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Seismic response of a soft soil deposit using non-linear and simplified models

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ABSTRACT: This study aims to improve the understanding of seismic behavior of soft soil deposits not susceptible to liquefaction, classified as NEHRP E and F sites, for which few empirical data is available. Non-linear analyses, considering the large strains expected, were performed to estimate the maximum values of horizontal acceleration and displacement for a soft soil deposit monitored by the seismic station TKCH07 of the KiK-net network of Japan. The main analysis used a recorded strong motion with maximum horizontal acceleration of 0.3 g and epicentral distance of 100 km from the 2011 Tohoku Earthquake. An additional comparison was made by applying the simplified model proposed by Carlton (2014). The estimations showed a de-amplification trend of acceleration, typically observed in soft soil deposits subjected to strong motions, on the other hand, amplification of displacement is predicted, suggesting this is a more representative parameter of the motion for this cases.

1 INTRODUCTION

Near surface soils can greatly influence the amplitude, duration, and frequency content of ground motions. Survey 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. Ordóñez (2012) believes that large ground displacement and consequent soil-structure interaction produces not-well understood mechanisms of failure and eventually collapse of structures usually observed when soft soil deposits are subjected to strong motions. In this sense, a more detailed analysis of the displacement is presented on this research.

The objective of this study is to improve the understanding of the seismic behavior of soft soils deposits not susceptible to liquefaction. For seismic design of structures, building codes as the International Building Code (IBC, 2012) 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 deposits of soft clays with high plasticity (lP>20), 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 on the seismic behavior 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. For the first part of this study, a validation analysis is made to be compared with a recorded motion from a vertical array in a soft soil deposit in the 2011 Tohoku earthquake (Mw=9). Equivalent linear and nonlinear analyses are performed for this stage using the programs SHAKE2000 (Ordóñez, 2012) and D-MOD2000 (Matasovic and Ordóñez, 2012) respectively. Furthermore, a non-linear simulation with a strong motion (PGA 0.3 g), was performed to estimate the behavior of the soil deposit at large strains and high levels of damping.

An additional comparison is made with the simplified method of Carlton (2014) developed specifically for NEHRP F sites, based in the one dimensional response analysis of seven soft soil deposits situated in the U.S., Canada, Ecuador and Japan.

The soft soil model for this study is based on the soft soil profile, classified as a NEHRP F site, of the station TKCH07 located in Hokkaido, Japan, which is part of the strong motion seismograph network KiK-net. KiK-net stations are vertical arrays 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).

2 THE SOIL PROFILE

The KiK-net TKCH07 soil deposit, classified as NEHRP F site, consists of the following sequential layers: a 14m high plasticity clay (PI=80), 24m of moderate plasticity silty sand, 10m of well graded silty sand and 55m of silt over the rock embedment situated 103m below surface. The geotechnical profile of the TKCH07 station (42°48’41”N, 143°31’13”E), shown in Figure 1, is available through the Japanese National Research Institute for Earth Science

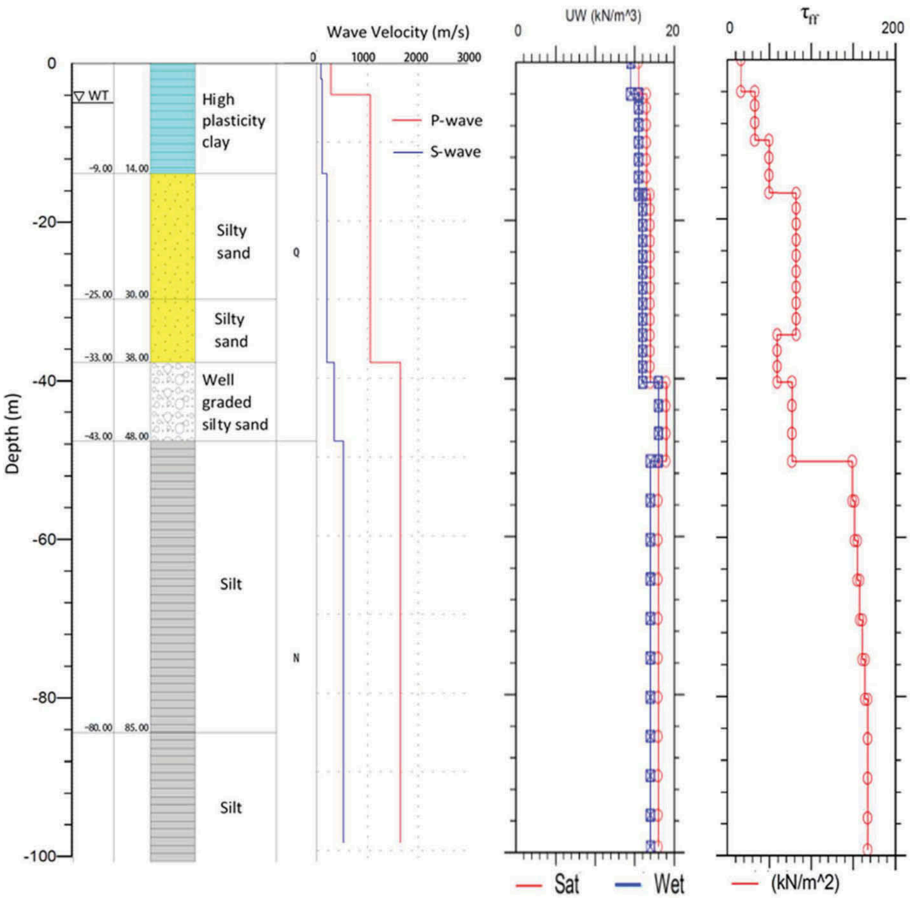

Figure 1. Geotechnical log at TKCH07 site including the distributions of unit weight (UW) and undrained shear strength (τ_{ff}).

and Disaster Prevention (NIED), including values of P and S wave propagation velocities, layer thickness and type of soil.

3 HYBRID SHEAR MODULUS REDUCTION AND DAMPING CURVES

Figure 2a shows the backbone curve constructed with the extreme values of the hysteretic loop for different values of cyclic shear strain. This curve approaches asymptotically the shear strength of the soil τ_{ff} for large strains.

Figure 2b depicts typical curves for shear modulus reduction and increasing damping with cyclic shear strain. Vucetic (1994) divided them into three regions separated by two characteristic levels of cyclic strain: the linear threshold shear strain (γ_{tl}) and the volumetric threshold shear strain (γ_{tv}). For shear strains smaller than γ_{tl} the soil exhibits linear elastic behavior with a constant value of shear modulus (G_{max}) and damping (D_{min}), while for shear strains between γ_{tl} and γ_{tv}, a region of non-linear elastic behavior, the shear modulus degrades and the damping ratio increases, but the amount of plastic deformation and pore pressure generation is still negligible or, in other words, the deformation may be recovered after unloading. For shear strains greater than γ_{tv}, it is expected that the soil will exhibit non-linear plastic behavior with volume change and pore pressure generation. For high plasticity soft clays, like the top layer of the profile analysed, the volumetric threshold shear strain γ_{tv} is around 0.1%, which is a relative large value compared to that of sands which are less than 0.02%.

For cohesive soils, the plasticity index (PI) is the parameter that most influences the shear modulus degradation. As it increases, the modulus reduction curve shifts to the right, consequently increasing the volumetric threshold shear strain (Darendeli, 2001).

Most of the proposed experimental models available in the literature for shear modulus reduction and damping curves are based on cyclic shear tests considering shear strains smaller than 0.5%. When they are extrapolated for larger values then the shear strength will not necessarily be matched (Stewart et al. 2008). To overcome the inconsistency in such models, the curves may be modified to account for the shear strength at large shear strains using the procedure given by Yee (2013). In the present research, the empirical curves proposed by Darendeli (2001) where used to represent the dynamic behavior of soil for small and intermediate cyclic shear strains. For values greater than 0.15, the Darendeli's curves were adapted by applying the method suggested by Yee (2013), thus generating the called hybrid curves to capture the shear strength at large strains (Figure 3a).

To numerically simulate the site response, each layer of the soil profile (Figure 1) was assigned with specific sets of shear modulus reduction (Figure 3b) and damping curves dependent on the plasticity index and the confining stress at the different depths.

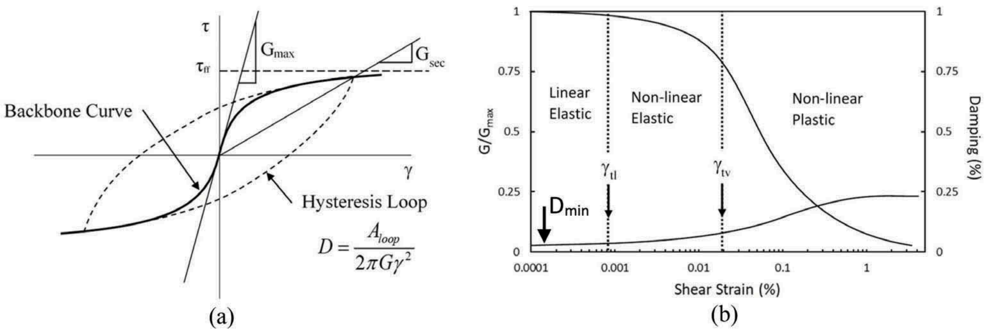

Figure 2. Typical (a) Hysteresis loop and backbone curve, and (b) shear modulus reduction and damping curves.

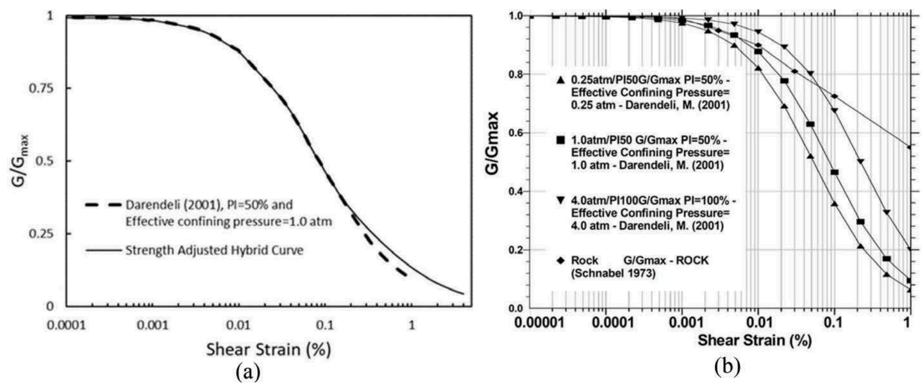

Figure 3. (a) hybrid shear modulus reduction curve for large strains and (b) Shear modulus reduction curves for small and intermediate strains.

4 CARLTON (2014) SIMPLIFIED MODEL

The simplified model proposed by Carlton (2014) to estimate the response spectrum for NEHRP F sites was based on the results of nonlinear analyses considering a combination of 15 sites and 12 scenarios of motion. Seven of the sites are based on actual sites situated in the U.S. (San Francisco Bay Area, New York City), Canada, Ecuador and Japan. The model was developed in two phases: the first one evaluated the effects of the different scenarios in the soil motion and the second phase determined the influence of the soil properties. Through regression analyses the results of both phases were combined to estimate the dynamic soil response through the following correlations:

$$\ln(Amp(T)) = f_1(T) + f_2(T) \times \ln\left(\left(Sa(T)_{Rock} + 0.1\right) \times 10\right) \tag{1}$$

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

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

where *Amp(T)* is the amplification defined as the ratio of the surface spectral acceleration at period T divided by the spectral acceleration that would be expected on a rock site at the same period, $Sa(T)_{rock}$ is the rock spectral acceleration, *Th* is the total thickness (m) of the soft layers (NEHRP F soil), $Vs_{,mean}$ is the mean shear wave velocity (m/s), $\gamma_{0.5,mean}$ is the mean shear strain when $G/G_{max} = 0.5$, CRR_{min} is the minimum value of the cyclic resistance ratio of the soft soil layers and c_1 to c_6 are period dependent coefficients.

5 RESULTS OF SITE RESPONSE ANALYSIS

5.1 *Validation analysis*

Figure 4 shows the horizontal acceleration spectra and the corresponding acceleration history of the measured and simulated ground responses at the site of station TKCH07. In general, the predicted responses have a similar trend and magnitude to the recorded motion. However, amplification peaks (Figure 4a) are observed for components of the calculated motion near the fundamental period of the soil profile. This mismatch of the frequency content is reflected in the acceleration history in the north-south (NS) direction, where slightly higher peaks are predicted during the shaking, being somewhat more notorious towards the end of the motion. Similar results were found for the acceleration history in the east-west (EW) direction.

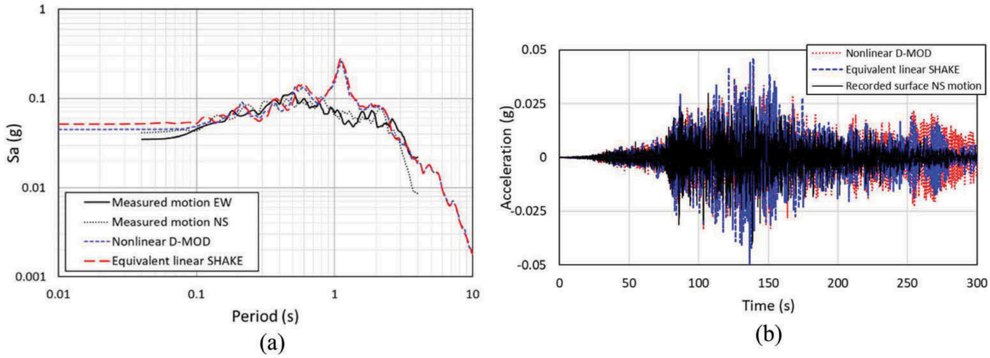

Figure 4. Response spectra (a) and acceleration history (b) of the measured and simulated ground motion at station TKCH07.

5.2 *Strong motion nonlinear analysis*

This section presents the results of a numerical analysis carried out with a strong motion (PGA=0.3 g) recorded in bedrock at a distance of 100 km from the earthquake epicenter.

Figure 5a shows the de-amplification of the maximum horizontal acceleration, decreasing from 0.3g (bedrock) to 0.16g (ground surface), but with a sudden increment in the shallow high plasticity clay layer. The maximum shear strain profile (Figure 5b) also indicates that large strains (2%) occur in this upper layer, implying a non-linear mechanical behaviour and high levels of hysteretic damping that were incorporated in the hybrid damping model previously described.

It was also determined the ground displacement motion (Figure 6) where it can be observed, at first glance, a maximum displacement amplitude of 15cm. However, considering the amplitude of the movement between extreme points for a given cycle, the actual absolute displacements reach values between 12 and 24 cm. These cycles of large displacement occur about 8 times with periods of 1 to 2 seconds, suggesting that displacement parameters might be as representative as acceleration parameters for strong motions in soft soil sites. Furthermore, the results also show that the displacements are amplified whereas accelerations are deamplified in this particular analysis.

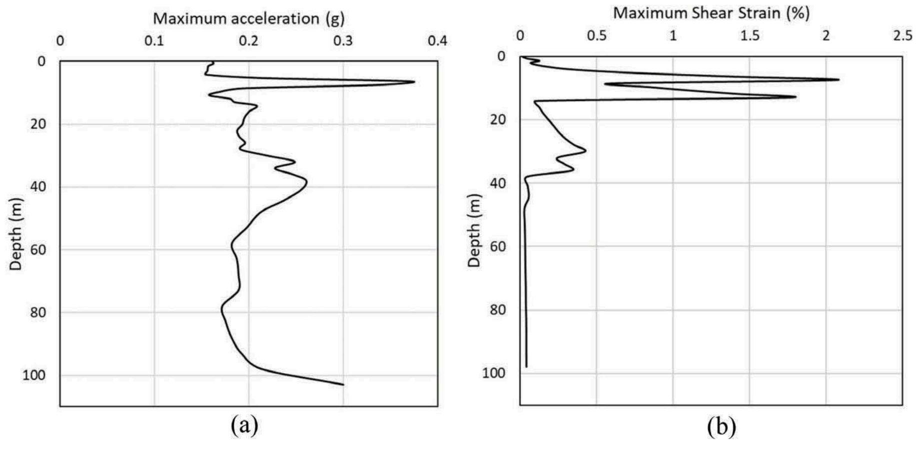

Figure 5. Profiles of maximum acceleration (a) and shear strain (b).

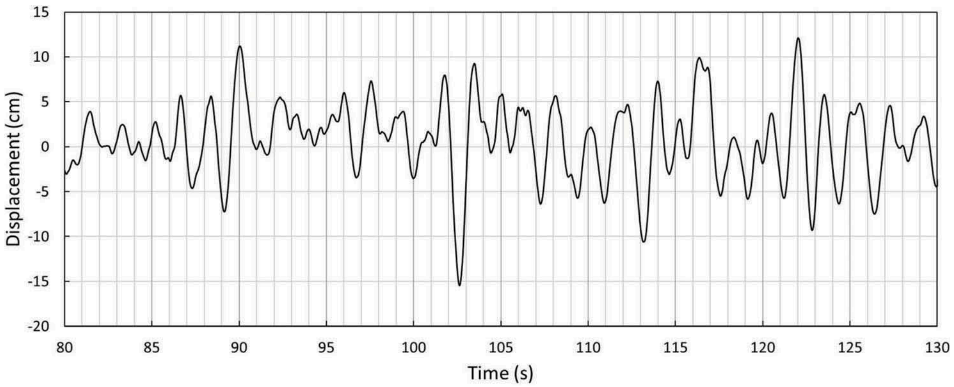

Figure 6. History of the ground horizontal displacements during the most intense 50 seconds of the motion.

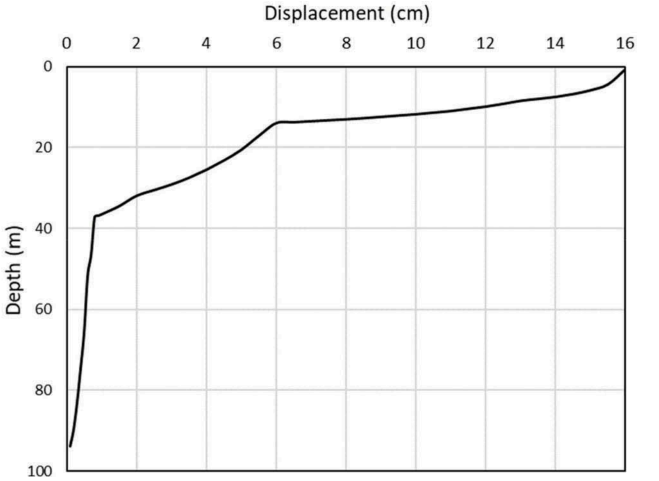

Figure 7. Profile of maximum horizontal displacement amplitudes at time t = 102.5 s.

Figure 7 shows the profile of horizontal displacement amplitudes at time t = 102.5 seconds when the maximum displacement at the surface occurs. It can be seen that the displacements become large from the surface until depth of 36m, due to low stiffness of the shallow strata (shear wave velocity of 90m/s in the clay layer and 191 m/s in the silty sand layer).

Figure 8 compares the amplification of the spectral acceleration considering the measured and predicted values, determined as the ratio between the 5% damped spectral amplitudes of soil and rock (Sasoil/Sarock) for each period. The depicted amplifications show a pronounced difference between the strong (PGA=0.3 g) and the weak (PGA=0.01g) motions for frequencies below 1 Hz. These results may explain the large displacements exhibited by soft soils subjected to strong motions, since displacements are controlled by the low frequency components of motion which, in this case, were amplified.

To study the variation of the peak horizontal acceleration at the surface of the soft soil deposit with respect to the maximum acceleration on the rock outcrop, also called the "bend-over" curves, a non-linear analysis was carried out for different values of maximum acceleration on the rock outcrop. The computed results were then compared with the curves proposed by Seed et al. (1997) and the simplified model developed by Carlton (2014).

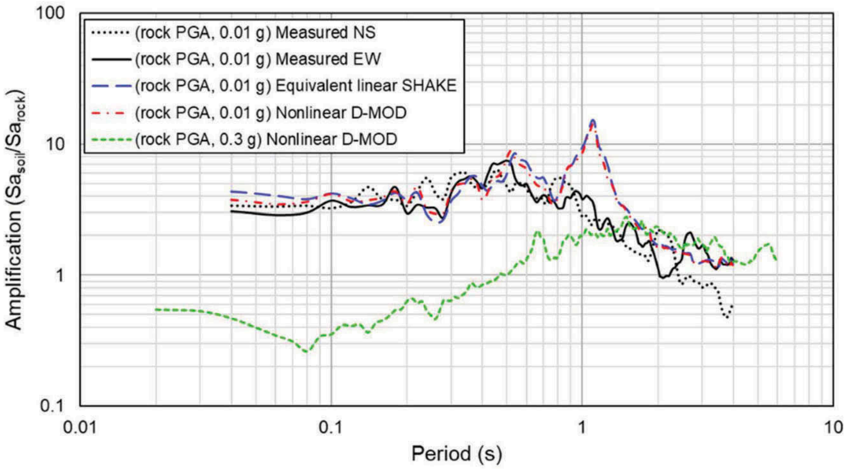

Figure 8. Amplification of spectral acceleration considering the measured motions and predicted results.

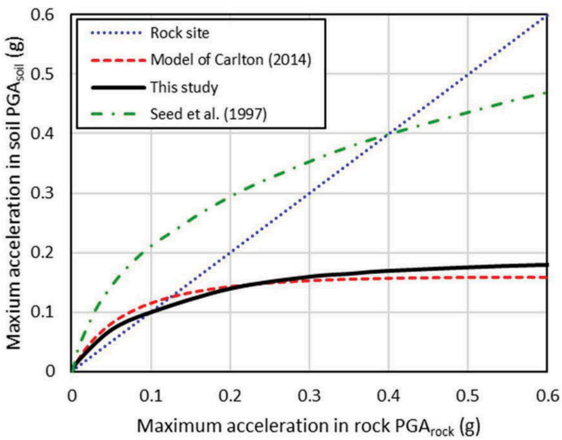

Figure 9. Variations of peak horizontal acceleration at soft soil sites with acceleration at rock outcrop.

The curve computed in this study (Figure 9) lies below the curve given by Seed et al. (1997), presenting a maximum value of PGAsoil that remains constant above a certain threshold of the PGA_{rock}. Carton (2014) found that the maximum PGA_{soil} for a given PGA_{rock} increases as the shear strength and volumetric threshold shear strain of the soil increase. Figure 9 shows that amplification occurs only for rock acceleration less than 0.1g, whereas for higher values the peak horizontal acceleration at the soft soil surface is de-amplified, reaching a maximum of 0.18 g. Both curves (Carlton simplified model and non-linear analysis) have shown a satisfactory agreement in this study.

6 CONCLUSIONS

As reported by Seed et al. (1997) and from events such as the 2011 Tohoku earthquake (Midorikawa, 2014), strong motions propagated through soft soil deposits, classified as NEHRP E and F, present in most cases a characteristic deamplification of the horizontal acceleration. In this study considering a NEHRP F soil layers, the predicted responses presented high levels of shear strains induced by the strong motion, resulting in greater soil damping and attenuation of accelerations.

Through non-linear analysis, the variation of the peak ground acceleration at the surface as a function of the horizontal acceleration in the bedrock, resulted in a "bend-over" curve that lies below a similar curve suggested by Seed et al. (1997) but it is quite similar to the curve calculated by applying the simplified model proposed by Carlton (2014).

For the strong motion analysis, the displacement history at the ground surface showed many cycles of large displacements, with periods of 1 to 2 seconds. The spectral amplification of the motion may explain the occurrence of these large values, given that displacements are controlled by the low frequency components which, in this case, are amplified. This behavior suggests that displacement parameters can be as representative as acceleration parameters for strong motions in soft soil sites.

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REFERENCES

Carlton B. 2014. An improved description of the seismic response of sites with high plasticity soils, organic clays, and deep soft soil deposits. A doctor of philosophy dissertation, University of California, Berkeley, 845 p.

Darendeli M. B. 2001. Development of a new family of normalized modulus reduction and material damping curves. Dissertation for the Ph.D. degree, The University of Texas at Austin, 362 p.

Matasovic, N. and Ordóñez, G.A. 2012. "D-MOD2000 – A Computer Program for Seismic Site Response Analysis of Horizontally Layered Soil Deposits, Earthfill Dams, and Solid Waste Landfills." GeoMotions, LLC; Lacey, Washington, USA, 196p.

IBC 2012. International Building Code, by International Code Council, Delmar Publishers.

Midorikawa, S.; Miura, E.H.; Atsumi, T., 2014.Characteristics of strong ground motion from the 2011 gigantic Tohoku, Japan earthquake, International Symposium for CISMID 25th Aniversary.

NIED. Base de dados online do Japanese National Research Institute for Earth Science and Disaster Prevention, http://www.k-net.bosai.go.jp/. Accessed in Janunary 2019.

Ordóñez, G.A. 2012. "SHAKE2000 – A Computer Program for the 1-D Analysis of Geotechnical Earthquake Engineering Problems." Geomotions L.L.C., Lacey, Washington, USA.

Ordóñez A. 2012. Aportes del Diseño Sísmico Basado en el Comportamiento (DSBC) a las Normas de Suelos y Cimentaciones y el Diseño Sismorresistente de Edificaciones. VI Congreso Internacional de la Construcción. ICG, Lima.

Schnabel, P.B. 1973. Effects of Local Geology and Distance from Source on Earthquake Ground Motions. Ph.D. Thesis, University of California, Berkeley, California.

Seed R.B., Chang S.W., Dickenson S.E., Bray J.D. (1997). "Site-dependent Seismic Response Including Recent Strong Motion Data," in Seismic Behaviour of Ground and Geotechnical Structures, Seco e Pinto, P. S., ed., Proc. of Spec. Session on Earthquake Geotechnical Engineering, XIV International Conf. On Soil Mechanics and Foundation Engineering, Hamburg, Germany, A.A. Balkema, pp. 125–134, Sept. 6-12.

Stewart, J.P., Kwok, A.O., Hashash, Y.M.A., Matasovic, N., Pyke, R., Wang, Z., Yang, Z. 2008. Benchmarking of Nonlinear Geotechnical Ground Response Analysis Procedures. Pacific Earthquake Engineering Research Center Report 2008/04.

Vucetic, M. 1994. Cyclic Threshold Shear Strains in Soils. Journal of Geotechnical Engineering, ASCE, 120(12),2208–2228.

Yee, E., Stewart J.P., Tokimatsu K. 2013. Elastic and Large-Strain Nonlinear Seismic Site Response from Analysis of Vertical Array Recordings. J. Geotech. Geoenviron. Eng. 139(10).