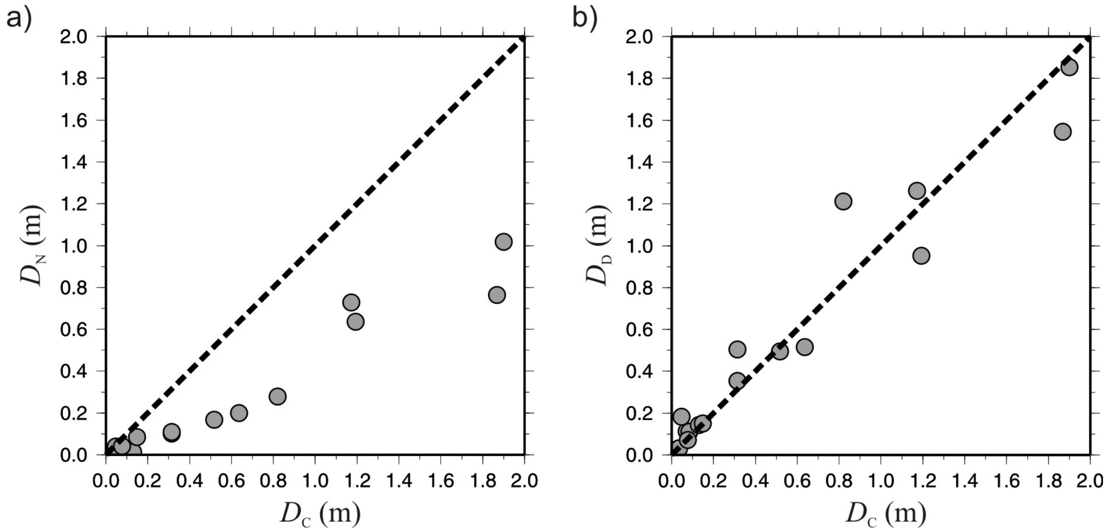


Fig. 7 - Comparison of both: a) Newmark (1965) D_N displacements and b) decoupled D_D displacements with coupled D_C displacements obtained for the Salcito (southern Italy) landslide (after Barani *et al.*, 2010).

Bray, 2007; Barani *et al.*, 2010). Among period-dependent IMs, the spectral acceleration at the slope fundamental period, $S_a(T_s)$, or at a degraded period equal to 1.5 times T_s (the degradation factor is introduced to account for soil nonlinearity) are generally the most informative IMs (e.g., Travasarou and Bray, 2003; Bray, 2007; Barani *et al.*, 2010). As a consequence, neglecting frequency-dependent parameters that carry implicit information on the soil fundamental frequency may result in a less accurate prediction of slope displacement. Generally, single regression models in either I_a, SI, $S_a(T_s)$, or $S_a(1.5T_s)$ allow sufficiently accurate predictions. However, in the case of complex resonance phenomena (e.g., landslide mass characterized by more than one resonant soil layer above the bedrock), multiple regression models incorporating information about the natural frequency of each resonant soil layer may yield a lower error in predicting soil displacement. In these cases, ground response results must be coupled with a vectorized version of PSDHA rather than its more conventional scalar counterpart. The mathematical formulations of both scalar and vectorized PSDHA are presented in the article of Rathje and Saygili (2008) and are not reported here for brevity.

4. Probabilistic fault displacement hazard assessment (PFDHA)

According to the definition by Coppersmith and Youngs (2000), "*fault displacement hazard is the hazard related to differential slip that occurs at the surface along a seismogenic fault or along secondary faults triggered by the seismogenetic rupture*". While deterministic approaches based on simple identification of active faults that may produce surface displacements are still largely applied for the siting of critical facilities, probabilistic methods for fault displacement hazard are rarely used due to their complexity, immaturity of analytical models [which leads to greater uncertainty in the computed hazard (Youngs *et al.*, 2003)] and, above all, to limited availability of fault-specific information based on post-seismic and paleoseismic data. Thus, the design of facilities against surface fault rupture is often guided simply by common sense by avoiding structures across active faults. Of course, this approach cannot be applied during the risk assessment of existing facilities, where PFDHA is the only sound option.

Definition and characterization of active faults are critical aspects of both deterministic and probabilistic techniques as they often require expensive site-specific investigations, particularly in those cases of blind deep sources. In these cases, the analysis of epicentre and hypocentre distributions resulting from the application of precise re-location techniques is a valuable tool for constraining fault locations and orientations and for studying fault behavior (e.g., Shearer, 1998, 2002; Astiz *et al.*, 2000; Waldhauser and Ellsworth, 2002; Grant and Shearer, 2004). Moreover, definition of fault activity is fundamental in PFDHA as it often triggers code requirements for special investigations or special design provisions (Kramer, 1996). Most definitions are based on the elapsed time since the most recent fault movement. Although many definitions exist, none of them assume elapsed periods larger than 100,000 years (e.g., Kramer, 1996). However, a typical assumption is to consider as active faults all earthquake sources that have produced surface displacement within the Holocene period (approximately, the past 10,000 years).

PFDHA has a relatively recent history. However, its formulation (Coppersmith and Youngs, 2000; Youngs *et al.*, 2003; Trifunac and Todorovska, 2005), which was introduced within the framework of the seismic hazard assessment for the nuclear waste repository at Yucca Mountain, USA (CRWMS M&O, 1998), is essentially based on the original methodology for ground shaking hazard assessment developed by Cornell (1968). Again, the final result is represented by a hazard curve, here expressing the annual frequency of exceedance of specific fault displacement amplitudes. The major difference with respect to the well-known ground-motion hazard formulation stands on the computation of the conditional probability term $P_j\,[C > c \mid s^{-}\ at\ l]$ in Eq. (1), which is expressed as (Coppersmith and Youngs, 2000; Youngs *et al.*, 2003):

$$P^{*}_{kn}\left(D > d \mid m, r\right) = P_{kn}\left(\mathrm{slip} \mid m, r\right) \times P_{kn}\left(D > d \mid m, r, \mathrm{slip}\right) \tag{7}$$

where the term P_{kn} (slip | m, r) defines the probability that some amount of slip occurs at site k as a result of an earthquake (fault slip at depth) on source n of magnitude m at a distance r from the site. The term P_{kn} ($D > d$ | m, r, slip) defines the conditional distribution of the amount of fault displacement given that some slip occurs. This conditional probability is computed using a continuous distribution whose parameter values, which are empirically derived, are functions of m and r as in PGMHA. Probability models for “principal faulting” (i.e., rupture along the main fault plane) and “distributed faulting” (i.e., rupture occurring on secondary faults in the vicinity of the principal rupture) are presented in the articles of Coppersmith and Youngs (2000) and Youngs *et al.* (2003).

Besides the previous approach (called “earthquake approach”), a further method has been presented in the articles by Coppersmith and Youngs (2000) and Youngs *et al.* (2003). This second approach, which is termed as “displacement approach” and is briefly summarized below, quantifies the hazard by using the characteristics of fault displacement observed at the study site without accounting for a specific causal mechanism. Thus, Eq. (1) reduces to:

$$\gamma\ (D > d) = \lambda_{DE} \times P\left(D > d \mid slip\right) \tag{8}$$

where λ_{DE} is the frequency of displacement events (it can be directly estimated from recurrence intervals derived from paleoseismic data or indirectly from fault slip rates) and $P(D > d \mid \text{slip})$ is the conditional probability that the displacement D will exceed a value d in a single slip event at a given site, which can be assessed using a gamma distribution with parameter values estimated from site-specific displacement measures.

5. Discussion and conclusions

The article has presented a review of methodologies that are available for carrying out a more holistic site-specific probabilistic seismic hazard assessment. These methodologies are suitable mainly for the design and assessment of critical facilities and structures with extreme high consequence potential. The article has explored theoretical and methodological aspects of different procedures for conducting a comprehensive PSHA at a site accounting for different earthquake-induced effects, such as ground-motion amplification due to soil and/or topographic characteristics, slope displacement, and fault displacement. Based on authors' experience, three major observations can be made. The first observation is very practical and concerns the level of effort that is required for the application of such a holistic PSHA approach for a critical facility. Based on SSHAC (1997) recommendations, four levels of effort (Table 1), which represent increasing levels of participation by technical experts in the development of a PSHA, can be defined. The first three levels rely on a single technical expert [called Technical Integrator (TI)] that is responsible for all aspects of the PSHA (however, other experts may be involved on a consulting basis). SSHAC (1997) recommendations point out that most site-specific studies make use of some type of TI approach. On the other hand, the most sophisticated level of study (Level 4) relies on the panel of experts (that coordinates the work of the Technical facilitator/Integrator, TFI) that evaluating proposed models, and discussing the different views in the scientific and technical community. Although SSHAC-Level 4 ensures the highest professional and regulatory standards, it is also by far the most demanding in terms of time and costs. Therefore, it seems suitable only for the design of NPPs and nuclear waste repositories. Two such cases are the PEGASOS project in Switzerland (PEGASOS, 2004; Studer, 2010) and the Yucca-Mountain-Project (CRWMS M&O, 1998) in the USA, which were the only two projects based on SSHAC-Level 4 ever performed to year 2009 (Studer, 2010). Besides SSHAC (1997), further guidances for performing a PSHA for an NPP can be found in Savy *et al*. (2002) and in the regulatory guide of the U.S. Nuclear Regulatory Commission (NUREG, 2007).

A second observation relates to policy aspects, in that deals with the acceptable level of risk that critical facilities should be designed for. In the customary design paradigm, this translates into setting an adequate annual rate of exceedance of the ground-motion adopted for design. This is a thorny issue that has been debated at length in the literature (e.g., Hank *et al*., 2006) and cannot be given justice in this article. It suffices to say that the design of critical facilities requires the quantification of the hazard of different earthquake effects to extremely low annual rates of exceedance or, conversely, very long mean return periods that range from a few thousands of years to more than a million years. For example, in the case of the Yucca Mountain Project, PSHA results are provided for MREs as small as 10^{-8} / yr (MRP of 10,000,000 years), corresponding to PGA levels of around 11 g (Hanks *et al*., 2006). As such, these values are often referred as

Table 1 - PSHA levels following the SSHAC (1997) recommendations.

Issue Degree	Decision Factors	Study Level
A Non-controversial; and/or insignificant to hazard		1 TI evaluates/weights models based on literature review and experience; estimates community distribution
B Significant uncertainty and diversity; controversial; and complex	• Regulatory concern • Resources available • Public perception	2 TI interacts with proponents & resource experts to identify issues and interpretations; estimates community distribution
C Highly contentious; significant to hazard; and highly complex		3 TI brings together proponents & resource experts for debate and interaction; TI focuses debate and evaluates alternative interpretations; estimates community distribution
		4 TFI organizes panel of experts to interpret and evalutate; focuses discussions; avoids inappropriate behaviour on part of evaluators; draws picture of evaluators' estimate of the community's composite distribution; has ultimate responsibility for project

"extreme ground-motions". Obviously this definition can be extended to other seismic hazards. The level of safety required by regulators has very distinct implications on the level of effort required during the design process. Regardless of the level selected, the methods presented here provide the basic building blocks necessary for establishing the desired levels of hazard for the different earthquake effects to be used in design.

As a final observation, we would like to stress the importance of uncertainty and sensitivity analyses in PSHA. As stated in Section 2, uncertainty and sensitivity analyses have become fundamental parts of PSHAs as they allow the identification of the models and parameter values with the largest impact on the hazard and its uncertainty and ensure that sufficient effort is devoted to developing a reliable representation of those models and parameter values before finalizing the PSHA calculations. Without going through this process of examining alternatives and sensitivities, "*the PSHA results will have little credibility*" (McGuire, 2004). Therefore, sensitivity and uncertainty analyses should be used as a preliminary step for the construction of logic trees focusing efforts on the parameters found to be most sensitive and excluding those unrealistic or with a negligible influence on the hazard and its uncertainty. This step would provide a reduction of the computation time, which in the case of logic trees with thousands of branches (these are typical in the case of PSHAs for NPPs) may be extremely long. More important, the knowledge of the parameters that have the highest influence on the hazard and drive its uncertainty is valuable in guiding focused research efforts to reduce such an uncertainty and in facilitating a correct understanding and use of PSHA results.

Finally, we want to spend some words on recent developments in PGMHA. In the last few years it has been recognized that some of the uncertainty in the PGMHA results and, in some cases perhaps some bias, may have been introduced by the so-called ergodicity assumption adopted in the derivation of GMPEs used in all hazard studies. In simple but somewhat loose words, that assumption trades space for time and uses ground-motion records from many sites and many earthquakes to predict the ground-motion at a single site. Hence, using traditional GMPEs implies accepting that the ground-motion variability computed from a global data set including recordings from multiple sites and from multiple earthquakes is an unbiased estimate of the variability of ground-motions at a single site (Anderson and Brune, 2009; Al Atik *et al*., 2010; Rodriguez-Marek *et al*., 2011). Repeatable and systematic effects of path and source and the effects of the same soil site conditions make the ground-motion variability at a single site smaller than that computed utilizing records from other sites with similar soil conditions affected by other earthquakes with different paths and sources. Relaxing the ergodicity assumption comes at a steep price, as is apparent from the study by Walling and Abrahamson (2012), but it is undoubtedly the new frontier of the future efforts in PGMHA.

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Corresponding author: Simone Barani
DISTAV, Università degli Studi
Viale Benedetto XV 5, 16138 Genova, Italy
Phone: +39 010 3538097; e-mail: barani@dipteris.unige.it