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NONLINEAR SITE EFFECTS: INTEREST OF ONE DIRECTIONAL - THREE COMPONENT (1D - 3C) FORMULATION

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ABSTRACT

Strong ground motions generally lead to both a stiffness reduction and a larger energy dissipation in the soil layers. Thus, in order to study such phenomena, several nonlinear rheologies have been developed in the past. However, one of the main difficulties of using a given rheology is the number of parameters needed to describe the model. In this sense, the multi-surface cyclic plasticity approach, developed by Iwan in 1967 is an interesting choice since the only data needed is the modulus reduction curve. Past studies have implemented this method in one-directional SH wave-propagation (1D-1C). This work, however, aims to study the local site effects by considering one-directional (1D) seismic wave propagation accounting for their three-dimensional nonlinear behavior. The three components (3C) of the outcrop motion are simultaneously propagated into a horizontal multilayer soil for which a three-dimensional constitutive relation is used. The rheological model is implemented using the Finite Element Method. The alluvial site considered in this study corresponds to the Tiber River Valley, close to the historical centre of Rome (Italy). The computations are performed considering the waveforms referred as the 14th October 1997 Umbria-Marche earthquake, recorded on outcropping bedrock. Time histories and stress-strain hysteretic loops are calculated all along the soil column.

The octahedral stress and strain profiles with depth and the modulus of acceleration transfer function (surface/outcrop spectral ratios) are estimated in the cases of combining three 1D-1C nonlinear analyses and of 1D-3C conditions, evidencing the influence of three-dimensional loading path.

INTRODUCTION

The undeniable significant role of local geologic site conditions in the response of infrastructures during a seismic event make the study of the site effects one of the important goals of earthquake engineering. According to collected seismic records, the local site condition emerges as one of the dominant factors controlling the variation in ground motion and determination of the site-specific seismic hazard. Any attempt of seismic zonation must take into account the local site conditions.

Soils are complex materials whose dynamical behavior can result strongly modified if modeled by a linear approach. The evidence of nonlinear soil behavior comes from experimental cyclic tests on soil samples, for different strain amplitudes, where it is observed departure from the linear state as well as hysteresis when ground deformations up to around 10^{-5} are attained (Hardin and Drnevich, 1972a, 1972b; Vucetic, 1990). The nonlinearity is manifested in shear modulus reduction and in the increase of damping for increasing strain levels. The effect on the transfer function produced by nonlinearity effects is a shift of the fundamental frequency toward lower frequencies, as well as an attenuation of the spectral amplitudes at high frequencies. For places where recorded data are not available, but soil layer parameters are known, it is necessary to estimate theoretically the transfer function based on the parameters of the soil layers.

One-directional wave propagation response analyses are used to estimate soil surface ground motions for use as input to the design of structures. Schnabel et al. (1972) introduced the equivalent-linear analysis as a way to approximate the computation of nonlinear site response through an iterative procedure. In this method, the resulting shear modulus reduction and increasing damping are independent of the stress-strain path. Nevertheless, the popularity of the equivalent linear method is perhaps due to the small number of parameters needed, its ease of use and its speed compared to time domain wave propagation. Although the equivalent linear method may produce similar results for site response studies in some particular cases, the equivalent linear method overestimates the peak ground acceleration for high strain regimes. This method is assumed to be reasonable for strain levels between 10^{-5} and 10^{-3} (Ishihara, 1996; Yoshida and Iai, 1998). Furthermore, the incorporation of hysteresis in any complete nonlinear analysis is fundamental. A complete nonlinear site response analysis requires the propagation of an earthquake record in a nonlinear medium by integrating the wave equation in the time domain and using an appropriate constitutive model. Inputs to these analyses include acceleration time histories at bedrock and nonlinear material properties of the various soil strata underlying the site. The main complication in nonlinear analysis is finding a constitutive model that reproduces faithfully the nonlinear and hysteretic behavior of soil with the minimum number of parameters. Realistic hysteresis behavior of soils is difficult to model because the yield surface can have a complex form. Some researchers adopt the theory of plasticity to describe the hysteresis of soil (Zienkiewicz et al., 1982; Chen and Baladi, 1985; Chen and Mizuno, 1990; Prevost and Popescu, 1996; Ransamooj and Alwash, 1997; Montans, 2000), others propose simplified nonlinear models (Kausel and Assimaki, 2002; Delephine et al., 2009) and other ones combine elasto-plastic constitutive equations with empirical rules (Ishihara and Towhata, 1982; Finn, 1982; Towhata and Ishihara, 1985; Iai et al., 1990a, 1990b; Kimura et al., 1993). Empirical rules that describe the loading and unloading paths in the stress-strain space are the so-called Masing rules (Masing, 1926), that reproduce quite faithfully the hysteresis observed in the laboratory (Vucetic, 1990). The main problem of these rules is that the computed stress may exceed the maximum strength of the material when an irregular load is applied (Pyke, 1979; Li and Liao, 1993). Several attempts have been done in order to control the dispersive property of these rules (Pyke, 1979; Vucetic, 1990).

The nonlinear site response analysis allows following the time evolution of the stress and strain during seismic events and the resulting ground motion at the surface. One-directional models for site response analysis are proposed by several authors (Joyner and Chen, 1975; Joyner et al., 1981, Lee and Finn, 1978; Pyke, 1979; Phillips and Hashash, 2009). Furthermore, Li (1990) incorporates the three-dimensional cyclic plasticity soil model proposed by Wang et al. (1990) in a finite element procedure, in terms of effective stress, to simulate the one-directional wave propagation; however, the rheology needs between 10 to 20 parameters to characterize the soil model.

The nonlinear rheology used in this research is a multi-surface cyclic plasticity mechanism that depends from few parameters that can be obtained from simple laboratory tests (Iwan, 1967). Material properties include the dynamic shear modulus at low strain and the variation of shear modulus with shear strain. This rheology allows the material to develop large strains in the range of stable nonlinearity in undrained conditions. Because of its three-directional nature, the procedure can handle both shear wave and compression wave simultaneously and predict not only horizontal motion but vertical settlement too. This formulation of soil hysteresis behavior can be used to examine case histories of known nonlinear soil response as well as to investigate the role of critical parameters in affecting the soil response.

In the present research, a finite element procedure to evaluate stratified level ground response to three-directional earthquakes is presented and the importance of the three-directional shaking problem is analyzed. The main feature of the procedure is that it solves the specific three-dimensional stress-strain problem with a one-directional approach.

A case study is analyzed and, at first, a 1D-1C simulation is compared to the nonlinear code NERA (Bardet and Tobita, 2001), to assess the reliability of the new hysteresis model, producing similar results. The comparison points out significant differences in peak ground parameters and dissipated energy if compared with EERA linear and equivalent linear models (Bardet J. P., Ichii K. and Lin C. H., 2000).

Since the theory is a three-dimensional strain condition, it can be developed to study two- and three-directional wave propagation problems. The extension of the proposed 1D-3C finite element numerical solution to two- and three-directional conditions are planned as further investigations to be able to study the effects of soil nonlinearity in cases of earth embankments, earth slopes and soil-structure interaction.

IMPLEMENTATION OF THE RHEOLOGICAL MODEL

The three components of the seismic motion are propagated into a multi-layered column of nonlinear soil from the top of the underlying elastic bedrock, by the use of a finite element scheme. Soil is assumed to be a continuous and uniform medium of infinite horizontal extent. Soil stratification is discretized into a system of horizontal layers, parallel to the xy plane, using quadratic line elements with three nodes. Shear and pressure waves propagate vertically in the z-direction. These hypotheses yield no strain variation in x- and y-directions.

According to a finite element modeling of a multilayer soil system assumed with an horizontal setting, the weak form of equilibrium equations, including compatibility conditions, three-dimensional nonlinear constitutive relation and the imposed boundary condition (Cook et al., 2002), is expressed in matrix form as

$$\mathbf{M}\,\ddot{\mathbf{D}} + \mathbf{C}\,\dot{\mathbf{D}} + \mathbf{F}_{\text{int}} = \mathbf{F} \qquad (1)$$

where $\mathbf{M}$ is the mass matrix, $\dot{\mathbf{D}}$ and $\ddot{\mathbf{D}}$ are the first and second temporal derivatives of the displacement vector $\mathbf{D}$, respectively, $\mathbf{F}_{\text{int}}$ is the vector of internal forces and $\mathbf{F}$ is the load vector. $\mathbf{C}$ is a matrix that derives from the fixed boundary condition, as explained below.

Discretizing the soil column into n nodes, having three translational degrees of freedom (d.o.f.) each, yields a $3n$-dimensional displacement vector $\mathbf{D}$ composed by three blocks whose terms are the displacement of the n nodes in x-, y- and z- direction, respectively. The assembled $(3n \times 3n)$-dimensional mass matrix $\mathbf{M}$ and the $3n$-dimensional vector of internal forces $\mathbf{F}_{\text{int}}$ respectively result from the assemblage of (9×9)-dimensional matrices like $\mathbf{M}^e$ and vector $\mathbf{F}^e_{\text{int}}$, corresponding to element e, which are expressed by

$$\mathbf{M}^e = \int_H \mathbf{N}^T \rho\, \mathbf{N}\, dz \qquad \mathbf{F}^e_{\text{int}} = \int_H \mathbf{B}^T \boldsymbol{\sigma}\, dz \qquad (2)$$

where H is the soil column height, $\rho(z)$ is the soil density and $\mathbf{N}$ is the (3×9)-dimensional matrix defined as follows:

$$\mathbf{N} = \begin{bmatrix} N_1 & N_2 & N_3 & & & & & & \\ & & & N_1 & N_2 & N_3 & & & \\ & & & & & & N_1 & N_2 & N_3 \end{bmatrix} \qquad (3)$$

N_1, N_2 and N_3 are the quadratic shape functions corresponding to the three-node line element used to discretize the soil column. The terms of the (6×9)-dimensional matrix $\mathbf{B}$ are the spatial derivatives of the shape functions, according to compatibility conditions (Cook et al., 2002). In equation (2), the stress components are terms of the 6-dimensional vector $\boldsymbol{\sigma}$.

The system of horizontal soil layers is bounded at the top by the free surface and at the bottom by the semi-infinite elastic medium which represents the seismic bedrock. The stresses normal to the free surface are imposed null and at the interface soil-bedrock the following condition, implemented by Joyner and Chen (1975) in a finite difference formulation, is applied:

$$-\mathbf{p}^T \boldsymbol{\sigma} = \mathbf{c}\left(\dot{\mathbf{u}} - 2\,\dot{\mathbf{u}}_b\right) \qquad (4)$$

The stresses normal to the column surface are $\mathbf{p}^T \boldsymbol{\sigma}$ and $\mathbf{c}$ is a (3×3) diagonal matrix whose terms are $\rho_b\, v_{bs}$, $\rho_b\, v_{bs}$ and $\rho_b\, v_{bp}$. The parameters ρ_b, v_{bs} and v_{bp} are the bedrock density and shear and pressure wave velocities in the bedrock, respectively. The three terms of vector $\dot{\mathbf{u}}$ are the velocities in x-, y- and z-direction respectively, at the interface soil-bedrock. The terms of the 3-dimensional vector $\dot{\mathbf{u}}_b$ are the input velocities, in the underlying elastic medium respectively in direction x, y and z. The boundary condition (4) allows energy to be radiated back into the underlying medium. According to equation (4), the matrix $\mathbf{C}^e$ and the load vector $\mathbf{F}^e$, of each element e, are defined by

$$\mathbf{C}^e = \int_H \mathbf{N}^T \mathbf{c}\, \mathbf{N}\, dz \qquad \mathbf{F}^e = \int_H \mathbf{N}^T \mathbf{c}\, \mathbf{u}_b\, dz \qquad (5)$$

The finite element model and nonlinearity of the soil requires, respectively, spatial and temporal discretization, to permit the problem resolution. The relation between stress and strain increments is linearized at each time step. Accordingly, Equation (1) is expressed by

$$\mathbf{M}\,\Delta\ddot{\mathbf{D}}^i_k + \mathbf{C}\,\Delta\dot{\mathbf{D}}^i_k + \mathbf{K}^i_k\,\Delta\mathbf{D}^i_k = \Delta\mathbf{F}_k \qquad (6)$$

where the subscript k indicates the time step t_k.

At each time step k, equation (6) requires an iterative solving to correct the tangent stiffness matrix $\mathbf{K}^i_k$. Starting from the stiffness matrix $\mathbf{K}^1_k = \mathbf{K}_{k-1}$, evaluated at the previous time step, the value of matrix $\mathbf{K}^i_k$ is updated at each iteration i (Crisfield, 1991) and the correction process continues until the difference between two successive approximations is reduced to a fixed tolerance, according to

$$\left|\mathbf{D}^i_k - \mathbf{D}^{i-1}_k\right| < \alpha \left|\mathbf{D}^i_k\right| \qquad (7)$$

where $\alpha = 10^{-3}$ (Mestat, 1993, 1998). The vectors of total displacement, velocity and acceleration are respectively defined by

$$\mathbf{D}_k^i = \mathbf{D}_{k-1} + \Delta\mathbf{D}_k^i \qquad \dot{\mathbf{D}}_k^i = \dot{\mathbf{D}}_{k-1} + \Delta\dot{\mathbf{D}}_k^i \qquad \ddot{\mathbf{D}}_k^i = \ddot{\mathbf{D}}_{k-1} + \Delta\ddot{\mathbf{D}}_k^i \tag{8}$$

The stiffness matrix $\mathbf{K}_k^i$ is obtained by assembling (9×9)-dimensional matrices like the following, corresponding to element e:

$$k_k^{e,i} = \int_H \mathbf{B}^T \mathbf{E}_k^i \mathbf{B}\, dz \tag{9}$$

The tangent constitutive (6x6) matrix $\mathbf{E}_k^i$ is evaluated by the constitutive incremental relationship given by

$$\Delta\boldsymbol{\sigma}_k^i = \mathbf{E}_k^i \,\Delta\boldsymbol{\varepsilon}_k^i \tag{10}$$

According to Joyner (1975), the actual strain level and the strain and stress values at the previous time step allow to evaluate the tangent constitutive matrix $\mathbf{E}_k^i$ and the stress increment $\Delta\boldsymbol{\sigma}_k^i = \Delta\boldsymbol{\sigma}_k^i\left(\boldsymbol{\varepsilon}_k^i, \boldsymbol{\varepsilon}_{k-1}, \boldsymbol{\sigma}_{k-1}\right)$.

The step-by-step process is solved by the Newmark algorithm, expressed as follows:

$$\begin{cases} \Delta\dot{\mathbf{D}}_k^i = \dfrac{\gamma}{\beta\,\Delta t}\Delta\mathbf{D}_k^i - \dfrac{\gamma}{\beta}\dot{\mathbf{D}}_{k-1} + \left(1-\dfrac{\gamma}{2\beta}\right)\Delta t\, \ddot{\mathbf{D}}_{k-1} \\ \Delta\ddot{\mathbf{D}}_k^i = \dfrac{1}{\beta\,\Delta t^2}\Delta\mathbf{D}_k^i - \dfrac{1}{\beta\,\Delta t}\dot{\mathbf{D}}_{k-1} - \dfrac{1}{2\beta}\ddot{\mathbf{D}}_{k-1} \end{cases} \tag{11}$$

Parameters $\beta = 0.3025$ and $\gamma = 0.6$ guarantee a conditional numerical stability of the temporal integration scheme with numerical damping (Hughes, 1987). Further research is required to investigate the influence of numerical damping on the solution and optimize the couple of chosen parameters. Equations (6) and (11) yield

$$\bar{\mathbf{K}}_k^i \,\Delta\mathbf{D}_k^i = \Delta\mathbf{F}_k + \mathbf{A}_{k-1} \tag{12}$$

where the modified stiffness matrix is defined as

$$\bar{\mathbf{K}}_k^i = \frac{1}{\beta\,\Delta t^2}\mathbf{M} + \frac{\gamma}{\beta\,\Delta t}\mathbf{C} + \mathbf{K}_k^i \tag{13}$$

and $\mathbf{A}_{k-1}$ is a vector depending to the response in previous time step, given by

$$\mathbf{A}_{k-1} = \left[\frac{1}{\beta\,\Delta t}\mathbf{M} + \frac{\gamma}{\beta}\mathbf{C}\right]\dot{\mathbf{D}}_{k-1} + \left[\frac{1}{2\beta}\mathbf{M} + \left(\frac{\gamma}{2\beta}-1\right)\Delta t\,\mathbf{C}\right]\ddot{\mathbf{D}}_{k-1} \tag{14}$$

After evaluating the displacement increment $\Delta\mathbf{D}_k^i$ by equation (12), using the tangent stiffness matrix corresponding to the previous time step, velocity and acceleration increments can be calculated by equation (11) and the total parameters are obtained according to (8). The strain increment $\Delta\boldsymbol{\varepsilon}_k^i$ is deduced by the displacement increment $\Delta\mathbf{D}_k^i$ and the stress increment $\Delta\boldsymbol{\sigma}_k^i$ and tangent constitutive matrix $\mathbf{E}_k^i$ are obtained by constitutive relationship (15). The modified stiffness matrix $\bar{\mathbf{K}}_k^i$ is calculated and the process restarts until condition (7) is verified. Afterwards, the next time step is analyzed.

FEATURES OF THE CONSTITUTIVE MODEL

Modeling three-component earthquake propagation in a soil column allows automatically taking into account effects of the second horizontal component and vertical component of motion in a one-directional site response analysis. The nonlinear soil behavior in a three-dimensional stress state is properly modeled by the constitutive model suggested by Iwan (1967) and applied by Joyner and Chen (1975) and Joyner (1975) in a finite difference formulation. This model takes into account the nonlinear hysteretic behavior of soils, using an elasto-plastic approach with hardening based on the definition of a series of nested yield surfaces. The main feature of this rheological model is its flexibility for incorporating laboratory results on the dynamic behavior of soils. The only necessary input data, to identify soil properties in the applied constitutive model, are the mass density ρ, shear and pressure wave velocities in the medium, v_s and v_p respectively, and the normalized shear modulus decay curve G/G_0 versus shear strain γ. $G_0 = \rho v_s^2$ is the elastic shear modulus, measured at the elastic behavior range limit $\gamma \cong 10^{-6}$ (Fahey, 1992).

The homogeneous cyclic soil response to one-component signal is evaluated to observe nonlinear effects. A sinusoidal shear strain input with increasing amplitude (Figure 1a) is applied in x-direction and the cyclic strain-stress behavior is displayed in Figure 1b. The undertaken analysis, presented in Figure 1, is based on the assumption that the behavior of soil can be adequately represented by a hyperbolic stress-strain curve. According to Hardin and Drnevich (1970), this assumption yields a normalized shear modulus decay curve, used as input curve representing soil characteristics, expressed as (Figure 1c)

$$G/G_0 = 1\Big/\left(1+\left|\gamma/\gamma_r\right|\right) \tag{16}$$

where γ_r is a reference strain provided by test data corresponding to a shear modulus reduction of 50%. The soil properties are $\rho = 1780\,\text{kg/m}^3$, $v_p = 560\,\text{m/s}$, $v_s = 300\,\text{m/s}$ and $\gamma_r = 0.1\%$ and the input frequency $3\,\text{Hz}$.

Applying cyclic shear strains with amplitude greater than the elastic behavior range limit gives open loops of shear stress versus shear strain, exhibiting some hysteresis. As strain amplitude is increased, the shear modulus reduces. This reduction is represented by a normalized shear modulus decay curve (Figure 1c).

The homogeneous cyclic soil response to a three-component seismic signal is evaluated to analyze nonlinear effect variation under triaxial stress state. The soil response to one-component signal is compared with the case of two equal shear strain components applied in x- and y-directions and a normal strain component reduced by a factor of 10 applied in z-direction (Figure 1d, e, f). The material strength is lower under triaxial loading than for simple shear loading. Figure 1c shows the normalized shear modulus decay curve used as input and the normalized tangent shear modulus decay and damping in the two cases of one- and three-component shear strain input. The material damping ratio D represents the energy dissipated by the soil. It is evaluated as

$$D = W_D / \left(4\pi W_S\right) \tag{17}$$

where W_D is the energy dissipated in one cycle of loading and W_S is the maximum strain energy stored during the cycle.

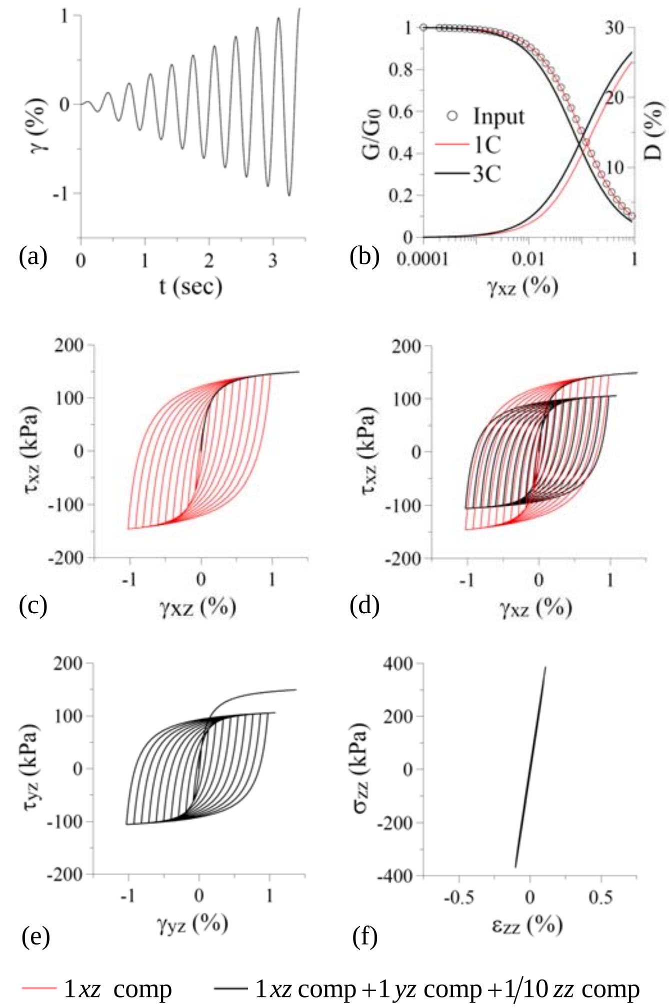

Fig. 1. Three-component effect in material behavior: a) Increasing amplitude sinusoidal strain input; b) Normalized shear modulus decay and damping computed in the two cases of one- and three-component shear strain input; c) Hysteretic response to one-component input; d, e, f) Comparison between the hysteretic response in x, y and z direction, respectively, in the two cases of one- and three-component input.

The shear modulus decreases and the dissipation increases, for increasing strain amplitude, due to nonlinear effects also when only one component is applied. From one to three components, for a given maximum strain amplitude, the shear modulus decreases and the dissipation increases.

The method does not depend on the hyperbolic initial loading curve. Different relationships satisfying Masing criterion can be represented by a model of the Iwan type and a purely empirical stress-strain curve derived from laboratory measurements can also be directly used. Stress-strain curves of real soils are not usually truly hyperbolic, thus it is more convenient to directly apply the real shear modulus decay curve obtained by test data.

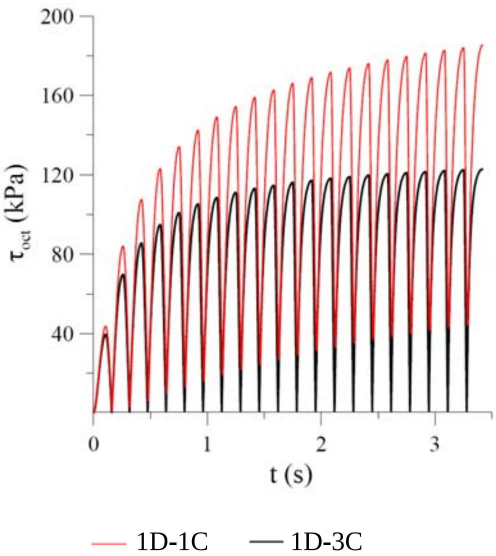

Fig. 2. Time history of the octahedral shear stress for the case of three combined 1D-1C seismic response analyses, in x-, y- and z-direction respectively, and for the 1D-3C case.

DESCRIPTION OF THE SITE AND INPUT

The stratigraphic setting of a soil column in the Tiber Valley of Rome (Italy) is used to analyze the seismic wave propagation in the cases of one- and three-component input and to compare the obtained results with those provided by public nonlinear codes, in the case of one-component input. The spatial description of stratigraphy and lithology of the alluvial deposits in the Tiber Valley of Rome, is described by Bonilla et al. (2010).

The soil column that is modeled in this study consists of five layers on a seismic bedrock, whose depth below the ground level z and physical properties, as density ρ, shear velocity in the medium v_s, and the elastic shear modulus G_0, are reported in Table 1 according to Bozzano et al. (2008). The pressure wave velocity in the medium v_p is deduced by imposing a Poisson's ratio of 0.3.

Site and laboratory testing of the Tiber alluvial deposits (Bozzano et al., 2000, 2008) shows a significant stiffness contrast between the sand layers (lithotypes R, A, B and D), silty-clayey alluvia (lithotype C) and sandy gravels of the bedrock. The basal gravels are considered as the local seismic bedrock, being characterized by a S-wave velocity $v_s > 700\,\text{m/s}$.

Table 1. Stratigraphic and geotechnical properties of the analyzed soil column (Tiber valley, Rome)

Layer	**z**	**ρ**	**v_s**	**G_0**
	m b. g. l.	kg/m^3	m/s	MPa
R	0 - 2.5	1830	220	89
A	2.5 - 12.5	1875	239	107
B	12.5 - 32.5	1865	417	324
C	32.5 - 52.5	1865	212	84
D	52.5 - 57.5	1957	417	340
Rock	> 57.5	2141	713	1088

The dynamic mechanical properties of the Tiber alluvial deposits come from laboratory data obtained by resonant column (RC) or from cyclic torsional shear tests. The normalized shear modulus decay curves employed in this work are shown in Figure 3.

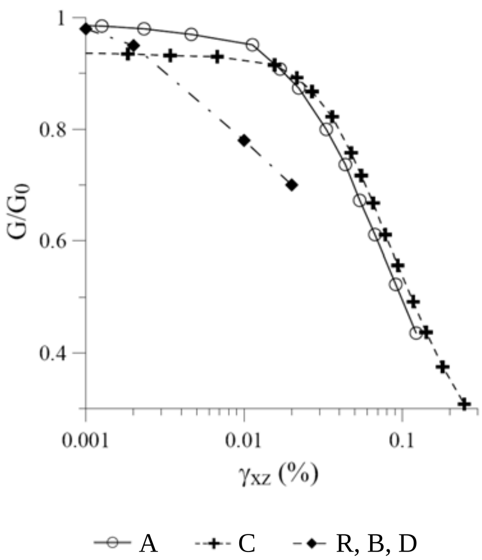

Fig. 3. Normalized experimental shear modulus decay curves for the soils present in the analyzed column.

The acceleration time histories in Figure 4 are the three components of the seismic event referred as the 14th October 1997 Umbria-Marche earthquake, characterized by a horizontal PGA of 0.3 g and local magnitude of 5.4 ML. These waveforms are provided by the ENEA ground motion database, recorded on outcropping bedrock by CODISMA digital accelerometer devices (Lenti et al., 2009). Accelerations are recorded at Cerreto di Spoleto (Perugia), about 25 km far from the earthquake epicenter. The three acceleration time histories in Figure 4 have been filtered using a cut-off frequency of 20 Hz. These acceleration signals are halved to take into account the free surface effect and integrated, to obtain the corresponding input data in terms of vertically incident velocities, before being forced at the base of the horizontal multilayer soil model.

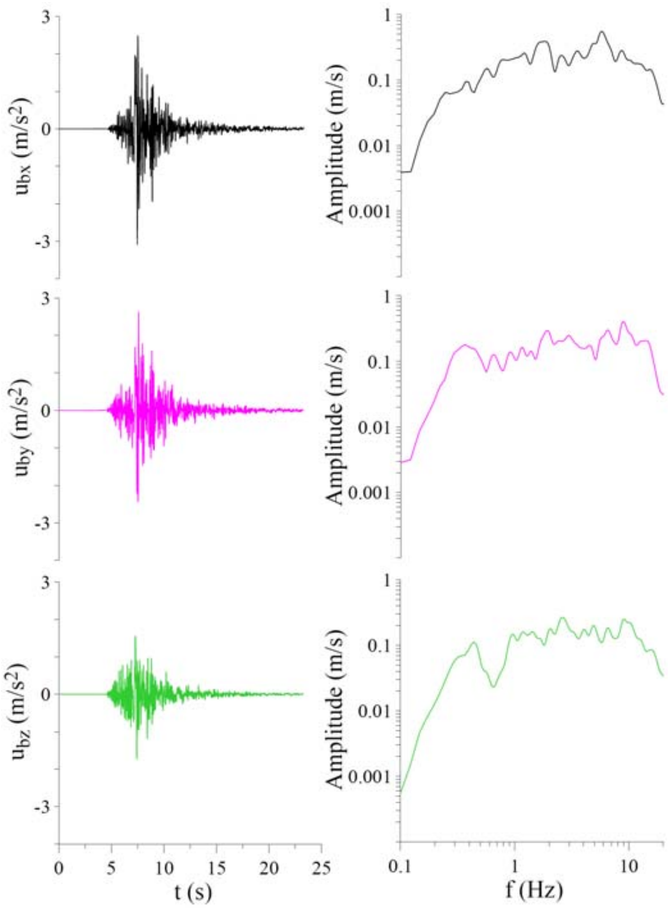

Fig. 4. Recorded acceleration time history and corresponding Fourier Transform in West-East, North-South and vertical directions, respectively.

COMPARISONS IN THE 1D-1C CASE

The influence of the implemented multi-surface plasticity approach on local seismic response is now analyzed. The presented finite element algorithm is assessed for a one-component wave propagation problem in a vertical profile.

As first test, a homogeneous layer having the same properties of the underlying elastic bedrock is used, to validate the implementation of the model. The x-component of the signal shown in Figure 4, registered on outcropping rock, is halved and it is propagated in the layer, in terms of velocity, up to the free surface. The acceleration signal at the free surface is, as expected, the acceleration corresponding to double velocity input respect to the one imposed at the column base.

The x-component of the signal in Figure 4 is also propagated along the system of horizontal layers modeling the stratigraphic setting present in the Tiber Valley of Rome (Bonilla et al., 2010), as described in section 4. Results obtained by this research (Figure 5) are corroborated by comparison with output data acquired by the software NERA (Bardet and Tobita, 2001).

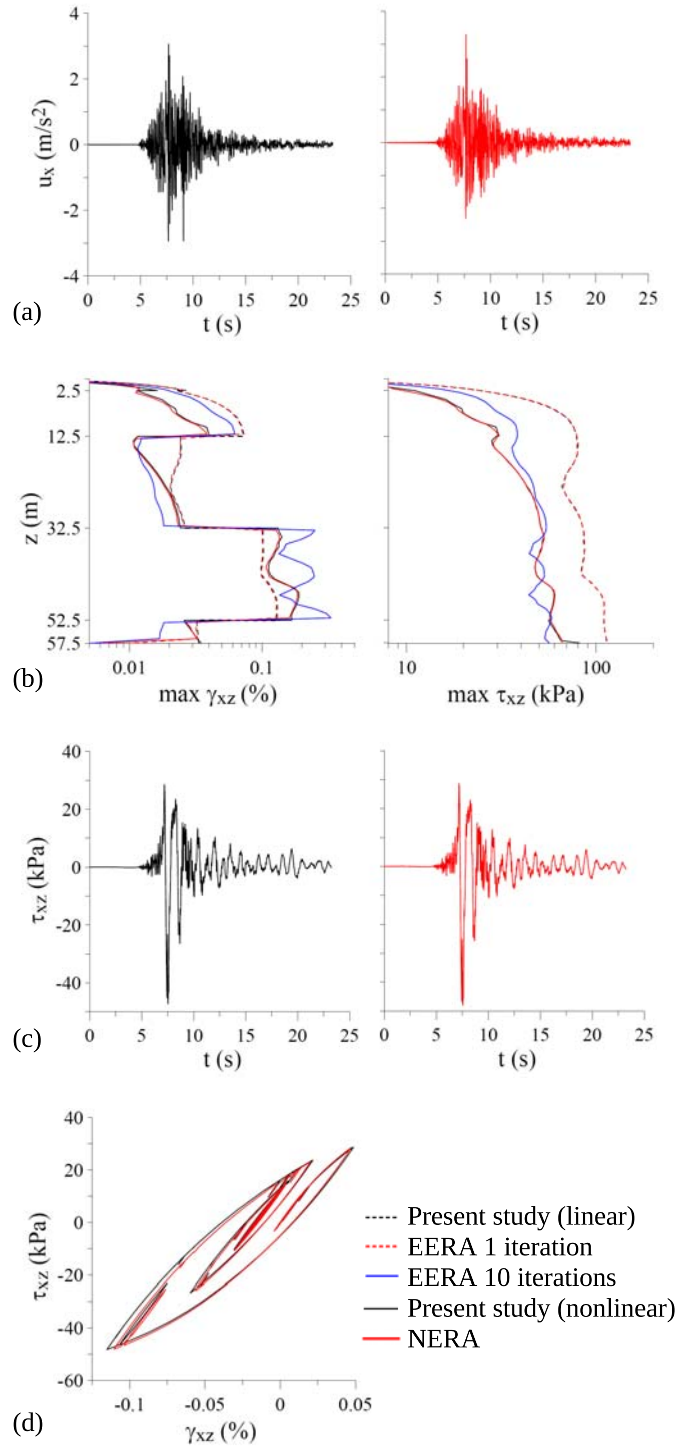

Fig. 5. Comparison with the results obtained by NERA and EERA for one-component seismic response analysis: a) Acceleration at the free surface; b) Maximum shear strain and shear stress profiles; c, d) Time history of the shear stress and hysteretic loop, respectively, computed at a 42.5 m depth.

NERA is a 1D-1C ground response analysis program where the one-component constitutive model developed by Iwan

(1967) is implemented in a finite difference formulation, using the boundary condition proposed by Joyner and Chen (1975). Nonlinear site response in time domain is also compared with the equivalent linear site response obtained by the software EERA that works in frequency domain (Bardet, Ichii and Lin, 2000).

The mechanisms of one-directional earthquake response to stratified level grounds are analyzed in terms of peak ground acceleration and the values obtained using the different employed formulations are presented in Table 2.

Table 2. Peak Ground Acceleration at the free surface of the soil column

Peak Ground Acceleration (m/s^2)				
EERA 1 it.	**Linear**	**EERA 10 it.**	**NERA**	**Nonlinear**
5.4	5.3	4.4	3.3	3.1

Acceleration time history at the free surface, stress time history and hysteresis loop, in the middle of the C clay layer, result coherent with the evaluations obtained by NERA (Figure 5a, c, d). Strain and stress profiles obtained by linear, equivalent linear and nonlinear earthquake analyses are compared in Figure 5b.

ANALYSIS OF THE LOCAL 3C SEISMIC RESPONSE

The effect of simultaneously propagating the three components of the input signal in a system of horizontal soil layers is then studied and compared to the cases of one-component input.

The three-component incident motion displayed in Figure 4, registered on outcropping rock, is propagated in the vertical profile of Tiber Valley, in Rome (Bonilla et al., 2010), described in section 4. The finite element procedure described in section 2 is applied for a one-directional wave propagation problem for local seismic response analysis.

Acceleration time history at the free surface, stress time history and hysteresis loop, in the middle of the C clay layer, are compared, in x - direction, for the case of one- and three-component input (Figure 6a, c, d). Strain and stress profiles are shown in Figure 6b.

The maximum octahedral shear strain and stress distribution, obtained along the vertical soil profile, is shown in Figure 7 for the two cases of three combined 1D-1C analyses in the three directions of input motion and of 1D-3C analysis. The low strain level that is attained in the analyzed soil column does not allow discerning the different achieved soil strength. Parametric studies for different applications are necessary to corroborate the model for higher deformation values.

The theoretical transfer function is a technique that is frequently used for site response estimation. This approach considers the ratio between the estimated acceleration spectrum at a site of interest and the recorded acceleration spectrum at a reference site, which is usually a nearby rock site. According to this criterion, the local site responses along the studied soil profile are compared in terms of horizontal acceleration amplification functions.

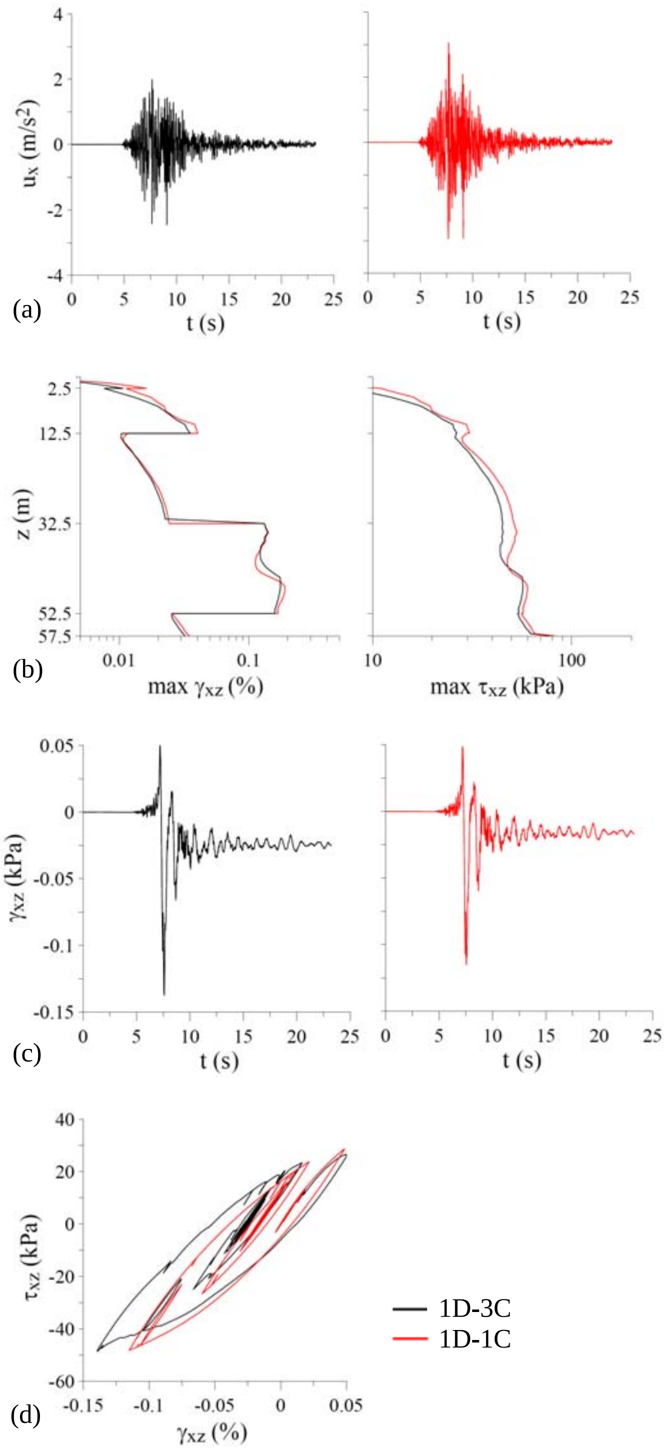

Fig. 6. Comparison between one- and three-component seismic response analysis in x-direction: a) Acceleration at the free surface; b) Maximum shear strain and shear stress profiles; c, d) Time history of the shear strain and hysteretic loop, respectively, computed at a 42.5 m depth.

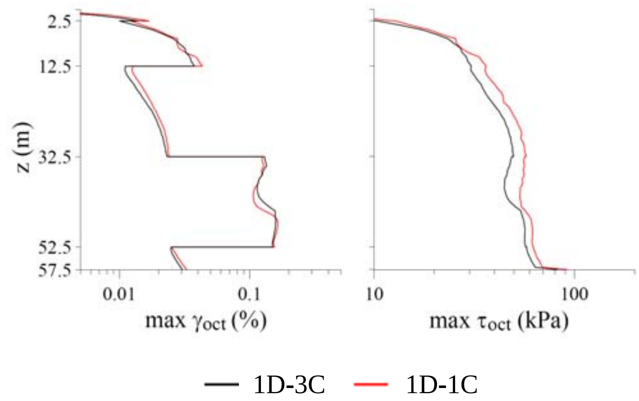

Fig. 7. Maximum octahedral shear strain and stress profiles for the case of three combined 1D-1C seismic response analyses, in x-, y- and z -direction respectively, and for the case 1D-3C.

The norm of the transfer functions of two horizontal accelerations, from the 3C results, is compared to the 1C transfer function for the linear and nonlinear computations (Figure 8). Nonlinearity effects produce a shift of the fundamental frequency toward lower frequencies, as well as an attenuation of the spectral amplitudes at higher frequencies. The effect on the transfer function produced by a three-component seismic input can not be defined using one case only, for this reason further research is necessary to compare the results obtained in different cases and to implement this nonlinear approach in two- and three-directional models.

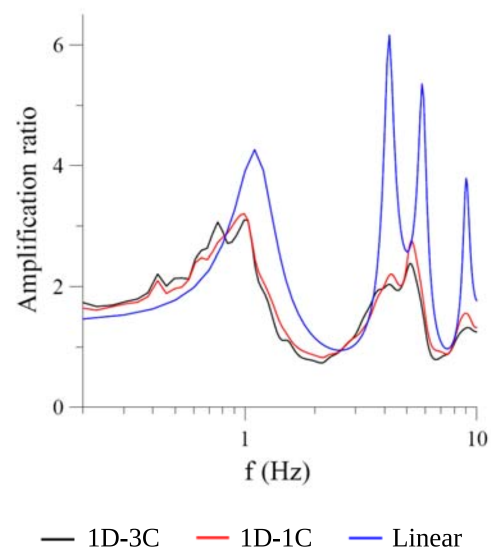

Fig. 8. Norm of horizontal acceleration transfer functions for soil linear behavior, for the case of the two combined 1D-1C seismic response analyses in x- and y-direction, and for the case 1D-3C.

CONCLUSIONS

In this paper, a mechanical model is proposed to analyze the 1D-3C seismic response of soil profiles. A finite element modeling of a horizontal multilayered soil is implemented, by adopting a three-dimensional constitutive relation that needs few parameters to characterize the hysteretic behavior of the soil.

The proposed method provides a promising solution for local seismic response evaluation and site effect analysis, useful for structural design. This work is a natural extension of the public nonlinear codes such as NERA.

Parametric studies and comparative analysis with experimental data are still necessary to calibrate the solution and to evidence the three-component effects in the 1D-3C approach.
Efficient finite element formulation of the proposed mechanical model for two- and three-directional cases motivates further developments.

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