Automatic definition of hazard-compatible accelerograms at non-rocky sites

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SUMMARY:
Nonlinear dynamic analyses require accelerograms as seismic input, possibly accounting for site effects. The rigorous approach consists in selecting real accelerograms recorded at rocky sites and propagating them through the soil profile to account for local site effects. Ground response analyses require a detailed geotechnical characterization of the site which might be unavailable. This paper presents a simplified, yet theoretically-based procedure to propagate real accelerograms recorded at rocky sites through 1D soil profiles, for which only the value of $V_{S,30}$ is required. A Monte Carlo simulation procedure is used to create a population of soil profiles compatible with $V_{S,30}$. Real accelerograms recorded at rocky sites are propagated using the concept of transfer function through each soil profile. The mean response spectrum computed accounting for site effects is then used for selecting spectrum-compatible time histories using the signals calculated by the described procedure which only require $V_{S,30}$ and not a detailed geotechnical characterisation of the site.

Keywords: Site response analyses, non-rocky soils, time histories selection, seismic input, transfer function

1. INTRODUCTION

Designers and practicing engineers are continuously facing the problem of assessing the seismic response of a structure sitting on a soft (or non-rocky) soil deposit. In nonlinear dynamic analyses of structures, the seismic input requires the definition of real/artificial time series. Despite being the most accurate tool for computing internal forces and deformations of a structure subject to severe ground motion, nonlinear dynamic analyses are generally used only in very important or strategic structures or geotechnical systems. The difficulty for practitioners of identifying an appropriate set of ground motion records is one of the reasons why this type of analysis is rarely applied in engineering practice.
At a given site, if the seismic input is known for standard ground conditions (outcropping bedrock), the filtering effects produced by the presence of soft sediments can be assessed from the knowledge of a geotechnical model of the subsoil. This is what is currently done in ground response analysis, but, even when simple 1D amplification is expected, it requires a detailed geotechnical characterization of the site under investigation (e.g. Rota et al. 2011). When this is not available and only a limited amount of information is accessible, such as the value of $V_{s,30}$ (i.e. the equivalent shear wave velocity of the top 30 m of the soil profile), it is not straightforward to identify a suitable method to define the seismic input taking into account local site conditions.
Several seismic codes (e.g. EC8, Italian Building Code NTC08, IBC 2009, etc.) allow to account for site effects using a simplified approach considering response spectra with modified spectral shape based on soil categories defined according to $V_{s,30}$. The spectral shape associated with each soil category is usually obtained as the envelope of response spectra of accelerograms recorded during past earthquakes at seismic stations with similar ground conditions, hence covering a wide range of V_S

profiles. These spectra are known to be very uncertain and several studies have shown that soil profiles belonging to the same soil category, and thus within the same $V_{S,30}$ range, exhibit a seismic response characterised by a large variability (e.g. Wald and Mori, 2000 and Boore, 2004). In addition, some studies (e.g. Lai et al. 2007; Barani et al. 2008; Gallipoli and Mucciarelli, 2009) have shown that the elastic acceleration response spectra prescribed by seismic building codes for ground-specific categories do not always reproduce correctly the expected seismic input at a site. For all these reasons, the simplified method to account for site effects allowed by seismic codes is oversimplified and it often leads to an over-conservative estimate of the seismic input. Moreover, these code-based spectra do not solve the problem if the seismic input is needed in terms of time histories for performing dynamic analyses of structures, as the selection of spectrum-compatible records using reference code-based spectra for non-rocky soils should be avoided since it is characterized by large uncertainties. Along these lines, the Italian building code (NTC08) and its commentary (Circ09) specify that, in case the seismic action is described by means of accelerograms, the simplified approach (consisting on the identification of soil categories, to which appropriate spectral amplification coefficients are associated) cannot be used. In this case, site-specific ground response analyses are required, with the seismic input defined in terms of real accelerograms representative of the reference expected seismic hazard (rocky site with flat topographic surface) and a geotechnical model characterized by an appropriate soil stratigraphy and set of parameters. However in many practical circumstances only limited geotechnical information is available which might be not sufficient to carry out a detailed, site-specific ground response analysis.

This paper proposes a practical, yet rigorous methodology for the definition of site-specific response spectra and time series at non-rocky sites when limited geotechnical information is available such as $V_{s,30}$. It is important to emphasize that the analyses conducted in this study are based on the assumption of linear viscoelastic soil behaviour, therefore the applicability of the proposed methodology is limited to the cases in which nonlinearity of soil response does not play an important role. Although the hypothesis of linear soil response may appear a strong oversimplification, the proposed algorithm could in principle be incorporated into a linear-equivalent program (SHAKE-type) which would allow to take into account moderate nonlinearities in soil behaviour. The accelerograms obtained with the proposed procedure can be easily obtained for any soil category and furthermore they comply with code requirements.

2. METHODOLOGY

This section describes the methodology proposed for the definition of site-specific response spectra and the selection of spectrum-compatible time histories for soil conditions different from rock when limited geotechnical information is available. In particular, the only required parameters are a range of $V_{s,30}$ values, few geotechnical parameters (i.e. soil density and damping ratio) which can be obtained from the technical literature and the seismic input represented by a set of time-histories appropriately defined to be representative of the reference seismic hazard at the site of interest (stiff soil with flat topographic surface). A stochastic approach, based on Monte Carlo simulations, is used to account for the uncertainty in site response associated with soil profiles, stochastically defined with the only constraint of belonging to a predefined range of $V_{s,30}$. For each randomly generated soil profile, the calculation of site effects is carried out on a deterministic model, whose input parameters are defined by a realization of a certain set of random variables.

As every seismic site response analysis, the proposed methodology requires as a main input a set of spectrum-compatible real records, representative of the reference seismic hazard for the site of interest, as discussed in the following section. At each run of the stochastic procedure, one accelerogram is randomly selected (according to a uniform distribution, i.e. assigning at each record the same probability of being sampled) from a prescribed suite of ground motions recorded on rock. This accelerogram is then convolved with the transfer function corresponding to one of the randomly generated soil profiles, hence producing an accelerogram at the free surface, obtained after the propagation of the signal recorded on rock through the selected soil profile. The procedure is applied a number of times (the number is pre-defined by the user), providing a set of accelerograms at the ground surface, from which a subset is selected, with the constraint of being spectrum-compatible with the average response spectrum computed at the free-surface considering the totality of the simulations. After the definition of the input data in terms of object motion and geotechnical parameters defining the soil profiles, the methodology is subdivided into the following three main steps:

- Stochastic definition of soil profiles;
- Site response and computation of the mean spectrum at the free-surface;
- Selection of spectrum compatible time-histories.

A flowchart of the proposed methodology is shown in Figure 2.1 . The model used for performing the site response analyses and the assumptions related to the other parameters required for the stochastic definition of the stratigraphy and geotechnical parameters are discussed in the following sections.

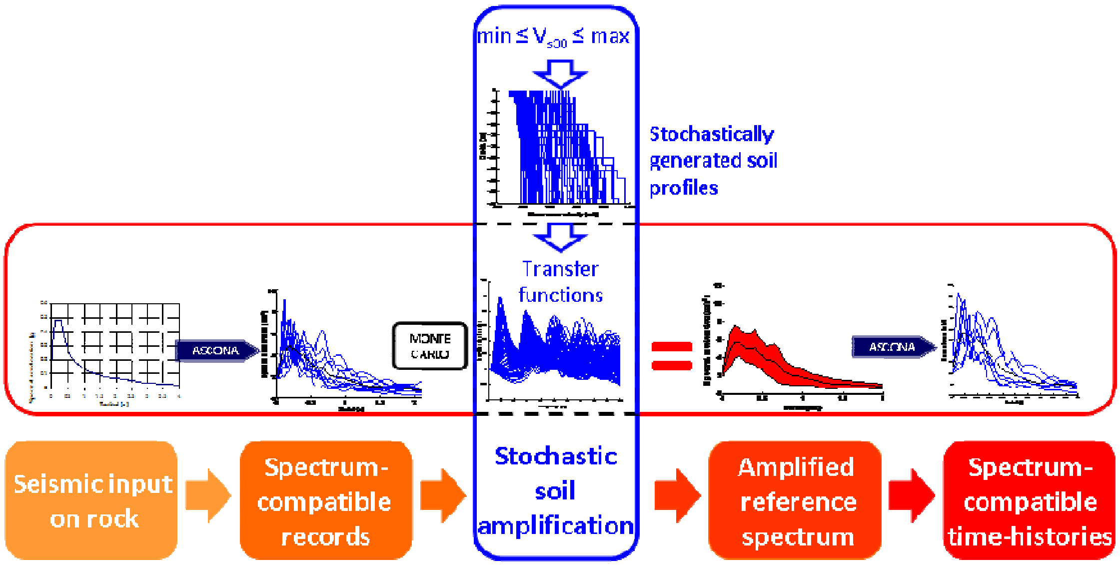

Figure 2.1 Flow chart of the proposed methodology

The proposed methodology will be illustrated through an example carried out for the city of Teramo (Central Italy), considering a range of $V_{s,30}$ between 360 and 800 m/s, which is equivalent to soil category B of NTC08 (2008) and EC8 (EN1998-1, 2004). Figure 2.2 shows the code-based response spectra according to NTC08 for the town of Teramo for different soil conditions.

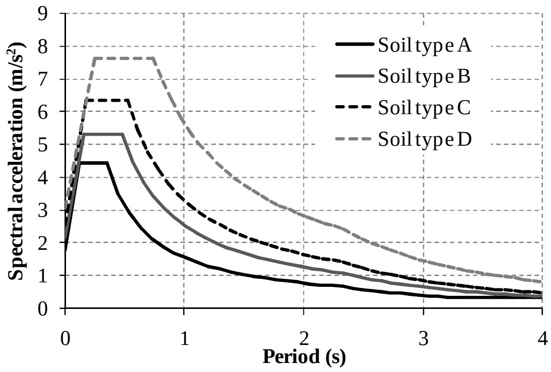

Figure 2.2 Response spectra for different soil conditions according to NTC08 for Teramo (Central Italy).

2.1. Definition of seismic input for ground response analyses

Site response analyses require the definition of seismic input in terms of accelerograms for stiff soil conditions. In this work, real accelerograms recorded on rocky sites have been used. As this study concentrates on the description of the proposed methodology to account for site effects, the criteria adopted for the selection of real accelerograms recorded on stiff soils will not be discussed. The interested reader may consult the paper by Corigliano et al. (2012) that is on press.

In the example below, real records were selected to be spectrum-compatible with the response spectrum prescribed by NTC08 for soil type A (rock), for the city of Teramo (Central Italy). Nevertheless, any other reference spectrum could be used instead. Spectrum-compatibility has been enforced according to the prescriptions of NTC08 for artificial accelerograms, i.e. by requiring that the negative difference between the average response spectrum computed from the selected records and the target spectrum does not exceed 10% in a specified range of periods, which in this case was set to be within 0.15 and 2.0 second. A set of 10 accelerograms recorded on stiff ground have been selected from a strong-motion database, using the algorithm implemented in ASCONA software (Corigliano et al., 2012). A relatively large number of input accelerograms is used to facilitate the selection of time-histories in the last step of the procedure. This is carried out with the constraint of avoiding having in the selected set of records, accelerograms resulting from different amplifications of the same real record. Since the input signals are recorded on outcropping rock, a simple deconvolution procedure, consisting in removing the free surface effect is automatically accounted for in the procedure.

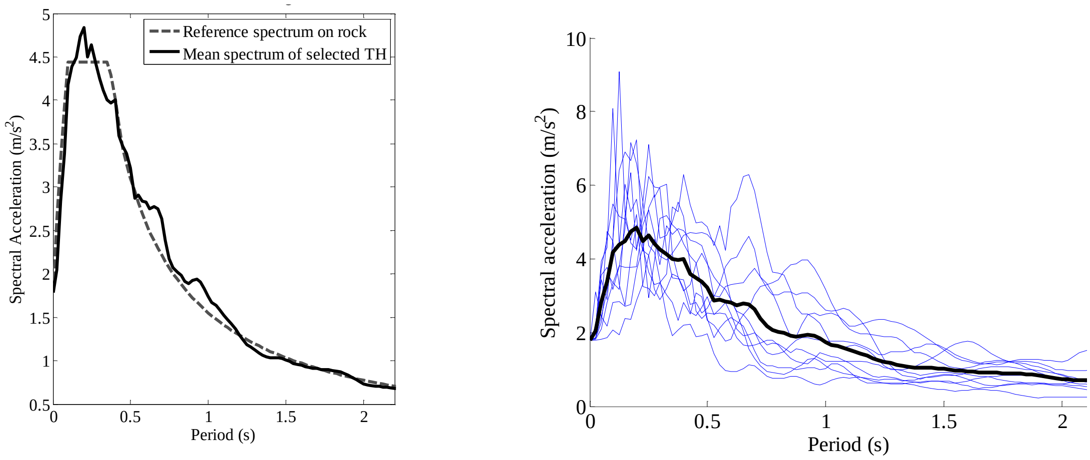

Figure 2.3 Left: comparison of NTC08 response spectrum for the town of Teramo (dashed line) and the average response spectrum (structural damping 5%) of the selected accelerograms. Right: elastic response spectra of the 10 accelerograms selected, along with their mean spectrum (thick line).

2.2. Definition of 1D site-specific soil profiles

The soil profiles used for site response analyses are defined based on a stochastic procedure using Monte Carlo simulations, after having defined an appropriate range of $V_{s,30}$ for the site of interest. The methodology considers 1D soil profiles and assumes a linear viscoelastic constitutive model for the soil. Under this hypotheses, the construction of the litho-stratigraphic model requires, for each layer of the soil profile, the definition of thickness, soil unit weight, shear wave velocity and damping ratio.

The bedrock is assumed to be characterised by a shear wave velocity V_s of 800 m/s (in accordance with EC8 and NTC08, as the object motion is supposed to be recorded on rock) and it is located at a depth specified by the user. For the example shown in this paper, the bedrock is assumed to be located at a depth of 50 m.–The values of V_s and the depth of the bedrock should be considered as random variables and their influence on the results of site response analyses will be assessed.

The first step for the definition of the shear wave velocity profile is the subdivision of the overall thickness of the soil deposit into a number of sublayers, using a procedure based on Monte Carlo simulation, which generates a number of sublayers (smaller than or equal to a predefined maximum number N_{max}), each one with a thickness randomly defined from a uniform distribution of values between the minimum and maximum value (h_{min} and h_{max}, respectively) and stops when the overall thickness of the already defined sublayers reaches the value defined by the user (i.e. the depth above the bedrock).

The second step of the procedure consists in defining a value of V_s to each sublayer. In this study, the variation of V_s with depth proposed by Santamarina et al. (2001) was assumed, i.e.:

$$V_S = V_{S0}\sigma_{vo}^{p} \tag{2.1}$$

where V_{S0} is the shear wave velocity at the free surface and p is a parameter of the model. The dependency on depth in the shear wave velocity is accounted for by the total vertical stress (σ_{vo}),

assuming a constant value of the unit weight of the soil (γ). It is emphasized that this equation can be easily replaced by any other relationship defining a shear wave velocity profile with depth, taking possibly into account other issues that have been neglected so far, such as for example the position of the water table. The values of V_{S0} and p used in Eq. (2.1) have been considered as uncorrelated random variables of the model, both characterised by a uniform distribution. V_{S0} is assumed to vary between $V_{s,30,min}/2$ and $V_{s,30,min}$, where $V_{s,30,min}$ is the minimum value of $V_{s,30}$ specified by the user for the case under examination, whereas p is extracted from a uniform distribution of numbers within the interval 0.01÷0.3. If the random combination of V_{S0} and p would generate a shear velocity profile with a value of $V_{s,30}$ outside the interval of interest specified by the user (i.e. outside $V_{s,30,\mathrm{min}} \div V_{s,30,\mathrm{max}}$), this profile is disregarded and a new profile is generated based on a new combination of V_{S0} and p.
The procedure stops when the required number of soil profiles with $V_{s,30}$ values falling in the interval of interest has been reached. This approach allows the generation of a continuous shear wave velocity profile, with values increasing with depth (i.e. non-inversely soil profiles). A single value of velocity is then attributed to each of the previously defined sublayers, corresponding to the value generated at the mid-depth of the layer. This means that, for each combination of V_{S0} and p values, depending on the previously generated soil profile, a different amplification of the input motion may be expected.
Figure 2.4 (left) shows an example of shear wave velocity profile defined according to the Monte Carlo simulation described above. The continuous line indicates the assumed variation of the shear wave velocity with depth, defined based on Eq. (2.1), whilst the staggered line represents the values of velocity assigned in the model to mid-depth of each sublayer. Figure 2.4 (right) shows an example of definition of category B soil profiles with values of $V_{s,30}$ between 360 and 800 m/s, thereby providing an idea of the variability in soil profiles that can be expected according to the proposed methodology.

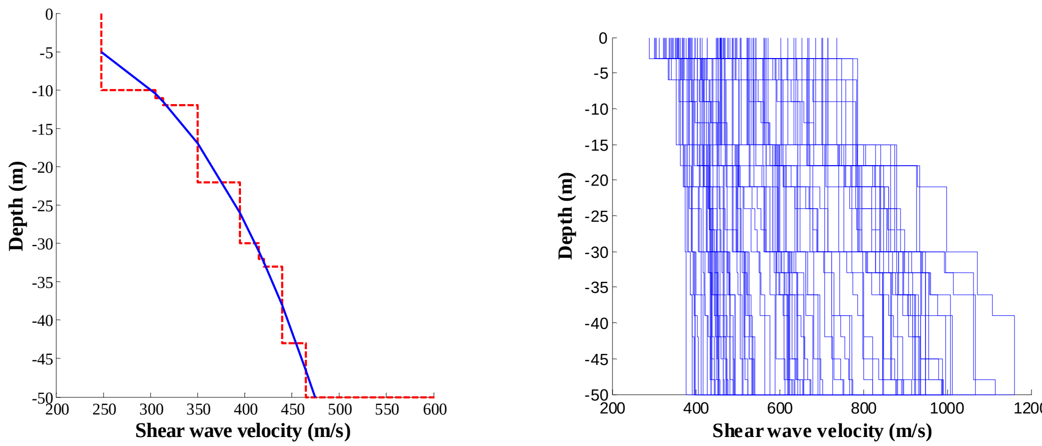

Figure 2.4. Example of shear wave velocity profile generated using Monte Carlo simulation (left) and random shear wave velocity profiles obtained using the proposed procedure for a soil category B, with values of $V_{s,30}$ ranging between 360 and 800 m/s, considering 100 soil profiles (right).

Summarising, the parameters required to create the stochastic shear wave velocity profiles are listed in the following, indicating in brackets the values adopted for each parameter for the case study presented in this study, which simulates a population of soil profiles corresponding to soil class B (V_S is monotonically increasing with depth) of EC8 (EN1998-1, 2005) and NTC08 (2008):

- number of soil profiles to be generated (100);
- minimum and maximum values of sublayer thickness (3 and 15 m);
- maximum number of sublayers (15);
- range of $V_{s,30}$ for the soil category of interest (360÷800 m/s).
- soil damping ratio of each sublayer (2%);
- soil mass density of each sublayer (1900 kg/m^3).

2.3. Assessment of site response

The acceleration time-histories at the surface accounting for site effects were calculated by exploiting the notion of transfer function, using 1D linear wave propagation theory. Assuming that the soil deposit consists of *N* viscoelastic horizontal sublayers obeying to the Kelvin-Voigt constitutive relation (e.g. Kramer, 1996), overlaying an elastic bedrock (see Figure 2.5), a monochromatic solution of the 1D wave equation can be expressed in the form:

$$u_j(z_j,t) = A_j e^{i\omega\left(t + z_j / V_{sj}\right)} + B_j e^{i\omega\left(t - z_j / V_{sj}\right)} \tag{2.2}$$

where A_j and B_j are the amplitudes of waves traveling in upward and downward respectively, z is the direction of propagation and V_{sj} is the shear wave velocity of layer *j*. The free surface condition yields A_1 equal to B_1.

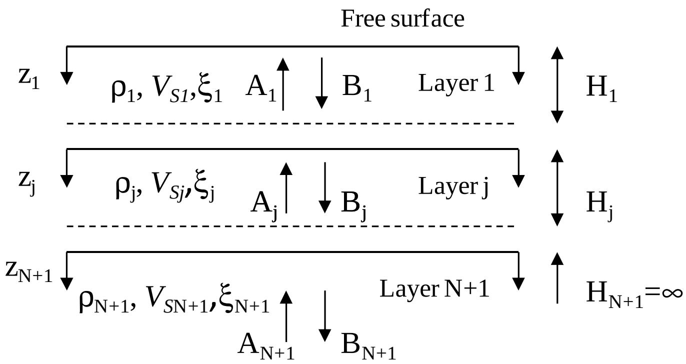

Figure 2.5. Viscoelastic layered soil deposit overlaying an elastic bedrock: geometry, geotechnical parameters and amplitudes of the waves traveling upward and downward (modified from Faccioli and Paolucci, 2005)

The amplitudes of the waves at layer *j* are related to those at layer *j+1* by the relation (Faccioli and Paolucci, 2005):

$$\begin{Bmatrix} A_{j+1} \\ B_{j+1} \end{Bmatrix} = \left[D_j\right] \begin{Bmatrix} A_j \\ B_j \end{Bmatrix} \tag{2.3}$$

where *[Dj]* is the propagation matrix of layer *j*:

$$\left[D_j\right] = \begin{bmatrix} \dfrac{1+\eta_j}{2} e^{\frac{i\varpi H_j}{V_{sj}}} & \dfrac{1-\eta_j}{2} e^{\frac{-i\varpi H_j}{V_{sj}}} \\ \dfrac{1-\eta_j}{2} e^{\frac{i\varpi H_j}{V_{sj}}} & \dfrac{1+\eta_j}{2} e^{\frac{-i\varpi H_j}{V_{sj}}} \end{bmatrix} \tag{2.4}$$

and η_j is the complex-valued impedance ratio:

$$\eta_j = \frac{\rho_j V_{sj}^*}{\rho_{j+1} V_{sj+1}^*} \tag{2.5}$$

where ρ is the soil density, $V_s^* \cong V_s \cdot (1 + i\xi)$ is the complex-valued shear wave velocity and ξ is material damping ratio. In the current version of the program, the value of damping ratio is selected by the user however it does not vary along depth, thus it is constant for different soil profiles. A recursive formula may be derived from the above relations relating the amplitude of the displacement at layer *j* to that of layer *N+1*:

$$\begin{Bmatrix} A_{N+1} \\ B_{N+1} \end{Bmatrix} = [D]_j \begin{Bmatrix} A_j \\ B_j \end{Bmatrix} \tag{2.6}$$

with

$$[D]_j = \begin{bmatrix} d_{11} & d_{12} \\ d_{21} & d_{22} \end{bmatrix} = [D_N][D_{N-1}]\cdots[D_j] \tag{2.7}$$

Knowledge of the wave amplitudes at layer *j* (A_j, B_j) allows computing the transfer function at the desired layer, and consequently the required time-histories. The transfer function relating the displacement amplitude at the free-surface (layer 1) to that at the rock outcrop is defined as the ratio of the wave amplitude at the surface and the wave amplitude at the rock outcrop, i.e.:

$$F(\varpi) = \frac{u_1}{u_0} = \frac{surface\ soil\ amplitude}{rock\ outcrop\ amplitude} \tag{2.8}$$

Figure 2.6 shows the amplitude of the transfer function for the soil profiles generated as described in section 2.1 and illustrated in Figure 2.4 (right). These transfer functions can then be convolved with the accelerograms recorded on stiff ground selected for the site of interest producing a database of real records filtered by the randomly-chosen soil profiles.

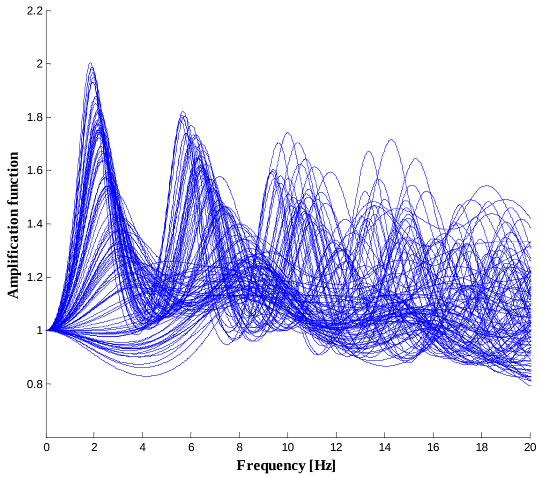

Figure 2.6. Amplitude of the transfer function for the soil profiles shown in Figure 2.4 (right)

The elastic acceleration response spectra computed at the free-surface by applying this procedure are shown in the left part of Figure 2.7, along with the mean spectrum. Each spectrum corresponds to a combination of one of the 100 soil profiles generated for this example and one accelerogram recorded on stiff ground randomly selected within the suite of 10. The right part of Figure 2.7 shows the mean acceleration response spectrum together with the mean plus/minus one standard deviation.

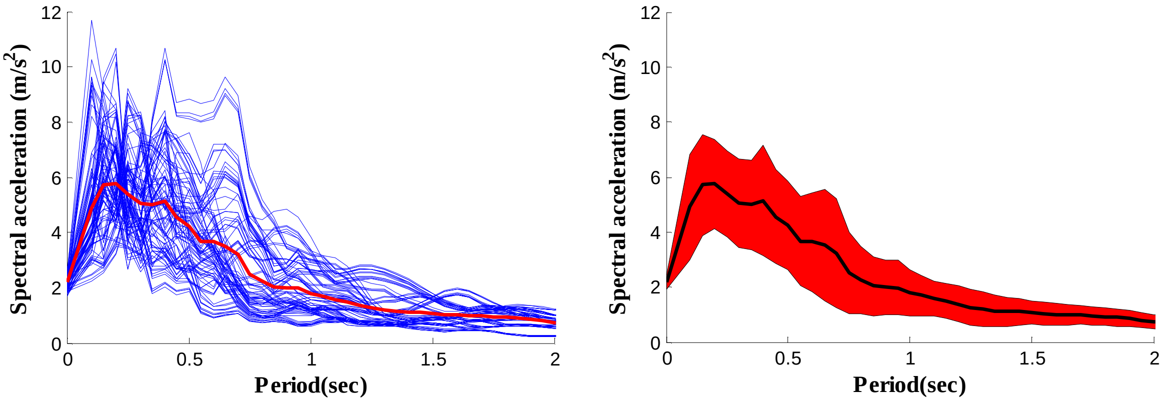

Figure 2.7. Left: acceleration response spectra computed for each of the 100 simulations and mean spectrum (thick red line). Right: mean and mean plus/minus one standard deviation (area) acceleration response spectra.

Figure 2.8 shows the comparison between the average response spectrum obtained at the free-surface from all the records after propagation through the randomly-generated soil profiles and the code spectra for soil type A (rock) and B according to NTC08 for the town of Teramo. It is noticed that the two response spectra for soil class B are comparable in the range of periods between 0 and 0.75 sec, whereas, as expected, the code spectrum for soil type B overestimates the average response spectrum obtained after site response analyses, especially in the high period range (beyond 1.2 s, the mean spectrum from site response analyses practically corresponds to the code spectrum for rock site).
In order to compare the results of the analyses with the code-based spectrum for soil category B, a

wide range of $V_{S,30}$ has been selected (i.e. 360÷800 m/s), hence generating a strong variability in the computed response spectra. Using the proposed approach, the $V_{S,30}$ range can be easily reduced, for example based on the availability of the results of geophysical tests at the site of interest, allowing to obtain a set of soil profiles which are better constrained to the actual ground conditions.

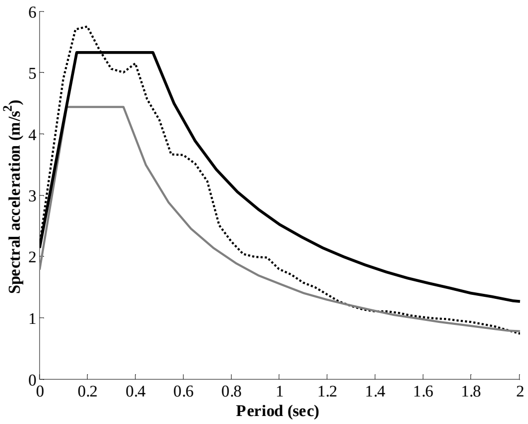

Figure 2.8. Comparison of the average response spectrum at the free-surface obtained from site response analyses (dotted line) and the code spectra for type A soil (rock, grey line) and for a type B soil (black thick line).

2.4 Selection of spectrum-compatible time-histories

The average spectrum computed at the free surface summarises the results of the stochastic ground response analysis. However, to carry out dynamic analyses of a structure, acceleration, velocity or displacement time histories are needed. Thus in the above procedure, a set of accelerograms is then selected among those propagated through the soil profiles, i.e. among those calculated at the free surface with the constraint of being spectrum-compatible with the average spectrum obtained from the stochastic site response analyses. The final set of accelerograms is identified using a selection procedure similar to that implemented in ASCONA (Corigliano et al., 2012), with the only difference that the time-series database is now constituted by all the accelerograms computed at the free surface of the soil deposit starting from a limited set of time histories recorded on stiff ground (e.g. 10 records in this example). A constraint has been added in the selection procedure to make sure that each of the accelerograms selected at the surface has been obtained by propagating a different input accelerogram recorded on rock. As an example, Figure 2.9 shows the comparison between the average spectrum obtained from the stochastic site response analyses and the average spectrum of the suite of 7 records selected for soil category B, while Figure 2.10 shows the elastic response spectra of the 7 accelerograms along with their mean spectrum, indicated by the thick line.

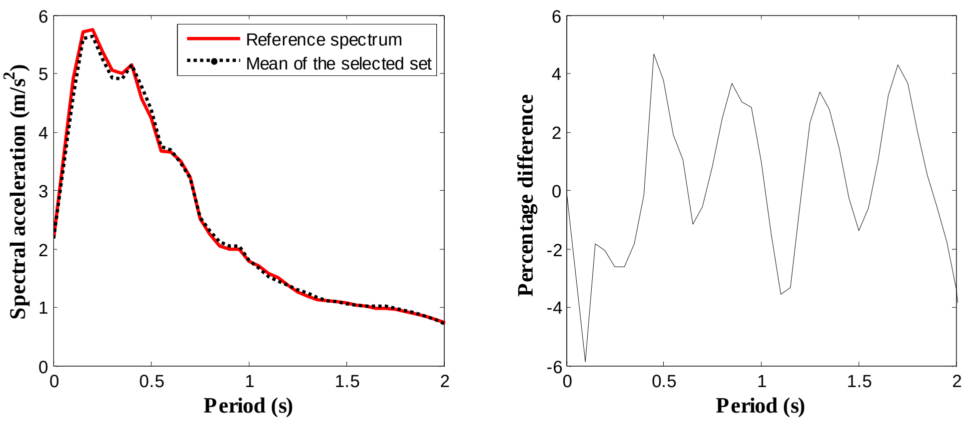

Figure 2.9. a) Comparison between the mean spectrum obtained from stochastic ground response analyses and the mean spectrum of the selected set of acceleration time histories to be used for dynamic analysis; b) percentage difference between the two spectra.

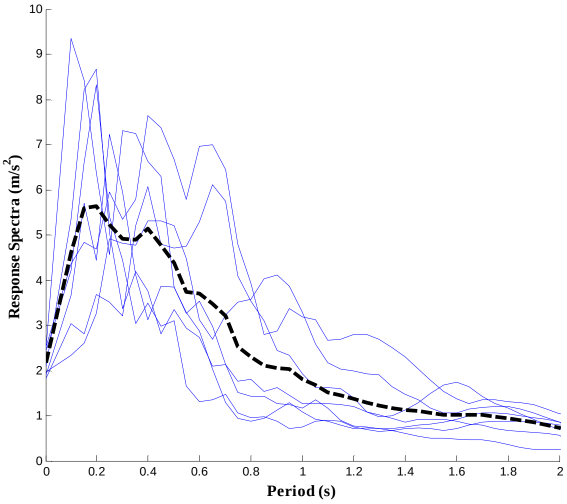

Figure 2.10. Elastic response spectra of 7 accelerograms for soil category B and their mean spectra (thick line).

3. CONCLUSIVE REMARKS

The paper describes a procedure for the definition of time histories to be used for dynamic analyses of structures accounting for site effects in a simplified yet rigorous way. The proposed approach is constituted by the implementation of the following a three steps:

- definition of shear wave velocity profiles using Monte Carlo simulation, based on the value of $V_{s,30}$ for the site of interest and literature information;
- computation of site response analyses using the transfer function for a layered viscoelastic soil deposit and definition of the mean response spectrum at the free-surface accounting for site effects. The seismic input used for site response analyses consists in real accelerograms recorded on stiff ground;
- selection, without scaling, of time histories among those obtained after convolution of the accelerograms recorded on stiff ground with the transfer functions for different soil profiles, with the constraint of being spectrum-compatible with the average response spectrum at the free-surface. These accelerograms consistently take into account site effects.

This simplified procedure complies with the building-code requirements, since accelerograms are selected on stiff-ground and then they are propagated through soil profiles to take into account site effects, thereby carrying out a simplified ground response analysis. At the same time, it allows to comply with cases for which only limited information is available such as the value of $V_{S,30}$ for the site of interest and hence detailed site response analyses could not be performed.

Obviously, this approach is based on simplifying assumptions, including the use of a 1D model, the adoption of a linear viscoelastic constitutive model for the soil, the absence of shear wave velocity inversions in the soil profile (the values of V_s are assumed to monotonically increase with depth, as suggested by the Italian building code and EC8-1 for category B soil deposits). The methodology could in principle be used also for soil categories B and C, where strong non-linearities are not expected, whereas for soil category D a more rigorous approach capable of capturing strong non-linear soil response should be adopted. However, some of assumptions of the method to carry out ground response analyses under limited geotechnical information can be easily removed or relaxed. For example the proposed algorithm could be incorporated into a linear-equivalent program (SHAKE-type) which could satisfactory handle moderate nonlinearities in soil behaviour. A further generalization allowed by the algorithm would be to consider different laws of variations of V_S with depth.

It must be clearly stated that the aim of the proposed procedure is not to replace site-specific, ground response analyses which should always be performed when detailed geotechnical information is available. Rather to propose a simplified yet theoretically-consistent approach to the definition of seismic input when only limited information is available at the site such as $V_{S,30}$.

The proposed procedure has several advantages, including the fact that it provides information on the variability associated to site response analysis and it allows to obtain a response spectrum which is more reliable and site-specific than a code-based spectrum (which often tends to overestimates the seismic input). If suite of accelerograms are needed for non-linear dynamic analyses of structures, the proposed algorithm overcomes the evident limitations of a seismic input based on a direct selection of records at non-rocky soils using as a reference the code spectrum for different soil categories. At this purpose it should also be remarked that some building codes including the NTC08, prescribes that the simplified approach of defining the seismic action based on soil categories cannot be adopted if the seismic input needs to be represented by time histories.

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REFERENCES

Barani, S., De Ferrari R., Ferretti, G. and Eva, C. (2008). Assessing the effectiveness of soil parameters for ground response characterization and soil classification. *Earthquake Spectra* **24:3**,565–597.

Boore, D. M. (2004). Can site response be predicted? *Journal of Earthquake Engineering* **8:SI1**,1-41.

Circ. NTC08 (2009). Circolare del Ministero delle Infrastrutture e dei Trasporti n. 617 del 2 febbraio 2009, supplemento ordinario n. 27 alla G.U. n. 47 del 26 febbraio 2009, "Istruzioni per l'applicazione delle Nuove norme tecniche per le costruzioni di cui al decreto ministeriale 14 gennaio 2008" – in Italian.

Corigliano, M., Lai, C.G., Rota, M. and Strobbia, C.L. (2012). ASCONA: Automated Selection of COmpatible Natural Accelerograms. *In press on Earthquake Spectra*.

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

Faccioli, E. and Paolucci, R. (2005). Elementi di sismologia applicata all'ingegneria, Pitagora Editrice, Bologna – in Italian.

Gallipoli, M.R. and Mucciarelli, M. (2009). Comparison of site classification from VS30, VS10, and HVSR in Italy. *Bulletin of the Seismological Society of America* **99:1**,340–351.

IBC (2009). International Building Code, International Code Council, Falls Church, VA.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering. Prentice Hall.

Lai, C.G., Corigliano, M., Bhuiyan, M.T.R. (2007). Comparison of real vs. artificial accelerograms in 1D linear-equivalent and nonlinear site response analysis of soil deposits. *Proc 14th Eur Conf Soil Mech Geotec Eng*, Madrid, Spain.

NTC08 (2008). Norme tecniche per le costruzioni, Ministero delle Infrastrutture e dei Trasporti, Decreto Ministeriale del 14 gennaio 2008, Supplemento ordinario alla G.U. n. 29 del 4 febbraio 2008 – in Italian.

Rota, M., Lai, C.G., Strobbia, C.L. (2011). Stochastic 1D site response analysis at a site in central Italy. *Soil Dynamics and Earthquake Engineering*, **31**,626–639.

Santamarina, J.C., Klein, K.A., Fam, M.A. (2001). Soils and waves. John Wiley and Sons, Chichester, UK.

Wald, L.A. and Mori, J. (2000). Evaluation of methods for estimating linear site-response amplifications in the Los Angeles region. *Bulletin of the Seismological Society of America* **90:6B**,S32-S42.