A Comparative Analysis of Seismic Site Response in Time and Frequency Domains

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ABSTRACT

This study aims primarily to perform a comparative analysis of the seismic response of a soil profile, in the time and frequency domains, in order to evaluate the seismic response of soil subjected to seismic excitation. After a few remarks made on the responses given by the linear elasticity method for this type of problem, it was considered necessary to use SHAKE 2000 and PLAXIS in this study. The obtained results were then compared with those of the available theoretical predictions. Rock elasticity, viscous damping and damping by hysteresis, and the nonlinearity of the ground were then taken into account. In addition, comparisons between recorded responses were also conducted.

Keywords-dynamic analysis; soil damping; finite elements; site response

I. INTRODUCTION

Dynamic Soil-Structure Interaction (SSI) problems involve determining the reaction of a building under the seismic effect and defined by the free surface of ground without any structure [1]. Variations in free surface seismic responses, which are used as the input motion, must satisfy the so-called free-field motion equations which can be obtained from the ground response analysis [2-3]. Thus, according to the literature, before studying the soil-structure interaction system, a free field response is required. It should be known that the dynamic finite element analysis can be considered as one of the most comprehensive available tools in geotechnical problems [4]. It is an approach capable of providing insight into the distribution of stresses in soil or the deformations, and the loads applied to structural components that interact with the soil. But, this technique requires the availability of an appropriate soil model, with sufficient information on soil properties, provided by performing different experiments. Here, the seismic input motion ought to be appropriately determined. The response of a finite element model [5] is also conditioned by the adjustment of several parameters that affect the sources of energy dissipation in the time domain analysis. Furthermore, the amount of damping exhibited by a discrete numerical system can be determined by the proper choice of the constituent model (material damping), the integration scheme of equations, and the boundary conditions (numerical damping). In addition, material damping models the viscous and hysteresis energy dissipation effects in soils. Note that the numerical damping appears as a consequence of the numerical algorithm for the dynamic equilibrium solution in the time domain [6]. Moreover, the boundary states can influence the numerical model and convey the specific energy of waves outside the time domain [7-8]. In this article, the free field surface is exclusively taken, and the responses provided for different domain analyses are interpreted and compared.

II. PROFILES OF THE SOIL UNDER STUDY

For the purposes of this study, it was considered necessary to examine different soil profiles that are characterized by an increasing level of heterogeneity. The soil includes a homogeneous viscoelastic layer (denoted HOM profile), a linear elastic layer with increasing stiffness at depth, and finally a non-linear soil layer (denoted SDS for Stress Dependent Stiffness profile) [9-10].

A. *Homogeneous Viscoelastic Layer*

The soil stiffness is displayed by a constant value of shear modulus G. In the present case, the homogeneous linear viscoelastic profile is placed on a rigid rock and is then examined. The soil profile characteristics are shown in Figure 1(a). Such a system can be entirely presented through its amplification function A(f). On the other hand, Figure 2 shows the graphical representation of the amplification function in relation to frequency, while admitting that the soil layer considered has the adopted characteristics. The two verticals refer to the first and second natural frequencies of the soil profile.

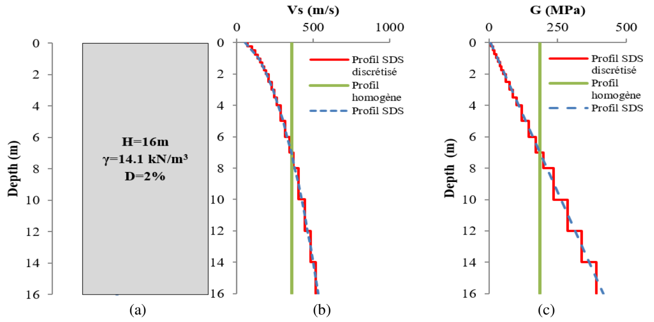


Fig. 1. Soil profiles: (a) Physical characterization, (b) shear wave propagation velocity of the profiles, (c) shear modulus of the soil.

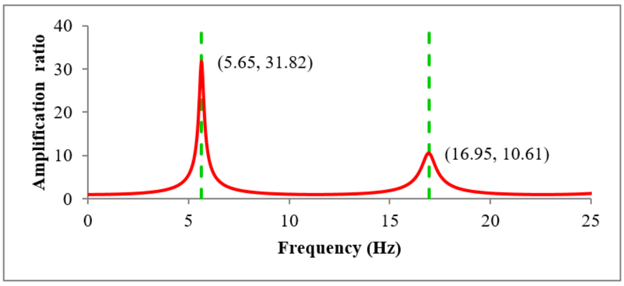


Fig. 2. Amplification function of a homogeneous viscoelastic soil layer on a rigid rock.

B. Stress Dependent Stiffness Layer

Stiffness is a function of the average effective stress p', and therefore of the depth. A linear relation describes the evolution of the shear modulus G as a function of depth z ($m = 1/2$):

$$G = G_0(1 + az)^{2m} = G_0(1 + az) \quad (1)$$

Afterwards, it was decided to adopt a single value for the density ρ. The velocity of shear wave propagation was then obtained by:

$$V_s = V_{s0}(1 + az)^{m} = V_{s0}(1 + az)^{1/2} \quad (2)$$

where V_{S0} is the shear wave velocity at the free surface, an a is a coefficient that represents the level of soil heterogeneity

C. Non-Linear layer

The average value curves for sand, similar to those proposed in [11], were then considered. Figure 3 shows the graphical representations of these curves. It is important to emphasize that the variation of the initial shear modulus according to depth is identical to that of the SDS profile.

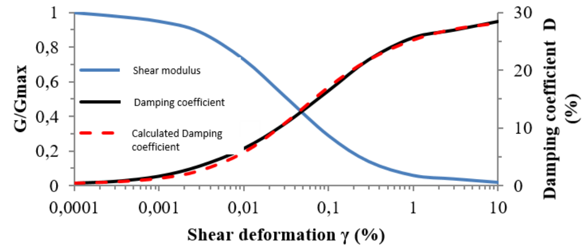


Fig. 3. Curves representing the adopted shear modulus (solid line) and damping factor (broken line), calculated by Hashash's MRDF model.

III. THE INPUT SIGNAL

In numerical calculations, the seismic loading is often imposed as an accelerogram at the base of the geometrical model (the rock). In order to clarify and better understand the impact of the seismic signal on the nonlinear soil response, two acceleration histories were used [12]:

- The first signal corresponds to the W-E component of the acceleration recording that was made at the Tolmezzo station for the main shock of the Friuli earthquake in Italy that occurred on May 6, 1976. It is denoted as TMZ-270. The data were recorded at a frequency of 200Hz, for a whole of 7279 recording points. The peak of horizontal acceleration, even to 0.315g was obtained after t = 3.935s. It is worth noting that most energy is within a frequency range among 0.8 and 5 hertz.
- The second signal corresponds to the W-E component of the acceleration recording that was made at the Station of Kedarra 1 for the main shock of the earthquake that occurred on May 21, 2003, in Boumerdès (Algeria). The data were recorded at a frequency of 200Hz for a total of 7000 recording points. The maximum horizontal acceleration, equal to 0.331g, was reached after t = 7.415 s.

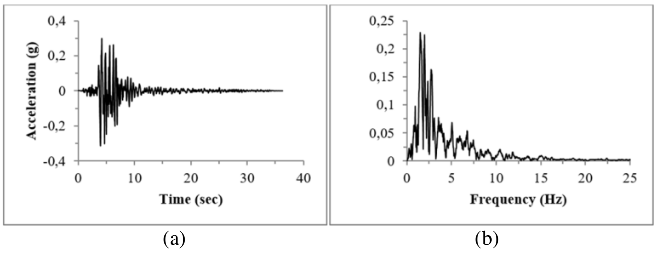


Fig. 4. TMZ-270 seismic input signal: (a) Acceleration history, (b) Fourier spectrum.

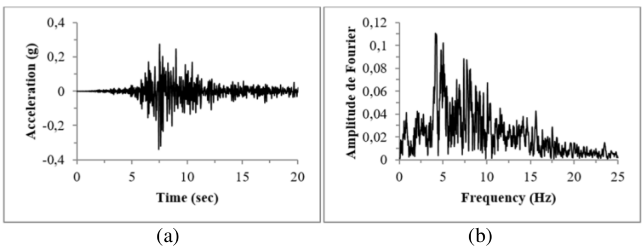


Fig. 5. Keddara-1 seismic input signal: (a) Acceleration history, (b) Fourier spectrum.

IV. RESULTS AND DISCUSSION

With regard to the soil profile interaction with the acceleration history, different analyses were performed in order to show the influence of some numerical parameters. In the present case, the response found from the HOM and the SDS profiles are presented. The equivalent linear approach for the time domain analyses with the PLAXIS code was used [13-14]. The obtained responses were compared with those resulting from the frequency domain analysis, carried out with SHAKE [15]. Lastly, comparisons of various calculated soil responses supplied were reviewed and interpreted.

A. Linear Analysis of the HOM Linear Profile

Linear analysis was carried out in the frequency and time domain. The obtained results were compared with of the homogeneous elastic profile resting on a rigid rock in order to show the effect of the various origins of energy dissipation on the finite element model and to explain the choices made for a good numerical modeling process. The frequency-domain and time-domain responses of linear analyses carried out in SHAKE and PLAXIS, respectively, are illustrated in Figure 6 which depicts the amplification function and the maximum acceleration profiles.

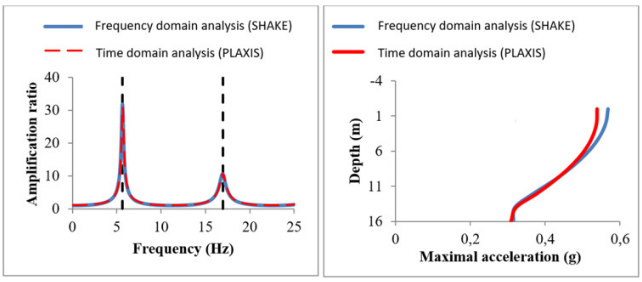


Fig. 6. Comparison between the numerical and the reference solutions: (a) Amplification functions, (b) maximum acceleration profiles for TMZ-270 input motion.

B. Linear Analysis of the SDS Profile

Linear analysis was performed in the frequency domain for TMZ-270 and Keddara-1 motions in SHAKE software. The obtained responses were used as a basic response for the finite element analysis. The Rayleigh coefficients (α_R = 1.224, β_R = 2.441×10) were determined arbitrarily by selecting the first natural frequency f_1 of the layer, and the mean $(f_2 + f_3)/2$, of the 2nd and 3rd natural frequencies of the soil [16-17]. Numerical damping was assumed to be equal to zero. Figure 7 shows specific gaps observed on the amplification functions, around the third natural frequency of the layer. Nevertheless, it turned out that there was a good agreement between the maximum acceleration profiles, which proves the effectiveness of the adopted lateral boundary conditions.

C. Analysis using the Equivalent Linear Approach for the SDS Profile

To simulate the non-linearity of the ground under periodic stress, it was deemed necessary to use the most common method, which in this case is the equivalent linear approach. This method was applied in the SHAKE software. The PLAXIS software does not allow equivalent linear analysis. In order to carry out an operation, the domain must be separated into sub-layers. A different material is indicated for each sub-layer. Consequently, the only eventual way out is to first perform equivalent linear analysis with the assistance of SHAKE including an auto-iterative procedure. The output profiles of $G(z)$ and $D(z)$, obtained from SHAKE, determine the PLAXIS material parameters. The initial shear modulus and damping constant profiles as well as those derived from the SHAKE analysis are plotted in Figure 8. The resulting curves represent the data profile for PLAXIS. Thus, the values accepted for the Rayleigh damping parameters are clearly reported.

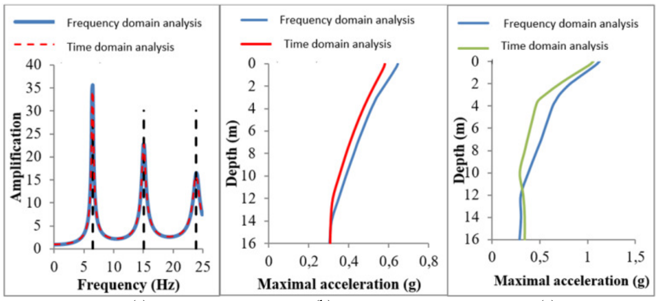


Fig. 7. Comparison between the linear analyses in the frequency domain and time domain of the SDS profile: (a) Amplification functions, (b) maximum acceleration profiles of TMZ-270 seismic motion, (c) maximum acceleration profiles of Keddara-1 seismic motion.

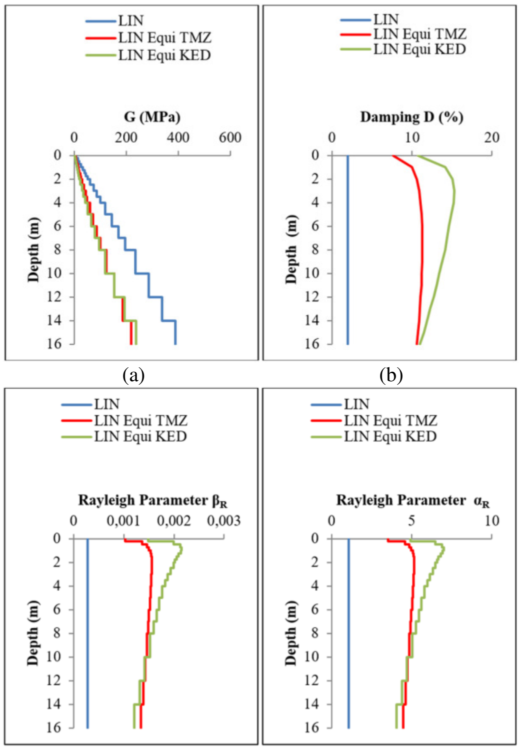


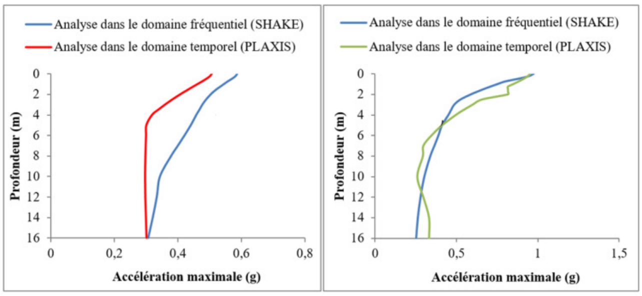
Fig. 8. Profiles of the soil parameters adopted by the equivalent linear approach: (a) Shear modulus, (b) damping, (c), (d) Rayleigh parameters.

Figure 9 presents the responses achieved from the numerical tests of the equivalent linear approach when using the time investigation scheme. In addition, the average constant acceleration and the Rayleigh parameters of each sub-layer were evaluated. The 1st and 2nd natural frequencies of the subsoil, as indicated in Table I, are considered as the reference frequencies. It should also be noted that the calibration of the analysis parameters in the time domain are based on the results of the frequency domain. Note also that the two types of equivalent linear approaches provide the same seismic excitation, particularly with regard to maximum accelerations.

TABLE I. REFERENCE FREQUENCIES FOR THE EVALUATION OF RAYLEIGH NUMERICAL DAMPING - SDS SOIL LAYER

Abbreviation	Type of analysis	f_1 (Hz)	f_2 (Hz)
Lin	Linear	6.48	15,15
Lin Equi	Linear equivalent	4.72	16,99

The maximum values of the amplification function determined with finite element software are near to those obtained with the SHAKE, particularly close to the reference frequencies. These responses show that, as indicated, the soil parameters determined for each sub-layer are in accordance with the deformation rate generated by the seismic movement.



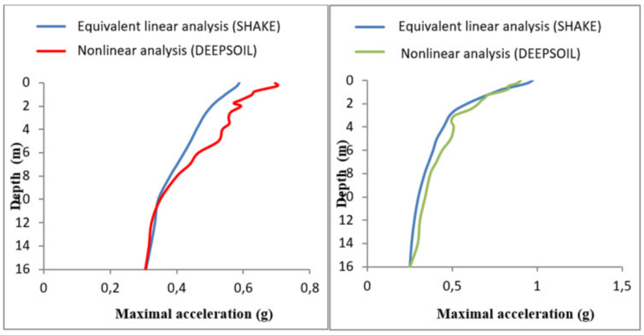
(a) (b)

Fig. 9. Comparison between the peak acceleration profiles obtained by equivalent linear analysis in the frequency and time domain of the SDS profile using (a) TMZ-270, (b) Keddara-1 seismic motions.

D. Nonlinear Analysis of the SDS Profile

It is recognized that the equivalent linear approach ensures satisfactory results, particularly for the site seismic effect, although this approach is an estimate of real nonlinear treatments. A new approach consists of analyzing the results of a real nonlinear profile, based on numerical integration in the time domain. DEEPSOIL [18] solves 1D problems related to wave propagation in a nonlinear medium. Estimates were computed in the time domain, and the constitutive modeling was implemented according to the modified hyperbolic modeling. Note that this software was used first to deal with the problem of vertically propagating S-waves in the non-linear SDS profile on a hard rock, and then to make a comparison of the obtained responses with those resulting from the equivalent linear model [19-21].

Figure 10 presents the peak acceleration profiles and response spectra that were computed by SHAKE and DEEPSOIL, by suggesting the shear modulus and damping constant curves shown in Figure 3 for hard rock.



(a) (b)

Fig. 10. Comparison between the maximum acceleration profiles obtained by the equivalent linear analyses (SHAKE) and by the non-linear analysis (DEEPSOIL) of the SDS profile, using: (a) TMZ-270, (b) Keddara-1 seismic motions.

V. CONCLUSIONS

The findings of this study showed that one-dimensional nonlinear soil response analysis provides a more accurate characterization of true nonlinear soil behavior in comparison with the equivalent linear procedures. However, the use of nonlinear software remains limited, resulting in insufficiently documented and imprecise choice of code application parameters and official procedures of used software.

In the current study, linear responses were employed in the frequency domain for the evaluation of energy dissipation sources in association with dynamic finite elements analysis. The calculations carried out using the PLAXIS indicated that the adopted numerical model was rightly chosen. Both were applied for the evaluation with Rayleigh damping parameters, while considering numerical dissipation for the time integration scheme, and also for lateral boundary conditions, to minimize the impact due to wave reflection. The obtained results are quite encouraging for using such methods whenever finite element seismic analyses are carried out in various types of geotechnical systems, i.e. retaining walls, tunnels, etc.

In finite element software, such as the PLAXIS, the filtering input for transforming the input motion from the rock outcrop to the interior is not taken into account. Besides, the input seismic motion can be simply converted to the outcrop interior with the help of other software (like the SHAKE) and by submitting the filtered signal to the base of the soil profile, in order to simulate an elastic rock using the finite element model.

Finally, the non-linear behavior of the soil was also examined. The numerical parameters of a two-dimensional soil profile using the finite element method developed in PLAXIS should be calibrated first using the equivalent linear approach analysis applied in SHAKE. Then, the non-linear approach, in DEEPSOIL, followed. The obtained responses were compared with those obtained by the equivalent linear approach. The maximum acceleration values at the surface of the equivalent linear analysis remain higher than those resulting from the non-linear analysis. This can be attributed to the greater amplification of the soil motion by frequencies near the first frequencies of the seismic excitations used.

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