Seismic Response of Soft Soil Deposit Using Simplified Models

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Abstract. Near surface soils can greatly influence the amplitude, duration, and frequency content of ground motions. Surveys of the damage caused by earthquakes indicates that the lowest levels of damage occur in structures founded on rock or hard soil, while most of the damage occurs usually in structures founded in soft soil sites. With the aim to understand better the seismic response of soft soils deposits, not susceptible to liquefaction, this study made a comparison between the real seismic response registered in soft soil deposit in the 2011 Tohoku earthquake (Mw=9.1), with the response predicted by a propagation analysis with the equivalent linear method using the computer program SHAKE2000 [1]. An additional comparison is made applying the simplified method of Carlton (2014), developed specifically for soft soils. The site chosen for this analysis was a soft soil deposit, with NEHRP site classification type F, monitored by the seismic station TKCH07 of the KiK-net network located in Hokaiddo, Japan. The estimated response showed and acceptable approximation with the real response, although the response calculated with SHAKE2000 predicted high levels of amplification near the natural frequencies of the soft soil deposit.

1 Introduction

Mechanical characteristics of soils deposits can greatly influence the amplitude, duration, and frequency content of ground motions. Surveys of the damage caused by earthquakes indicates that the lowest levels of damage occur in structures founded on rock or hard soil, while most of the damage occurs usually in structures founded on sites composed by soft soils.

The objective of this study is to have a better understanding of the seismic behaviour of soft soils deposits not susceptible to liquefaction. For seismic design of structures, building codes as the International Building Code (IBC, 2012) [2] and the provisions of the National Earthquake Hazard Reduction Program (NEHRP) classify soft soils deposits as sites of the types E and F, which include sites with more than 7.5 meters of soft clay with high plasticity (PI>75), highly organic clays and soft to medium stiff clays more than 37 meters thick, with undrained shear strength less than 50 kPa and average shear wave velocity over the top 30 meters of the soil deposit less than 180 m/s.

There is little empirical data the seismic behaviour of NEHRP E and F sites. Generally, its necessary to perform site specific numerical simulations called site response analyses, based on the propagation of shear waves through the entire profile of the soil deposit, originated in the bedrock. In this study a comparison was made between the real seismic response registered in a soft soil deposit in the 2011 Tohoku earthquake (Mw=9) with the response predicted by the propagation analysis using the total stress equivalent linear method implemented in the computer program SHAKE2000. An additional comparison is made with the simplified method of Carlton (2014) [3], developed specifically for soft soils E and F.

The soft soil model for this study is based on the soft soil profile, classified as a NEHRP F site, monitored by the station TKCH07 located in Hokaiddo, Japan, which is part of the strong motion seismograph network KiK-net. KiK-net stations consist of pairs of strong-motion seismographs installed in a borehole in the contact with bedrock as well as on the ground surface. The database of strong motion acceleration time series and the properties of the soil profile are available for public access on the web site of the National Research Institute for Earth Science and Disaster Prevention (NIED) [4].

2 TKCH07 Station from KIK-NET seismographic network

KIK-NET is a seismographic network operated by National Research Institute for Earth Science and Disaster Prevention (NIED) [4] in Japan, consisting of pairs of high-sensitivity seismographs installed at the bottom and on the surface of drilled holes. The geological and geophysical data registered in each station are available on the internet. The selected station for the present study was TKCH07, located in a soft soil deposit, classified as a NEHRP F site, whose properties are described in section 4.2.

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The time history of accelerations used in the propagation analyses of SV waves corresponds to the 2011 Tohoku earthquake of magnitude Mw=9, registered in the bottom of the bore hole, to a depth of 103 m, in the contact with the bedrock (Figure 1). The accelerogram was baseline corrected and filtered by bandpass between the frequencies of 0.1 to 25 Hz.

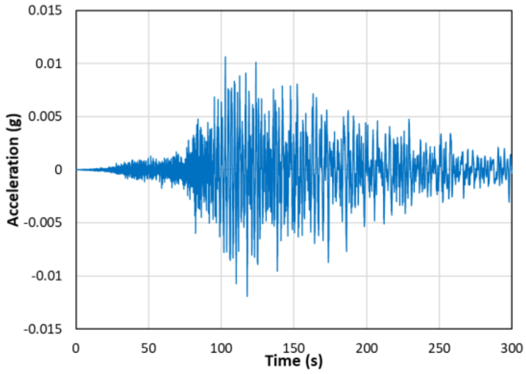

Fig. 1. Acceleration record of the 2011 Tohoku earthquake, at bedrock bottom of the soil profile.

To estimate the response with the Simplified Method of Carlton 2014 it was used the acceleration spectrum obtained through a deterministic analysis with the attenuation law of Atkinson and Boore [5] for subduction sources of the interface type. It was considerate a distance of 300 Km and a depth of 24 km, according to the real characteristics of the Tohoku earthquake, to produce the spectrum of accelerations response on the surface of the soil profile. The spectral accelerations (Sa) are shown in Figure 2 as well the spectrum of real response on the rock at the profile base.

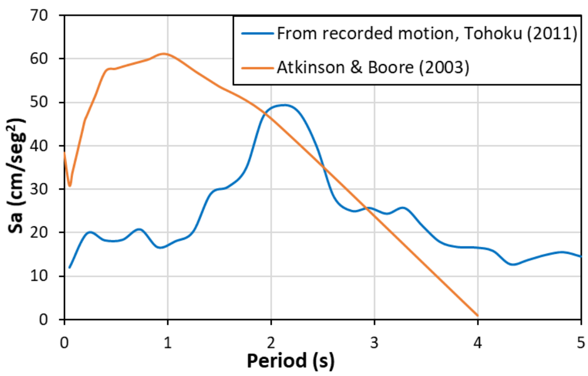

Fig. 2. Acceleration response spectrum of the recorded ground motion and from prediction equations of Atkinson and Boore [4].

3 Shear Modulus Reduction and Damping Curves

Stiffness and damping are most important soil properties for dynamic site response analysis (Kramer) [6]. In the equivalent linear model (Figures 3 and 4), the degradation of the shear modulus is represented as the decrease of the normalized secant modulus in relation of the maximum shear modulus (G_{sec}/G_{max}) as cyclic shear strain (γ) increases. On the other hand, soil damping is represented as the associated damping ratio D (not frequency dependent) proportional to the area of the hysteresis loop, which increases as cyclic shear strain (γ) increases.

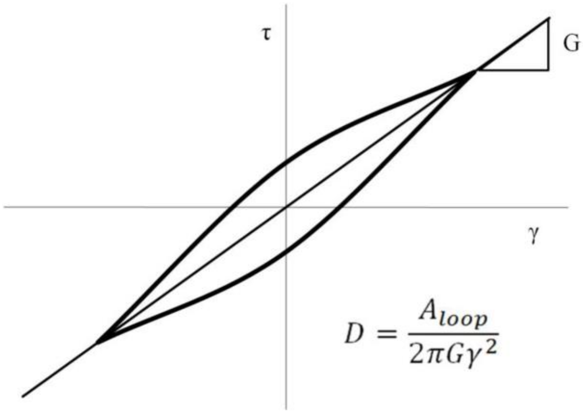

Fig. 3. Hysteresis loop of soil, showing secant shear modulus G and the damping ratio D for the equivalent linear model.

Figure 4 shows the backbone curve corresponding to the extreme values of the hysteretic cycle to different levels of cyclic shear strain. The curve approaches asymptotically the shear strength of the soil τ_{ff} for larges deformations, while the inclination from the origin corresponds to the maximum shear modulus (G_{max}).

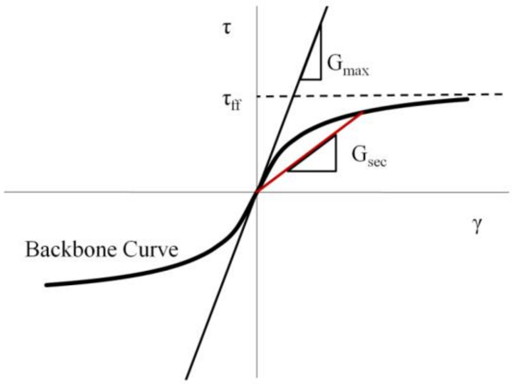

Fig. 4. Backbone curve in the stress (τ)–strain (γ) space.

Figure 5 shows typical shear modulus reduction and damping curves. Vucetic [7] divided these curves into three regions separated by two shear strain values: the linear cyclic threshold shear strain (γ_{tl}) and the volumetric cyclic threshold shear strain (γ_{tv}). For shear strains less than γ_{tl} soils exhibit an elastic linear behaviour, the shear modulus is a constant maximum value, G_{max}, and the soil damping is constant minimum value, D_{min}. For shear strains between γ_{tl} and γ_{tv} soils presents non-linear elastic behaviour, the shear modulus degrades and the damping increases, but the amount of plastic deformation and pore pressure generation are negligible so that are deformations being recoverable upon unloading.

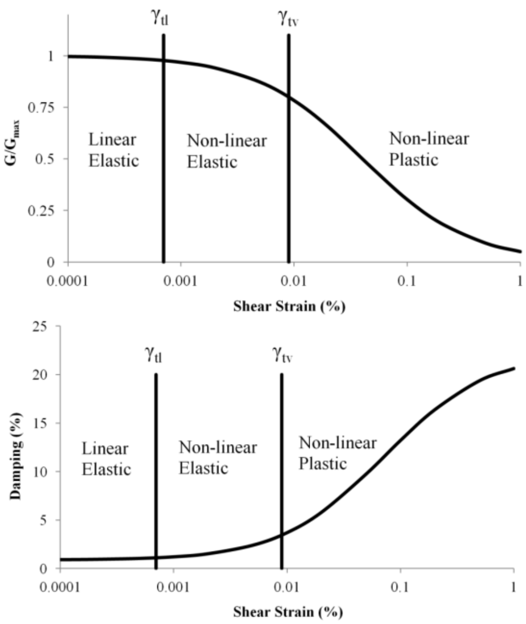

Fig. 5. Shear modulus reduction (above) and damping (below) curves, showing the linear threshold (γ_{tl}) and the volumetric threshold (γ_{tv}) shear strains.

At shear strains greater than γ_{tv} soils exhibit nonlinear elastoplastic behaviour with volumetric variation and pore pressure generation observed.

For cohesive soils, plasticity index (PI) is the parameter with more influence on the definition of the modulus reduction curve. As PI increases the G/G_{max} curves shift to the right, and the volumetric cyclic threshold shear strain (γ_{tv}) increases (Darendeli) [8]. In the current research, to estimate the shear modulus and the damping ratio empirical correlations, proposed by Darendeli [8], was used.

4 Site response analysis

4.1. Equivalent Linear Method

The most common site response analysis method is the equivalent linear method for their robustness, simplicity, flexibility, and low computational requirements. It has the advantage of applying the principle of overlapping linear solutions, which makes possible the analysis in the frequency domain. In addition, the input parameters for equivalent linear programs such as SHAKE2000 are physical parameters that are readily understood, like shear wave velocity, specific weight, shear modulus reduction and damping curves for the different types of soil in the profile. These parameters can be obtained from field or laboratory tests, or by correlations with other geotechnical parameters.

This linear model takes an acceleration time series in the time domain and convert it to the frequency domain using a Fast Fourier Transform (FFT). The FFT determines the amplitude of harmonic waves at many different frequencies whose summation would be the acceleration time series. The Fourier series is then multiplied by a transfer function that determines how each frequency in the input motion is either amplified or deamplified to produce the Fourier series of the output motion. The Fourier series of the output motion is then transformed back to the time domain using the inverse FFT. Transfer functions are solutions to the wave equation of a vertically propagating horizontal shear wave. They are dependent on frequency and the stiffness, damping, and density properties of the soil profile, Kramer [6].

In the Equivalent linear method, the seismic response is calculated by an iterative process in which the shear modulus and damping are updated in each step for the corresponding value of effective shear strain, generally taken as 65% of the maximum shear strain obtained from the calculated strain history. The iterations are performed until the effective shear strain is compatible with the values of shear modulus and damping adopted in the current calculation step within a pre-stablished tolerance.

Note that for linear methods, the shear modulus and damping, remain constant throughout the analysis. In the actual behaviour of soils, dynamic parameters are nonlinear and change constantly depending on the shear strain level for each point of the profile along the duration of the ground motion. This simplification of the actual non-linear behaviour is a major drawback of the linear method and can lead to results that are not seen in empirical data, because of the following reasons:

- When the peak shear strain is much greater than the shear strain at other time intervals, it may result in an underestimation of stiffness and an overestimation of the damping.
- When the shear strain is approximately uniform over time it may result in an overestimation of stiffness an underestimation of damping.
- As stiffness and damping do not change over time, high amplification levels can be predicted near the natural frequency of the soil deposit. These large resonances are not seen in empirical observations, because the stiffness and damping in real deposits change during the motion.
- The equivalent linear method is formulated in terms of total stress, therefore it is not possible to predict the pore pressure generation, which may eventually reduce the stiffness and strength of the soil layers, resulting in failure for cohesive soils or liquefaction for granular soils.
- The method is not adequate to predict the response of the soil at large strains because of the highly nonlinear behaviour involved, with significant changes in stiffness and damping during the motion.

4.2 Properties of the soil profile

The KIK-NET soil deposit consists in 14 meters of high plasticity clay, which is the layer that made this deposit be classified as a NEHRP F site, followed by a highly over consolidated crust, 14 meters of moderate plasticity sand, 10 meters of low plasticity sand and 55 meters of silt on a rocky base to 103 meters of depth. The geotechnical profile of the TKCH07 station (geographic coordinates 42° 48’ 41’’ N, 143° 31’ 13’’ E) is available on the Japanese National Research Institute for Earth

Science and Disaster Prevention, NIED [4], including S wave propagation velocity data, layer thickness and type of soil.

Table 1. Properties of the soil profile in the TKCH07 station.

Depth (m)	USCS	Thickness (m)	γ (kN/m^3)	Vs (m/s)	OCR	PI	τ_{ff} (atm)	G_{max} (atm)
0.7	CH	0.7	15.5	80	5	80	0.030	99.9
1.4	CH	0.7	15.5	80	4	80	0.049	99.9
2.1	CH	0.7	15.5	80	3.5	80	0.074	99.9
3.1	CH	1	16.5	110	3	80	0.100	201.1
4.2	CH	1.1	16.5	110	2	80	0.106	201.1
5.3	CH	1.1	16.5	110	1.5	80	0.112	201.1
6.4	CH	1.1	16.5	110	1.3	80	0.124	201.1
7.5	CH	1.1	16.5	110	1.15	80	0.135	201.1
8.6	CH	1.1	16.5	110	1.15	80	0.157	201.1
9.7	CH	1.1	16.5	110	1.15	80	0.180	201.1
10.8	CH	1.1	16.5	110	1.15	80	0.202	201.1
11.9	CH	1.1	16.5	110	1.15	80	0.225	201.1
13	CH	1.1	16.5	110	1.15	80	0.247	201.1
14.1	CH	1.1	16.5	110	1.15	80	0.269	201.1
16	SM	1.9	17	200	2	60	0.469	685
18	SM	2	17	200	1.5	60	0.425	685
20	SM	2	17	200	1.3	60	0.428	685
22	SM	2	17	200	1.3	60	0.476	685
24	SM	2	17	200	1.3	60	0.524	685
26	SM	2	17	200	1.3	60	0.572	685
28	SM	2	17	200	1.3	60	0.620	685
30	SM	2	17	200	1.3	60	0.669	685
32	SM	2	17	200	1.3	60	0.717	685
34	SM	2	17	200	1.3	60	0.765	685
36	SM	2	17	200	1.3	60	0.813	685
38	SM	2	17	200	1.3	60	0.862	685
41	SW-SM	3	19	350	1.5	35	1.254	2345
44.5	SW-SM	3.5	19	350	1.3	35	1.238	2345
48	SW-SM	3.5	19	350	1.3	35	1.368	2345
53	ML/MH	5	18	500	3.5	100	3.345	4533
58	ML/MH	5	18	505	3	100	3.279	4624
63	ML/MH	5	18	510	2.75	100	3.358	4716
68	ML/MH	5	18	515	2.5	100	3.390	4809
73	ML/MH	5	18	520	2.5	100	3.668	4903
78	ML/MH	5	18	525	2.5	100	3.946	4998
83	ML/MH	5	18	530	2.5	100	4.224	5093
88	ML/MH	5	18	530	2.5	100	4.502	5093
93	ML/MH	5	18	530	2.5	100	4.780	5093
98	ML/MH	5	18	530	2.5	100	5.058	5093
103	ML/MH	5	18	530	2.5	100	5.336	5093

4.3 Darendeli Model (2001) [8]

Darendeli [8] presented shear modulus reduction and damping curves based on experimental results of 110 samples of soil, from 20 sites, using the bayesian method. An equipment combining a resonant column and torsional shear was used to measure the dynamic properties of the soil for low and large shear strains. These curves, incorporated in the SHAKE2000, are available for a range of plasticity index values and effective confining stress expressed in units of atmosphere.

The soil profile studied was subdivided into 40 layers whose properties are listed in Table 1. In order to perform the analysis in SHAKE2000 the layers were grouped into three major layers, according to the average values of plasticity index and effective confining stress, to model the dynamic properties by representative shear modulus reduction and damping curves.

The bedrock at the base of the soil profile was characterized by shear modulus reduction and damping curves developed by Schnabel [1] for rock.

Figure 6 and 7 show the modulus reduction and damping curves of the Darandeli Model [8] used for the site response analysis, indicating of the values of the average effective confining stresses and plasticity index considered.

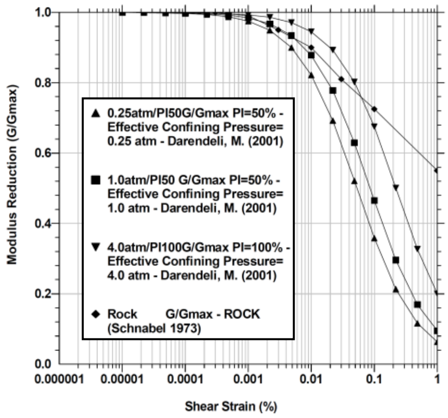

Fig. 6. Modulus reduction curves for soil (Darendeli [8]) and rock (Schnabel [1]).

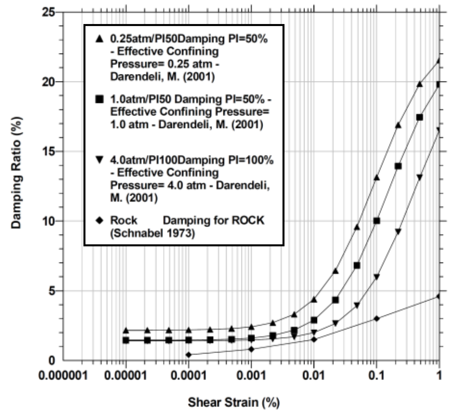

Fig. 7. Damping curves for soil (Darendeli [8]) and rock (Schnabel [1]).

5 Simplified model of Carlton (2014) [3]

The simplified method of Carlton [3] to estimate the response spectrum for deposits of soft soil type NEHRP E and F was based on the results of nonlinear analysis for a combination of 15 sites and 12 ground motion scenarios.

The method was developed in two phases, the first evaluating the effects of the different ground motion scenarios and the second determining the influence of the site. Through regression procedures the results of both phases were combined to estimate the coefficients of the final model, expressed by equations 1, 2 and 3:

$$\ln\left(Amp\left(T\right)\right) = f_1\left(T\right) + f_2\left(T\right) \times \ln\left(\frac{Sa\left(T\right)_{Rock} + 0.1}{0.1}\right) \quad (1)$$

$$f_1(T) = c_1(T) + c_2(T) \times \ln(Th) + c_3(T) \times \ln(Vs_{mean}) + c_4(T) \times \ln(\gamma_{0.5,mean}) \quad (2)$$

$$f_2(T) = c_5(T) + c_6(T) \times \ln(CRR_{\min}) \quad (3)$$

Where *Amp(T)* is the amplification defined as the ratio of the superficial spectral acceleration in *T* period divided by the expected spectral acceleration on the rocky base for the same *T* period. $Sa(T)_{rock}$ is the spectral acceleration in rock for the *T* period. *Th* is the total thickness of the soft layers (class NEHRP E and F), in meters. Vs_{mean} is the average velocity of shear wave on the soft layers, in m/s. $\gamma_{0.5,mean}$ is the shear effective strain for $G/G_{max}=0.5$ in the layers of soft soil, in percentage. CRR_{min} is the minimum value of cyclic resistance ratio of the soft layers. c_1 to c_6 are period-dependent coefficients.

6 Results

Figure 8 shows the acceleration history of the recorded motion and the calculated by the program SHAKE2000 on the surface of the soil deposit. It can be seen that the acceleration history calculated with SHAKE2000 presents the same shape compared the measured motion, however the predicted response contains more peaks than the real response, indicating a higher intensity on the predicted ground motion. The maximum acceleration of the real motion was 0.04 g while the maximum acceleration computed with SHAKE2000 was 0.05 g, which is an acceptable approximation for this parameter.

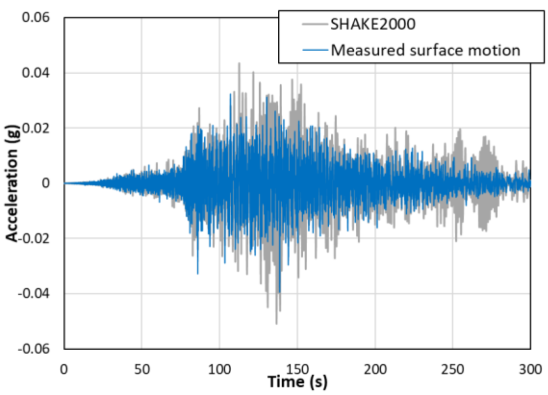

Fig. 8. Real and Predicted acceleration record on surface.

Figures 9, 10 and 11, show the amplification, Fourier spectrum and acceleration response spectrum, respectively. Each figure compares the response obtained from recorded ground motion with those predicted by the equivalent linear analysis and the simplified method of Carlton [3]. In general, the response calculated with SHAKE2000 are satisfactory in relation to the real response, however amplification peaks due resonance effects are observed near the natural period of the soil deposit (1.1 seconds).

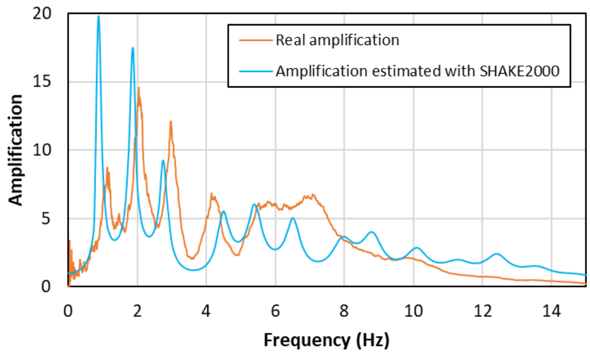

Fig. 9. Recorded and predicted amplification between motions at bedrock and surface of the soil profile.

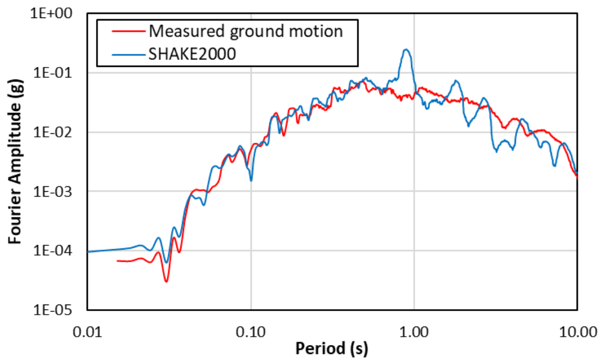

Fig. 10. Fourier spectrum for recorded and predicted ground motions at the surface of the soil profile.

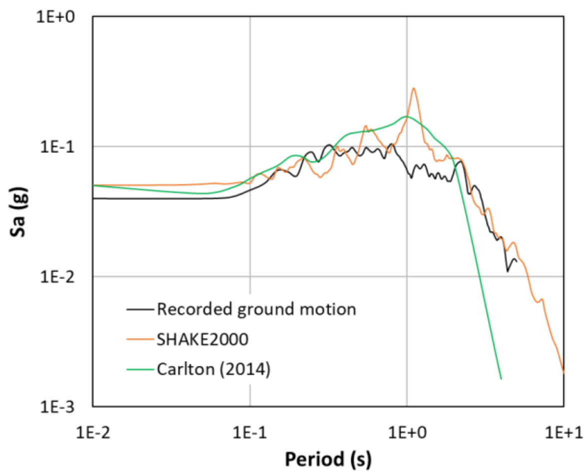

Fig. 11. Acceleration response spectra from recorded and predicted ground motions.

Figure 11 compares the response spectrum of acceleration. For the model of Carlton [3], the spectral acceleration in rock, $Sa(T)_{rock}$, was determinate based on the prediction equation of Atkinson and Boore [5]. The predicted spectra show similar tendency and magnitude to that obtained for the recorded motion.

Finally, Figures 12 and 13 show the predicted profiles of maximum horizontal acceleration and maximum shear strain, respectively. It shows that the maximum horizontal acceleration was amplified from a value of 0.01 g to 0.05g at the ground surface. The maximum shear strain profile show peaks of 0.07% near the surface, in the softer layer, which is considerable high in relation the intensity of the motion, which is relatively low, exceeding the linear elastic threshold shear strain γ_{tl}.

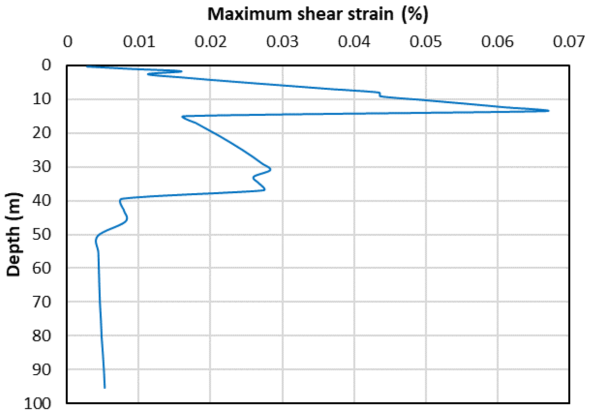

Fig. 12. Maximum horizontal acceleration profile.

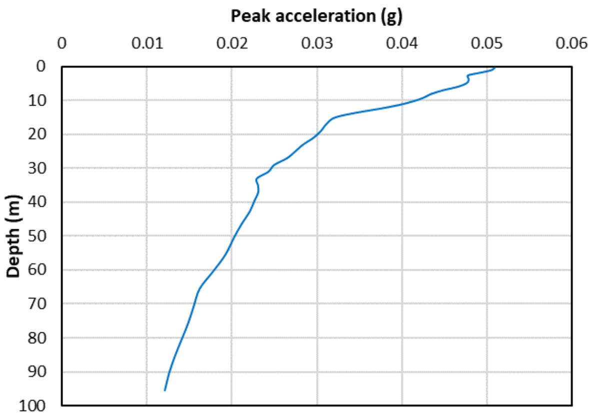

Fig. 13. Maximum shear strain profile**.**

7 Conclusions

This study verified an important limitation of the linear equivalent method in the site response analysis for soft soil deposits, which derives of the fact that the stiffness and damping used to calculate the response are maintained constant along the duration of the motion; this results in the prediction of peaks of amplification near the natural frequency of the soft soil deposit. This high level of resonance is not observed in empirical data because stiffness and damping for actual soils change depending on the level of strain during the motion.

The time history acceleration at the ground surface calculated with SHAKE2000 showed a fair approximation to the real motion, however the predicted motion showed a few more peaks of maximum acceleration than de real motion. Comparing the maximum acceleration predicted (0.05g), with the actual maximum ground acceleration (0.04g) is considered an acceptable estimation.

The acceleration response spectrum calculated using the simplified method of Carlton [3] showed a reasonable approximation to the real response spectrum with a tendency to overestimate the response which can be useful to make conservative preliminary prediction of ground surface motion.

A relevant characteristic observed in the response of the soft soil deposit (NEHRP F site) studied is that a relative low intensity motion (0.01 g) is enough to produce relatively large shear strains due to the high compressibility of this type of soils.

The high levels of shear strain and damping, also observed in this study, are the main characteristic of the seismic response of soft soil deposits, classified for seismic design of structures as a NEHRP E and F sites. This behaviour is comparable to that of stiffer soils, which exhibit relatively lower levels of shear strain and damping.

Finally, considering the large shear strain that strong ground motions cause on surface of soft soil sites, it is recommended to analyse the response of the soil in terms of displacements, rather than accelerations, to understand better the real effect of this parameter on the seismic response of structures.

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