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Clay Behavior, Ground Response and Soil-Structure Interaction Studies in Mexico City

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SYNOPSIS This paper focuses on the most relevant results of recent investigations carried out on the behavior of Mexico City clays under dynamic loading, on the effects of soil site conditions and on the evaluation of the dynamic soil-structure interaction phenomenon. The paper shows the impact of these studies upon foundation engineering practice in earthquake-prone regions where clayey deposits exist, and advances simple, yet accurate, procedures to develop site-dependent, building-specific input motions for the design of structures in the Valley of Mexico. It discusses recent seismic observations that clarify the origin of the long coda observed in several records obtained in the lake zone in Mexico City.

INTRODUCTION

One of the prime problems in earthquake engineering is the definition of the seismic environment to be used in the analysis and design of structures. It is now widely accepted that local site conditions may affect considerably the rock-like motions and that soil-structure interaction phenomena may, in turn, alter the free field ground motions. The most spectacular cases of soil amplification and soil-structure interaction effects, during recent times, have been observed in Mexico City during the 1985 and more recent events. This paper centers mainly on the problem of developing appropriate seismic input motions for the the analysis of existing buildings and the design of new structures in the Valley of Mexico. Emphasis is put on buildings located within the clayey soil deposits. The procedures developed to compute site-dependent, building-specific seismic environments for analysis purposes are included and discussed in the paper. It is shown that if soil behavior modeling is properly accounted for, the proposed approaches are capable of reproducing both free field and building base seismic motions.

DYNAMIC BEHAVIOR OF MEXICO CITY CLAYS

Clays usually exhibit a nonlinear behavior within a broad range of strains. For low strains (i.e. $\gamma \leq 10^{-4}\%$) clays behave as a viscoelastic material. The stiffness and strength of clayey materials become stress dependent for strains exceeding values of $10^{-2}\%$, although there are clays that show this dependency only for strains above $10^{-1}\%$, depending on their plasticity index, I_p, and their relative consistency, I_r. From experimental studies carried out for Mexico City clay and other more consistent clays it has been observed that these two index properties are key parameters in the behavior of clayey materials (Romo, 1990, 1991; Romo and Ovando, 1993).

Low-Strain Shear Modulus

It has been broadly accepted that the low strain shear modulus of clayey materials may be obtained in the laboratory from resonant column tests and in the field from shear wave propagation procedures. It has also been recognized that shear moduli evaluated by these two alternatives may yield different results mainly because of aging effects and remolding that may be induced by retrieval and handling of soil samples (Harding and Drnevich, 1972; Afifi and Woods, 1971; Anderson and Richart, 1976; Anderson and Sotokoe, 1978). The study results reported below show that for highly plastic clays the differences between laboratory and field determinations are less significant that those reported by other researchers.

Results from resonant column tests on samples retrieved from different sites and depths are presented in figure 1. The characteristics of the soils tested as well as the testing conditions are included in Table I.

The results of figure 1 show that the maximum shear modulus, $G_{máx}$, is a function of the effective consolidation stress, σ'_c, of the plasticity index, I_p, and the relative consistency, I_r.

$$\left(I_r = \frac{\omega_L - \omega_n}{I_p} \;;\; \omega_L = \textit{liquid limit},\ \omega_n = \textit{natural water content.} \right)$$

From least square regression studies the following analytic expression has been obtained (Romo and Ovando, 1994)

$$G_{máx} = 122 p_a \left(\frac{1}{I_p - I_r} \right)^{(I_p - I_r)} \left(\frac{\sigma'_c}{p_a} \right)^{0.82} \tag{1}$$

where p_a is a constant with stress units to define the $G_{máx}$ units (i.e. 1kg/cm^2; 10t/m^2), the other parameters were defined previously. Equation (1) is valid for (I_p-I_r) positive, where I_p is in decimals (i.e. I_p=0.95, instead of 95%).

Figure 1 compares the experimental results with the values computed using Equation 1. The approximation is very good, showing that this analystical expression may be used reliably to estimate maximum shear modulus of soft clays. It should be mentioned that all samples included in the study were normally or slightly overconsolidated. One would expect that for heavely overconsolidated clays an additional parameter multiplying the right hand side of equation (1) of the type R^{α} should be included (R=overconsolidation ratio; α=volumetric strain ratio parameter of the Critical State theory). It is worth mentioning that most clays in Mexico City are normally or slightly overconsolidated ($R<3$) thus the consideration of the effect of R has relatively low impact in practical applications.

After the 1985 seismic events shear wave velocities were measured in a number of sites using both down hole and suspension P-S logging

Table I Soils characteristics and testing conditions

Sample	Depth (m)	ω_L (%)	ω_P (%)	I_p (%)	ω_n (%)	I_r	G_ω (%)	γ_s (gr/cm^3)	e_i	Ss	σ'_c (kg/cm^2)
1	18.50	198.50	48.8	149.7	155	0.290	1.10	1.262	4.24	2.70	1.0
2	8.20	293.0	73	220	266	0.123	1.01	1.198	6.04	2.32	1.0
3	18.50	198.5	48.8	149.7	155	0.290	1.01	1.251	4.31	2.70	0.3
4	14.20	245	98	147	221.8	0.163	1.02	1.210	6.20	2.76	1.0
5	14.20	245	98	147	221.8	0.163	1.02	1.210	6.20	2.76	2.0
6	14.20	245	98	147	221.8	0.163	1.02	1.186	6.20	2.76	1.5
7	14.20	245	98	147	221.8	0.163	0.90	1.248	5.99	2.76	0.3
8	14.20	245	98	147	221.8	0.163	0.90	1.453	5.99	2.76	1.2
9	11.40	257	77	180	210	0.269	0.98	1.195	5.43	2.53	1.0
10	19.10	245	95	150	226	0.133	0.84	1.163	5.97	2.64	1.0
11	17.90	156.50	64.70	91.80	154.48	0.022	1.03	1.267	4.00	2.58	1.0
12	8.73	322.50	89.60	232.90	360.20	-0.162	1.02	1.128	9.22	2.56	1.0
13	8.73	322.50	89.60	232.90	366.00	-0.188	1.09	1.158	9.37	2.56	1.5
14	17.30	345.60	108.0	237.60	328.60	0.072	1.04	1.158	9.18	2.79	1.0
15	17.50	272.00	81.90	190.10	234.30	0.198	0.99	1.214	6.41	2.74	1.0
16	17.50	272.00	81.90	190.10	230.20	0.219	1.08	1.195	6.30	2.74	0.3
17	24.50	281.90	94.11	187.79	288.06	-0.033	0.98	1.185	7.92	2.75	2.0
18	24.50	281.90	94.11	187.79	272.68	0.049	1.10	1.163	7.51	2.75	1.5
19	24.50	281.90	94.11	187.79	272.68	0.049	1.10	1.163	7.03	2.75	2.0
20	24.50	281.90	94.11	187.79	272.68	0.049	1.10	1.163	6.47	2.75	3.5
21	40.14	252.50	65.37	187.13	156.53	0.513	1.02	1.286	4.16	2.66	2.0
22	40.14	252.50	65.37	187.13	156.53	0.513	1.02	1.286	4.08	2.66	2.5
23	40.14	252.50	65.37	187.13	156.56	0.513	1.02	1.286	4.02	2.66	3.5

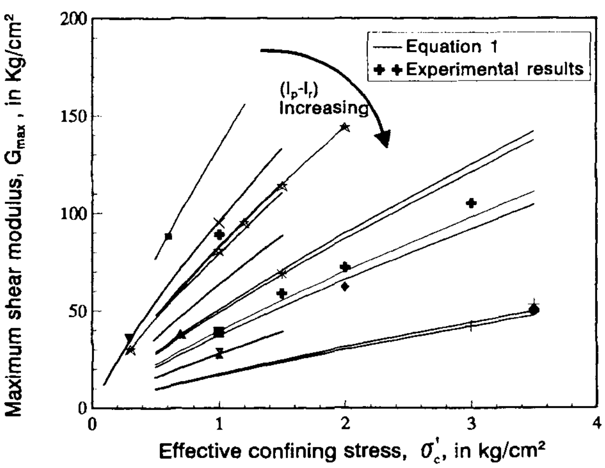

Figure 1. Effect of σ'_c and $(I_p\text{-}I_r)$ on maximum shear modulus

procedures. At same sites CPT tests were also carried out with the purpose of studying possible correlations between shear wave velocities and resistance cone penetration values. In figure 2 a typical result from these investigations is depicted. It is seen that the velocity profile resembles the cone penetration profiles indicating that these two quantities are susceptible of beeing correlated.

Using expansion cavity theory coupled with a hyperbolic model for clay stress-strain representation, Ovando and Romo (1991) developed analytical expressions that allow the evaluation of shear wave velocities, V_s, from cone penetration resistances, q_c. For the clayey and silty sand materials found in Mexico City the equation they propose is

$$V_s = \eta \sqrt{\frac{q_c}{N_k \gamma_s}} \tag{2}$$

where η and N_k are parameters that depend on the soil type. The values for Mexico City soils are given in Table II. In equation (2); V_s is in m/sec, q_c in ton/m^2 and γ_s, the saturated unit weight, in ton/m^3.

Large-Strain Shear Modulus

The effects of strains, γ, and confining effective stresses, σ'_c, on shear modulus are depicted in figure 3, and in figure 4 are shown the corresponding G/G_{max}-γ curves. The results of figures 3 and 4 show that Mexico City clays have a nearly linearly elastic behavior up to shear strains values that vary from about 0.2 to 0.5% depending on the value of the parameter $(I_p\text{-}I_r)$. This finding is slightly different from that reported previously in the sense that both the shape of the G/G_{max}-γ curves and the magnitude of the threshold strain were believed to be affected only by the plasticity index (Dobry and Vucetic, 1987; Romo and Taboada, 1988).

It is well known that the plasticity index is related to the initial microstructure of the clay, which is basically dominated by compositional factors such as mineralogy, particle size and shape, pore water chemistry and depositional medium (water, ice, air). However, the initial soil microstructure varies through time due to chemical and physical processes. While the former may also modify the plasticity characteristics of the clay the latter usually induces no appreciable changes to the initial plasticity index of the clay. Accordingly, the introduction of I_r is an attempt to take into account in a simple fashion any modifications the initial microstructure of the soil might have undergone since its formation by physical processes (Romo and Ovando, 1993; Romo and Ovando, 1994).

Table II Parameter values of equation (2)

Soil type	N_{kh} values			η values
	maximum	mean	minimum	
Clays from Texcoco Lake	14.0	9.5	6.7	23.33
Clays from Xochimilco-Chalco Lake	14.0	9.9	7.0	26.40
Silty-sands of the hard layers in the Valley of Mexico	16.0	11.1	8.0	40.00

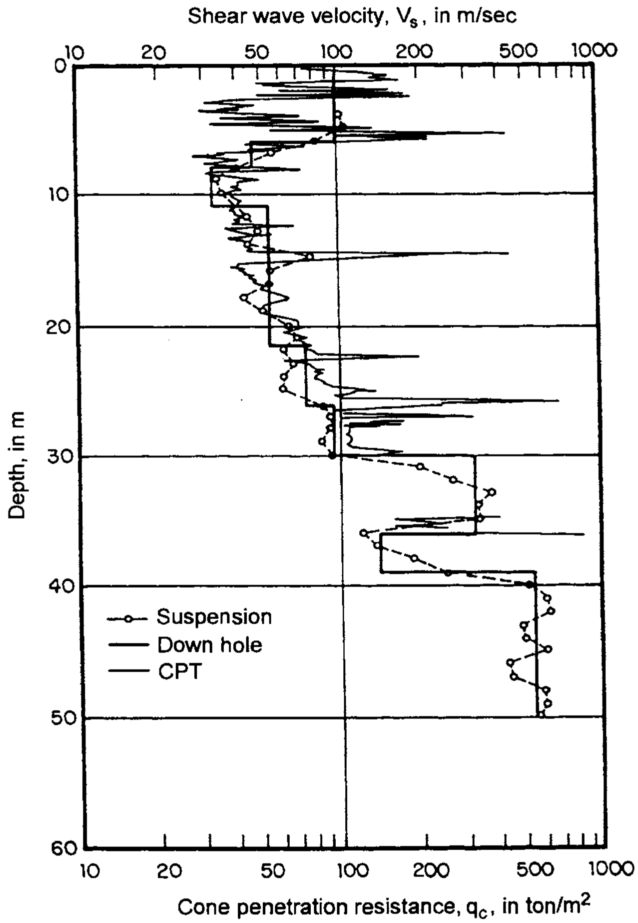

Figure 2. Velocity and CPT profiles in the lake zone of Mexico City

The results of figures 3 and 4 may be modeled using a Masing type model given by the following expression (Romo, 1990)

$$G = G_{m\acute{a}x}(1 - H(\gamma)) \tag{3}$$

where

$$H(\gamma) = \left[\frac{(\gamma/\gamma_r)^{2B}}{1 + (\gamma/\gamma_r)^{2B}} \right]^{A'}$$

and

$$A' = A + I_r$$

here G is the shear modulus for any shear strain, γ; G_{max} is the low shear strain modulus; γ_r, A and B are soil parameters that are plasticity index dependent. The values of these three parameters are given in figures 5 to 7.

The accuracy of equation (3) has been proven to be very good for the soils found within the Valley of Mexico (for range of soil types see Table II). As an example, comparisons between equation (3) and the experimental results are shown in figure 8.

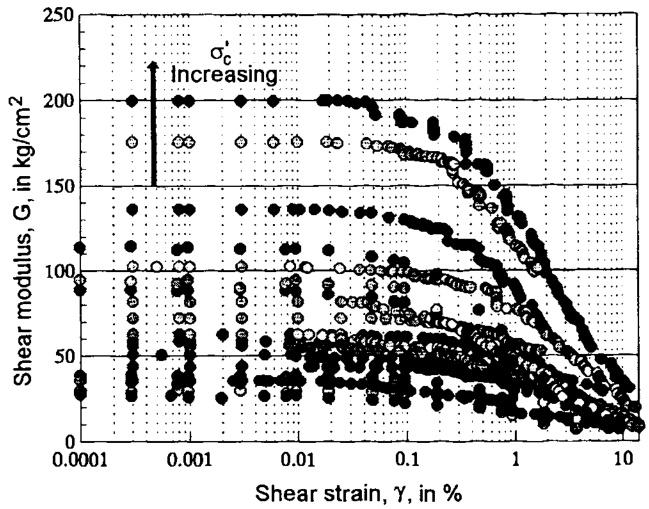

Figure 3. Effect of confining stress on shear modulus-shear strain curves

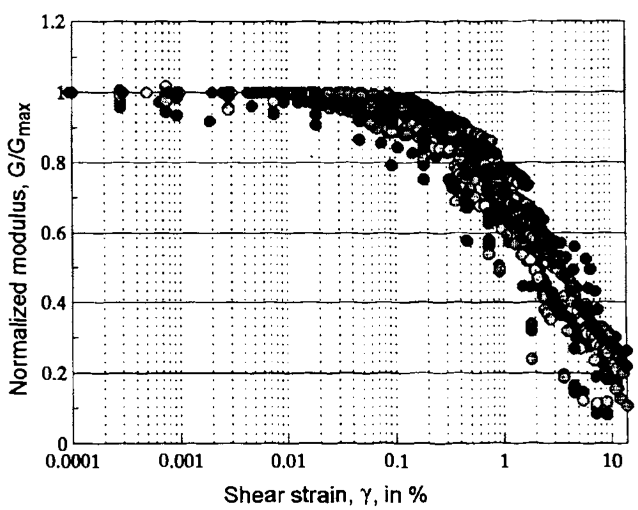

Figure 4. Normalized shear modulus curves

Shear Modulus Degradation by Fatigue

Soils fatigue develops mainly because of the cyclic shear distortions caused by seismic loading at microstructural level. The importance of this phenomenon depends on a number of factors such as type and state of soil, effective confining stresses, magnitude of cyclic shear stresses and number of times the cyclic stress is applied. Degradation

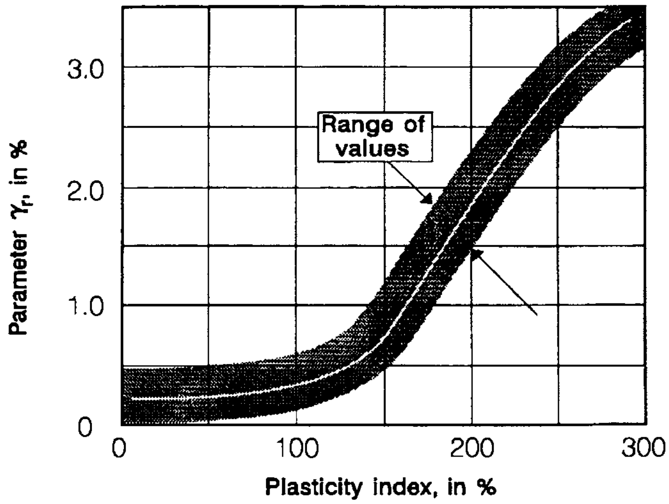

Figure 5. Effect of I_p on parameter γ_r of equation 3

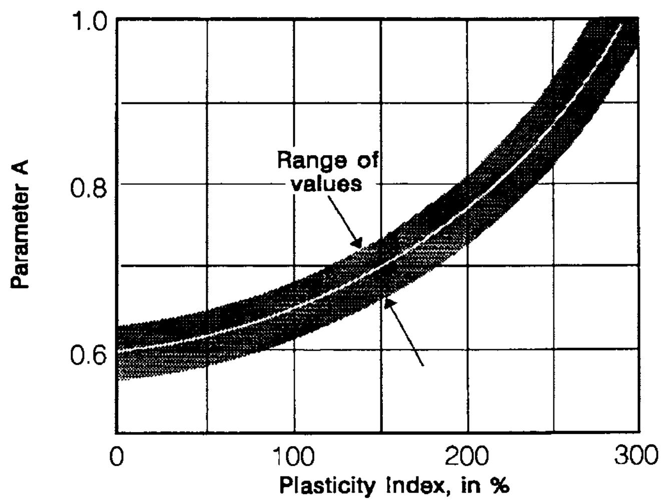

Figure 6. Effect of I_p on parameter A of equation 3

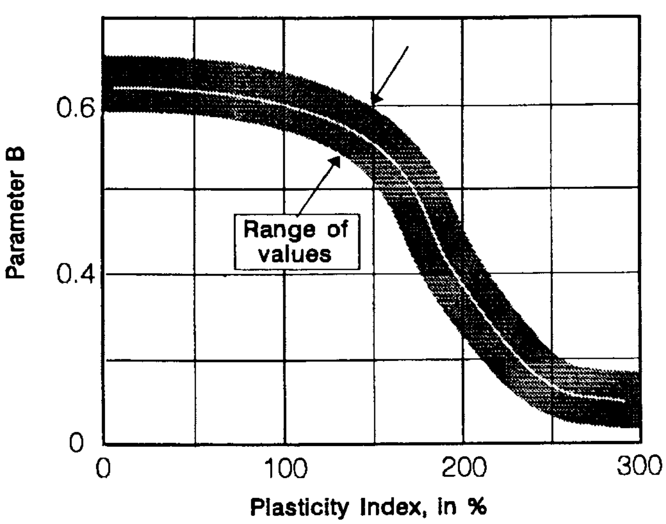

Figure 7. Effect of I_p on parameter B of equation 3

rate is increased when pore water pressures develop during cyclic loading. For soils where dynamic pore water pressures are negligible the fatigue phenomenon is of little importance except when the cyclic shear stress (plus the sustained, static, shear stress) is similar to the soil undrained shear strength. Such is the case of clays with high (larger than 150%) plasticity indeces. Some examples of the variation of G as a function of the number of cycles, N, are shown in figure 9, where it is seen that the degradation rate is negligible even for total shear stress (cyclic plus static) values close to soil shear strength.

The effect of fatigue may be evaluated using the relation proposed by Idriss et al (1978)

$$G_N = G_0 \, N^{-t} \tag{4}$$

where G_N is the shear modulus for load cycle N, G_0 is the initial shear modulus, N is the number of cycles and t is the degradation parameter.

There exists a bulk of experimental information showing that the degradation parameter depends on the magnitude of the cyclic strain, the stress path followed in sample consolidation and over consolidation ratio (Romo, 1991; Dobry and Vucetic, 1987). For the normally consolidated clays found (within Texco Lake in Mexico City), the parameter t varies with cyclic shear strain according to the following approximate relations (Romo, 1991):

$$t = \frac{2}{1+\nu}\,(0.0299)\ \gamma;\text{for anisotropic consolidation and } \gamma < 2\%$$

$$\tag{5}$$

$$t = \frac{2}{1+\nu}\,(0.0122)\ \gamma;\text{for isotropic consolidation and } \gamma < 6\%$$

where υ is the Poisson ratio.

The above results seem to indicate that anisotropically consolidated soil samples are less susceptible to the fatigue phenomenon (rate of degradation is lower). This is understandable since the shear stresses induced during the consolidation stage modify more significantly (than isotropic stresses) the microstructure leading to a somewhat more stable soil structure; furthermore, the reversal of cyclic shear stresses on the potential failure plane may be precluded by the initial static shear stress, thus decreasing the damaging effects on soil stiffness.

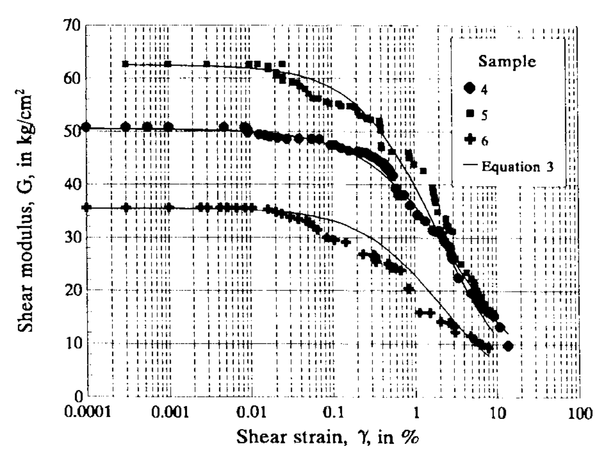

Figure 8. Comparison between experimental results and equation 3

Damping Ratio

Following Harding and Drnevich (1972) it may be shown that for

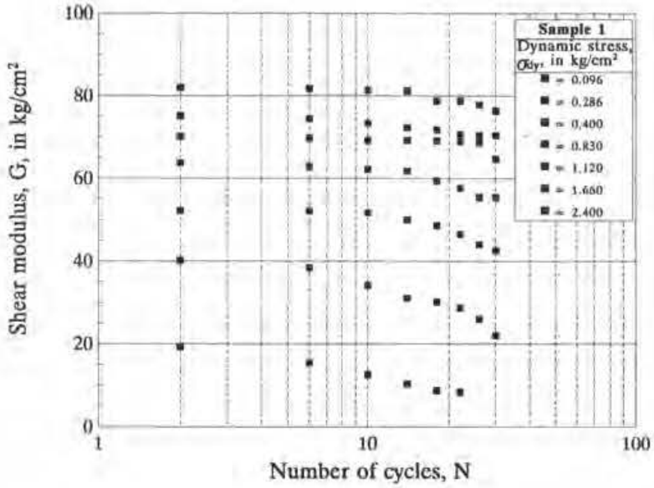

Figure 9. Shear modulus degradation by fatigue

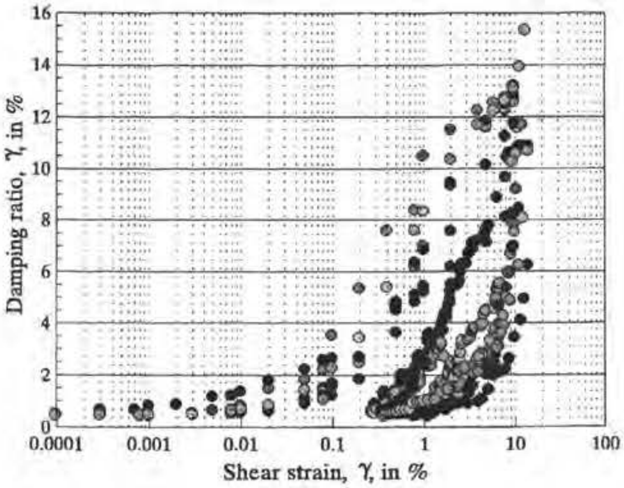

Figure 10. Damping ratio curves

viscoelastic materials and assuming a Masing-type behavior, the damping ratio, λ, is related to shear modulus by the next equation

$$\lambda = \lambda_{max}\,(1 - G/G_{max}) \tag{6}$$

where λ_{max} is defined as the maximum value of λ that the soil may attain before reaching failure under dynamic loading. From a large amount of laboratory testing results (see figure 10) it has been found that λ_{max} is for all practical purpouses equal to 13% for Mexico City clays. This value is much lower than the values reported for other clays (range between 20 and 26%) with lower plasticity indeces, indicating the dependence of λ_{max} on I_p (and presumably I_r).

Replacing equation (3) in equation (6) and using a lower boundary value for λ, (i.e. λ_{min}) the following relationship has been found to represent adequately the variation of λ with shear strains

$$\lambda = \lambda_{max}(H(\gamma)) + \lambda_{min} \tag{7}$$

The results obtained from numerous laboratory tests show that a value of 0.5% for λ_{min} is appropiate for Mexico City clays (see figure 10). It should be noted that this value is significantly lower than the values reported for other clays (range between 3 and 6%).

Stress-Strain Behavior

As indicated in figure 11, when a soil specimen is subjected to cyclic loading it undergoes a transient cyclic deformation and after a number of load applications the sample accumulates permanent deformations. For a given soil, the former basically depends on the magnitude of the cyclic stress and the latter on the cyclic stress magnitude and loading duration.

Test results gathered throughout the years have pointed out that these two types of deformation may be correlated. A correlation between cyclic and permanent strains is depicted in figure 12 for a typical clay sample (I_p=250%) of Mexico City. These curves show a distinctive strain value where the rate of permanent deformations increases indicating the existance of a threshold of cyclic strains beyond which permanent (plastic) deformations accumulate faster. According to the results of figure 12 the critical axial strain is about 3% (the corresponding shear strain is γ=0.75 (3%)=2.25%). This threshold strain is one order of magnitude higher than the critical strain defined previously as boundary between linear and nonlinear clay behavior. This difference between both thresholds seems to indicate that eventhough the clay behaves as a nonlinear material the plastic permanent deformations remain negligible until a cyclic shear deformation of the order of 2.25% is reached. This implies that in the case of highly plastic clays (like Mexico City's) permanent deformations will develop significantly only when the soil is close to failure under dynamic loading. Such behavior is manifested in figure 13 where a typical total stress-permanent strain is plotted. The results show that permanent deformations (axial deformations) accumulate only when the total shear stress (cyclic plus static) exceeds the strength of the clay. They also point out at the fact that the dynamic strength is higher than the static strength (S_u). In the case of the saturated clays of Mexico City the dynamic strength may be about 60% higher than the static undrained strength.

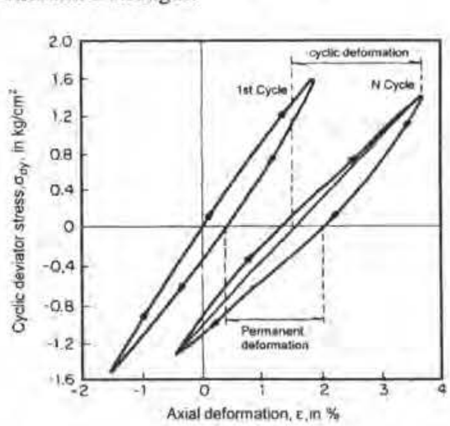

Figure 11. Dynamic strain components

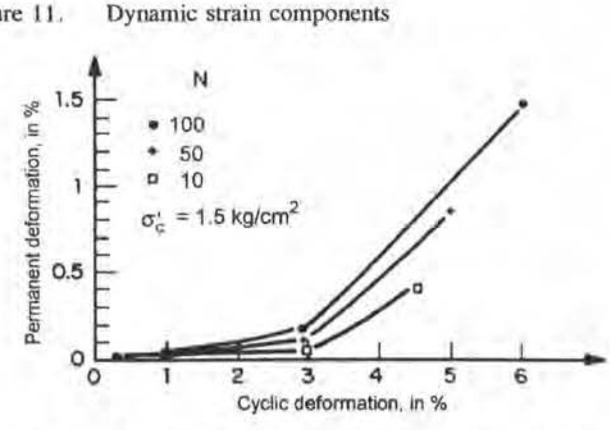

Figure 12. Effect of cyclic deformation on permanent deformation

Pore Water Pressure

Results of cyclic triaxial tests on clay samples ($I_p > 250\%$) consistently show that the dynamic pore water pressure developed is very small even near sample failure (Romo, 1991). The maximum pore water pressures recorded during cyclic loading (and after a 48-hour rest period) do not exceed $0.30(\sigma'_c)$. These results are reazonable considering the fact that plastic deformations do develop only until large (2%) cyclic shear strains are induced in the clay sample, which means that microstructural permanent distortions are not significant within a very broad range of cyclic shear strains. Since these permanent distortions are the main cause of pore water pressure generation then it is understandable why Mexico City clays develop very low pore water pressures under dynamic loading. These results seem to indicate that the amount of dynamic pore water pressure that a given clay may develop depends on its plasticity index (and presumably I_l). This is an aspect that should be pursued thoroughly in view of the results reported for other clays ($I_p < 60\%$) where large pore water pressures accumulate during cyclic loading.

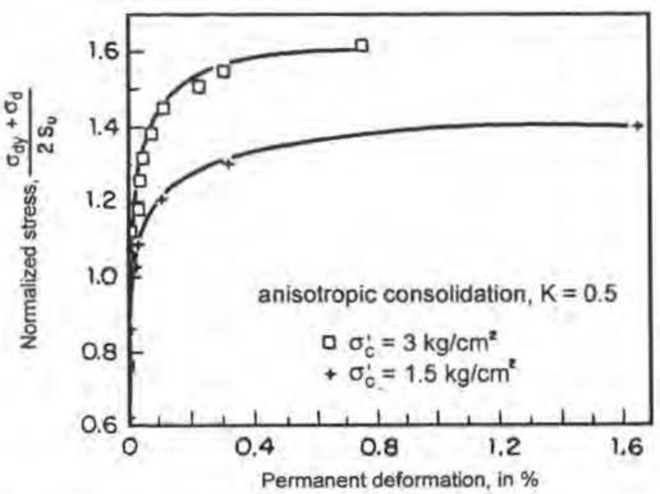

Figure 13. Dynamic stress-plastic deformation relationship

FREE FIELD RESPONSE STUDIES

From a geotechnical point of view, Mexico City has been divided in three regions (see figure 14): a) the lake zone, which consists of a 20 to more than 100m deposit of highly compressible, high water content clay underlain by the so-called deep deposits formed by very stiff layers of cemented silty sands (see figure 2), b) the hill zone formed by volcanic tuffs and lava flows and c) the transition zone composed by erratic stratifications of alluvial sandy and silty layers interlaced with clay layers. Extensive field studies have shown that the hard deep deposits underlay the aluvial compressible deposits in most of the basin of Texcoco Lake and outcrop towards the west side of the hill zone (Romo, 1987).

Local site effects on ground motions have been recognized since the begining of seismology (Reid, 1910; Gutenberg, 1957; Rosenblueth, 1952); however, it was until the 1985 seismic events that the influence of local soil conditions was so dramatically evidenced since even within the same clay deposits, relatively small variations in their thicknesses was sufficient to modify significantly the surface ground motions. An example of their variability is shown in figure 15 where the response spectra of motions recorded at different sites during the 1985 seismic events and their corresponding shear wave velocity profiles are plotted. (Location of these sites is indicated in figure 14.)

The velocity profiles show that the shear wave velocities in the clay are as low as 30m/sec and in the deep deposits are higher than 600m/sec. The information included in this figure clearly shows that relatively small variations in velocity values and in depths to the deep deposits significantly affect the ground motions in the lake zone. Accordingly, failure to properly characterize the clay deposits may lead to erroneous definitions of the seismic environment for designing buildings in the lake zone. It is important to stress the fact that the motions on firm ground, (CU site) may be amplified some 13 times (for 5% damping spectral accelerations, for lower dampings the amplification is even larger) by the clay deposits (SCT site) for periods in the range of 2.0 seconds. These drastic amplifications have no parallel in any part of the world to the author's knowledge. These unprecedent amplifications have their origin in the dynamic properties of the clayey materials found in Mexico City. Namely, nearly linear elastic behavior up to large shear strains (1%) and extremely low damping ratios (3 to 5%) for this shear strain levels.

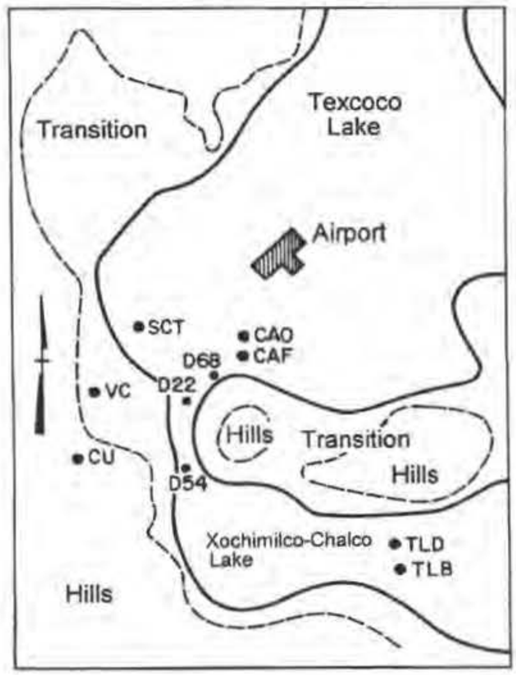

Figure 14. Mexico City geotechnical zoning

An additional aspect of relevance in foundation engineering is the vertical variation of motions from the deep deposits up to the ground surface. The spectra included in figure 16 correspond to motions recorded by a vertical array of accelerographs at three depths in the lake zone. As it would be expected for a deposit where the soil stiffness increases with depth, the intensity of the ground motions grows toward the ground surface. It is important to note that motions are amplified more in the upper 30m of the stratigraphy than in the deeper 72m. The maximum amplification occurs at the soil deposit natural period (2.3sec) and is about 12 times, which is similar to the maximum amplification between CU and SCT sites (see figure 15).

Having in mind the above observations, it would seem to be a good engineering practice to define site-specific spectra for the seismic design (or retrofitting) of buildings placed within the lake and transition zones. To accomplish this objective it is necessary to have available procedures to compute the response of stratified soil deposits. There exists a number of analytical methods that can perform the neccesary tasks to evaluate ground seismic response analyses. Any procedure that proves to be reliable could be used for this purpose.

After the seismic events of 1985 struck Mexico City, a number of analytical procedures have been used to explain the observations made during these and more recent events. Early studies focused on one dimensional models. The degree of approximation with which these procedures reproduce the motions recorded at different sites and due to various earthquakes, ranges from good to excellent as may be observed in figures 17 to 20 (Romo, 1991). Similar approximations have been obtained at some 50 sites located both in the lake and transition zones (at present time there are in use about 110 strong motion recording stations scattered throughout the Valley of Mexico). The particular features of the procedure used to compute the responses included in these figures are as follows. The input motion is assigned at an outcrop (for the cases shown, CU site), in terms of acceleration response spectra. Because of the stiffness characteristics a CU (see figure 15) the input spectra is deconvolved down to a layer with similar shear wave velocity to that of the deep deposits, so that the site effects are removed from the recorded motions at CU. Then, this computed motion is used as input (within the site stratigraphy) at a layer that has the deep deposits stiffness characteristics, that in turn are considered as the halfspace on which the soil stratigraphy rests (see figure 21). The soil deposit is idealized as a horizontally stratified one dimensional model where SH waves with any incidence angle are considered. The analytical procedure is based on the Thomson-Haskell approach and makes use of random vibration and extreme value theories to solve the problem. An equivalent (piece-wise continous) approximation is used to account for the nonlinear response of the clays, according to the procedures to model clay dynamic behavior discussed previously. A more detailed discussion of the analytical method is presented elsewhere (Bárcena and Romo, 1994).

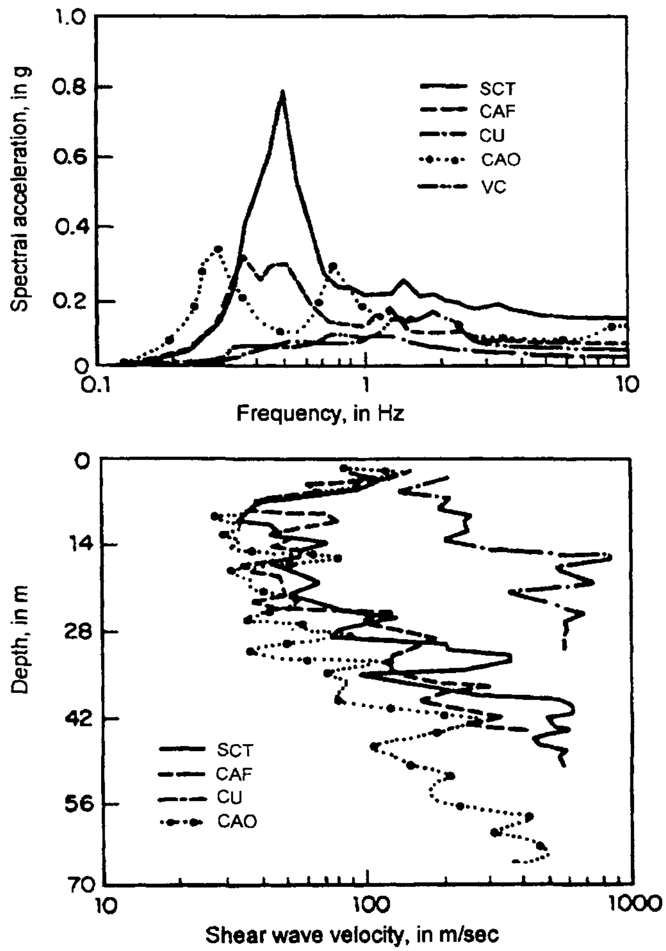

Figure 15. Ground motion variations and velocity profiles

In spite of the accurate reproduction of response spectra for such a large number of sites there were some aspects of the responses at several sites that apparently one dimensional models were not able to reproduce. The main arguments against one dimensional idealizations of the soil deposits was that when excited by a hill-zone record the duration of the records obtained in the lake zone was not reproduced, in fact the total length of the computed motions was about half the

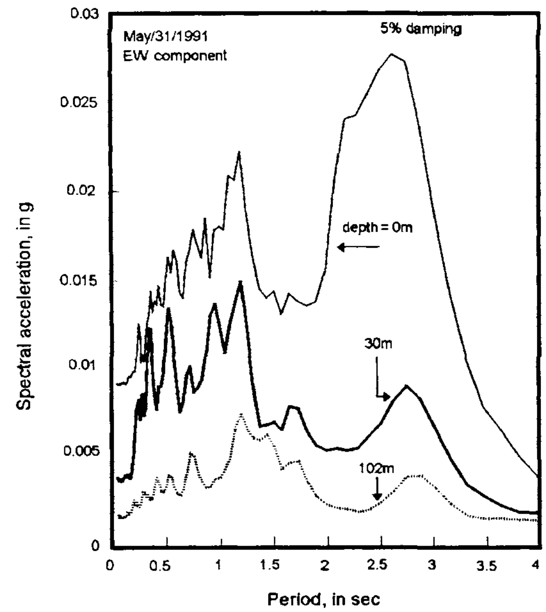

Figure 16. Ground motion variation with depth

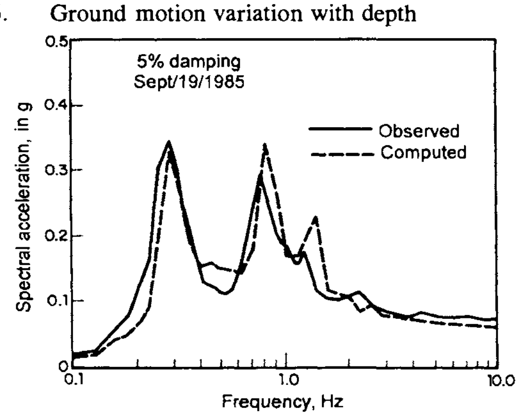

Figure 17. Response spectra at CAO site. Lake zone

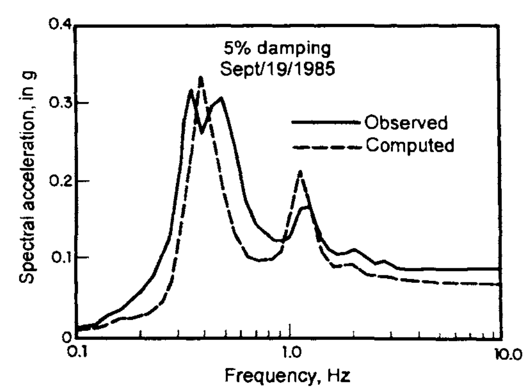

Figure 18. Response spectra at CAF site. Lake zone

length of the recorded motions. Fuerthermore, it was argued that the observed trace at different sites showed a harmonic beating (Sánchez-Sesma et al, 1988; Kawase and Aki, 1989). The first argument was answered by noting that the sensitivity of accelerographs that were operating in the hill zone was not sufficient to record the small amplitudes towards the end of the rock-like motions. Thus, it was postulated that if the hill-zone records were longer, the difficulty in modeling the observed long coda would be overcome. Results of analyses with one dimensional models using records of the hill zone, with durations increased in a somewhat arbitrary fashion, showed that, in effect, if the accelerographs were capable of recording the total duration of the firm ground motions, the durations of motions recorded in the lake zone could be reproduced with one dimensional models (Romo, 1986).

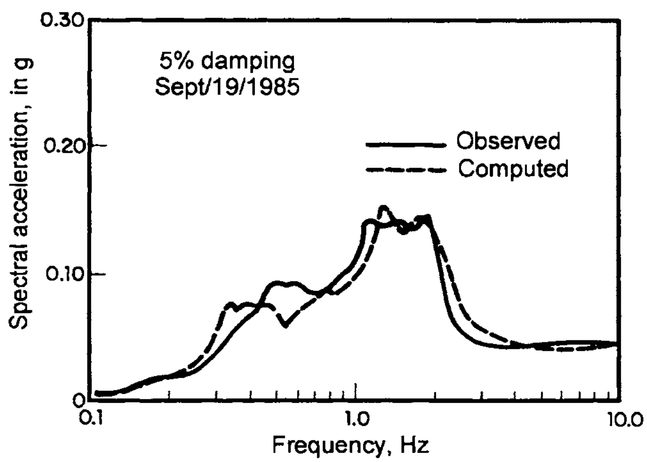

Figure 19. Response spectra at VC. Transition zone

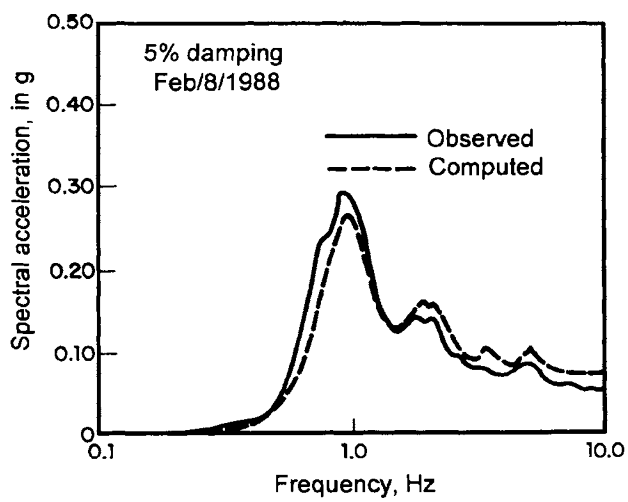

Figure 20. Response spectra at D54 site. Lake zone

At the same time, numerous investigations were carried out to explain the above objections. Several alternative models have been advanced, among them: 1) a two dimensional valley (Sánchez-Sesma et al, 1989); 2) a large-scale two dimensional valley within which a small-scale lake-bed zone is included (Bard et al, 1988; Kawase and Aki, 1989); 3) local small-scale variations in the stratigraphy near the recording sites (Faccionli et al, 1989; Chávez-García and Bard, 1989); and 4) resonance of horizontally propagating P waves in the laterally confined clay layer of the lake zone (Seligman et al, 1989). In a comprehensive study Chávez-García (1991) concluded that the models

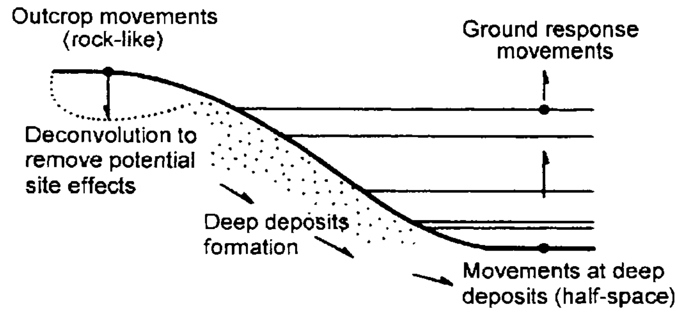

Figure 21. One dimensional modeling for ground response analyses in Mexico City

of the type 1 to 3 mentioned above are not viable unless the shear-wave Q (half of the inverse of the damping ratio) in the lake zone sediments is of the order of 200 to 300. These values correspond to damping ratios ranging from 0.16 to 0.25% which seem a little too low even for Mexico City clays (see figure 10). Model 4 appeals to lateral resonance of P waves partly to avoid a high Q required for S waves in reproducing the long coda.

In a recent paper Singh and Ordaz (1993) have shown that the origin of long coda with beating observed in the lake zone strong-motion records of Mexico City, lies in the characteristics of the ground motions in the hill zone. They conclude that the long coda is most probably composed of multipathing from the source to the site and/or multipathing within the Valley of Mexico. Their explanation eliminates the need for unrealistic low damping ratios in the clayey sediments, complicated two-and three dimensional models and/or small scale variations in the soil deposit properties to increase the coda duration and to reproduce the beating observed in the lake zone sites. An example of their results is shown in figure 22, where they compute the ground response at CAO site using a one dimensional model and the stratigraphic characteristics used in other studies (Romo, 1986). (See velocity profile in figure 15.) The input motion was the record obtained at CU site with a broadband seismograph during a 4.9 magnitude (M_c) earthquake generated by the subduction mechanism near the port of Acapulco. It may be seen that although the beating is not clearly observed in the motion recorded at CU site, it is evident in the computed response at CAO site, indicating that the soil deposit is capable of amplifying hard-to-see details of the rock-like motions.

On the basis of the results and comments included above it may be concluded that the onde dimensional model is adequate enough to evaluate the response of the deposits found within the lake and transition zones of Mexico City. Thus, if properly defined the stratigraphic characteristics and the soil dynamic properties, the definition of site-dependent (free-field) spectra for building design purposes is a feasible task using one dimensional, SH wave propagation procedures. It is important to stress the need of using proper soil site conditions and soil behavior according to what it was presented above under the heading of dynamic behavior of Mexico City clay.

Deep Deposits Seismic Movements

Since 1990 the first accelerographic network comprising instruments installed within boreholes has been operational (Quaas et al, 1990). It includes three component accelerographs located at different depths. On May 31 that same year, a M_c=5.5 subduction earthquake that originated about 300km south from Mexico City was recorded by some of the network instruments located in the deep deposits provinding instrumental evidence on the characteristics of the motions

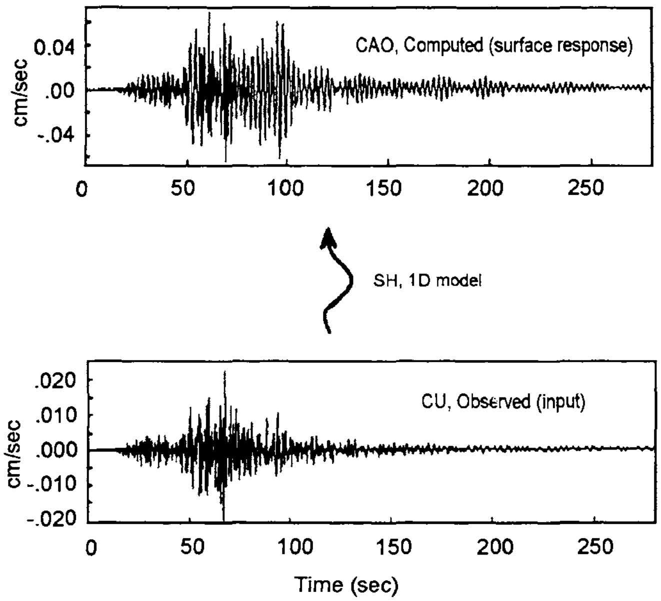

Figure 22. Response computed at CAO site showing beating and long coda (modified from Singh and Ordaz, 1993)

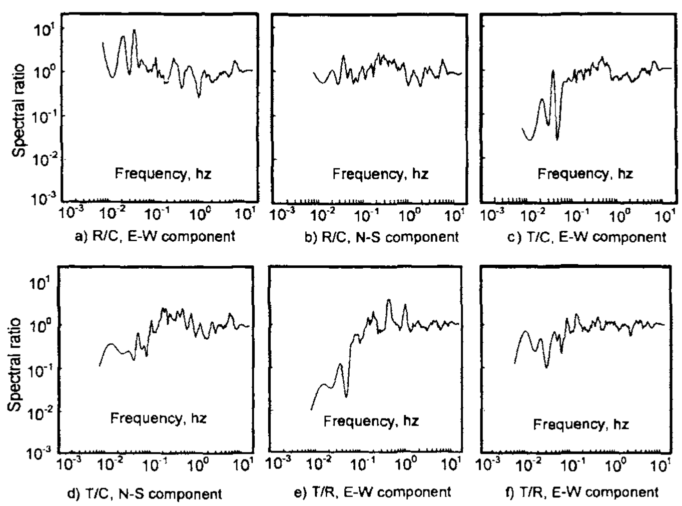

Figure 23. Spectral ratios among various points in the deep deposits, May/31/1990 (taken from Ovando et al, 1992)

in these rock-like deposits. One important aspect that has to be verified regarding the validity of one dimensional modeling is the degree of uniformity of the motions in the deep deposits that are considered the halfspace on which the soil strata lie. To this end, it is a matter of computing Fourier amplitude spectral ratios among the various recording stations. The 31 May event was recorded by three instruments located at 70, 102 and 86m of depth, well into the deep deposits. The approximate horizontal distance between each site is 12km. One of them (V, 70m deep), is located in the transition zone (near CU site) and the other two in the lake zone, one (R, 102m deep) lies within one of the areas in the city that has recurrently been more damaged during severe seismic events (near SCT site) and the other (T, 86m deep) is in the eastern sector of Texcoco Lake in a less densely populated area (close to the international airport).

The spectral quotients obtained from the horizontal components recorded at the above three stations are given in figure 23 (Ovando et al, 1992). These ratios were calculated by dividing the smoothed Fourier amplitude spectra of each pair of signals. The results show that the movements at the base of the three stations are fairly uniform, thus, rendering support to the assumption of uniform movements in the half space implicit in one dimensional modeling.

SOIL STRUCTURE INTERACTION STUDIES

In general, the dynamic interaction phenomenon involves three aspects. First, the spatial variation of the ground movements induced by late wave arrivals tends to be uniformed by stiff foundations; second, the inertial interaction that develops when the foundation soil is compressible may affect significantly the vibration charactersitics of the building; and third, the kinematic interaction that evolves when the foundation is embedded deep into the soil deposits may modify the free field motions and thus the charateristics of the excitation the structure is actually subjected to may differ appreciably from those of the free field.

In the lake zone of Mexico City these three aspects of the interaction phenomenon are most likely present with varios degrees of importance depending on soil, foundation, building and ground motion characteristics. It is therefore a matter of practical importance that potential effects be properly evaluated.

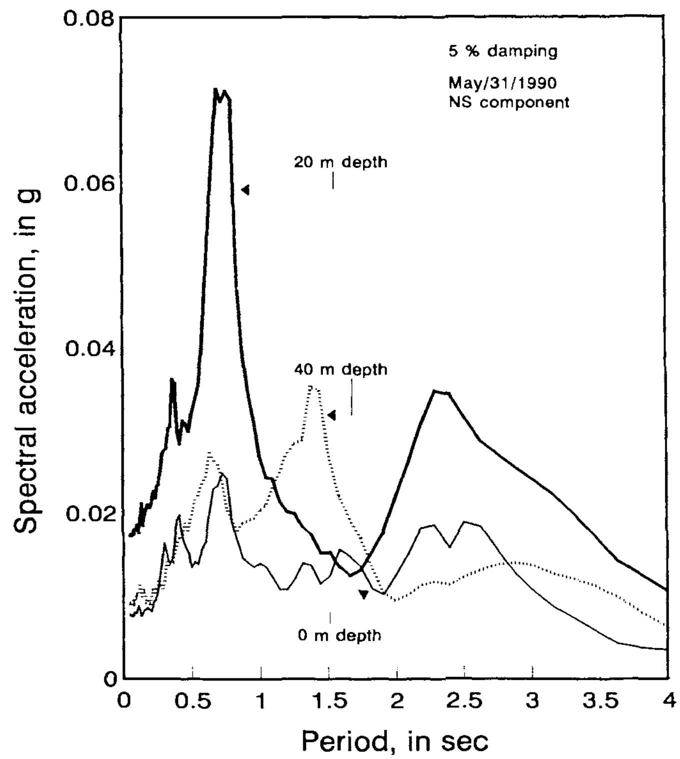

Figure 24. Ground movements at Bernardo Quintana building

Case Histories

Two well documented case histories and the motions recorded at the building base and free field are presented below. Both buildings are located in the lake zone of Mexico City.

1. Bernardo Quintana building. It is an eight story concrete structure supported by a rigid box that is embedded eight meters into the soil deposit. Although the 1985 seismic events did not caused structural damage to the building, it was necessary to reinforce both the

structure and the foundation in accordance to the new seismic coefficients in the 1987 Construction Building Code. Ambient vibration studies carried out before and after the rehabilitation works showed that the frequency of the foundation-building system varied from 1.11 to 1.68Hz in the transversal direction and from 0.86 to 1.19Hz in the longitudinal direction (Rodríguez, 1991).

The seismic instrumentation was carried out in two stages. The first involved the installation of a vertical array of accelerographs into the soil ground located some 10m from the foundation building. The instruments are located at ground level, at 20m of depth in clay material and at 40m of depth in the deep deposits. In the second stage accelerographs were installed at the building base and in three different stories. The second stage was recently accomplished and as to now no records have been obtained in the building.

Since the installation of the vertical array a series of minor earthquakes have been recorded. The most intense was the May 31, 1990 event and thus it is reported here. The acceleration response spectra of the motions recorded at the three elevations are shown in figure 24. It may be seen that spectral ordinates are amplified from 40 to 20m and from 20m to ground surface the motions are significantly attenuated. Even at some interval periods, the ground surface motions are less severe that the corresponding motions in the deep deposits. This significant attenuation of the motions was shown to be due to the kinematic interaction developed between the deep box foundation and the surrounding soil (Romo y Bárcena, 1993).

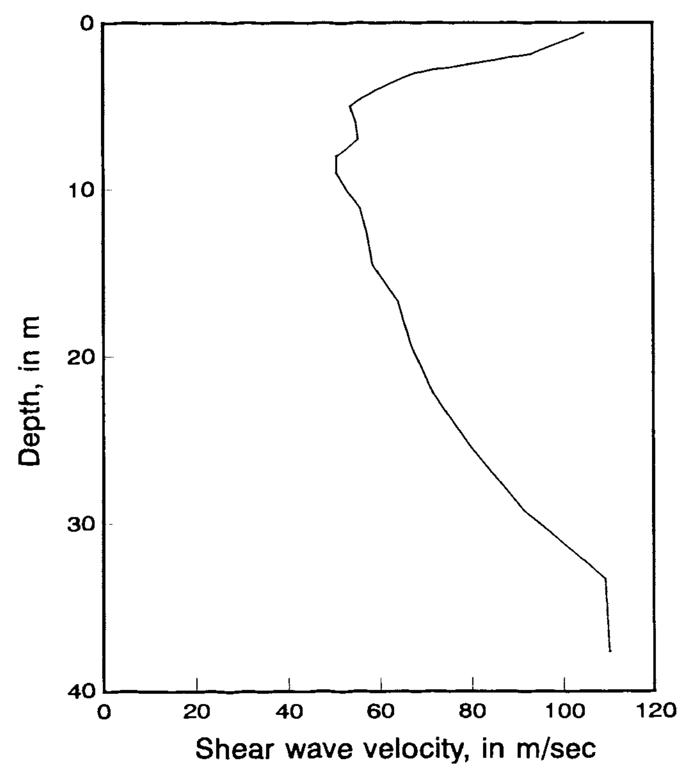

Figure 25. Shear wave velocity profile at Bernado Quintana building site

In order to see if it was possible to reproduce the observed motions a finite element computer code based on random vibration and extreme value theories (Romo et al, 1980) was used to model the soil-foundation-building system. The shear wave velocity profile of the site is given in figure 25. The soil behavior was assumed to follow the response characteristics of Mexico City clays explained above.

The input motion assumed in the analysis was the acceleration response spectrum of the movements recorded at 40m of depth, and the control point was considered at this depth (at the deep deposits, assumed to be the halfspace boundary of the finite element model). The observed and computed responses are compared in figures 26 and 27. The approximation is excellent for the motions at 20m and good for the motions at ground surface.

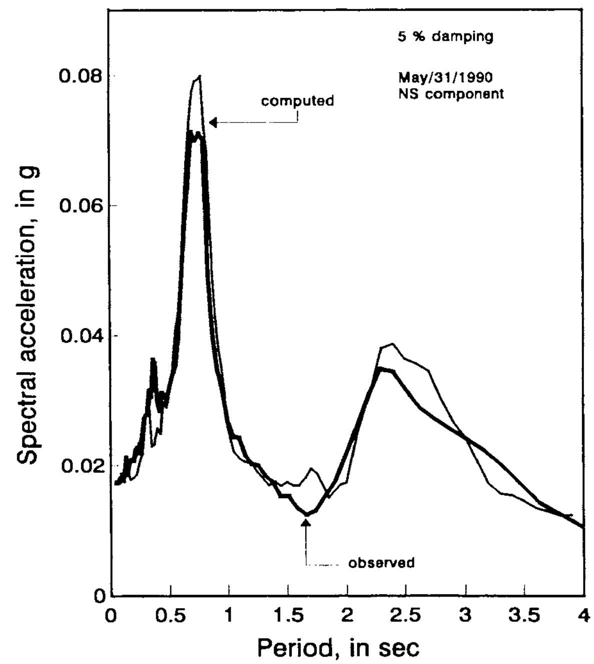

Figure 26. Observed and computed motions at 20m deep at Bernado Quintana building site

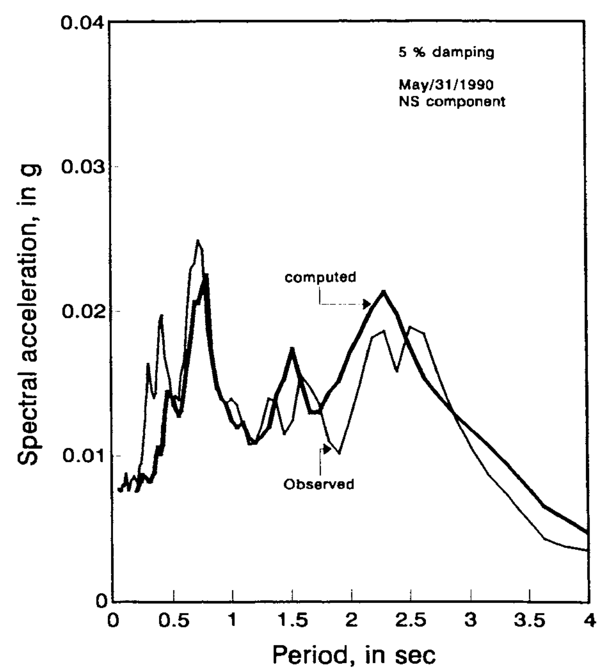

Figure 27. Observed and computed motions at ground surface at Bernardo Quintana building site

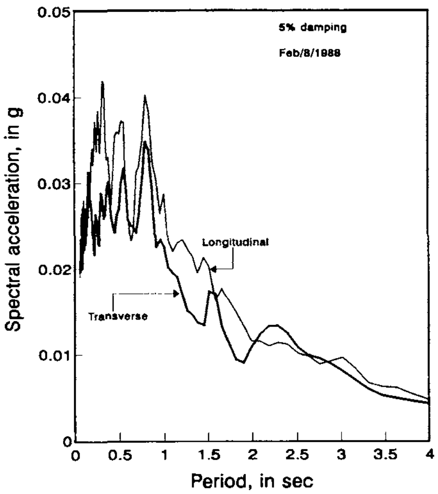

Figure 28. Free field ground motions at Secundaria 3 site

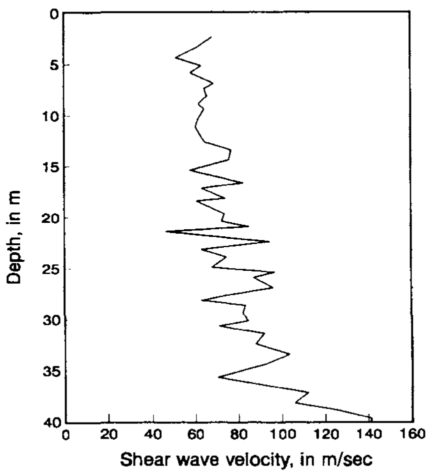

Figure 29. Shear wave velocity profile at Secundaria 3 site

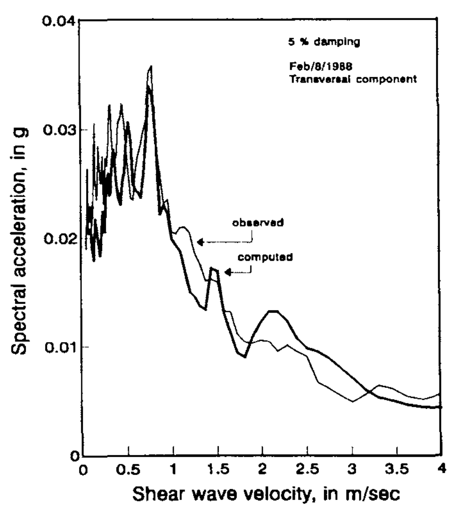

Figure 30. Observed and computed motions at Secundaria 3 building base

2. Secundaria 3 building. It is a three story steel structure with concrete walls supported by a superficial mat foundation. The plan dimensions are 9 by 29m with a 3.05m story height. Ambient vibration studies have shown that the soil-foundation-building has natural periods of 0.36sec and 0.23sec in the longitudinal and transverse directions, respectively. The seismic instrumentation includes an accelerograph at the free field on the ground surface, one instrument at building base and other at building roof. The three instruments have been in operation since 1987. On February 8, 1988 all three acclerographs recorded a 5.7Mc seismic event that was generated by the subduction mechanism in the Pacific coast, some 290km south of Mexico City. The free field motions recorded at this site are shown in figure 28. The spectra show that the energy of this event was concentred mainly on the high frequency range.

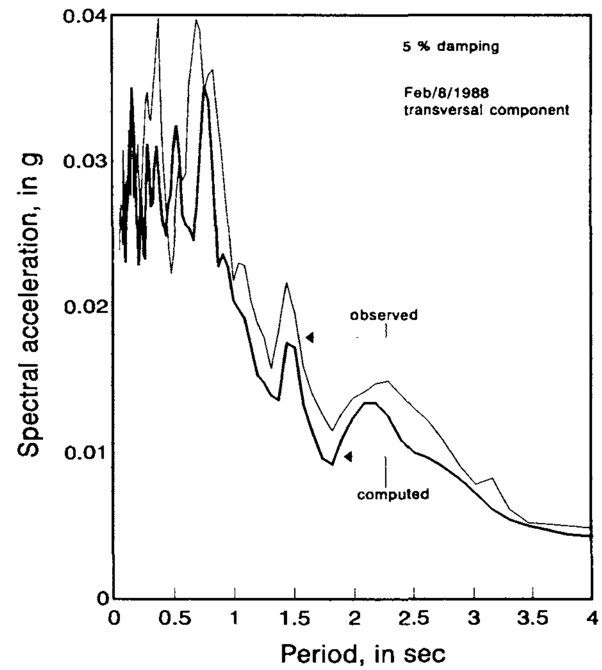

Figure 31. Observed and computed motions at Secundaria 3 building roof

Using the finite element computer code mentioned above (Romo et al, 1980) the interaction problem was studied. The velocity profile of the site is given in figure 29. The base of the model (halfspace) was assigned at the deep deposits (45m of depth), the input motion considered was the observed spectra of the free field (figure 28) and the control point was estipulated at the ground surface of the free field.

In figures 30 and 31 are compared the observed and computed motions (in the transversal direction) at building base and building roof, respectively. The agreement is in general good for the period range analysed. Similar agreement was obtained from the analyses carried out in the longitudinal direction.

The above included comparisons show that with available analytical tools it is feasible to reproduce the interaction phenomenon with sufficient degree of accuracy for practical applications. Thus, it would be expected that the above numerical procedure could be used for the definition of site-specific, building-specific interaction spectra for design purposes.

Theoretical Studies

The finite element method mentioned above has been used to evaluate different aspects related with foundation engineering. A brief account of the main results is given below.

1. Effect of foundation embedment depth. In the studies it was assumed that the foundation was a rigid box embedded at different depths within a clay deposit. The embedment depth effect on maximum ground accelerations is depicted in figure 32. These results show that the severity of the motion decreases as the foundation depth becomes larger.

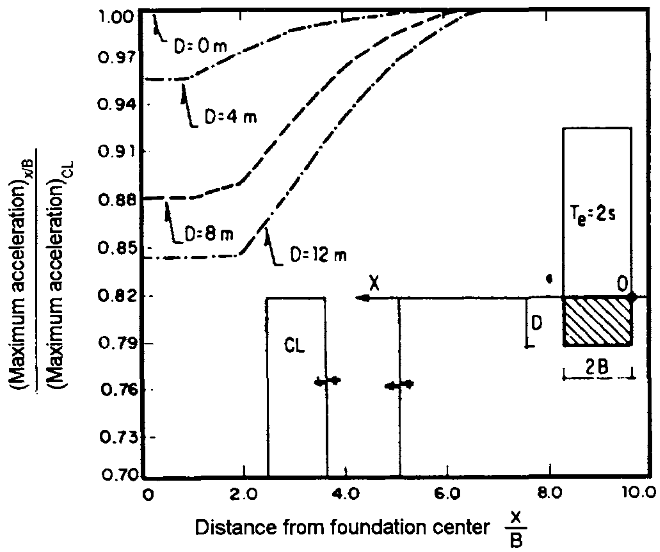

Figure 32. Effect of foundation depth on ground motion

2. Effect of foundation type. Two types of foundations were considered. One was a box-type caisson (2.5m deep) with friction piles (23m long). The other was a 14m deep rigid box-type foundation. Both foundations were equivalent from the static design point of view. The soil deposit had a natural frequency of 0.43Hz and the structure a natural frequency of 1.0Hz.

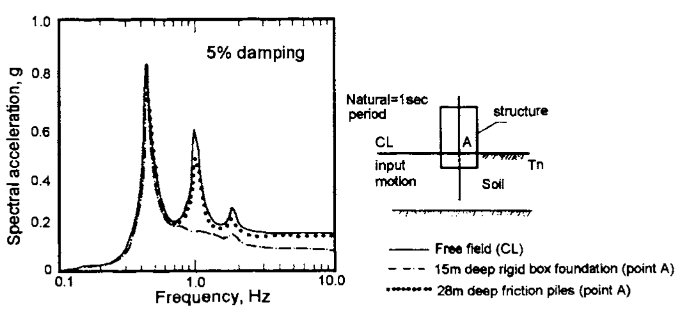

Figure 33. Effect of foundation type on floor response spectra

The results of the finite element analyses, shown in figure 33, indicate that the friction-pile foundation does not modify significantly the free field motions. On the other hand the deep box foundation attenuates the base building motions appreciably as compared to the free field movements. This points out at the fact that friction piles move with the soil, hence the free field motions are barely affected by the presence of the structure. Alternatively, the rigid box effectively interacts (kinematic interaction) with the soil, producing significant reductions in the motions at the natural (and larger) frequency of the building.

On the basis of the above results it may be concluded that deep, rigid box-type foundations are a better solution than friction piles in clayey deposits located in seismic zones like Mexico City. The box-type foundation not only provides the building with important fixity at its base but attenuates the severity of the motions that act upon the base of the building.

CONCLUSIONS

Extensive field and laboratory investigations have provided significant amount of valuable information that shows different aspects of the dynamic behavior of Mexico City clays. On one hand, it has been possible to establish reliable correlations between CPT results and shear wave velocity measurements, providing the profession with an economic procedure to estimate the velocity profiles required in ground response and soil-structure interaction analyses. On the other, from the laboratory results it has been possible to obtain expressions to compute the low-strain and large-strain shear modulus. It is shown that G/G_{max}-γ curves are mainly dependent of the plasticity index, I_p, and the relative consistency, I_r, of the clayey materials. Likewise, an analytical relationship is proposed to compute damping ratios as a function of shear strains and G/G_{max} values. The results show that, contrary to what has been dubbed as atypical behavior, Mexico City clays constitute what seems to be the upper and lower boundaries of clay dynamic behavior. Similar conclusions have been reached regarding the static behavior of Mexico City clays (i.e. Mendoza and Romo, 1991).

Observations of rock-like seismic motions in recent earthquakes show that the long coda present in several records obtained in the lake deposits has its origin precisely in the motions that arrive at the Valley of Mexico via multipathing of long-period surface waves, through a predominantly oceanic path as explained by Ewing et al (1957)) and Capon (1970). These observations and others indicating that the rock-like motions at the deep deposits are fairly uniform provide further support to the validity of one dimensional modeling for response analysis of the soil deposits in Mexico City.

The study of two case histories has shown that it is feasible to reproduce their seismic response using available finite element codes based of random and extreme value theories. Accordingly, a general procedure to compute site-dependent, building-specific seismic environments in Mexico City is devised and advanced in this paper. This procedure constitutes a relatively simple tool that may be used in practice to designing the foundation in accordance with the site specific seismic conditions. This philosophy design has proved to render economic benefits in the total costs of buildings located in the compressible clays of the lake zone. It would be expected that similar benefits be attained for buildings founded on the erratic deposits of the transition zone and other parts of the world where the general geotechnical and seismic conditions are similar to those prevailing in the Valley of Mexico.

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