International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

30 Mar 2001, 1:30 pm - 3:30 pm

Evaluation of Mexico City Clay Dynamic Properties Using a Parameter Identification Approach

Juan C. Carvajal U.
Instituto de Ingeniería, UNAM, Mexico

Víctor M. Taboada U.
Instituto de Ingeniería, UNAM, Mexico

Miguel P. Romo O.
Instituto de Ingeniería, UNAM, Mexico

Recommended Citation

Carvajal U., Juan C.; Taboada U., Víctor M.; and Romo O., Miguel P., "Evaluation of Mexico City Clay Dynamic Properties Using a Parameter Identification Approach" (2001). *International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. 25.
https://scholarsmine.mst.edu/icrageesd/04icrageesd/session03/25

EVALUATION OF MEXICO CITY CLAY DYNAMIC PROPERTIES USING A PARAMETER IDENTIFICATION APPROACH

Juan C. Carvajal U.
Assistant Researcher
Institute of Engineering-UNAM
México D.F. – MÉXICO

Víctor M. Taboada U.
Research Professor
Institute of Engineering-UNAM
México D.F. – MÉXICO

Miguel P. Romo O.
Head Research Professor
Institute of Engineering-UNAM
México D.F. – MÉXICO

ABSTRACT

Laboratory-determined soil dynamic properties are always (to different degrees) affected by sample disturbance, scale effects, deficient modeling of in situ conditions, and so on. The installation of vertical arrays of strong motion instruments and the ensuing records obtained during various seismic events, have opened the opportunity to explore other alternatives to evaluate soil dynamic properties by solving the inverse problem. In this paper, an analytical procedure that allows the solution of this problem in a simple way is presented and applied to a case history in Mexico City. The model assumes 1-D propagation of shear waves throughout homogeneous viscoelastic soil deposits. The results obtained here are compared with the velocities measured by means of field studies at Central de Abasto Oficinas (CAO) site with a P-S logging system. These comparisons show the potential of this procedure.

INTRODUCTION

The dynamic seismic response of a soil deposit depends on the material dynamic properties, the deposit geometry and the earthquake excitation characteristics. Herein, the response of the ground is computed using a 1-D wave propagation model and assuming a homogeneous, viscoelastic soil deposit. These approximations have been shown to yield adequate results when dealing with the deep clay deposits in Mexico City [e.g. Romo and Seed, 1986; Romo, 1995]. Regarding the evaluation of soil dynamic properties, it is common practice to resort either to laboratory or field test. As discussed in Romo et al [2000], these procedures involve uncertainties that may lead to erroneous evaluations of shear modulus and damping ratio values.

A procedure that has been gaining popularity and the approval of the profession in recent years is based on the observation of the response of sites properly instrumented. Once the ground motions are known at various points within the soil deposit for different earthquakes, the equivalent properties of the materials may be estimated by solving the inverse problem (system parameter identification). There exist a number of procedures to solve the inverse problem (e.g. Ljung, 1987).

In this paper, a simple procedure to achieve this purpose is proposed. It basically consists in comparing the experimental and analytical responses. Once the amplitude Fourier spectrum of the response is computed, it is compared with the corresponding spectrum of the measured response. Then, an optimization procedure is activated that is based on the minimization of the overall error between both spectra. In doing this, sets of shear modulus (G) and soil viscosity (η) are varied until the minimum error is obtained. The corresponding values are the in situ equivalent properties.

Herein, this approach is applied to a case history in Mexico City (Central de Abasto Oficinas - CAO) and the results are discussed.

CAO SITE

This site is located according to the Mexico City geotechnic zoning within the lake zone, as indicated in Fig 1.

The vertical array at CAO site consists of a superficial accelerometer and three more located at depths of 12, 30 and 60 m. The horizontal distance between the surface accelerometer and the downhole accelerometers is 3 m and the horizontal distance between the downhole accelerometers (12, 30 and 60 m) is 1 m. The identification codes of these instruments according to the Mexican Database of Strong Earthquakes (BMDSF, 1997) are: CDAO, C166, C266 and C366, respectively. The downhole accelerometers are installed inside aluminium tubes of flexible walls without vertical confinement (e.g. borings were not backfilled). This arrangement has been shown to be advantageous because it allows the retrieval of the instrument for maintenance purposes. Furthermore, the aluminium casing is flexible enough to follow ground motions and preclude casing-soil interaction effects on recorded accelerations.

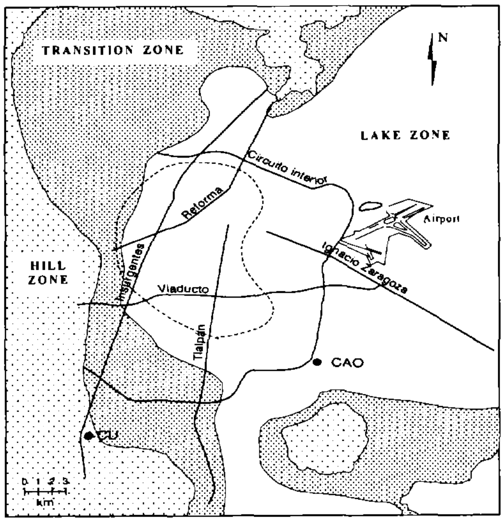

Fig. 1. Mexico City geotechnical zoning.

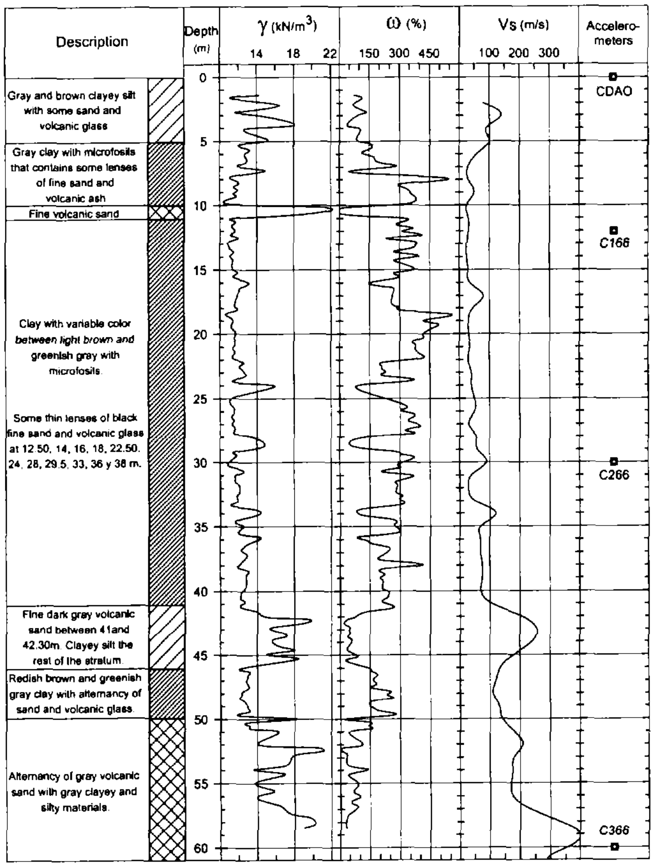

Fig. 2. Stratigraphic profile of the CAO site.

DYNAMIC RESPONSE OF CONTINUUM

The analytical model that represents the undimentional propagation of transversal waves in a viscoelastic continuos medium is:

$$G\frac{\delta^2 x}{\delta z^2} + \eta\frac{\delta^3 x}{\delta z^2 \delta t} = \rho\frac{\delta^2 x}{\delta t^2} \qquad (1)$$

where G is the shear modulus [MPa], η is the viscosity coefficient [MPa*s], ρ is the medium density [kg/m^3], x is the transversal displacement [m] and z is the coordinate along the wave train propagates [m]. The solution is:

$$x_{(z,t)} = Ae^{i(\omega t+K^*z)} + Be^{i(\omega t-K^*z)} \qquad (2)$$

where A and B are the amplitudes of the transversal waves that go in the z negative and positive directions, respectively and ω is the circular frequency of the harmonic excitation.

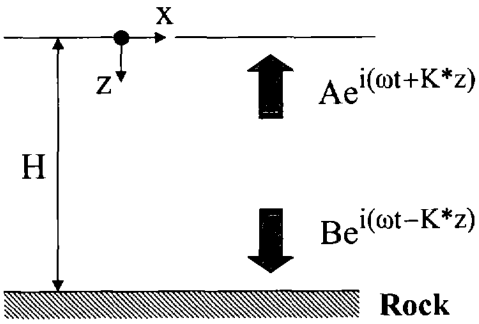

Fig. 3. Schematic interpretation of Equation 2.

K* is the complex wave number and is defined as follows:

$$K^* = r_z^{1/2}\left[\cos(\phi_z/2) + \sin(\phi_z/2)i\right] \qquad (3)$$

where

$$r_z = \omega^2\rho \Big/ \sqrt{G^2 + (\omega\eta)^2} \qquad (4)$$

and

$$\phi_z = \tan^{-1}\left(-\omega\eta/G\right) \qquad (5)$$

This expression is slightly different from that adopted in soil dynamics. As seen in Eq 3, K* is expressed in terms of η (consequently in terms of ω), instead of ξ as it is done conventionally. In doing this, it is attempted to consider directly the fundamental parameters of the soil: G and η. Note (see Eq 6) that since ξ depends on the frequency, consequently it is not a fundamental soil parameter.

$$\xi = \frac{\omega\eta}{2G} \qquad (6)$$

To solve the wave equation it is assumed that the stratum is homogeneous and the upper boundary is a free surface and the lower boundary is rigid, as is shown in Fig 3. Therefore, the equation that determines the lateral displacement, x, at any point z of the medium is:

$$x_{(z,t)} = 2A\cos(K^*z)e^{i\omega t} \qquad (7)$$

and represents a stationary transversal wave of amplitude equal to 2Acos(K*z).

Equation 7 implies that points of the medium along z are not out of phase in time with respect to the excitation. This solution also establishes that the excitation must be harmonic with amplitude A, so the response will be a harmonic with amplitude 2Acos(K*z). Taking into account this, the response at a point of the medium at z=h, can be easily obtained using the transfer function, FT:

$$x_{(z=h,t)} = (FT)\,Ae^{i\omega t} \qquad (8)$$

$$FT = \cos(K*h) \big/ \cos(K*H) \qquad (9)$$

where $Ae^{i\omega t}$ is the excitation, which can also be $Ae^{i(\omega t+\alpha)}$, if it has an initial phase α different from zero.

The transfer function FT depends on the viscoelastic parameters, medium density, excitation frequency, location of the excitation control point and depth where the response is estimated.

In view that all data is been collected in terms of acceleration time histories, it is convenient to obtain the response in terms of accelerations. To achieve this, Eq 8 is derived twice with respect to time.

Thus, the expression used to compute de accelerations at any depth and time in given by

$$\frac{\delta^2 x_{(z=h,t)}}{\delta t^2} = (FT)\,Be^{i\omega t} \qquad (10)$$

The amplitude of the excitation in terms of the acceleration is B, which is equivalent to $-A\omega^2$. Note that FT is time invariant.

The process followed to evaluate the analytic response of a soil deposit is schematically indicated in Fig 4.

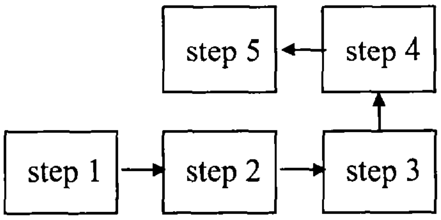

Fig. 4. Process to evaluate the analytic response.

The first step is to assign the input motion in the time domain. Then, in step 2, this signal is mapped to the frequency domain applying the Discrete Fourier Transform. In step 3 the transfer function is computed using Eq 9. Then, in step 4 this transfer function is multiplied by the input motion to obtain the response in the frequency domain. Finally, step 5 performs the transformation of the response computed in step 4 to the time domain.

It is important to stress that the control point of the input motion not necessarily has to be assigned at the rigid boundary. It may be considered at any depth within the soil deposit.

PARAMETER IDENTIFICATION SYSTEM PROPOSED

To determine the dynamic properties of the soil deposit, a Dynamic Parameter Identification System, DPIS, was implemented.

The DPIS takes the input record that corresponds to the excitation and calculates the theoretical response as indicated in Fig 4, considering a pair of values of G y η. Then compares the theoretical response with the recorded response in the frequency domain and estimates an error based on the differences in areas (for each frequency increment) between the amplitude Fourier spectra of the recorded and computed motions. The absolute value of this difference is always considered.

Applying the same procedure, the theoretical response is calculated for G values between 0.1 and 50 MPa and for η between 0 and 2.0 MPa*s. The optimum dynamic parameters are considered as those that induce the minimum error between the theoretical response and the one measured in the field. Although the soil density was considered constant in this study, it can easily be included in the optimization procedure.

The DPIS calculates the transfer function, considering the dynamic parameters constant with the frequency. It is worth mentioning that this parameter identification system does not need to determine angular strains, because they are the product of the dynamic parameters of the medium and the excitation, therefore, the system identifies the parameters based on the variables measured in the field (e.g. accelerations).

CAO SITE CASE HISTORY

To evaluate the dynamic properties of CAO site clays, the recorded motions by instruments C166 and C266 were processed. Both horizontal components were considered. Seismic events included here are shown in Table 1. The location of C166 and C266 are depicted in Fig 2. The unit weight considered in the analyses was the average (12 kN/m^3) value obtained from the unit weight profile included in Fig 2.

It is worth mentioning that there are some thin lenses of volcanic ash embedded in the clay layer. This could affect somehow the evaluation of the dynamic properties of the clay reported in this paper.

It is also important to point out that due to the low intensity of the earthquakes, the amplitude of the motions recorded at 30 m of depth were very small. This may have caused that in some instances, the sensibility of the sensors was not adequate to pick clear firms.

To eliminate spurious frequencies caused by the "square pulses", a "box-type" filter was used to filter out frequencies below 0.1 Hz and above 6.0 Hz. In doing this, one may be confident that the potential effects of these "square pulses" have negligible impact on the analyses reported below.

The idealization of CAO site for the analyses carried out in this study is show in Fig 5.

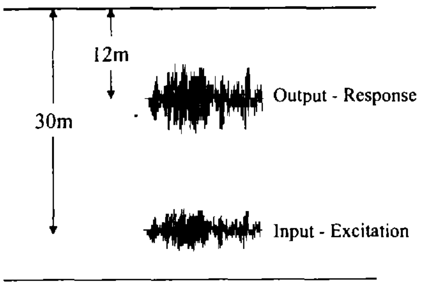

Fig. 5. Idealization of the CAO site according to the DPIS.

Some of results obtained from the analyses are included in Figs 6 to 17.

Figures 6 to 9 show (in time and frequency domain) the motions recorded at 30 m and 12 m of depth, and in Fig 10 and Fig 11 the responses computed with the procedure proposed are included.

Comparing the results in these two figures with those where the measured responses are depicted, it may be seen that the analytic approach reproduces with high degree of accuracy the actual responses.

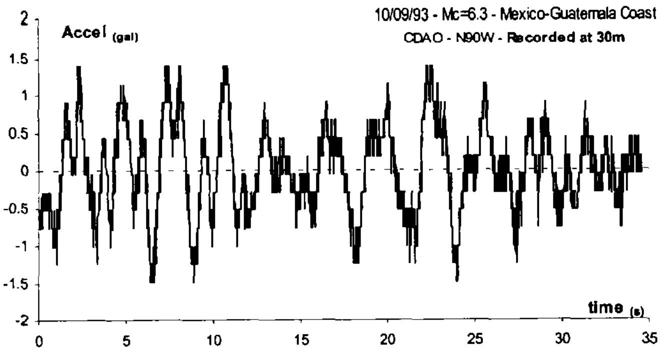

Fig. 6. Record in time domain at 30 m – 10/09/93.

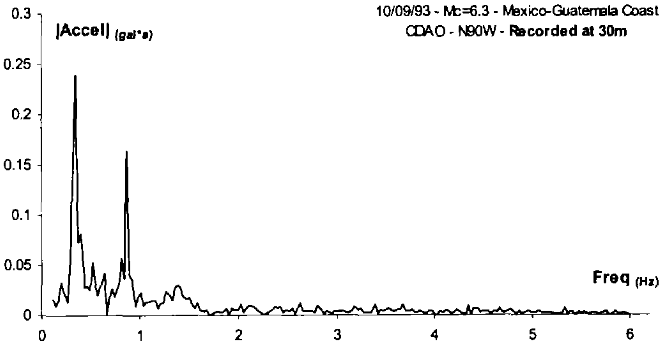

Fig. 7. Record in frequency domain at 30 m – 10/09/93.

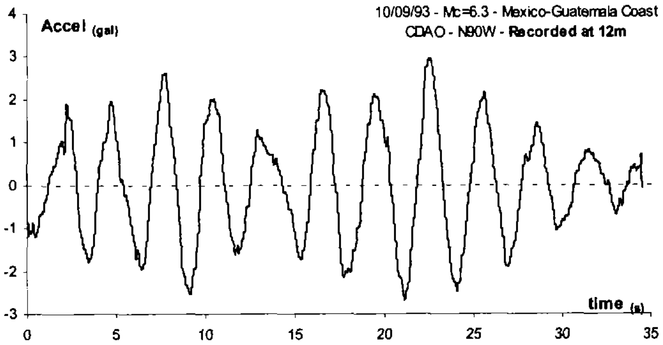

Fig. 8. Record in time domain at 12 m – 10/09/93.

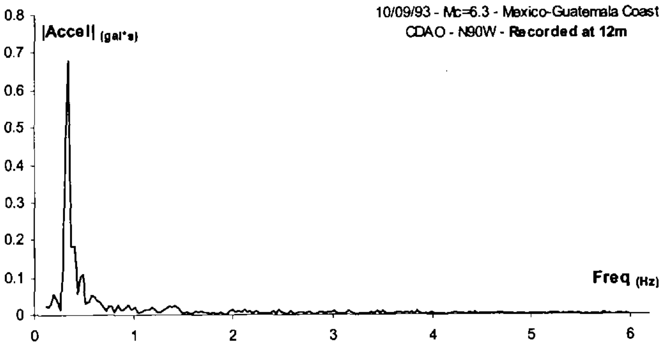

Fig. 9. Record in frequency domain at 12 m – 10/09/93.

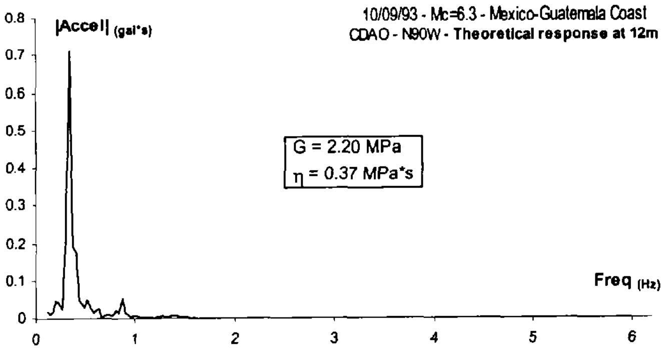

Fig. 10. Response in frequency domain at 12 m – 10/09/93.

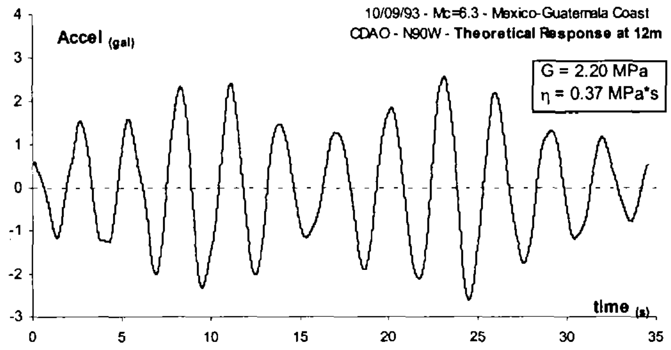

Fig. 11. Response in time domain at 12 m – 10/09/93.

With the purpose of evaluating further the procedure, it was applied to the same site subjected to another excitation. The results of this exercise are included in Figs 12 to 17.

Comparisons between computed and actual responses indicate that the procedure reproduces fairly well the field measurements.

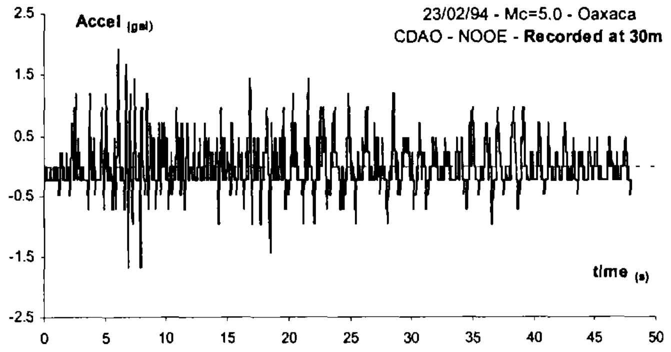

Fig. 12. Record in time domain at 30 m – 23/02/94.

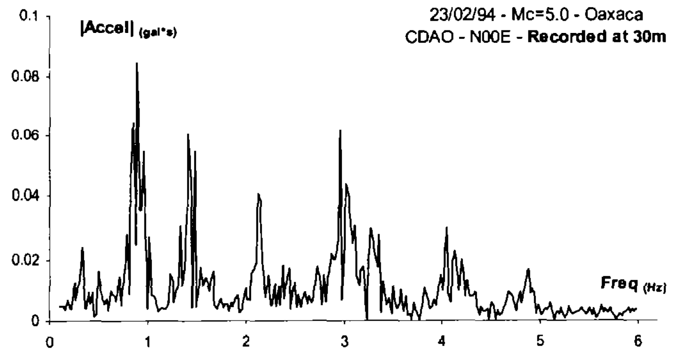

Fig. 13. Record in frequency domain at 30 m – 23/02/94.

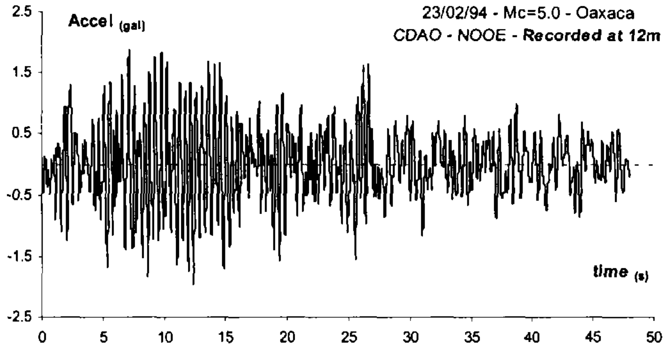

Fig. 14. Record in time domain at 12 m – 23/02/94.

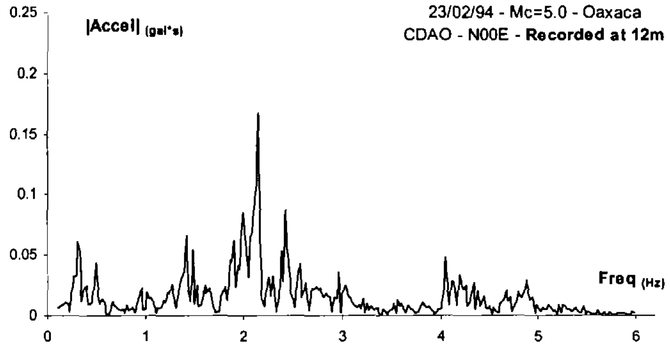

Fig. 15. Record in frequency domain at 12 m – 23/02/94.

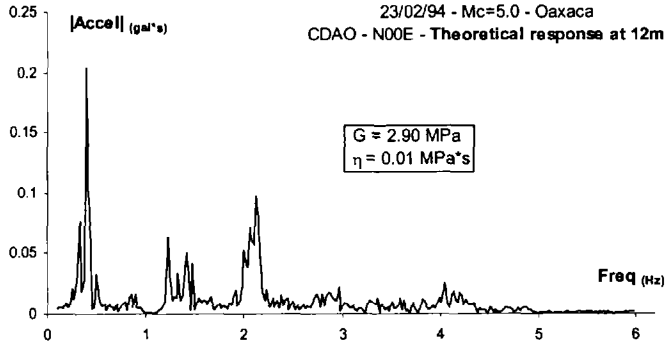

Fig. 16. Response in frequency domain at 12 m – 23/02/94.

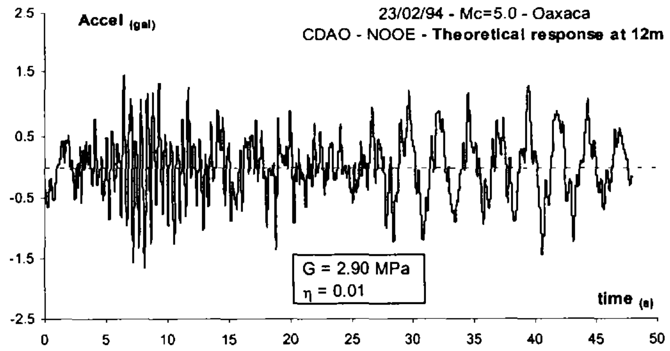

Fig. 17. Response in time domain at 12 m – 23/02/94.

From these examples and ten more (see Table 1) not included here due to space limitations, it was observed that when the input motion had low frequency content (as that of Fig 7), the predicting capability of the procedure was higher than in the cases where the input motion had high frequency content (as that of Fig 13). Since this was systematically observed, it may be argued that the explanation for these discrepancies could be found in the dependence of G on the frequency. Aspect that has been observed in laboratory testing for high frequencies and supported by preliminary results obtained in the curse of this investigation. It seems that the G is affected even for the range of frequencies found in earthquakes. This should not be considered as a big surprise for Mexico City clays, since their behavior is highly rate dependent due to their notorious viscous properties. Thus, it may be expected that the effect of input frequency content on shear modulus be plasticity index dependent.

The optimum shear modulus and viscosity values obtained from the twelve case histories are included in Table 1.

Table 1. Clay dynamic properties obtained of CAO site.

Event	Component	G (MPa)	η (MPa*s)
31/03/93	N00E	2.8	0.01
	N90W	2.9	0.01
10/09/93	N00E	2.6	0.11
	N90W	2.2	0.37
23/02/94	N00E	2.9	0.01
	N90W	3.2	0.03
04/07/94_1	N00E	2.5	0.01
	N90W	2.6	0.01
04/07/94_2	N00E	3.3	0.12
	N90W	3.5	0.04
30/10/95_1	N00E	3.1	0.02
	N90W	1.8	0.05

The corresponding average values are $\overline{G}$ = 2.78 MPa and $\overline{\eta}$ = 0.07 MPa*s. This $\overline{G}$ agrees well with the average value of G (2.5 MPa) computed from the shear wave velocity profile (between 12 m and 30 m) depicted in Fig 2.

CONCLUSIONS

The method proposed in this paper is based on a 1-D wave propagation model. Thus, should other wave types be of importance in modeling the phenomenon, the approach is not applicable. However, for the case of the lake zone in Mexico City the 1-D hypothesis has been shown to hold.

According to the results included in this paper it may be concluded that the procedure proposed to estimate the equivalent in situ dynamic properties of Mexico City clays yields accurate estimations of the shear modulus. Thus, it represents an alternative for the evaluation of this property. Regarding the viscosity, the values computed with this procedure fall within the range of magnitudes of clay viscosity determined from laboratory tests (e.g. Garcia, 1995).

On the basis of the studies reported here, it may be argued that even for frequencies within the range of values found in most earthquakes, the dynamic shear modulus of Mexico City clays vary also with the frequency. Preliminary analyses of recorded responses indicate that the shear modulus increases with ground motion frequency. Which conforms with the general behavior of viscoelastic soils.

ACKNOWLEDGMENTS

This research was supported by the National Council of Science and Technology (CONACYT), and the National University of Mexico (UNAM), under the program PAPIIT. This support is very gratefully acknowledged.

REFERENCES

Alcántara, L. [1997]. *Base Mexicana de datos de sismos fuertes*. Universidad de Colima, México.

García, L.H. [1995]. *Reproducción de vibraciones en arcillas mediante modelos viscoelásticos*. Tesis de maestría. UNAM, México.

Kramer, S.L. [1996]. *Geotechnical earthquake engineering*. Prentice-Hall, USA.

Ljung, L. [1987]. *System identification, theory for the user*. Prentice-Hall, New Jersey, USA.

Newland, D.E. [1975]. *An introduction to random vibrations and spectral analysis*. D.E. Newland, London, England.

Romo, M.P. and Seed, H.B. [1986]. *Analytical modeling of dynamic soil response in the Mexico earthquake of September 19, 1985*. Proceedings of the ASCE Specialty International Conference on The Mexico Earthquakes – 1985, pp. 148-162, Sep 19, Mexico.

Romo, M.P. [1995]. *Clay behavior, ground response and soil-structure interaction studies in Mexico City*. State of the Art paper, Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis Missouri, Vol. 2, pp. 1039-1051, April.

Romo, M.P., Mendoza, M.J. and García, S.R. [2000]. *Geotechnical factors in seismic design of foundations*. State of the Art report, Proc. 12th World Conference on Earthquake Engineering, Paper # 2832, Auckland, New Zealand, February.

Thomson, W.T. [1982]. *Theory of vibration with application*. Prentice-Hall, Mexico.