Regional subsidence effects on seismic soil-structure interaction in soft clay

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ABSTRACT
Regional subsidence effects on dynamic soil properties and layering configuration, when dealing with seismic soil-structure interaction analyses, are often ignored in practice. These, however, can substantially change the frequency content and spectral accelerations in both free field and in the soil-structure system. Pore pressure variations over the project economic life is due to both regional subsidence as well as dissipation of excess pore pressure caused by the structure weight. These variations lead to changes in effective stresses, which in turn, modify the dynamic properties such as shear wave velocity distribution and modulus degradation and damping curves, as well as soil layer thickness and configuration. These changes can be substantial in highly compressible very soft cay, such as that found in Mexico City valley. This paper presents a numerical study on the seismic response of a conventional five-story building supported by a compensated box foundation built in soft clay, considering these effects. Three dimensional finite difference models were developed with the software FLAC^{3D}. Initially, the evolution of effective stresses with pore pressure was established based on in-situ piezometer measurements of an instrumented site, and laboratory data. Then, changes in dynamic properties were taking into account based on series of resonant column tests conducted for several effective confining stresses and suspension logging tests results. The static behavior of the soil-structure system was assessed. The free field model response was calibrated comparing the fully no-linear analyses results with equivalent linear analyses, considering an extreme subduction event associated to a 2475 years return period. Finally, the seismic performance of the soil-structure system was studied to evaluate the impact of the changes in dynamic properties on the seismic response. Insight, was gained regarding the complexity of the interplay of the effective stress history, and static and seismic soil-structure performance during an extreme earthquake.

1 INTRODUCTION

Seismic performance evaluation of soil-structure systems built on very soft high plasticity clays is a complex problem, especially when expected changes in effective stresses during the economic life of the structure due to dissipation of excess pore pressure caused by the building gravity loads and regional subsidence are very large. These changes in effective stresses lead, in turn, to large settlements. This is particularly important in urban areas located in highly compressible clays, such as Mexico City, where the settlement rate of regional subsidence reaches about 10 cm/ year in average, but can go as large as 35 cm/year is some areas. Thus, it is common to have ground settlements about 40 to 90 cm, due to load consolidation, and around several meters due to regional subsidence (Merlos and Romo 2006). These settlements produce changes in both soil profile configuration (i.e. layer thickness and geometry), as well as dynamic properties, such as shear wave velocity distribution with depth and modulus degradation and damping curves. Thus, effecting the seismic response of the soil-structure system. In particular, the effect of dynamic properties changes in the seismic response of sites located in soft clay, has been marginally studied by other researches (Ovando-Shelley et al. 2007), finding that the evolution of shear wave velocities and modulus degradation and damping curves can modify significantly the computed response. Nevertheless, the impact of these variations in the seismic performance of soil-structure systems has not been addressed, neither the effect of changes in the soil profile configuration after several meters of sinking. These effects however, can drastically modify both free field, near field and structural response over time. This paper presents a numerical study of the seismic response of a conventional five-story building supported by a partially compensated box foundation built in highly compressible soft clay, considering these effects. Three dimensional finite difference models were developed with the software FLAC^{3D}. Initially, the evolution of effective stresses with the pore pressure was established based on in-situ piezometer measurements, and laboratory data. Then, variations in dynamic properties were taking into account based on series of resonant column tests conducted for several effective confining stresses and suspension logging tests. The static behavior of the soil-structure system was assessed. The free field model response was calibrated comparing the fully no-linear analyses results with equivalent linear analyses carried out with the program SHAKE (Schnabel et al. 1972). Finally, the seismic performance of the soil-structure system was studied to evaluate the impact of the changes in dynamic properties in the seismic response, considering an extreme subduction event associated to 2475 years return period. Insight was gained regarding the complexity of the interplay of the effective stress history, and static and seismic soil-structure performance during an extreme earthquake.

2 CASE STUDY

A conventional five story building supported by a compensated box foundation located on the Texcoco Lake area, was considered in the numerical study.

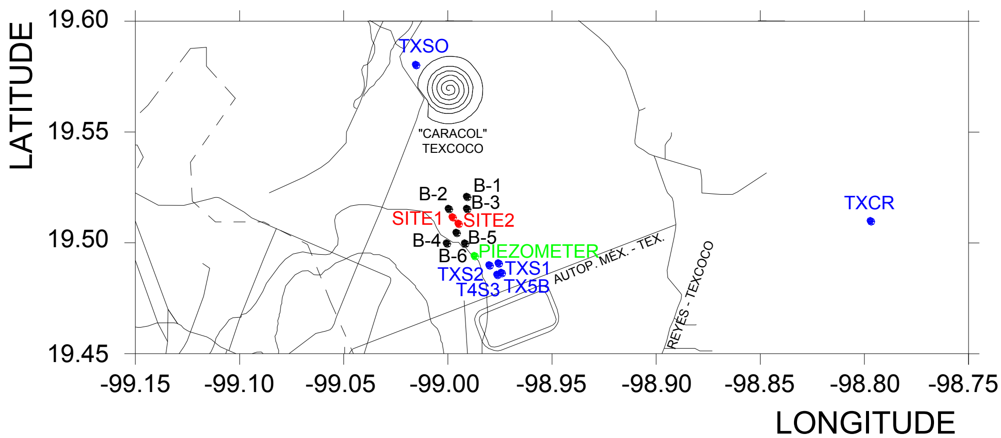


Figure 1. Seismological stations locations

Due to the particular characteristics of Mexico City clay having high plasticity index, no significant reduction in shear modulus is observed even for shear strains as high as 0.1%. Similarly, there is no significant increase in the damping ratio until angular distortions of the order of 0.3% are reached. Thus, the response of clayey soil deposits is nearly elastic even for shear strains as high as 0.3%, which leads to a high potential of amplification of the seismic waves. Indeed, amplification factors up to 5 (between peak ground acceleration, PGA, observed at soft soil with respect to those of rock outcrops) were observed during the 1985, Michoacan earthquake.

2.1 Seismic Instrumentation

The study area is instrumented with six seismic stations located in soft clay TXSO, TXS1, TXS2, TXCH, TX5B and T4S3. The corresponding UTM coordinates of the stations are summarized in Table 1. The nearest station that recorded the devastating 1985 Michoacan earthquake, is TXSO, located approximately about 2 km away from the studied site, as shown in Figure 1. In addition, there is a rock outcrop station, identified as TXCR, which is around 20 km northeast.

Table 1. UTM coordinates of the seismological stations

Stations	UTM coordinates East	North
TXSO	498007 m	2165005 m
TXS1	502827 m	2155384 m
TXS2	502419 m	2155267 m
TXCH	505248 m	2148407 m
TXCR	500456 m	2160789 m
TX5B	503148 m	2155046 m
T4S3	503114 m	2155021 m

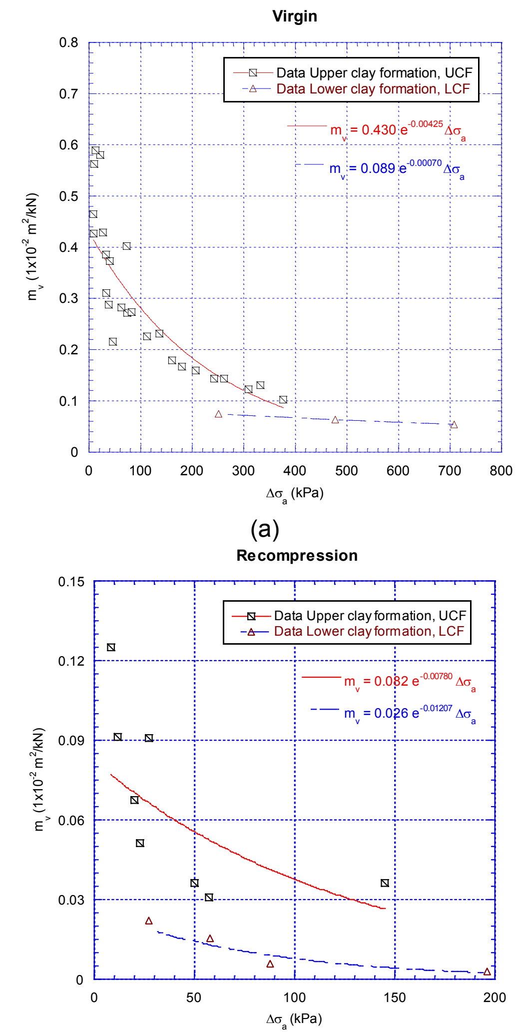


Figure 2. Variation of the coefficient m_v, in the studied site for (a) “virgin” and (b) “recompression" soil stages

2.2 Experimental data

Series of one dimensional consolidation tests were conducted on undisturbed specimens gathered from the borings B-1, B-2, B-3, B-4, B-5 and B-6. The volumetric compressibility m_v of the “virgin” and “recompression” branch vary depending on the increase of the effective stresses applied, as shown in Figure 2. Figure 2 also include a numerical regression equation obtained to calculate the coefficient m_v for the normally consolidated (i.e. virgin branch) and overconsolidated soil stages (i.e. recompression branch), in the upper clayey formation. For the deep stiffer clay deposits, the coefficients m_v were considered constants.

2.3 Piezometric measurements

A piezometer was installed at the instrumented site, as indicated in figure 1. It can be clearly noticed that prevailing pore water distribution is not hydrostatic (Fig. 3). The pore pressures distribution readings, allowed performing estimation of water pore pressure evolution for several time intervals (i.e. 5, 10, 30 and 60 years).

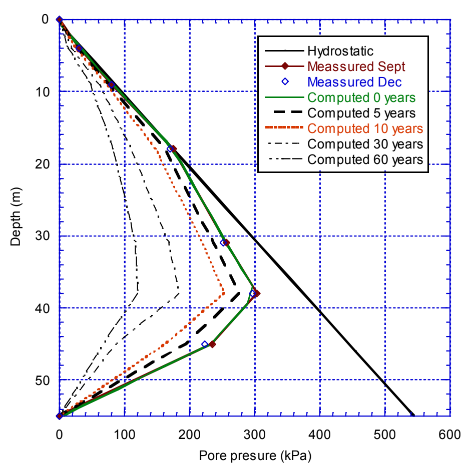


Figure 3. Pore pressure distribution with depth

Commonly the soil profile at this zone presents a desiccated crust of clay at the top extending down to a depth of 1.0m, which is underlain by a soft clay layer approximately 28.0m thick, with interbedded lenses of sandy silts and silty sands. The plasticity index varied from 139 to 265%. Underlying the clay there is a 6.0m thick layer of very dense sandy silt, sandy stiff clay and silty sands, which rests on top of a soft to stiff clay layer, with interbedded lenses of sandy silts and silty sands which goes down to a 55.0m of depth. The plasticity index varied from approximately 59 to 106%. Underneath this elevation (i.e. about 55m), permeable layers of sandy silt and silty sand intercalated with stiff clay are found (figure 4). Due to regional water extraction from deep water-wells, the pore pressure reduces down to zero in this layer, as can be seen in figure 3.

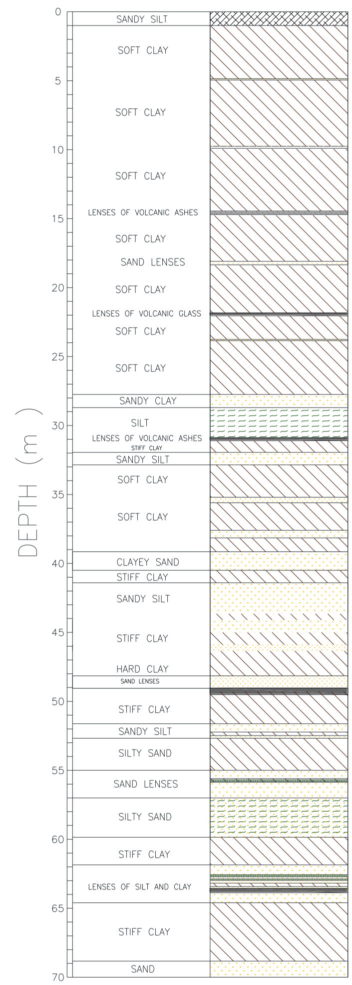


Figure 4. Soil profile at the site analyzed

2.4 Shear wave velocity distribution

The variation of the dynamic properties over time has been estimated based on the effective stresses state evolution calculated with the software FLAC^{3D}. The evolution of the shear wave velocity with the considered consolidation times (i.e. 5, 10, 30 and 60 year) was characterized from the calculated effective stresses with the numerical simulation in each layer, correlating them with specific laboratory tests, carried out at different effective confinement pressures. A correction factor was applied based on the initial stresses conditions and the corresponding results of the suspended logging test. The inferred values of the shear wave velocity for each confining stress associated with a consolidation time are shown in Figure 5. It can be seen that there is an increase of the initial shear wave velocity of around 10, 20, 30 and 40% corresponding to 5, 10, 30 and 60 years respectively. This phenomenon is depicted in Figure 5, which includes the velocity distribution obtained for each considered consolidation time.

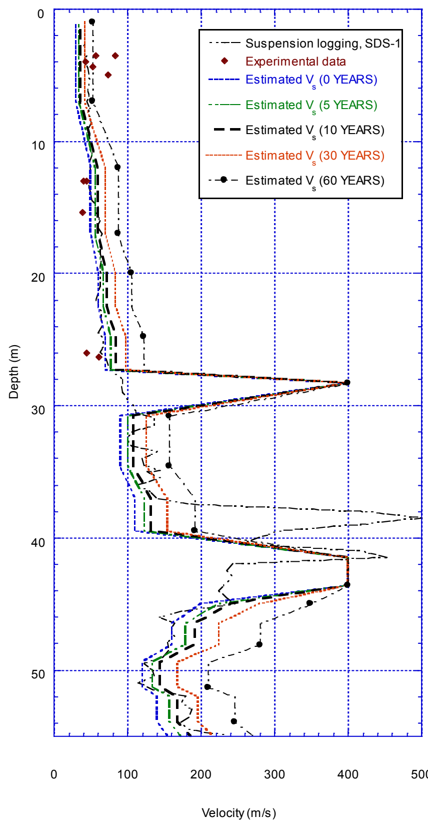


Figure 5. Shear wave velocity profile evolution

3 MODULUS DEGRADATION AND DAMPING CURVES

Romo and González (2011) proposed to match the shear modulus degradation and damping curves separately. This model is defined by the following expressions:

$$G=(G_{\min}-G_{\max})H_G(\gamma)+G_{\max} \quad [1]$$

$$\lambda=(\lambda_{máx}-\lambda_{\min})H_\lambda+\lambda_{\min} \quad [2]$$

$$H_G=\frac{\left(\gamma/\gamma_{rG}\right)^{2B_G}}{1+\left(\gamma/\gamma_{rG}\right)^{2B_G}} \quad [3]$$

$$H_\lambda=\frac{\left(\gamma/\gamma_{r\lambda}\right)^{2B_\lambda}}{1+\left(\gamma/\gamma_{r\lambda}\right)^{2B_\lambda}} \quad [4]$$

where G_{max} and λ_{min} are the small strain shear modulus and damping respectively, these are unique for each soil type. The parameters G_{min} and λ_{max} are the values that reach G and λ before dynamic failure. The parameters γ_{rG} y $\gamma_{r\lambda}$ are the strains references corresponding to 50% of degradation of the shear modulus, and 50% of increase in damping ratio respectively. In this model, these deformations depend only of the plasticity index, which is a source of uncertainty. The parameters B_G y B_λ are parameters that define the geometry of the curves G–γ and λ–γ respectively, which are a function of the plasticity index.

$$B_G=-2*10^{-6}IP^2+0.0014\ \ IP+0.2846 \quad [5]$$

$$B_\lambda=-7*10^{-6}IP^2+0.0038\ \ IP+0.3282 \quad [6]$$

3.1 Analysis Results

To calculate the normalized modulus degradation and damping curves for each consolidation time, the proposed model described above by Gonzalez and Romo (2011) was used. Effective stresses obtained from the numerical simulation were considered as the effective confinement stress for the model. It was assumed that the plasticity index does not change significantly over time.

3.2 Seismic environment characterization

A uniform hazard spectrum determined to characterize the seismic environment for the subduction seismogenic zone of the Mexican Pacific Coast, was developed at the same location as the rock station TXCR, at about 18.70 km from the site, to be able to compare it directly with measured responses. As it is well known, the uniform hazard spectra, UHS, is a representation of the relationship between the natural vibration period, T, and spectral acceleration, Sa, for a given exceedance probability associated with a return period. An extreme event with a return period of 2475 years was considered in the study presented herein (Figure 6).

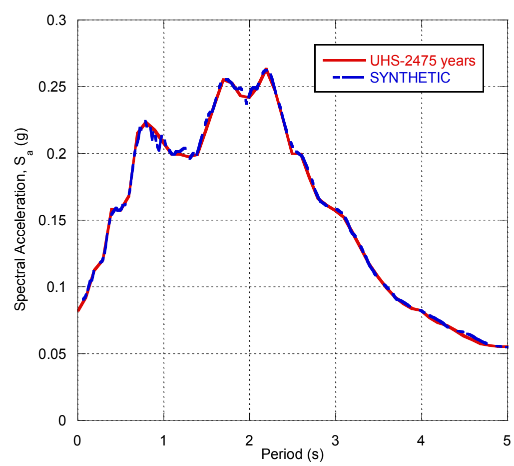


Figure 6. Computed uniform hazard spectra at TXCR and synthetic ground motion response spectra

3.3 Synthetic ground motion

To develop time histories which response spectrum reasonably match the design response spectrum, the selected (recorded) time history was modified using the method proposed by Lilhanand and Tseng (1988) as modified by Abrahamson (1993). This approach is based on a modification of an acceleration time history to make it compatible with a user specified target spectrum. The 5% damped response spectra calculated for the modified time histories are compared with the target UHS in figure 6. It can be seen that the response spectrum calculated from the modified time histories reasonably match the target spectrum. The seed is a long duration record (i.e. 175 seconds) typical of the subduction Pacific Coast, measured during the 1985 Michoacan earthquake, in firm soil.

4 SITE RESPONSE ANALYSIS

Initially, the computer code SHAKE (Schnabel et al., 1972) was used to conduct the 1D equivalent linear site response analysis. Then, a fully non-linear site response analysis was carried out with the program FLAC^{3D} (ITASCA, 2009), to further study soil nonlinearities. The finite differences model of the free field has a depth of about 53 m for the initial soil conditions (i.e. right after building construction), and a square section of 10 m by 10 m. The free field boundaries implemented in FLAC were used along the edges of the model. A rigid base was considered at the bottom of the model, to simulate the large dynamic impedance contrast existing at the site, in which a low shear wave velocity clay overlaid a high shear wave velocity bedrock. This model was calibrated against the results obtained with the program SHAKE considering a linear-elastic model in FLAC, characterized with the equivalent linear properties derived from the SHAKE analyses. For the nonlinear analysis, due to the lack of experimental data, the practical-oriented hysteretic model available in FLAC^{3D} (ITASCA, 2009) denominated as "sig3" was used to approximately deal with both modulus stiffness degradation and damping variation during the seismic event. This model considers an ideal soil, in which the stress depends only on the deformation and not on the number of cycles, with these assumptions an incremental constitutive relationship of the degradation curve can be described by $\tau / \gamma = Ms$, where τ is the shear stress normalized, γ is the angular deformation by shear and *Ms* the normant secant modulus. The sig3 model is defined according to equation [7]:

$$Ms = \frac{a}{1+\exp(-\frac{L-x_0}{b})} \qquad [7]$$

The model used for the analysis uses three parameters (a, b, and x_0). For the case studied, the parameter "a" ranges from 0.99 to 1.00, "b" from -0.43 to -0.6, and "x_0" from -1.15 to 0.21. Accordingly, to find the ground motions in the underlying bed rock to be used in the time domain site response analyses, the synthetic ground motions computed at the rock outcrop were deconvolved with the program SHAKE to the hardlayer found at the base of the soil profile.

4.1 Pore pressure evolution with time

Static stress-strain and pore pressure conditions for each time are the starting point for the dynamic analysis. These included state of stresses, ground settlements and soil layering configuration. Figure 7 shows the actual stresses state. Where u_r is the pore water pressure reduced by the extraction of water in this area of Mexico city.

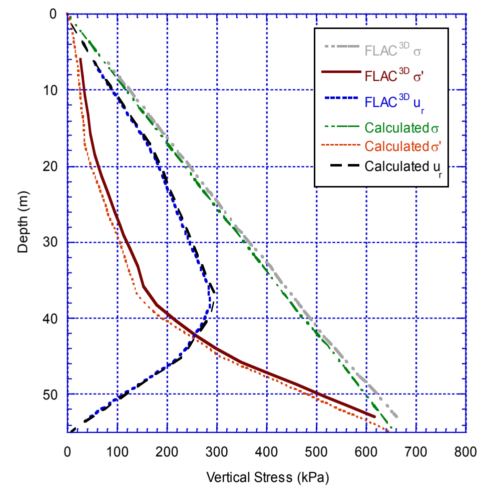


Figure 7. Actual stresses state

The one-dimensional consolidation problem, assuming that the soil matrix is homogeneous and behaves elastically, is given by equation [8]. The applied pressure is initially taken by the fluid but, as time goes on, the fluid drains throughout the soil to the layer surface, transferring the load to the soil matrix. The solution to this one-dimensional consolidation problem is commonly expressed in the framework of Biot theory (Detournay and Cheng, 1993).

$$\frac{\partial p}{\partial t} - c\frac{\partial^2 p}{\partial z^2} = 0 \qquad [8]$$

Where p is pore pressure, t is consolidation time, z is soil depth, and c is permeability over storage capacity. This equation considers constant vertical stress, and the boundary conditions.

$$P = 0 \text{ at } z = H, \text{ and } \frac{\partial p}{\partial t} = 0 \text{ at } z = 0 \qquad [9]$$

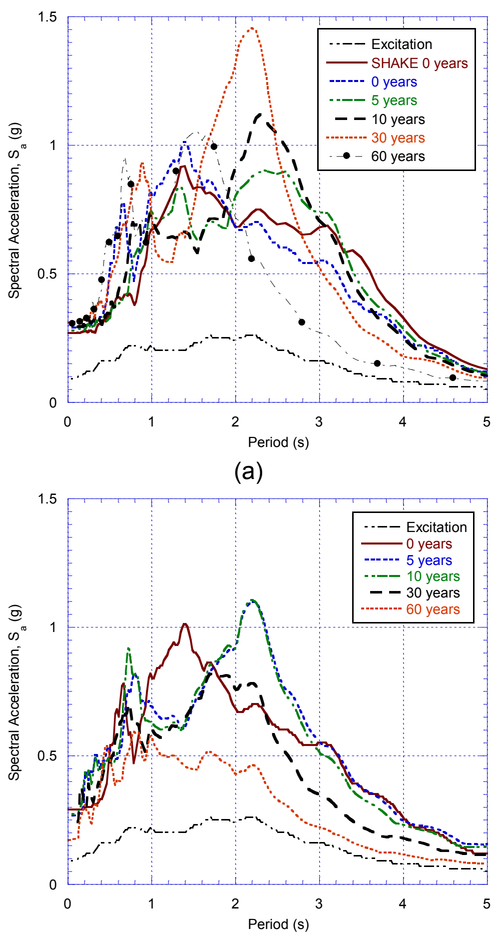
Equation [8] was solved using the subroutine available in FLAC^{3D}, and the results were validated comparing them with the piezometer in-situ pore water measurements, as depicted in figure 7.

4.2 Free field response

Site response analysis were carried out for each static ground conditions, accounting for changes both in the dynamic properties, as well as in soil thickness (Figure 4) for the studied site. Figure 8 presents the results obtained for the initial conditions (i.e. 0 years) in each studied site, with SHAKE and FLAC assuming linear properties characterized through the equivalent linear properties computed with the program SHAKE, and the FLAC non-linear model.

Thus, ten soil profiles were utilized in each studied site. Figure 8a depicts a comparison between the results obtained with SHAKE and FLAC for initial conditions (i.e.

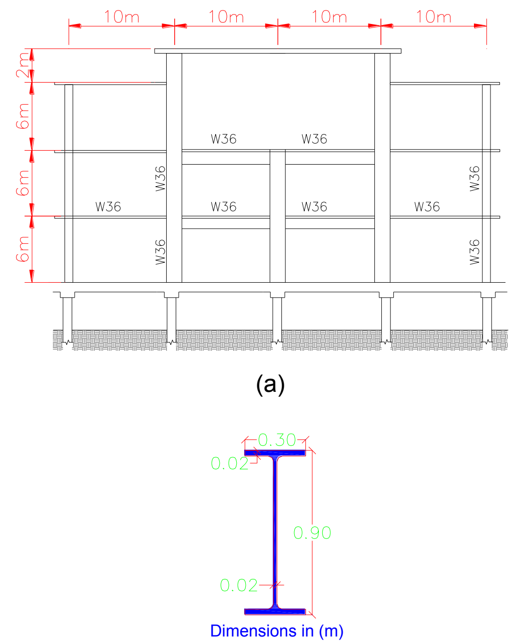
0 years), considering equivalent linear properties. It can be seen that there is good agreement between the results obtained with SHAKE and FLAC. Figure 8b, presents the results obtained with FLAC considered the fully non-linear model. It can be noticed that consolidation changes both the frequency content and spectral accelerations. After 5 and 10 years, although consolidation effects overall tends to reduce the predominant period of the soil deposit, these are still minor. Thus, soil non-linearity prevails and the period shifts towards 2 s, which corresponds to where the seismic excitation has its energy concentrated. Thus, the maximum spectral acceleration increases. After these critical year when reaching 30 years of consolidation, the dynamic properties and layering configurations has drastically change, so the predominated period has reduce down to 1.8 s, and the spectral acceleration have attenuated. Finally after 60 years, the spectral acceleration is about 0.9 s respectively, and the spectral acceleration have reduced.



(b)

Figure 8. Free field computed response spectra with (a) equivalent linear and (b) non-linear analyses

A 3D finite difference large strain model was developed using the program FLAC^{3D} (ITASCA, 2009) to analyze the seismic soil-structure interaction system response. The stress-strain relationship of the soil was assumed elastoplastic, following the Mohr-Coulomb failure criterion. In the finite difference model the columns and slabs, were modeled with beam and shell elements respectively. Figure 9 shows the building dimensions and the transversal section of the columns and beams considered. The thickness slab was considered 0.15 m. Table 2 summarizes the corresponding material properties considered in the numerical model.



(b)

Figure 9. (a) Elevation view of building analyzed and (b) Cross section W36x135

Table 2. Structural material properties

Section	Material	E (GPa)
Column	Steel	206.84
Beam	Steel	206.84
Slab	Concrete	24.25

The box foundation is excavated to a depth of 5 m. A 5% structural damping was considered for the building and foundation. The model have 10360 solid elements, 12065 nodes, 1577 elements and 759 nodes, as depicted in Figure 10. The modeled soil strata have the following abbreviations, surface crust, SC, upper clay formation, UCF, first hard layer, FHL, lower clay formation, LCL and second hard layer, SHL. The soil-structure system seismic response was monitored at the control points depicted in Figure 11. The depth of the control points is shown in Table 3.

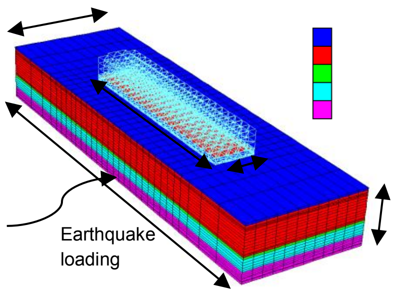


Figure 10. 3D model for the soil-structure analysis

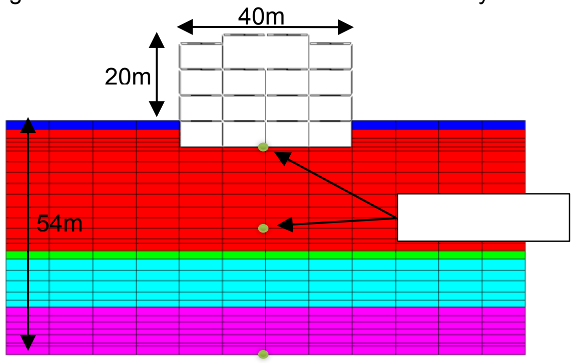


Figure 11. Control points locations in the 3D model

Table 3. Control points heights

Height (m)	SSI
-5.00	A
-20.00	B
-41.50	C

5.1 Static behavior

To assess the effect that ground consolidation have in the seismic soil-structure-interaction response, the total settlements generated for each analysis time considered (5, 10, 30 and 60 years) were obtained and presented in Figures 12 through 15 respectively. Ground movements due to 1) elastic settlements due to weight of the structure applied right after the completion of the building construction, 2) consolidation settlement due to the dissipation of the excess pore pressure generated by the building weight, and 3) regional consolidation due to water extraction were computed. It can be noticed that despite the large settlement amount expected at 30 years due to ground consolidation (i.e. around 4.20 m), the differential settlement is negligible. Thus, the soil settlement is almost uniform. A similar conclusion is withdrawn for 60 years, where the total settlements is around 5.91m, but the differential settlement is quite small.

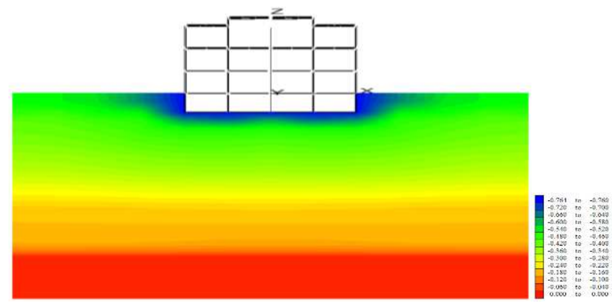


Figure 12. Elastic settlements + consolidation 5 years (m)

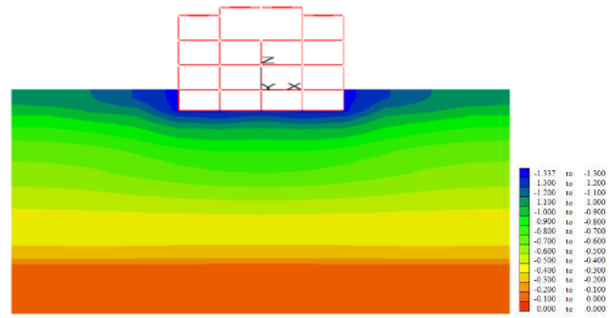


Figure 13. Elastic settlements + consolidation 10 years (m)

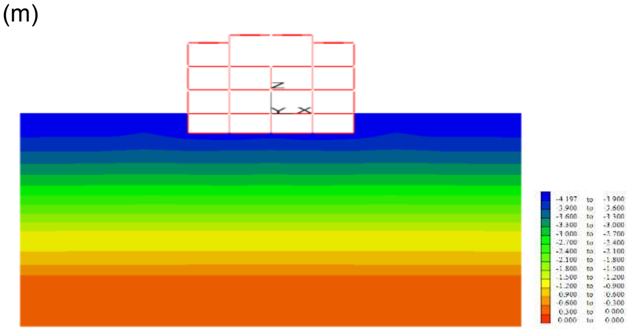


Figure 14. Elastic settlements + consolidation 30 years (m)

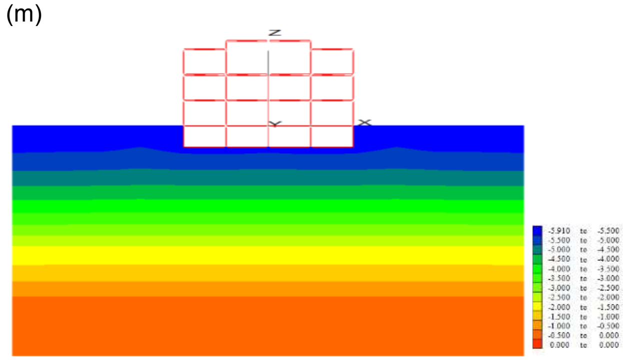
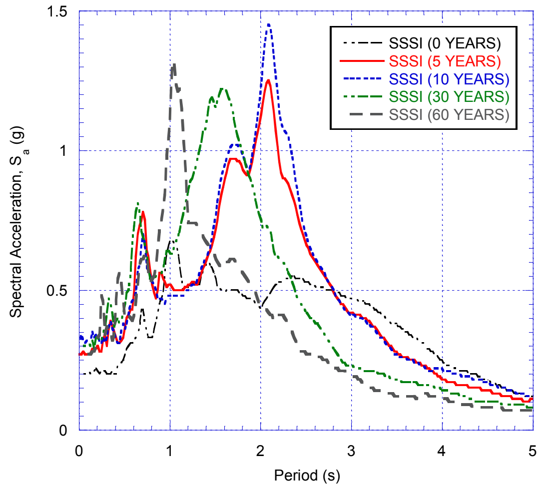


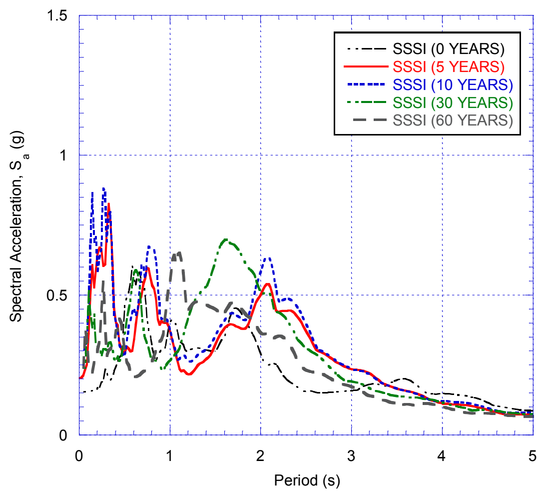
Figure 15. Elastic settlements + consolidation 60 years (m)

1.1 Seismic response

Seismic soil-structure interaction analysis were carried out for current conditions and after 5, 10, 30 and 60 years. Changes in dynamic properties and soil profile configuration due to soil consolidation were accounted for, as well as soil nonlinearities. Response spectra at the control points A, B and C, are depicted in figure 16. These are located below the foundation building at a depth of -5, 20 and -41.5 m (Fig 11).



a)



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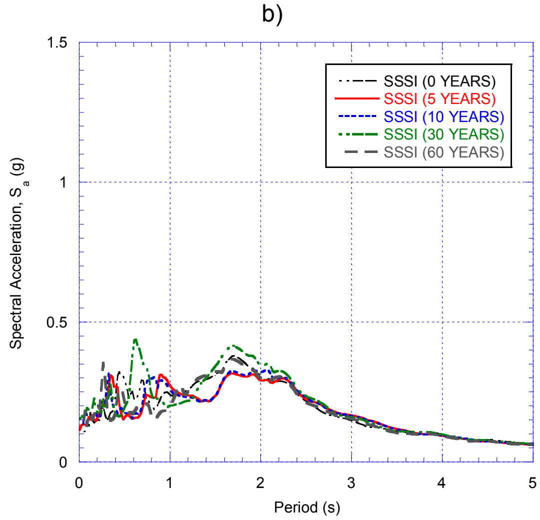

c)

Figure 16. Evolution of the response spectra with time in a) control point A, b) point B and c) point C

The static conditions for each consolidation time were the starting point of the dynamic analysis. As can be noticed, both frequency content and spectral acceleration change over time. Overall, the structural response is similar to that observed in the free field. Thus, for site 1 at 5 and 10 years the nonlinear soil behavior control the soil-structure seismic response and the spectral ordinates are significantly amplified when the deposit predominant period gets closer to 2, with respect to those obtained considering ground conditions right after construction. After 30 and 60 years, the soil gets stiffer; shear wave velocity distribution increases and the soil thickness decreases. Therefore, the effect of soil nonlinearities is overcome, and the predominant soil deposit moves towards 1.1 seconds, having a relative reduction in the spectral ordinates.

2 CONCLUSIONS

Soil effective stresses changes during the economic life of a structure built in highly compressible clays, leads to important ground deformations, and changes in the dynamic properties (i.e. shear wave velocity distribution and modulus degradation and damping curves). These effects, which are commonly ignored in practice, can substantially modify the frequency content and spectral accelerations in both free field and in the soil-structure system. Pore pressure variations over the project economic life is due to both regional subsidence as well as dissipation of excess pore pressure caused by the structure weight. For the cases studied herein, a complex interplay between soil nonlinearities, which tend to elongate the predominant period of the soil deposit, T_p, and the overall tendency of ground consolidation to shorten it, led to variations in the spectral ordinates depending on how close T_p is of the predominant period of the excitation, T_{pe}. For subduction events recorded at Mexico City firm soils T_{pe} ranges between 1.5 to 2.5 s. A similar trend was observed in the soil-structure system response for the cases analyzed inhere. The non-linear simulation does not account for pore pressure generation due to cyclic loading, considering that seismic-induced pore pressure is expected to be small in these very high plasticity clays, even for ground motions as strong as the devastating1985 Mexico City earthquake. On the other hand, the high compressibility of Mexico City clays seems to play a major role in the effective stress distribution, changing both dynamic properties as well as soil profile configuration over time. Furthermore, currently there is a lack of experimental data to properly characterize the stress-strain-pore pressure relationship during cyclic loading for Mexico City clays.

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