ENVIRONMENTAL GEOTECHNOLOGY

3rd. INTERNATIONAL SYMPOSIUM

SAN DIEGO, CAL. JUNE 9-12, 1996

F-DEPFI

Simposium 3

1996

3rd INTERNATIONAL SYMPOSIUM on ENVIRONMENTAL GEOTECHNOLOGY

Holiday Inn on the Bay
June 9-12, 1996

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KEYNOTE SPEAKERS:

Dr. James Mitchell, Via Professor of Civil Engineering, Virginia Polytechnic Institute and State University will deliver the Keynote address for the Opening Session on June 10, 1996 on the topic: "Geotechnology - The Environmental Prospective."

Dr. Leonardo Zeevaert, Professor Emeritus of Civil Engineering, Universidad Nacional Autonoma de Mexico, Mexico, D.F. will deliver the Special Technical Lecture on June 11, 1996 on the topic: "The Seismic-geodynamics in the Design of Foundations in Difficult Subsoil Conditions."

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SCHEDULE:

June 9 (Sunday)	Registration and Welcome Reception.
June 10 (Monday)	Registration, Opening Session Address by Dr. James K. Mitchell, Technical Sessions, Evening Free.
June 11 (Tuesday)	Opening Session Address by Dr. Leonardo Zeevaert, Technical Sessions, Evening Free.
June 12 (Wednesday)	Panel Discussions, Closing Banquet.

OFFICIAL LANGUAGE:
The official language of the symposium is English.

S A N D I E G O

THE SEISMIC-GEODYNAMICS IN THE DESIGN OF FOUNDATIONS IN DIFFICU LT SUBSOIL CONDITIONS

Leonardo Ze[illegible]aert

C O N T E N T S

L I S T   O F   F I G U R E S

THE SEISMIC-GEODYNAMICS IN THE DESIGN OF FOUNDATIONS IN DIFFICULT SUBSOIL CONDITIONS

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ABSTRACT: The experience in the misbehavior of foundations because of destructive earthquakes calls for the necessity to develope methods in engineering practice to verify the order of magnitud of the seismic stability of foundations. Since, the foundation is responsible for the good behavior of the building

The history of foundations behavior has demostrated to the engineers the need to give priority to the soil seismic analysis for building foundations during the final design

The paper presented has the aim to give the foundations designer a practical tool to analyse the seismic stability of foundations. The paper contains statistical values of soil dynamic parameters in case of Mexico City subsoil. They were obtained by the author during a long time in the design practice of foundation engineering. The methods of foundation computation given in the paper have been used by the author successfully during many years

The seismic configuration in the subsoil of strains, stresses, accelerations and displacements with depth are treated

An example of the action of the seismic waves on a box type foundation is analysed from a practical engineering point of view

The engineers designing foundations have the need to know the order of magnitud of the seismic wave action in the subsoil, and learn on their effects at the ground surface and building foundations in relation to the seismic soil strength. The importance of the "Seismic

Pore Water Pressure" is emphasized. Since, this pressure is responsible for the dual effect in the reduction of the confining soil stresses and the bearing capacity in soils containing silt and fine sand

I) INTRODUCTION

The experience in the foundation behavior because of the action of the seismic waves in earthquake areas can not be overlooked in the design of building foundations.(1)

The need to learn on this complicated problem from the practical engineering point of view, is important to the foundation engineer to be able to vizualize the order of magnitud of the strains, stresses and displacements in the soil mass created by the seismic waves.

The subsoil behavior is of primary importance to analyse the foundation and building behavior, since no matter how well and strong the foundation and building structure are designed, when the supporting soil fails because of seismic wave action, the foundation and structure will also fail from the assumed forseen behavior.

II EARTHQUAKE MAIN CHARACTERISTICS

The characteristics of an earthquake are highly dependent on the subsoil physical conditions and soil dynamics properties. This statement may be easily understood if we compare the history of seismic records of the ground surface response in different earthquake areas.

The two main body seismic waves originated by the earthquake at the focus, travel out from the generation areas as a train of waves with a specific celerity and variable orbital physical characteristics which are a function of the subsoil stratigraphycal and soil physical conditions.

In firm ground the waves travel at a high celerity with short wave periods and corresponding orbital velocities and accelerations. In contrast, in soft subsoil conditions they travel with low celerities and large periods and orbital accelerations.

Consequently, because of the conservation of energy, an important action takes place when the seismic waves pass gradually from a rigid soil medium (n) to a soft medium (m). In fact, the celerities and orbital velocities are for energy transfer in the following proportion.[2,3]

$$(\rho C)_n \cdot (V_n^2 - \bar{V}_n^2) = (\rho C)_m \cdot V_m^2 \qquad (1)$$

Where $\bar{V}_n$ is the orbital velocity of the reflected wave induced at the boundary between the two mediums, for a gradual energy transfer we obtain the ratio of accelerations between the surface (a_S) and the rigid base stratum (a_B). We can write, according to equation[(1)] for a stratified subsoil, and from the rigid base up, the following:

$$\left(\frac{a_S}{a_B}\right)^2 \cong e^2 \cdot \frac{(\rho C_B)}{(\rho C)_1} \cdot \frac{(\rho C)_1}{(\rho C)_2} \cdot \frac{(\rho C)_2}{(\rho C)_3} \cdots \frac{(\rho C)_n}{(\rho C)_S} = \frac{(\rho C)_B}{(\rho C)_S} \cdot e^2$$

Therefore, from the above expression we obtain

$$\frac{a_S}{a_B} \cong e \sqrt{\frac{(\rho C)_B}{(\rho C)_S}} \qquad (2)$$

Where (e) is a coefficient of effective energy transfer, furthermore we define.

$(\rho C)_S$, the unit mass and celerity of the wave in the surface soft soil stratum

$(\rho C)_B$, the unit mass and celerity of the wave in the base rigid stratum

Therefore, since the ratio $(\rho C)_B/(\rho C)_S > 1$, hence it is obtained also: $a_S/a_B > 1$. Notice, that this ratio may be variable in a region depending on the wave transfer from the base stratum to the soft soil deposit. However, from the above physical consideration we obtain an approximate idea of the phenomenom involved. The acceleration in the soft medium increases importantly. This phenomenon takes place in the lacustrine area of the soft and high deformable sediments of the subsoil of the Valley of Mexico.The seismic waves arriving at the edge of the Valley in the hard soils pass gradually to the soft subsoil increasing their acceleration. The ratio of the acceleration records of soft to firm ground shows a value on the order of two or somewhat greater.One has to take in consideration that Mexico City typical silty clay behaves quasi-elastic up to high stress levels(2). Therefore, it is an ideal medium for wave transmission, Fig 1, shows tentative ratios of accelerations and masses vs celerities for different values of (e),for a ratio of unit masses on the order of 1.5. For Mexico city subsoil the value of (e) ranges on the order of 0.8 - 0.9.

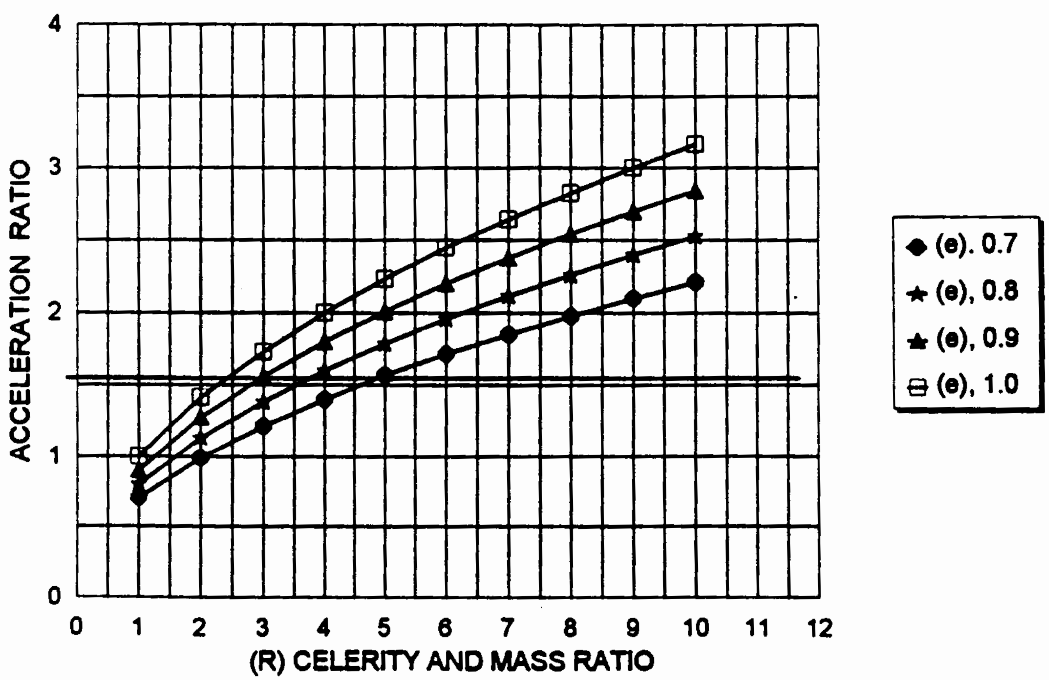


FIG 1. ACCELERATION RATIOS vs CELERITY RATIOS
(e) COEFFICIENT OF ENERGY TRANSFER

Other important characteristics of the earthquake may be noticed in the history of the accelerograms registred at the ground surface.The effective energy of the earthquake action is only a fraction of the total motion recorded, since the total recorded acceleration history includes the motion after the earthquake has come to an end. To study the effective earthquake action, one has to expand the time in the records. The active earthquake action may be detected from the interaction with other small amplitud seismic waves that enter the basic wave train, Afterwards the waves show a practically clean record corresponding to the oscilations of the earth mass dissipating the balance of the input earthquake energy.

The Figure 2 shows an acceleration record of one component of the earthquake 1985 in Mexico City where the action mentioned above may be clearly detected. The seismologists may assist the foundation engineers in the interpretation of the acceleration records.

The seismo-geotechnical behavior of the subsoil may be analysed using the typical dynamic soil properties of the soil profile in Mexico City, based on a unit surface acceleration of 100 gal.

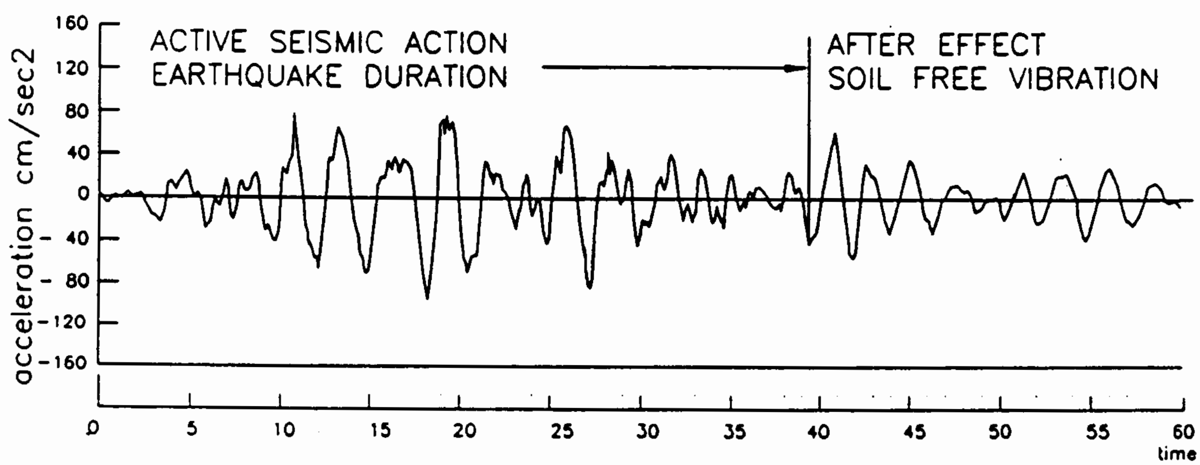


FIG 2. WAVE TRAIN RECORDED ACCELERATION N90° E. HORIZONTAL COMPONENT, MEXICO CITY 1985

From statements mentioned in the preceding paragraphs it can be realized the importance of the subsoil stratigraphy and the necessity to investigate in the field, not only the ground surface seismic motions and accelerations, but those taking place with depth in every soil stratum with different dynamical properties. A typical subsoil profile of a certain place in Mexico City, is shown Fig 3.

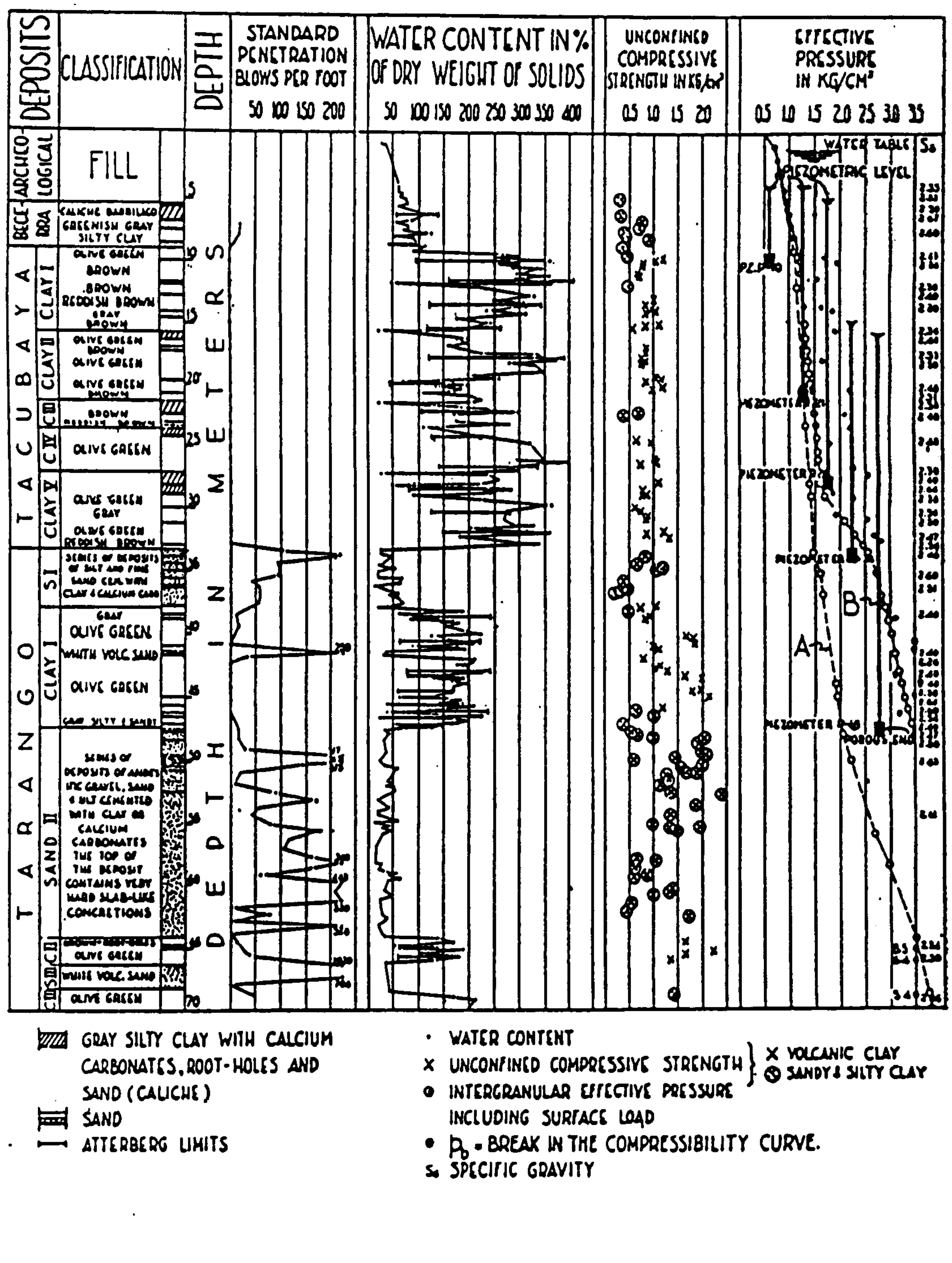


FIG 3. TYPICAL SUBSOIL PROFILE MEXICO CITY

Very important is to investigate the horizontal component of the surface wave, SR-wave. This seismic wave may be considered to have a celerity very close to the shear wave, S-wave, and for all practical purpose the celerity of the S-wave in used for the SR-wave. Ref. (3,6). The vibration of the SR-wave takes place along the path of wave propagation, producing a compression and expansion of the soil. The compressional waves produce in saturated soils high pore water pressure, "Seismic Pore Water Pressure" (SPWP), reducing the soil shear strength in soil containing fine sand and silt. Therefore, we are obliged to estimate from the engineering point of view the "SPWP".[7] The horizontal component of the shear wave and surface wave may induce cracking at the soil surface because of passing the soil shearing strength.

III SEISMIC WAVES[1]

The two body waves are know as "P" and "S" waves. The P-wave, or irrotacional wave, has a much higher celerity than the S-wave, it vibrates in the direction of propagation, compressing and expanding the soil. In case of saturated soils, the celerity is that of water, on the order of 1460 m/seg. The S-wave is also know as transverse, shear or equivolumetric wave. This wave originates shear stresses in the soil, independently of the degree of soil water saturation. The orbital actions take place normal to the direction of propagation.

The horizontal component of the SR-wave, surface wave,with celerity assumed to be equal to the S-wave celerity,compresses and expands the soil in the direction of its propagation. The P-wave and SR-wave create positive and negative strains and pressures in the soil.

From wave equations, satisfying the differential equations of motion, we find the basic characteristics related with the wave orbital velocity and the celerity, respectively, and for every soil stratum comprising the soil mass. The strain and stress for the three important wave types may be obtained from the following equa tions.

A) THE IRROTATIONAL VERTICAL PLANE WAVE[1]

$$\delta_z = \delta_o \cos(\pi Z/2H) \sin P_d (t-x/C_d) \qquad (3)$$

Where δ_o is the surface amplitud and p_d the circular frecuency, H the soft deposit thickness, (z) the average depth of the stratum and C_d the wave celerity.

The strain is $\frac{\partial \delta_z}{\partial z} = \varepsilon_z$, and considering that $\frac{\pi}{2H} = \frac{p_d}{C_d}$ and $V_{do}=\delta_o P_d$ we obtain for the maximum strain orbital action.

$$\varepsilon_z = \frac{V_d}{C_d} \sin\left(\frac{\pi}{2H} z\right) \qquad (4)$$

Moreover we know that for the plane wave: $p_z = \frac{2\ (1-\nu)}{(1-2\nu)} \mu \varepsilon_z$ hence to obtain the pressure we multiply by

$$\frac{2\cdot(1-\nu)}{(1-2\nu)} C_s^2\ \rho = C_d^2\ \rho$$

therefore

$$p_z = \rho(V_d \cdot C_d) \sin\left(\frac{\pi}{2H}\cdot Z\right) \qquad (5)$$

B) THE SHEAR OR EQUIVOLUMETRIC PLANE WAVE[1]

$$\delta_{yz} = \delta_{yo} \cos\frac{\pi z}{2H} \cdot \sin p_s \left(t - \frac{x}{C_s}\right) \qquad (6)$$

The maximum shear distortion in vertical planes normal to the direction of propagation is $\gamma_{yz} = \partial\delta_{yz}/\partial z$ and in the horizontal planes $\gamma_{xy} = \partial\delta_{xy}/\partial x$ hence:

$$\gamma_{yz} = \pm\left(\frac{V_s}{C_s}\right) \sin\frac{\pi}{2H} \cdot z \qquad (7)$$

and

$$\gamma_{xy} = \pm\left(\frac{V_s}{C_s}\right) \cos\frac{\pi}{2H} \cdot z \qquad (8)$$

The corresponding shear stress is obtained multiplying be $C_s^2 \cdot \rho$, we obtain

$$\tau_{yz} = \pm \rho \, (V_s \cdot C_s) \, \sin \frac{\pi}{2H} Z \tag{9}$$

and

$$\tau_{xy} = \pm \rho \, (V_s \cdot C_s) \, \cos \frac{\pi}{2H} \cdot Z \tag{10}$$

C) THE SURFACE PLANE WAVE HORIZONTAL COMPONENT[1,2,6]

$$\delta_{xz} = \pm \delta_{xo} e^{-rz} \sin P_R \, (t - \frac{x}{C_R}) \tag{11}$$

here δ_{xo} is the wave amplitud at the ground surface, in the direction of propagation, P_R and C_R are the corresponding wave circular frecuency and celerity respectively. The value of (r) is an atenuation factor with depth (z), and (x) is the horizontal coordinate position of the wave.

The maximum strain is $\varepsilon_{xz} = \pm \frac{V_{xo}}{C_{xo}} e^{-rz}$. To obtain the pressure we multiply by the compression modulus of the plane wave, in terms of the celerity: $2\rho C_{xo}^2 / (1-\nu)$, and obtain

$$P_{xz} = \frac{2\rho}{1-\nu} (V_{xo} \cdot C_{xo}) e^{-rz} \tag{12}$$

where V_{xo} and C_{xo} are the orbital velocity and celerity of the wave at the ground surface. The value of the atenuation factor is, Apendix II:[3]

$$r_z = (\frac{P}{C})_R \sqrt{1 - \alpha^2 \frac{1-2\nu}{2 \, (1-\nu)}}$$

we call the radical $a(\nu)$, and considering that $\frac{P_R}{C_R} \approx \frac{P_S}{C_S}$ we write

$$r_z = P_s \cdot a(\nu)/C_{sz} \tag{13}$$

From above discussions we find that the characteristic values for the seismic wave equations are

$$\begin{array}{lll} 1) & \text{for strain} & (\frac{V}{C})_z \\ 2) & \text{for pressure} & (V \cdot C)_z \end{array} \qquad (14)$$

Therefore, we are compelled to know the wave orbital velocity and celerity with depth and in every soil stratum comprissing the soft soil mass. For this purpose we have to investigate the dynamic soil rigidity (μ)

By means of the Free Vibration Torsion Pendulum[8,9] Fig 4, the values of (μ) are investigated for different confining stresses (σ_C), and plotted for each (σ_C) as shown in Fig 5, in a dimension

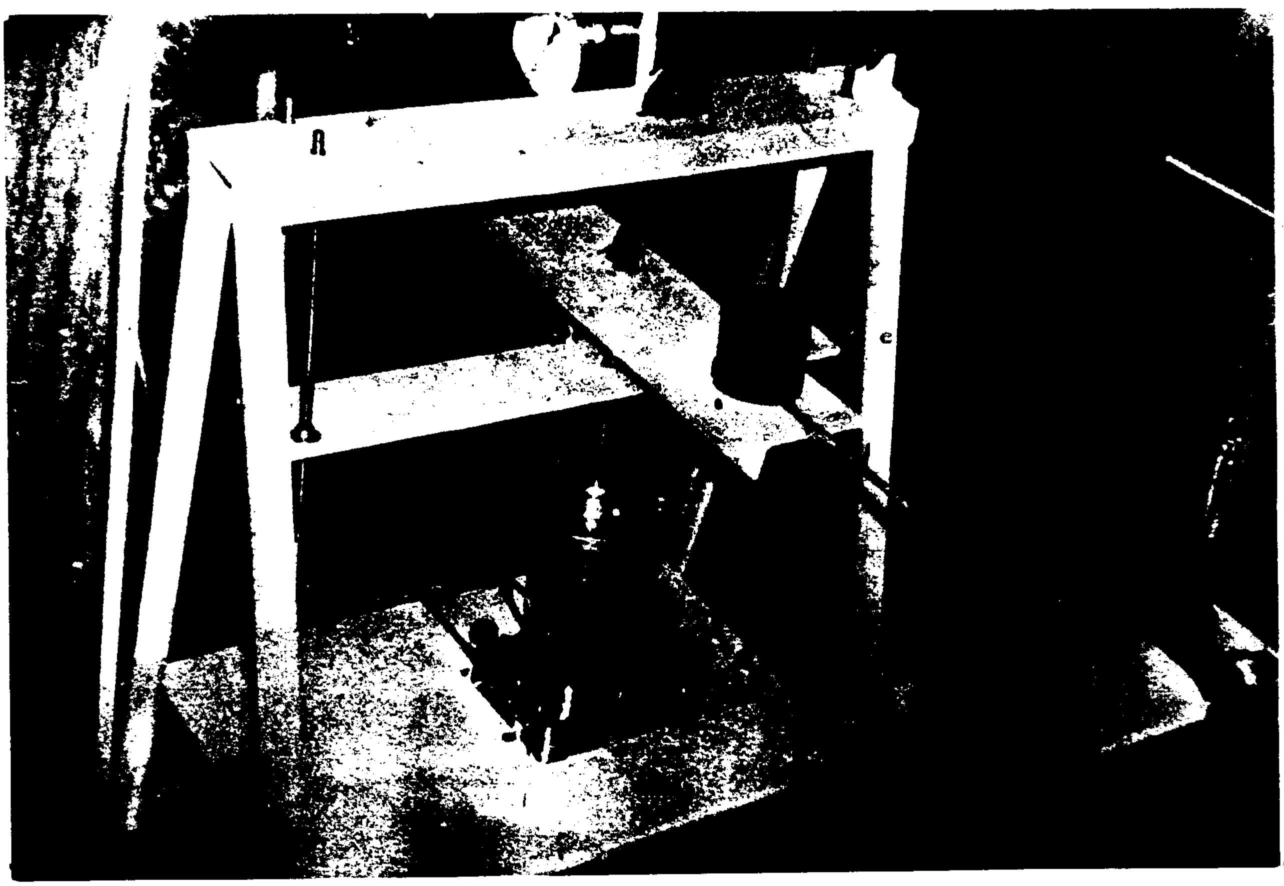
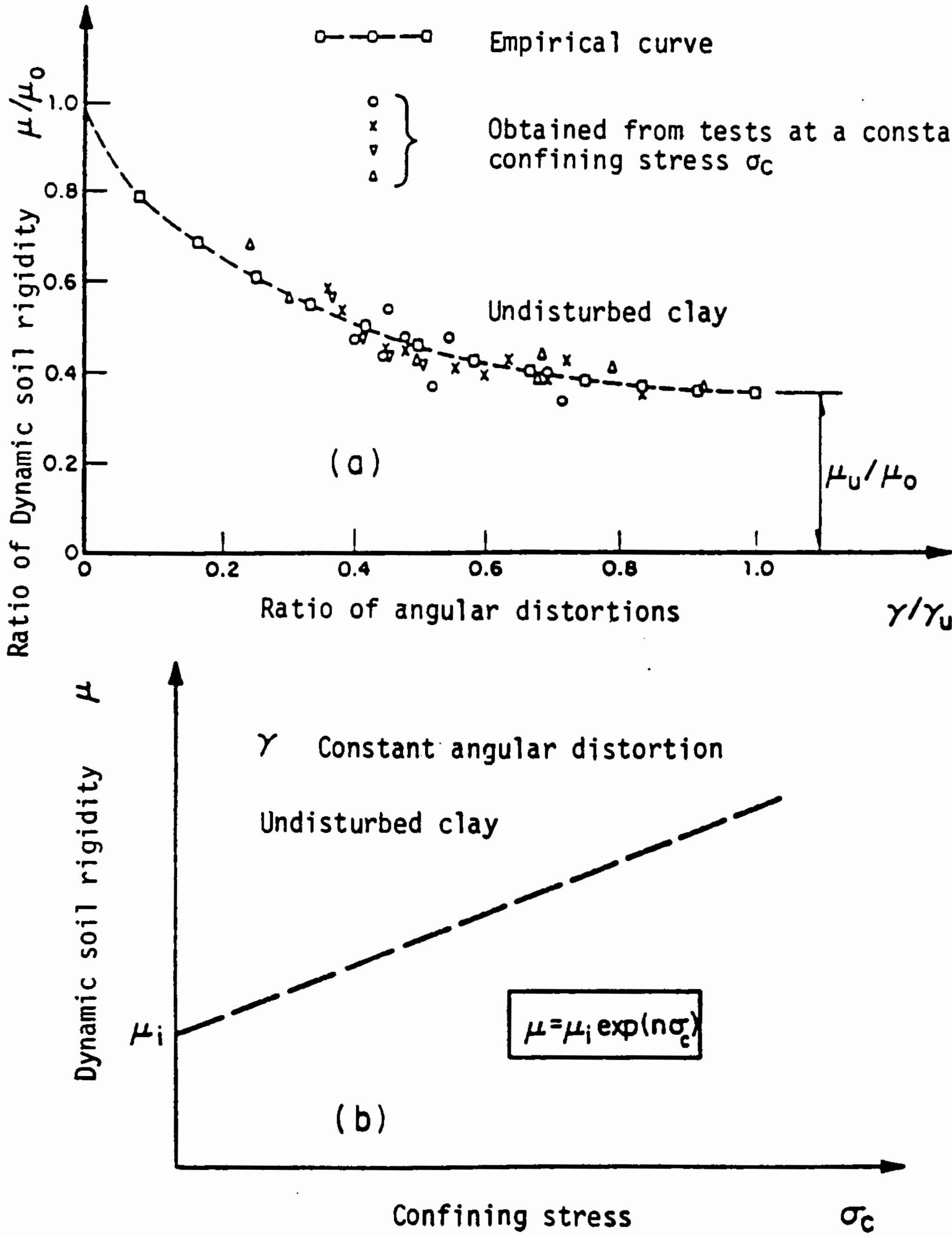


FIG 4. FREE VIBRATION TORSION PENDULUM, (FVT)



Note:

Undisturbed Mexico City typical silty clay

Fig 5. (a) DYNAMIC SOIL RIGIDITY vs ANGULAR DISTORTION

(b) DYNAMIC SOIL RIGIDITY vs OVERBURDEN STRESS, MEXICO CITY less form hence we find

$$\frac{\mu}{\mu_o} = \{1-(1-\frac{\mu_u}{\mu_o}) \sin^n (\frac{\pi}{2} \cdot \frac{\gamma}{\gamma_u}) \} \tag{15}$$

in which

(μ) , dynamic soil rigidity for a specific confining volumetric stress and distortion (γ)

μ_u , dynamic soil rigidity for final angular distortion (γ_u)

μ_o , virtual dynamic soil rigidity when $\gamma \rightarrow 0$

n , 1/3 for Mexico City volcanic clay

When values of (μ) are plotted for a fix value of (γ) we obtain the dynamic soil rigidity as a function of the confining volumetric stress (σ_c), Fig 5 . For Mexico City clay(6) we obtain

$$\mu = \mu_i \cdot \text{Exp.}(n_c \cdot \sigma_c) \tag{16}$$

It may be noticed from equation (15) that for a large distortion (γ_u), the ratio (γ/γ_u) $\rightarrow$ 1, and $\mu = \mu_u$ remains constant provided the soil does not reach failure. This action is important to observe for large distortions induced by the seismic waves. Hençe, equation (15) is valid for (γ/γ_u) $\leq$ 1.

The value of the celerity may be investigated from the dynamic soil rigidity (μ) and with the unit soil mass (ρ), we calculate the shear wave celerity

$$C_s^2 = \mu/\rho \tag{17}$$

assigning a Poisson ratio (ν) we calculate the irrotational wave celerity

$$C_d = \sqrt{\frac{2\ (1-\nu)}{(1-2\nu)} \cdot \frac{\mu}{\rho}} \tag{18}$$

In case of Mexico City the author has investigated for a long time the dynamic properties of Mexico City's high compressible silty clay. It has been found statistically that within field water contents on the order of 200% to 400%, the dynamic soil rigidity has an a verage value of

$$\mu \cong 10 \cdot e^{0.92\sigma_z}, \; K/cm^2 \qquad (19)$$

in which σ_z is the effective overburden stress in the soil deposit to a depth (z), given in kg/cm^2

Considering an average unit soil mass of 0.135 $ton \cdot sec/m^4$ the shear wave celerity in the typical Mexico City clay has the value.

$$C_s = (27.22)\; e^{0.46\sigma_z} \; m/sec \qquad (20)$$

here σ_z is given in kg/cm^2

A graphical representation of (μ) vs (σ_z) is given in Fig 6, The va

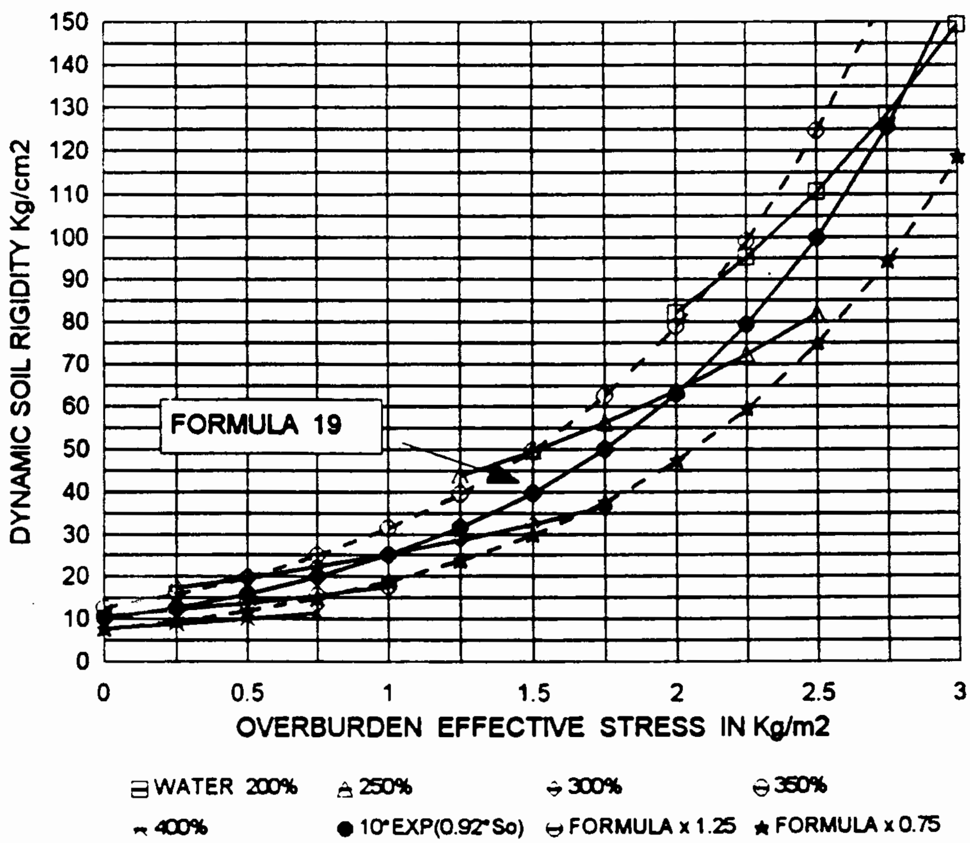


Fig 6. DYNAMIC SOIL RIGIDITY MEXICO CITY

lues were obtained from a great number of dynamic tests in undisturbed soil specimens.

Also it was found statistically a relation between (μ) and the compression dynamic lineal strain modulus M_C, that reads as follows.

$$(\mu)\cdot M_C \cong 0.38 \qquad (21)$$

The values of (μ) in k/cm^2 and (M_C) in cm^2/kg. The average ratio of M_C with respect to the response lineal strain expansion modulus M_e is on the order of $M_e/M_C = 0.85$

The subsoil of México City is highly stratified and interbedded with volcanic sand strata Fig 3. It has been found the following values for the dynamic soil rigidity of the fine silty sand.

a) For loose sand

$$\mu = 154\ \sigma_z^{0.76}\ kg/cm^2 \qquad (22)$$

b) For compact sand

$$\mu = 430\ \sigma_z^{0.5}\ kg/cm^2 \qquad (23)$$

Here σ_z is the overburden effective stress in kg/cm^2

The values reported above may be used for estimates with an accurancy on the of ± 25%. However due to the variable stratigraphycal conditions of the subsoil in Mexico City it is neccesary, for accurancy, to carry on tests in good undisturbed soil specimens, and for each stratum building up the soil deposit to the depth of the firm stratum.

The "FVTP", device Fig 4, designed by the author several decades ago to investigate the dynamic soil rigidity,[4] has given reliable results for practical engineering purposes in soil dynamics problems. The calculations performed using the results of (μ) have been verified with field observations recorded in the accelerographs intalled at the ground surface

IV SUBSOIL SEISMO- GEODYNAMICS BEHAVIOR[1,2]

The interest of the foundation engineer is to learn from a practical engineering point of view the order of magnitud of the soil strains, stresses and displacements, that the seismic waves induce in the soil mass, and most important close to the ground surface. Equally important is the need to estimate the periods of vibration of the foundation and building and their critical damping characteristics.

The practical procedure proposed by the author to estimate the soil behavior is illustrated in two typical cases at different locations in Mexico City. The theoretical procedures are contained in Appendix I, for the shear wave and in Appendix II for the horizontal component of the surface wave. These cases are the following

CASE I

a) Shear wave in layered subsoil Fig 7
b) Surface wave in layered subsoil Fig 8

CASE II

a) Shear wave in layered subsoil Fig 9
b) Surface wave in layered subsoil Fig 10

A ground surface acceleration of 100 gal was used for the two cases presented. Notice, that the stratigraphy and the (μ) profile values are different en each case.

Case Ia and IIa Figs 7 and 9 respectively, give the configuration with depth of the vertical and horizontal shear stresses, the accelerations and displacements, correspond to the soft soil deposit investigated to a depth of 30.40 m. The foundamental periods of the soft soil deposits are 2.2 and 2.25 seconds, respectively.

SHEAR WAVE IN LAYERED SUBSOIL

SIMBOLS

d: Stratum thickness
mu:Dynamic soil rigidity
rho:Soil density
(Dis)yz, displacemet plane YZ
Acc: Orbital acceleration
(tau)yz, Shear stress in plane YZ
(tau)yx, Shear stress in plane YX
z: Depth in mtrs

Surf. Acc. m/sec2.. 1.00 Period 2.220 sec Frec. c 2.83

SUELO	Estr	d m	z m	mu Ton/m2	rho	(Dis)yz m	(tau)yz Ton/m2	(tau)xy Ton/m2	DIS.cm 1/10	Ac m/sec2
Silty clay	0	0.00	0.00	130.00	0.19	0.1250	0.000	1.76	1.25	1.00
Silty clay	1	3.00	3.00	130.00	0.19	0.1250	0.570	1.76	1.25	1.00
Silty clay	2	2.00	5.00	180.00	0.20	0.1187	0.960	2.01	1.19	0.95
Clay-tipical	3	3.00	8.00	135.00	0.18	0.0979	1.428	1.36	0.98	0.78
Clay-tipical	4	4.20	12.20	165.70	0.13	0.0626	1.778	0.82	0.63	0.50
Clay-tipical	5	3.40	16.60	208.70	0.12	0.0340	1.936	0.48	0.34	0.27
Clay-tipical	6	1.50	18.80	418.40	0.12	0.0271	1.980	0.54	0.27	0.22
Clay-tipical	7	3.40	22.90	501.70	0.12	0.0138	2.047	0.30	0.14	0.11
Clay-tipical	8	2.40	27.20	620.60	0.12	0.0059	2.070	0.14	0.06	0.05
HARD STR.	9	4.00	30.40	1500.00	0.22	0.0004	2.092	0.02	0.00	0.00

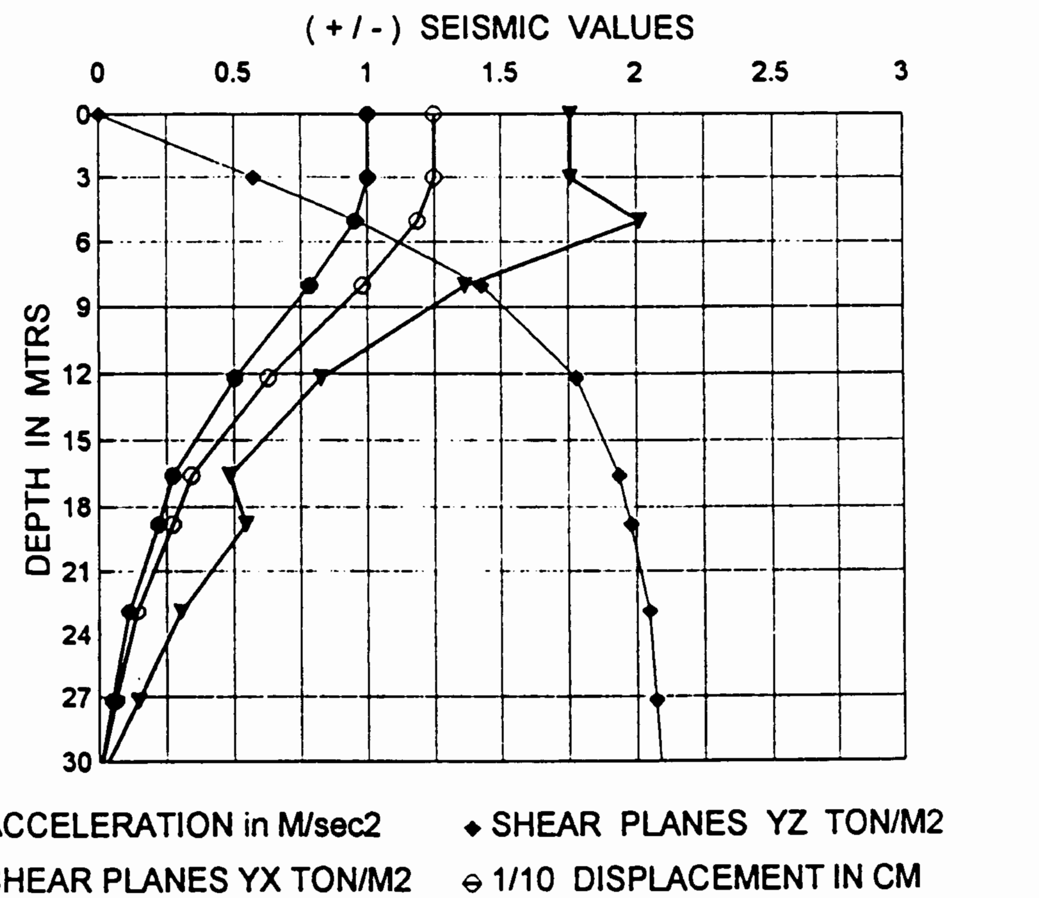


FIG.7 SHEAR WAVE IN LAYERED SUBSOIL

SURFACE WAVE IN LAYERED SUBSOIL   NAME: SWLSCP1.WK3

LIST OF SYMBOLS:

(d),Stratum thickness.	(M),Strain Modulus	SUM,Summation of (rd)
(mu)zSoil ,Dyn.Rigidity	(M)e, For Traction	(STR)z,Strain at (z)
rho.Unit mass	(M)c For Compression	(P)z,Aver.Pressure
(v),Poissons ratio	Beta, Response factor	(S)z,Aver.Stress
(C)z,Celerity at depth z	Beta, (M)e/(M)c	(Ac), Acceleration
a(v),Parameter at depth z	(r)z,Attenuation factor	Displ. Soil Displacement

PERIOD 2.22 sec.

Acc.m/s2 1.00   Circular frecuency 2.83   Orb.Vel 0.35 m/sec   Surface Strain 0.01351

SURFACE WAVE CELERITY 26.16 m/sec

SOIL	Estr	d m	(mu)z Ton/m2	(rho)z Unit.m	v nu	(C)z m/sec	a(v) —	(Beta)c —	1/M Ton/m2	(r)z 1/m	rd —	SUM —	Depth m	STRAIN —	Pz Ton/m2	Stress Ton/m2	Ac x 10	Displ. cm
													0.00	0.01351	4.68	4.68	10.00	12.49
													2.50		3.55	3.55		
Silty clay	1	3.00	130.00	0.190	0.25	26.16	0.85		346.67	0.0919	0.2758	0.2758	3.00	0.01025	4.92	2.26	7.59	9.48
Silty clay								0.85					5.00		4.19	1.93		
Silty clay	2	2.00	180.00	0.200	0.25	30.00	0.85	0.85	480.00	0.0802	0.1603	0.4362	5.00	0.00874	3.69	1.69	6.47	8.08
Silty clay								0.85					8.00		2.83	1.30		
Silty clay	3	3.00	135.00	0.180	0.36	27.39	0.85	0.85	421.88	0.0878	0.2635	0.6996	8.00	0.00871	3.42	1.57	4.97	6.20
Silty clay								0.85					12.20		2.54	1.21		
Silty clay	4	4.20	165.70	0.130	0.35	35.70	0.89	0.90	509.85	0.0705	0.2963	0.9959	12.20	0.00499	3.20	1.52	3.69	4.61
Silty clay								0.90					16.60		2.61	1.24		
Silty clay	5	3.40	208.70	0.120	0.35	41.70	0.89	0.90	642.15	0.0604	0.2053	1.2012	16.60	0.00408	5.23	2.48	3.01	3.76
Silty clay								0.90					18.80		4.91	2.33		
Silty clay	6	1.50	418.40	0.120	0.35	59.05	0.89	0.90	1287.38	0.0426	0.0640	1.2652	18.80	0.00381	5.10	2.42	2.82	3.52
Silty clay								0.90					22.90		4.49	2.13		
Silty clay	7	3.40	501.70	0.120	0.25	64.66	0.85	0.90	1337.87	0.0372	0.1265	1.3916	22.90	0.00336	6.42	3.04	2.49	3.11
Silty clay								0.90					27.20		5.90	2.62		
Silty clay	8	2.40	620.60	0.120	0.35	71.91	0.89	0.80	1909.54	0.0350	0.0840	1.4757	27.20	0.00309	12.38	5.49	2.29	2.88
Silty clay								0.80					30.40		10.92	5.17		
HARD STR	9	4.00	1500.00	0.220	0.25	82.57	0.90	0.90	4000.00	0.0308	0.1234	1.5990	30.40	0.00273	10.92	5.98	2.02	2.52

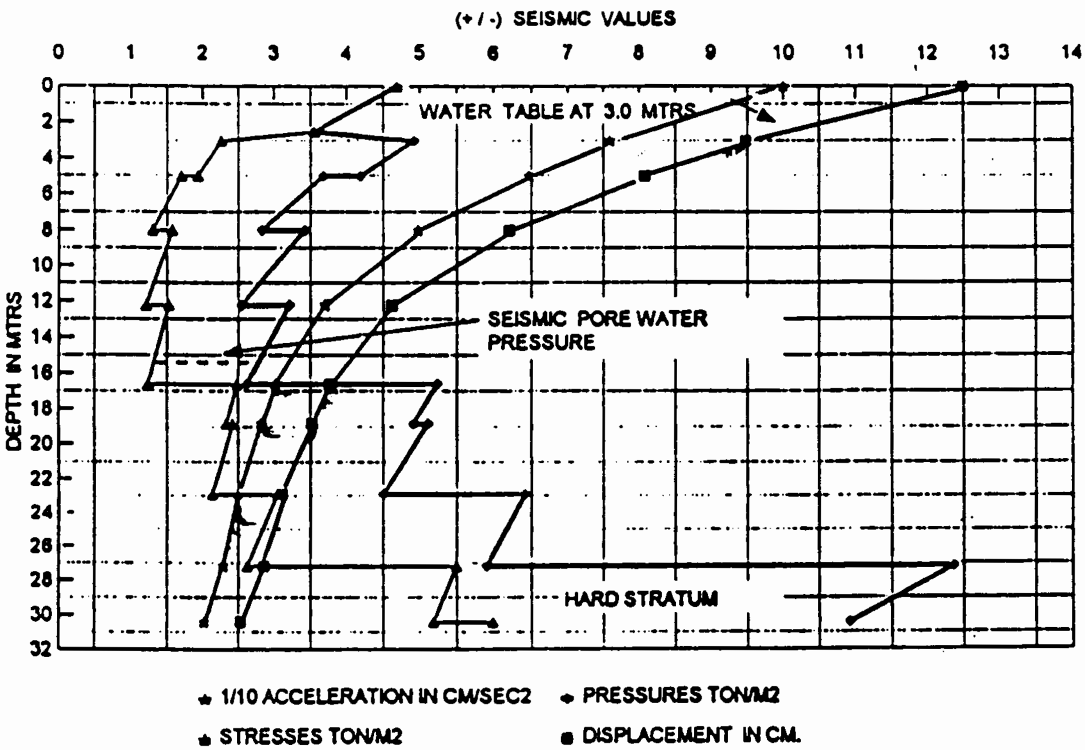


FIG8. SURFACE SEISMIC WAVE IN LAYERED SUBSOIL

SHEAR WAVE IN LAYERED SUBSOIL, SHWLS2

d: Stratum thickness
mu: Dynamic soil rigidity
rho: Soil unit density
(Dis)yz: Displacement in YZ

Acc: Orbital acceleration
(tau)yz: Shear stress in YZ
(tau)yx: Shear stress in YX
(z), Depth

Surf. Acc. m/sec2 1.00 Period 2.250 sec Cir. Fre. rad 2.79

SOIL	Estr	d m	z m	mu Ton/m2	rho	(Dis)yz m	(tau)yz Ton/m2	(tau)xy Ton/m2	Acc. cm/s2 X 10	Displ. cm
Sandy clay	0	0.00	0.00	0.00	0.151	0.128	0.00	3.83	10.00	12.84
Sandy clay	1	1.10	1.10	757.00	0.151	0.128	0.17	4.58	10.00	12.84
Sandy clay	2	1.00	2.10	1080.00	0.187	0.128	0.35	4.43	9.99	12.82
sily clay	3	2.40	4.50	820.00	0.141	0.127	0.69	3.33	9.91	12.72
sily clay	4	2.10	6.00	626.00	0.116	0.125	0.93	1.42	9.73	12.49
sily clay	5	2.90	9.50	144.00	0.118	0.106	1.24	1.48	8.29	10.64
sily clay	6	5.00	15.10	210.00	0.118	0.078	1.66	2.69	6.06	7.77
SAND	7	1.40	16.50	1300.00	0.210	0.076	1.84	1.22	5.92	7.60
silty clay	8	3.50	20.00	158.00	0.119	0.036	2.02	0.61	2.80	3.60
silty clay	9	2.00	22.00	310.00	0.119	0.023	2.07	0.49	1.79	2.30
silty clay	10	2.00	24.00	496.00	0.130	0.015	2.11	0.38	1.14	1.47
silty clay	11	1.00	25.00	648.00	0.150	0.011	2.13	0.35	0.89	1.14
silty clay	12	2.70	27.70	814.00	0.175	0.004	2.16	0.22	0.34	0.44
HARD STRAT	13	3.80	30.40	1880.00	0.200	0.000	2.17	0.00	0.00	0.00

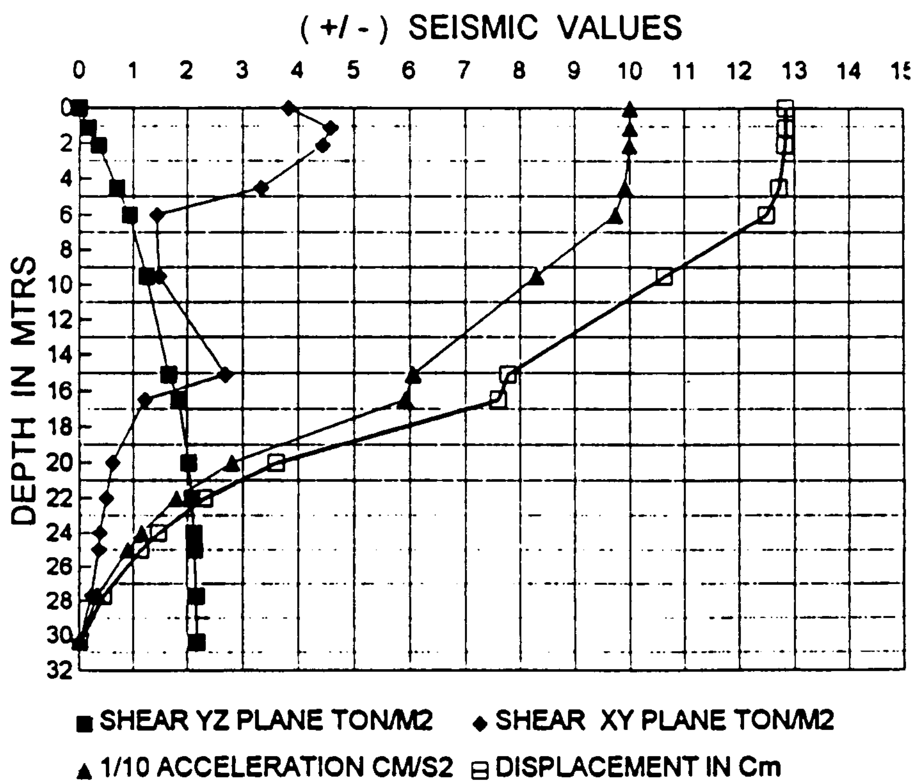


FIG.9 SHEAR WAVE IN LAYERED SUBSOIL

SURFACE WAVE IN LAYERED SUBSOIL : SWLSCP2.WK3

LIST OF SYMBOLS:

(d),Stratum thickness.	(M),Strain Modulus	SUM,Summation of (rd)
(mu)z,Dyn.Shear Mod.	(M)e, For Traction	(Str) ,Strain at (z)
rho.Unit mass	(M)c For Compression	(P)z.Aver.Pressure
(v),Poissons ratio	Beta, Response factor	(S)z.Aver.Stress
(C)z,Celerity at (d)	Beta, (M)ex/(M)cx	(Ac), Acceleration
a(v),Parameter	(r)z.Attenuation	(pc).Circ.frecuency

PERIOD 2.25 SEC.

Initial surface : Acc.m/ 1.00 Circular frecuency 2.79 Orb.Vel 0.36 m/sec Surfac Strain 0.00567

Surface wave celerity 63.12 m/sec

SOIL	Estr	d m	(mu)z Ton/m	(rho)z U.mas	v nu	(C)z m/sec	a(v)	Bcx	1/M Ton/m2	(r)z 1/m	rd	Sum	depth m	Strain	Pz Ton/m	Stress Ton/m	Ac x 1/10	Displ. cm
SANDY CLAY.	SURFACE							——	2018.67				0.00	0.00567	11.46	11.46	10.00	12.83
SANDY CLAY.								——					1.10		10.99	10.99	9.59	12.31
SANDY CLAY.	1	1.10	757	0.190	0.25	63.12	0.85	——	2018.67	0.0376	0.0414	0.0414	1.10	0.00544	15.68	15.68	9.59	12.31
SANDY CLAY.								——					2.10		15.18	15.18	9.29	11.92
SANDY CLAY.	2	1.00	1080	0.200	0.25	73.48	0.85	0.85	2880.00	0.0323	0.0323	0.0736	2.10	0.00527	11.53	5.30	9.29	11.92
SILTY CLAY								0.85					4.50		10.71	4.92	8.63	11.07
SILTY CLAY	3	2.10	820	0.180	0.25	67.49	0.85	0.85	2186.67	0.0352	0.0738	0.1475	4.50	0.00490	9.43	4.33	8.63	11.07
SILTY CLAY								0.85					6.00		8.75	4.02	8.00	10.27
SILTY CLAY	4	2.10	626	0.130	0.35	69.39	0.89	0.90	1926.15	0.0358	0.0752	0.2227	6.00	0.00454	2.01	0.95	8.00	10.27
SILTY CLAY								0.90					9.50		1.63	0.77	6.50	8.34
SILTY CLAY	5	2.90	144	0.120	0.35	34.64	0.89	0.90	443.08	0.0717	0.2080	0.4307	9.50	0.00369	2.38	1.13	6.50	8.34
SILTY CLAY								0.90					15.10		1.77	0.84	4.83	6.20
SILTY CLAY	6	5.00	210	0.120	0.35	41.83	0.89	0.90	646.15	0.0594	0.2970	0.7277	15.10	0.00274	9.50	4.50	4.83	6.20
SILTY CLAY								0.90					16.50		9.11	4.32	4.63	5.94
SAND	7	1.40	1300	0.210	0.25	78.68	0.85	0.90	3466.67	0.0302	0.0422	0.7699	16.50	0.00263	1.28	0.61	4.63	5.94
SILTY CLAY								0.90					20.00		1.01	0.48	3.64	4.67
SILTY CLAY	8	3.50	158	0.120	0.35	36.29	0.89	0.80	486.15	0.0685	0.2397	1.0096	20.00	0.00207	1.97	0.88	3.64	4.67
SILTY CLAY								0.80					22.00		2.84	1.26	3.27	4.20
SILTY CLAY	9	2.00	310	0.140	0.35	47.06	0.90	0.85	953.85	0.0534	0.1068	1.1164	22.00	0.00186	2.84	1.30	3.27	4.20
SILTY CLAY								0.85					24.00		2.60	1.19	3.00	3.85
SILTY CLAY	10	2.00	496	0.150	0.35	57.50	0.90	0.85	1526.15	0.0437	0.0874	1.2037	24.00	0.00170	3.40	1.56	3.00	3.85
SILTY CLAY								0.85					25.00		3.27	1.50	2.89	3.71
SILTY CLAY	11	1.00	648	0.140	0.35	68.03	0.90	0.85	1993.85	0.0369	0.0369	1.2407	25.00	0.00164	4.11	1.89	2.89	3.71
SILTY CLAY								0.85					27.70		3.74	1.72	2.63	3.37
SILTY CLAY	12	2.70	814	0.160	0.35	71.33	0.90	0.85	2504.62	0.0352	0.0951	1.3358	27.70	0.00149	7.48	3.44	2.63	3.37
HARD STRAT.								0.95					30.40		6.75	3.29	2.37	3.04
HARD STRAT.	13	3.80	1880	0.220	0.25	92.44	0.90	0.95	5013.33	0.0272	0.1033	1.4391	30.40	0.00135			2.37	3.04

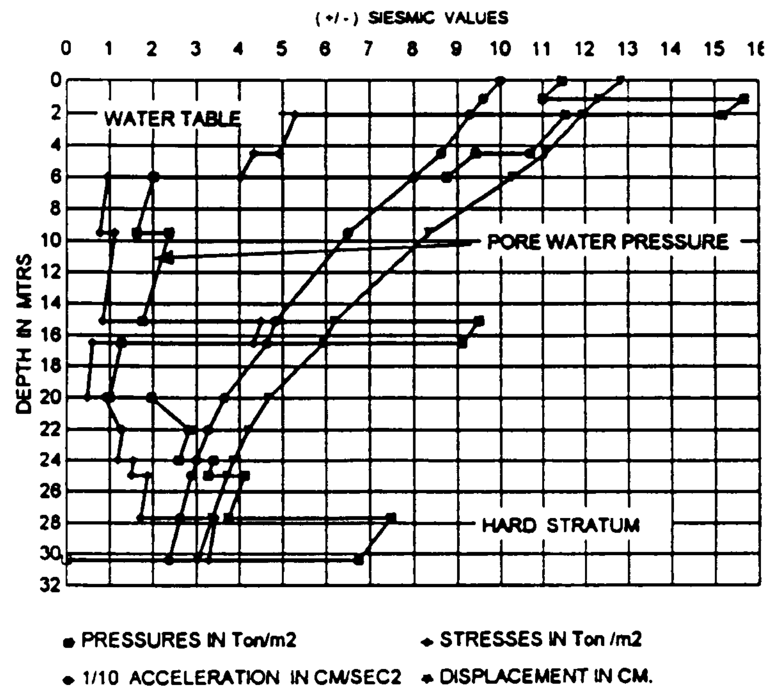


FIGURE 8

FIG.10 SURFACE WAVE IN LAYERED SUBSOIL

The configurations with depth for the surface waves are given respectively in Figs 8 and 10 for Case Ib and Case IIb. Notice, how the rigid soil strata relieve from pressure the soft soil strata. In other words the soft strata is protected from high pressures by the rigid strata. The difference in value of the pressures and stresses shown, represent the "Seismic Pore Water Pressure" induced by the wave. Notice the large pore water pressure in the hard sand stratum.

It is very important to learn on the order of magnitud of the "Seismic Pore Water Pressure" (ω), since its value has an important application in soil stability seismic problems. The value of (w) may be estimated by means of the response factor $(\beta)_{e/c} = M_e/M_c$. In which M_e is the response vertical strain modulus and M_c is the compressional strain modulus.

In case of the plane surface wave[10]

$$\omega = \pm \frac{\bar{K}_{ex}}{3K_a + \bar{K}_{cx}\ (1+\beta_{e}/_{c})} p_X \qquad (24)$$

here:

$\bar{K}_{cx}$ = $3(1-2\nu)\cdot(1+\nu)\cdot M_{cx}$

K_a = soil air-water compressibility

ν = Poisson's ratio

The soil air-water compressibility has the following theoretical value according to Boyle-Marriote and Henry laws[10]

$$K_a = \frac{n[1-S\cdot(1-\alpha)]}{(1+\alpha)\cdot(p_a+U\pm w)} \qquad (25)$$

here

n, soil porosity

S, degree of saturation

U, in situ piezometric water pressure

w = seismic pore water pressure

α = Henry's coefficient 17cc per liter at 20° centrigrade and a pressure of one atmosphere. For contaminated soil water with other substances we may assume α = 10 cc per liter.

We recognize from equation (25), that for a larger content of microscopic air bubbles in the soil pores, the soil air-water compressibility increases. See Ref 6, chapter V. In practice we may assume from be safe side.

$$K_a = \frac{n\cdot(1-S)}{P_a + U \pm w} \tag{26}$$

On the other hand, notice that the response factor (β) is an approximate measure of the fraction of the soil critical damping, hence:

$$\zeta = (1-\beta_{e/c}) \tag{27}$$

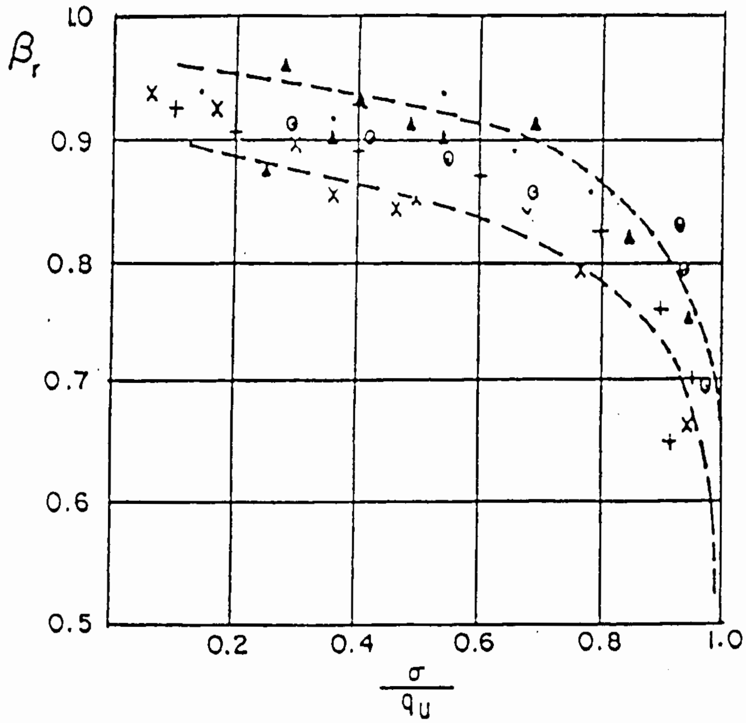
From cyclic compression and expansion tests the value of M_e and M_c are determined in undisturbed soil specimens. The tests are performed for the expected stress levels, because of the seismic action, and the values $\beta_{e/c} = M_e/M_c$ obtained. For high stress levels in relation to the shearing strength the value of (β) has the tendency to decrease rapidely, Fig 11.[11,12]

The compressions and extensions produced by the wave induce craking of the soil because of traction

When the seismic wave compression on the soil in larger than the passive earth pressure,the ground surface suffers extrusion as shown in Fig 13. (photo), and when the traction is larger than the soil resistence with depth cracks develop,and the ground surface subsides, forming a "Graben" Type depression as shown in Fig 12.

The computed wave stresses shall be compared with the dynamic soil strength, and the factor of safety for soil failure determined.

The displacement configuration of the soil mass with depth shown in



σ , Vertical Stress

q_u , Strength from Unconfined Compression Test

FIG 11. RESPONSE FACTOR β vs VERTICAL STRESS LEVEL

FIG 12. SEISMIC SETTLEMENT OF PAVEMENT (GRABEN)
Court. Ing. Carlos E. Castañeda N

Case Ib and IIb, shall be used to study in pile foundations the soil mass drift on the piles. The estimate is performed with a pile-soil interaction method at the maximum amplitud of the seismic motion.(14,15)

Finally, it may be mention that the method herein explained to analize the soil mass behavior can be readily used in earthquake areas similar in stratigraphical conditions as described for Mexico City subsoil. Many cities are expanding to softer grounds underlain by strata of high strength and low deformational characteristics.

FIG 13. HEAVE OF PAVEMENT
Court. Ing. Carlos E. Castañeda N

V SEISMIC STABILITY OF SURFACE FOUNDATIONS(8,9)

The application of the seismic concepts mentioned in the preceding paragraphs will be illustrated in the design of a rigid box type foun dation usually used in Mexico City.(16,17)

Let us consider a foundation to a depth (D) into the lacustrine silty clay deposit, allowing to acommodate two basements and the foundation structure. Once the foundation has been statically designed, we are compelled to verify the seismic stability according to the following items.

1) Determine the seismic vertical pressure at the foundation grade elevation, due to the amplitud of rocking of the building. For this purpose it is necessary to know the period of vibration and the equivalent critical damping of the system, soil-foundation and building frame, Fig 14

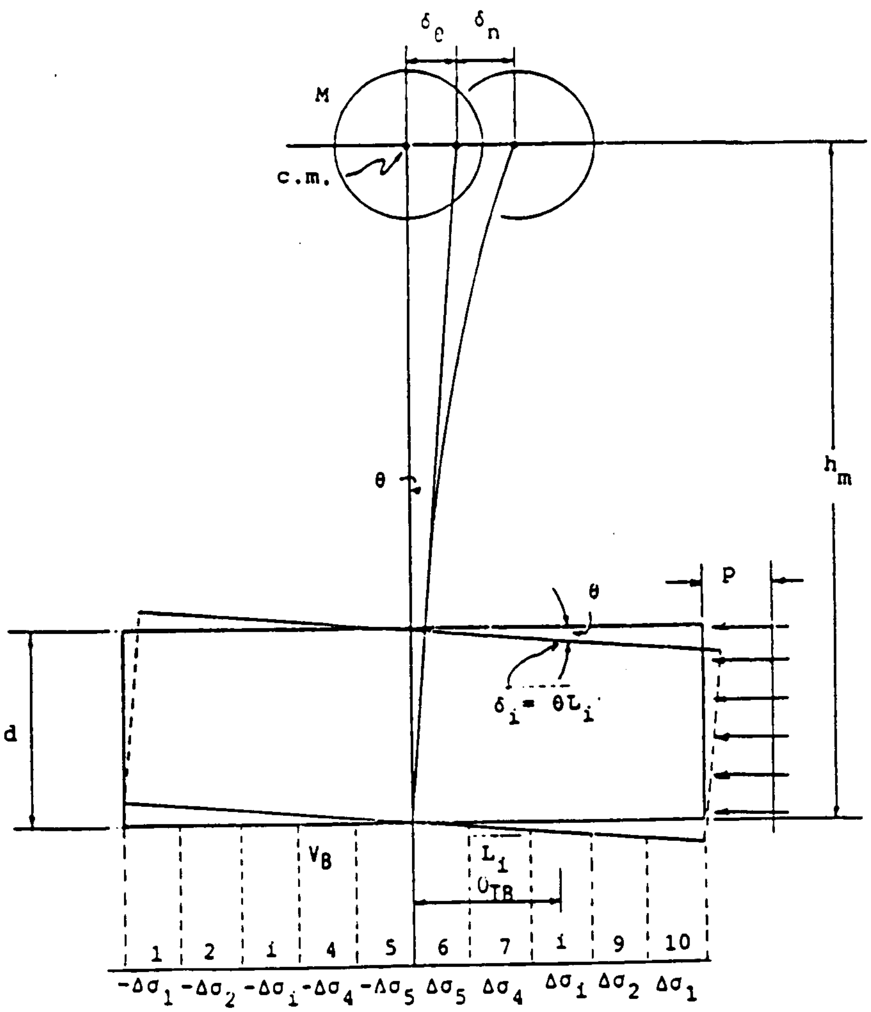


FIG 14. MAXIMUM AMPLITUD OF ROCKING IN BOX TYPE FOUNDATION

2) With the vertical pressures induced in the soil mass because of rocking of the building and the horizontal pressures induced by the seismic wave, the shear in the soil foundation is computed and compared with the dynamic shear strength of the soil, Fig 15.

3) Analyze the possibility of cracking of the soil surface close to the foundation structure and the probable depth of the cracks. When cracks take place, the lateral confinement of the box type foundation is lost. Fig 16.

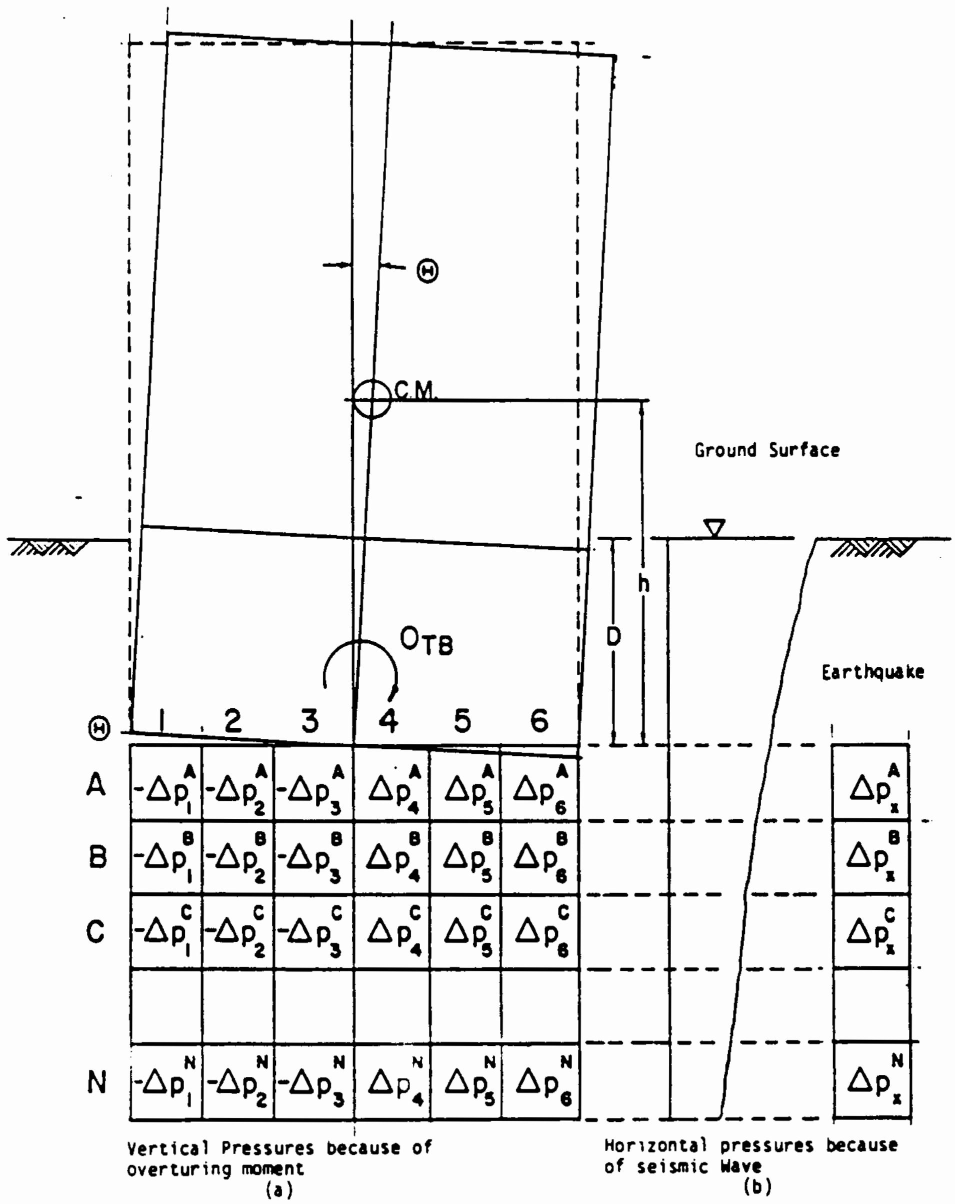
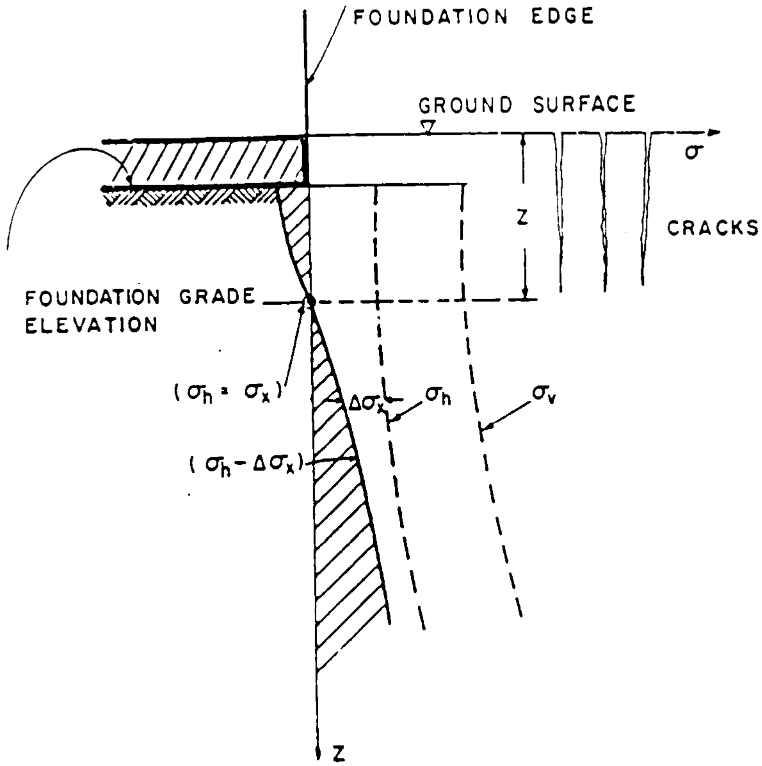


FIG 15. SEISMIC VERTICAL AND HORIZONTAL PRESSURES



σ_h , Pressure induced by the building seismic rocking added by the weight of the building, and soil

$-\Delta\sigma_x$, Reduction of horizontal pressure because of the surfase wave action

FIG 16. CRACKS INDUCED BY SEISMIC WAVES

4) From results of (2) and (3) the seismic stability of the foundation is revised

5) The factors of safety are determined for failure and for permanent rotation of the foundation, consequently the tilting of the building.

The foundation design method according to the afore mentioned statements in 1 to 5, will be discussed further in the following paragraphs.

Knowing the equivalent coupled period of vibration of rocking of the foundation and that of the building frame (T_O), the period of vibration of the soil mass (T_S), me obtain the ratio (T_O/T_S). Furthermo re we need to know the fraction of critical damping of the building frame (ζ_B), and that of the soil foundation (ζ_s), in order to calcu late the equivalent value of the system (ζ_o).

With (T_o/T_s) and (ζ_o) we enter the "Design Acceleration Envelope Spectrum" (DAES), Fig 17 and determine the amplification factor of the acceleration (f_a), to be applied at the center mass of the buil ding

The "Design Acceleration Envelope Spectrum" (DAES) shown in Fig 17, was construted to obtain the maximun response acceleration of a structural frame of one degree of freedom vs the fraction of critical damping, Fig 17.

The Maximum response of the system building and foundation, is obtai ned when the dominant period of the ground is coincident with the coupled period of vibration (T_o) of the building frame and founda-

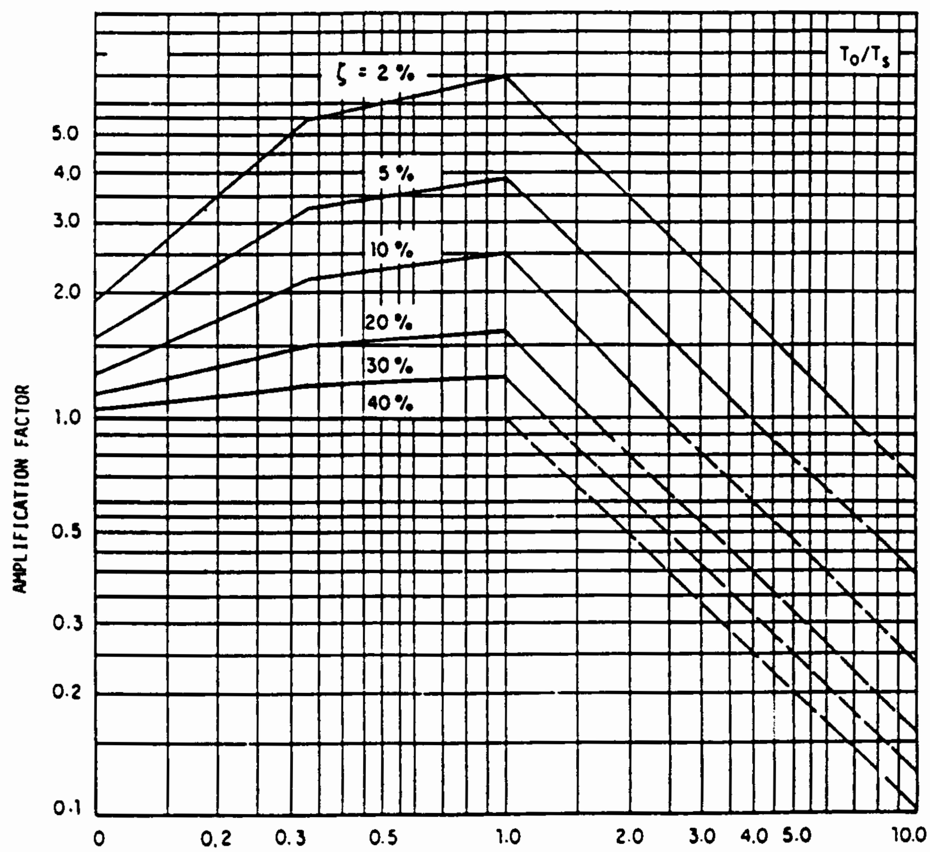


FIG 17. DESIGN ACCELERATION ENVELOPE SPECTRUM (DAES)

tion. The amplification (f_a) is defined as the ratio of seismic response acceleration at the center of mass of the building to the maximum acceleration recorded or assigned at the ground surface. This maximum takes place when $(T_o/T_s) = 1$.

The author has investigated in several sites the amplification factors of the ground surface acceleration against the fraction of the critical damping (ζ_o), and found a dimensionless relation for the maximum amplification factor when $(T_o/T_s) = 1$, hence:

$$(f_a)_{max} = A\zeta_o^n \tag{28}$$

in which (ζ_o) is the percent of critical damping of the system. For Mexico City subsoil in the lacustrine area A = 10.6 n = -0.61. It was found that for different stratigraphical conditions, the coefficient (A) and (n) have a variation on the order of ±20%[19]

In Mexico City for the second mode the coefficients are A = 6.42 and n = - 0.47, respectively.

As an example of the extreme variation of the afore mentioned coeficients the author found for city of San Salvador, El Salvador, C.A., with subsoil conditions completely different from the lacustrine soft clay deposit in Mexico City, the following values A = 8.86 and n = - 0.591[19]

Therefore, conclusions may be drawn for the construction of DAES for general use. Since it appears that the above mentioned coefficients are fairly independent from the stratigraphycal conditions of the site. The author has proposed the following empirical equations for general use when $T_o/T_s = 1$, Fig 18 hence.

$$f_{a1} \cong 9.7\ \zeta_o^{(-0.616)} \tag{29}$$

and for the second mode of vibration at $T_o/T_s \cong 0.33$

$$f_{a2} = 5.8\ \zeta_o^{(-0.477)} \tag{30}$$

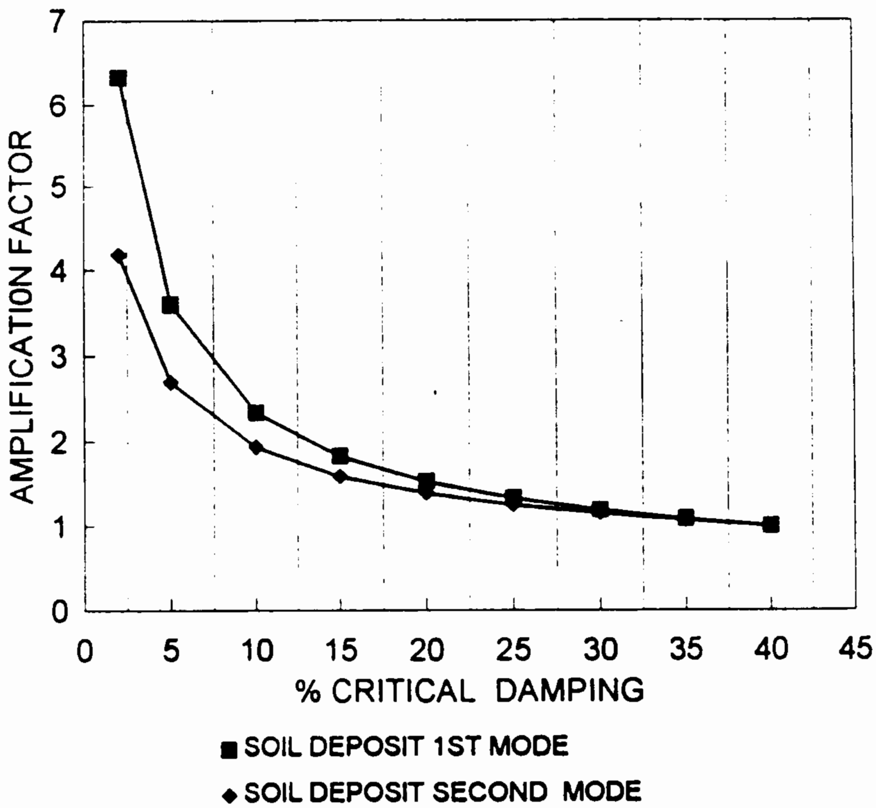


FIG 18. PROPOSED ACCELERATION AMPLIFICATION FACTOR FOR $T_o/T_s = 1$

Using the value of (f_a) from DAES we calculate the inertia force at the equivalent center of mass of the building

Hence

$$V_m = f_a \cdot (a_D M) \tag{31}$$

Here M is the total mass of the building and its foundation, and (a_D) is the maximum acceleration at the foundation grade elevation

The value (a_D) is less in magnitud as the acceleration recorded or assigned at the ground surface (a_s). This may be recognized from the analysis of the shear and the surface waves reported in Fig 7 to 10. Therefore, $a_D = f_d \cdot a_s$, hence from equation (31) we obtain

$$V_{cm} = f_a \cdot f_d \ (a_s M) \tag{32}$$

At the maximum amplitud angle of rocking (θ), the dynamic equilibrium of the system is established. With the use of Fig 14 we create the dynamic maximum rocking moment of the rigid box type foundation, respectively.

$$O_T = V_{cm}\, h_m, O_T = O_{TB} + O_{TW} \tag{33}$$

The reaction moments are defined as follows

$$\begin{aligned} &\text{a) for the base,} \quad O_{TB} = K_{\theta B} \cdot \theta \\ &\text{b) for the wall,} \quad O_{TW} = K_{\theta W} \cdot \theta \end{aligned} \tag{34}$$

The moment shown require the knowledge of the "spring contants" $K_{\theta W}$ and $K_{\theta B}$ respectively. These values are calculated with a soil-structure interaction method devised by the author (11,14,20). The method takes in consideration the soil stratigraphy and the dynamic physical properties of every stratum up to firm ground. The equivalent spring constant of the system is:

$$K_\theta = K_{\theta W} + K_{\theta B} \tag{35}$$

therefore, the period of vibration of rocking of the foundation is

$$T_\theta = 2\pi h_m \sqrt{\frac{M}{K_\theta}} \tag{36}$$

here h_m is the height of the equivalent center of mass of the building from the foundation grade elevation and M the total mass of the system

With the period of vibration of the building frame (T_B) and that of the foundation (T_θ), the coupled period of vibration of the system is computed by means of the equation:

$$T_O^2 = T_B^2 + T_\theta^2 \tag{37}$$

and the approximate equivalent critical damping by

$$\zeta_O^2 \cong \frac{\zeta_\theta^2 T_\theta^2 + \zeta_B^2 T_B^2}{T_B^2 + T_\theta^2} \tag{38}$$

As mentioned before with the values (T_O/T_S) and (ζ_O) we find from DAES, de value of (f_a), Fig 17. With (32) we calculate the overturning moment O_T, and with K_θ we calculate the amplitud of the rocking angle

$$\theta = O_T/K_\theta \tag{39}$$

for the wall we apply the following spring constant [6]

$$K_{\theta w} \cong (1+\nu)\ \mu \cdot D^2 \tag{40}$$

At the foundation grade elevation we use a method of soil structure interaction to calculate the value of $K_{\theta B}$ [14,20]

The foundation base is divided in a par number of strips, Fig 14 and 15, and so many as necessary to obtain a satisfactory accuracy. A unit load is applied on each strip at a time. The influence matrix of unit pressures is computed for the soil mass under the foundation [14,19]. The influence unit pressure matrix $[I_{ji}]$ is multiplied by the vector of the strata compressibilities with the following value

$$\left|\alpha_d^n\right| = \left|\left(\frac{d}{2(1+2\nu)\mu}\right)\right|_n \tag{41}$$

in which (n) is the stratum number, (d) the thickness and (μ) the dynamic soil rigidity. Therefore, for each strip (i) we find

$$[I_{ij}] \cdot \left|\alpha_d^n\right| = \left|\bar{\delta}_{ij}\right| \tag{42}$$

From the matrix equations for each strip we form the flexibility matrix of the soil mass under the foundation structure. this is multiplied by th vector $|P_i|$, representing the unknown unit pressures on each strip, because of the rocking of the building at its maximum amplitud, hence

$$[\bar{\delta}_{ij}] \cdot \left|P_i\right| = \left|\delta_i\right| \tag{43}$$

However, since the rocking produces an anti-symetrical physical action, Fig 14, we calculate $\delta_i = L_i \cdot \theta$, and introducing this value in equation (43) we obtain

$$[\bar{\delta}_{ij}] \cdot \left|(\frac{P_i}{\theta})\right| = \left|L_i\right| \qquad (44)$$

Equation (44) is solved for the values of (P_i/θ)

Therefore, the seismic anti-symmetrical unit pressures (P_i) at the foundation grade elevation because of the maximum amplitud of rocking are obtained with the following equation

$$\left|P_i\right|_{sis} = \pm \left|(\frac{P_i}{\theta})\right| \cdot (\frac{O_T}{K_\theta}) \qquad (45)$$

The total interacting stresses are determined adding the static values Fig 19. The maximum pressures in the subsoil under the foundation structure because $(\pm P_i)_{sis}$ may be calculated with[14]

$$[P_{ij}{}^n] = \Sigma\ [I_{ij}] \cdot \left| P_i^n \right| \qquad (46)$$

The vertical (P_{vz}) pressures because of rocking of the building and those of the surface wave respectively, Fig 7 to 10, will induce at a certain time a maximum shear stress in the soil, in planes XZ at the average depth of the stratum (n), Fig 14

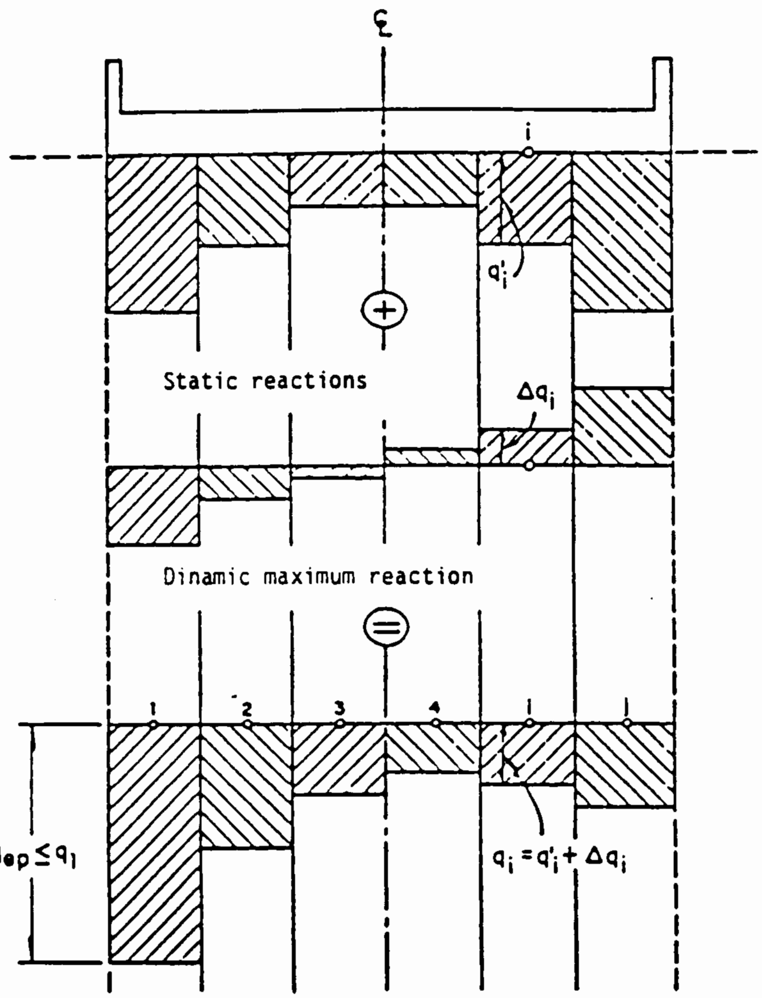
$$\tau^n_{sis} = \frac{1}{2}\ (P_{vz}{}^n - P^n_{hz})_n \qquad (47)$$

Moreover, the building vibrates with a frecuency (ω_B) different from the surface wave frequency (P_s), therefore it is necessary to investigate the possible maximum value of the shear stress during the seismic action. We consider from the safe side a certain number of periodic vibrations hence

$$(\tau_{sis})max = \frac{1}{2}\ (P_{vz} \cdot \sin \omega_B t - P_{hz} \cdot \sin P_s t)max \qquad (48)$$

The values of P_{vz} and P_{hz} are the amplitudes of the pressures respectively. An example of this is given Fig 20

The factor of safety against shear is determined by



Total reactions during seismic action

FIG 19. STRESS DISTRIBUTION IN CONTACT WITH THE FOUNDATION BASE

$$F_S = \frac{S}{\tau} \tag{49}$$

in which (S) is the shear strength of the soil

The cracks that may take place at the ground surface are estimated using the stress profiles given in Fig 7 to 10. For this purpose the horizontal static stress (σ_{xz}) is calculated in the soil mass under the box type foundation grade elevation at depth (z). The difference of the resulting horizontal stresses induced by the surface wave from the static horizontal stresses is calculated with

$$\Delta\sigma_{xz} = \sigma_{xz} - (P_{xz} - w_z) \tag{50}$$

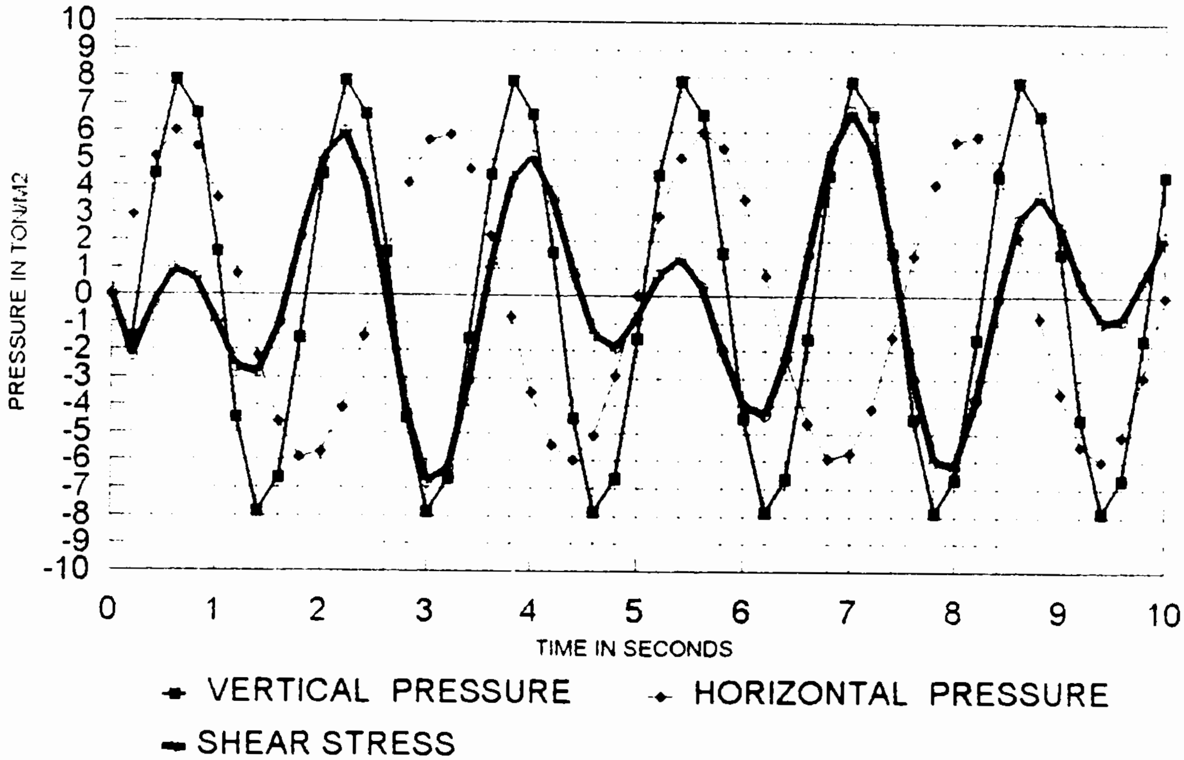


FIG. 20 FOUNDATION SEISMIC SHEAR STRESS

here (w_z) in the "Seismic Pore Water Pressure" according to equation (25), and (24) of Appendix II. if $\sigma_{xz} < (P_{xz}-w_z)$, cracks will take place to depth when $\sigma_{xz} = (P_{xz}-w_z)$ Fig 16. Accordingly the box type foundation will lose its lateral support. Moreover, the stability because of the vertical pressures exerted by the rocking effect shall be verified at the foundation grade elevation by means of the bearing capacity of the soil under the foundation, using the corresponding seismic shear parameters of the soil, and the factor of safety against tilting of the building shall be also determined Fig 21

It is important to mention that when estimating the soil seismic bearing capacity, it is necessary to consider the "Seismic Pore Water Pressure", Which has a dual effect, in the confining pressure and in the seismic friction angle $(\phi)_{sis}$ in soils containing silt and sand [20]. The Figs 22, 23, show one of the many examples of bearing capacity foundation failures because of the aforesaid dual

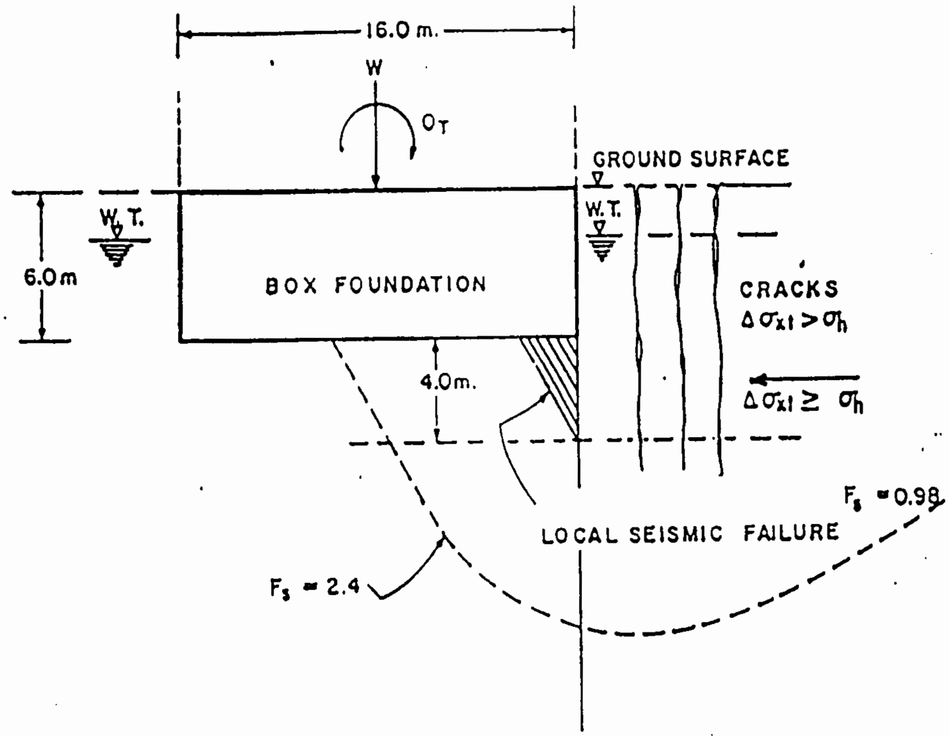


FIG 21. FOUNDATION BEARING CAPACITY ANALYSIS

effect exerted by the "Seismic Pore Water Pressure" in the confining stress and in the ultimate soil bearing capacity

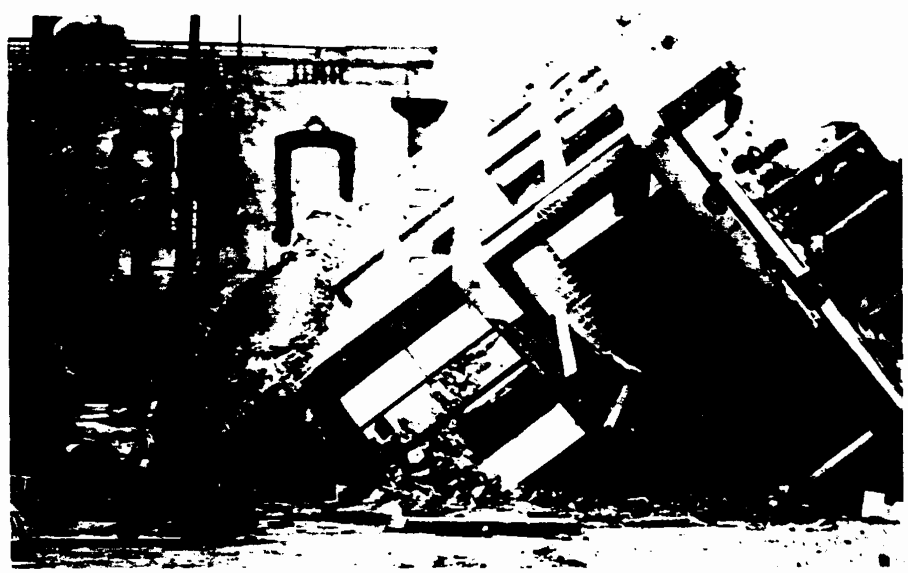


FIG 22. BEARING CAPACITY FOUNDATION FAILURE

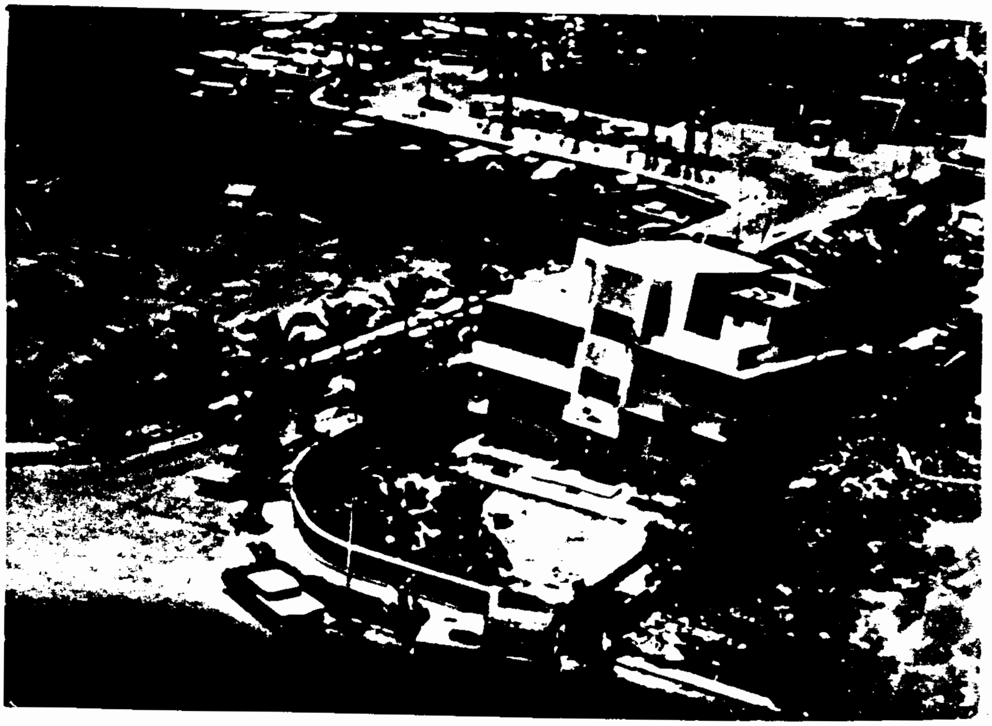


FIG 23. BEARING CAPACITY FOUNDATION FAILURE

VI CONCLUSIONS

The defective behavior of building foundations in earthquake areas of difficult subsoil conditions, may be mitigated when the design is verified with a seismic analysis of stability of the foundation, as explained in the chapters contained herein, and corrections made.

It has been shown that to assign a certain ground surface acceleration from past history or city regulations, is not sufficient to learn on the subsoil behavior. The foundations engineer is compelled to investigate carefully the stratigraphical and dynamical characteristics of the soil strata to a depth not influencing in a mayor way the ground surface and foundations.

The waves orbital velocity and acceleration as well as the celerity shall be investigated for every stratum of the subsoil, and up to the depth of a base hard stratum.

It may be realized, from the contents of the text, that a poor soil seismic behavior is responsible for a poor foundation and building behavior.

The soil, as any other material has a limiting strength and deformational properties. During seismic action wave strains, stresses and displacements take place. This action has to be supported by the limiting strength properties of the soil. Since if the soil fails or deforms excessively no matter how well and strong the foundation and building structure is designed, it will misbehave.

The author considers that in earthquake areas the study of the soil seismic behavior and corresponding factors of safety against seismic soil failure or excessive deformation are of primary importance in foundation engineering. This has been demostrated in Mexico City, and so many other cities around the globe in regions subjected to strong and destructive earthquakes

The author will be greatly gratified if the general philosophy and information given in this paper may serve as an example to the foundations engineering profession.

APPENDIX I

SEISMIC SHEAR WAVE BEHAVIOR OF THE SUBSOIL

The seismic design of the foundation is achieved by means of a quantitative analysis of the seismic behavior of the subsoil. In order to proceed the analysis it is necessary to know the stratigraphy, the hidraulic conditions and dynamic soil rigidity (μ), representative of each one of the strata forming the subsoil. The dynamic parameter (μ) of the soil may be determined my means of the simple "Free Vibration Torsión Pendulum" designed by the author for this purpose, Fig 4.

The definition of the dynamic soil rigidity at certain stress level is

$$\mu = \tau / \gamma \qquad (1)$$

in which τ is the shear stress and γ the angular distortion induced in the soil by the seismic equivolumetric or shear waves that travel from the firm soil to the surface. The seismic waves have different celerities (C_s) according to the values of (μ) for each stratum. The wave celerity is given by $C_{si} = \sqrt{\mu_i / \rho_i}$, in which ρ_i is the unit soil mass. Hence, the time taking by the wave to travel the stratum i of thickness d_i is d_i / C_{si}, and to travel all the soft soil strata will take a time equal to 1/4 of the fundamental period of vibration of the soil deposit that is

$$T_s = 4 \sum_{1}^{n} \frac{d_i}{C_{si}} \qquad (2)$$

The value of T_s represents the largest free period of vibration of the ground, that also creates the largest shear stresses and displacements of the soil mass. Therefore, any rigid element constructed in the subsoil will by subjected to the dynamic drift induced by the horizontal displacements of the soil mass[1]

Let us consider, Fig 1, the relative displacements of the soil mass supported on the firm base. We observe that the stratum at certain depth is disturbed by the seismic waves producing shear stresses in the YZ plane. The dynamic equilibrium of an element of thickens d_i requieres

1) For distortion

$$\frac{\delta_i - \delta_{i+1}}{d_i} = \frac{\tau_i + \tau_{i+1}}{2\mu} \qquad (3)$$

2) For the inertia force

$$(\tau_{i+1} - \tau_i) = (\rho d)_i \, p_n^2 \, \frac{1}{2}(\delta_i + \delta_{i+1}) \qquad (4)$$

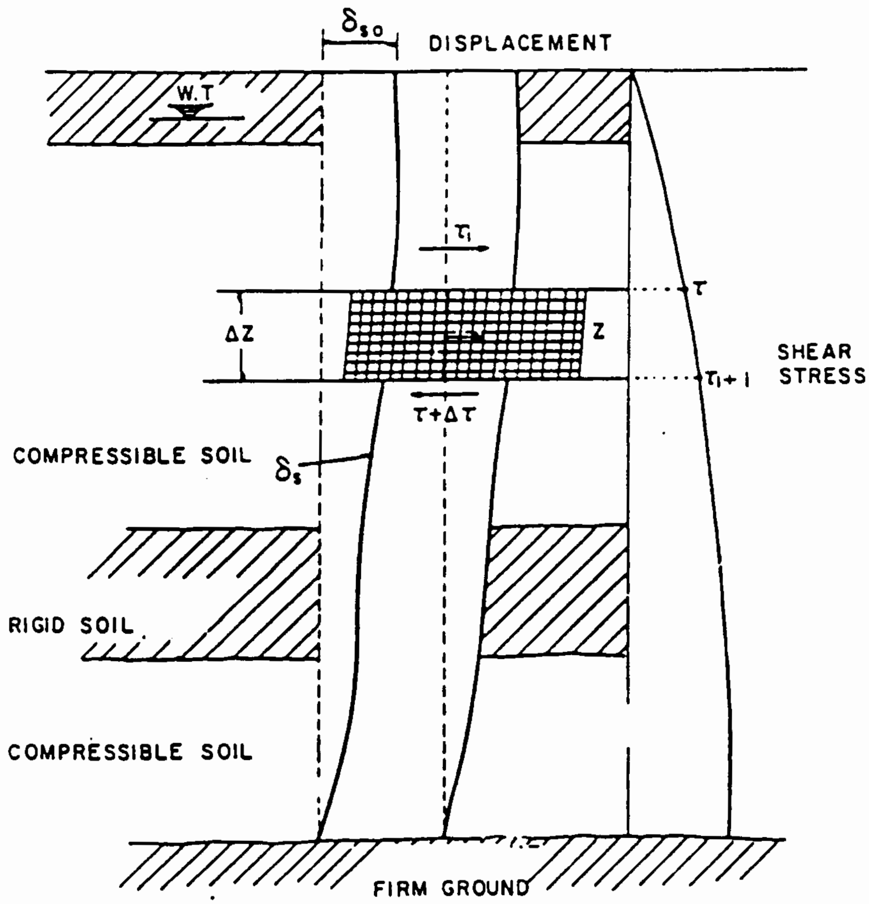


FIG 1. SEISMIC BEHAVIOR OF SUBSOIL

Making algebraic arrangements we find the expressions governing the subsoil movements with circular frecuency (P_s) for the assigned surface acceleration[13], we obtain:

$$\delta_i+1 = A_i\delta_i - B_i\tau_i \qquad (5)$$

$$\tau_i+1 + C_i(\delta_i+\delta_{i+1})+\tau_i \qquad (6)$$

in which

$$A_i = \frac{1 - N_i}{1 + N_i} \qquad B_i = \frac{1}{1 + N_i}\;\frac{d_i}{\mu_i}$$

$$C_i = \frac{1}{2}\,(\rho d_i)\;P_n^2 \qquad N_i = \frac{\rho d_i^2 \cdot P_n^2}{4\mu_i} \qquad (7)$$

With equations (5) and (6) and knowing the assigned acceleration at the ground surface we can find the vibration configuration of the subsoil mass during the seismic motion. The maximum acceleration of the ground surface we designate a_m and p_s the circular frecuency, therefore the surface displacement will be $\delta_{so} = a_m/p_s^2$. With this information we can proceed with the integration step by step by means of equations (5) and (6) until we find that the relative displacement is zero at the firm base where the seismic waves are generated. We obtain with this method the free circular frecuency of the soil mass, the displacements and the shear stresses configuration created by the assigned surface acceleration

APPENDIX II

HORIZONTAL COMPONENT OF THE PLAIN SURFACE SEISMIC WAVE IN STRATIGRAPHICAL SUBSOIL CONDITIONS

The seismo-geodynamics behavior of the subsoil because of the action of the compression and dilatation of the surface seismic waves, is very important to take into account and shall be included in the design of foundations, and any other engineering proyects located under the ground surface as piles, ducts and other underground instalations (13)

A particular solution for the horizontal behavior of the seismic horizontal surface wave in the isotropic half space medium, takes place along a vertical plane (XZ) in the direction of propagation (X). The strain conditions for the plane wave are $\varepsilon_x \neq 0$, $\varepsilon_y = 0$ and $\varepsilon_z \neq 0$ and the pressures $p_x \neq 0$, $p_y \neq 0$, $p_z = 0$ (6,13). The action of this wave is represented by the equation

$$\delta_{xz} = \pm\ \delta_{xo}\ e^{-rz} \sin p_R \left(t - \frac{x}{C_R}\right) \qquad (1)$$

In equation (1) we define

- δ_{xz} , horizontal displacement of the wave at a depth (Z)
- δ_{xo} , horizontal displacement at Z=0
- x , coordinate of reference
- C_R , celerity of the surface wave
- p_R , circular frecuency
- t ; time
- r . atenuation factor with depth

The differential equation of motion (2) shall be satisfied by equa tion (1)

$$C_d^2 \left(\frac{\partial^2 \delta x}{\partial x^2} + \frac{\partial^2 \delta x}{\delta z^2}\right) = \frac{\partial^2 \delta x}{\partial t^2} \tag{2}$$

in which C_d is the compressional wave celerity, hence substituting (1) in (2) we obtain

$$-\frac{P_R^2}{C_R^2} C_d^2 + C_d^2 r^2 = -P_R^2$$

and arranging terms we have

$$r^2 = \frac{P_R^2}{C_R^2}\left(1 - \frac{C_R^2}{C_d^2}\right) \tag{3}$$

From the general theory of the plane surface wave we find[3]

$$\frac{C_R}{C_d} = \alpha^2 \cdot \frac{1-2\nu}{2(1-\nu)}, \text{ and } \frac{C_R}{C_s} = \alpha \tag{4}$$

Substituting (4) in (3) we obtain the atenuation factor (r),

$$r = \frac{P_R}{C_R} \cdot \sqrt{1 - \alpha^2 \frac{1-2\nu}{2(1-\nu)}} \tag{5}$$

We call the radical in equation (5) $a(\nu)$, which is a function of Poisson ratio (ν), it is obtained assigning a value of (ν), between the following limiting values: for $\nu = 0.25$, $a(\nu) = 0.85$, when $\nu \to 0.5$ then $a(\nu) \to 1.0$ on the other hand, we can consider for practical purposes that the ratio of the wave frecuency and celerity, of the shear and surface wave, hold to the following ratio

$$\frac{P_R}{C_R} = \frac{P_s}{C_s} \tag{6}$$

Here p_s and C_s are the circular frequency and celerity of the shear plane wave.

The strain $\partial\delta_{xz}/\partial_x$ in the direction of propagation of the wave and to a depth (z) is

$$\Delta\varepsilon_{xz} \cong \pm \frac{\delta_{xo}p_s}{C_s}e^{-rz} \cos p_s(t- \frac{X}{C_s})$$

but $\delta_{xo}\cdot p_s = V_{xo}$, is the orbital velocity of the wave at the ground surface. Therefore, we obtain

$$\Delta\varepsilon_{xz} \cong \pm \frac{V_{xo}}{C_x} e^{-rz} \cos p_s (t - \frac{X}{C_s}) \qquad (7)$$

Futhermore, we call M_{xz} the plane strain modulus in the (X) direction at depth (z), and the pressure originated by wave is

$$\Delta p_{xz} = \frac{\Delta\varepsilon_{xz}}{M_{xz}} \qquad (8)$$

In terms of the dynamic soil rigidity $M_{xz} = (\frac{1-\nu}{2\cdot\mu})$ the pressure at depth (z) of the plane surface wave is

$$\Delta p_x = \frac{2\mu}{(1-\nu)} \Delta\varepsilon_x \qquad (9)$$

Moreover,considering that

$$\mu = \rho C_s^2 \qquad (10)$$

we find, the dynamic strain modulus as a function of the wave celerity

$$M_{xz} = \frac{(1-\nu)}{2\rho\ C_{sz}^2} \qquad (11)$$

Finally, using equation (7) we find the pressure originated by the wave

$$\Delta p_{xz} = \{\frac{2\rho}{(1-\nu)} C_s^2\}_n \frac{V_{xo}}{C_{so}} e^{-rz} \cos p_s (t- \frac{x}{C_s}) \qquad (12)$$

From equation (12) we may recoqnize that when the soil is isotropic, the maximum configuration of the orbital velocity with depth is given by

$$V_{xz} = V_{xo}\ e^{-rz} \qquad (13)$$

and if $(C_s)_z = C_s$ is considered constant with depth we can write

$$\Delta p_{xz} = \frac{2\rho}{1-\nu} \cdot (C_{so}\ V_{xz})\ \cos p_s\ (t- \frac{x}{C_s} \qquad (14)$$

for the maximum pressure at depth (z), we consider $\cos\ (t- \frac{x}{C_s}) = 1$ then

$$(\Delta p_{xz})\ \max = \frac{2\rho}{1-\nu}\ (C_{so} \cdot V_{xo})\, e^{-rz} \qquad (15)$$

The action of equation (15) shows the wave behavior for an isotropic soil mass.

The intension of the author up to this point, was to present the basic principles for the particular solution of the horizontal behavior of the plane surface wave.

Nevertheless, the subsoil in nature is not isotropic, in contrast we find that the subsoil is stratified having each stratum different dynamic soil parameters. Therefore we need to find an approximate method from the practical engineering point of view to solve this problem, so that the foundation engineer may have a practical and easy tool to estimate the order of magnitud of pressures and displacements produced by this wave. The proposed method described ahead gives the configuration with depth of the soil pressures, accelerations and displacements. The stratigraphy shall be determined as accurately, as possible, also the soil dynamic parameters for all the strata involved in the specific problem, from the ground surface to firm ground

The compatibility conditions call for equal strains at the interface of the strata. Therefore, we establish the conditions shown in Fig 1.

ESTRATA	PARAMETERS ρ	ν	a_ν	C_s	r	d	MAXIMUM STRAIN	COMPATIBILITY
GROUND SURFACE								$\Delta\bar{\epsilon}_1$
1	ρ_1	ν_1	a_1	C_1	r_1	d_1	$\Delta\bar{\epsilon}_1 = V_1/C_1$ $\Delta\epsilon_1 = (V_1/C_1)\,e^{-r_1 d_1}$	$\Delta\epsilon_1 = \Delta\bar{\epsilon}_2$
2	ρ_2	ν_2	a_2	C_2	r_2	d_2	$\Delta\bar{\epsilon}_2 = V_2/C_2$ $\Delta\epsilon_2 = (V_2/C_2)\,e^{-r_2 d_2}$	$\Delta\epsilon_2 = \Delta\bar{\epsilon}_3$
3	ρ_3	ν_3	a_3	C_3	r_3	d_3	$\Delta\bar{\epsilon}_3 = V_3/C_3$ $\Delta\epsilon_3 = (V_3/C_3)\,e^{-r_3 d_3}$	$\Delta\epsilon_4 = \Delta\bar{\epsilon}_4$
.	.	.	.	.	.	.	$\Delta\bar{\epsilon}_4 = V_4/C_4$	
n-1	ρ_{n-1}	ν_{n-1}	a_{n-1}	C_{n-1}	r_{n-1}	d_{n-1}	$\Delta\epsilon_{n-1} = (V/C)_{n-1}\,e^{-(r\,d)_{n-1}}$	$\Delta\epsilon_{n-1} = \Delta\bar{\epsilon}_n$
n	ρ_n	ν_n	a_n	C_n	r_n	d_n	$\Delta\bar{\epsilon}_n = V_n/C_n$ $\Delta\epsilon_n = (V/C)_n\,e^{-(r\,d)_n}$	$\Delta\epsilon_n = \Delta\bar{\epsilon}_{n+1}$
n+1	ρ_{n+1}	ν_{n+1}	a_{n+1}	C_{n+1}	r_{n+1}	d_{n+1}	$\Delta\bar{\epsilon}_{n+1} = (V/C)_{n+1}$	

FIG 1. COMPATIBILITY OF STRAIN BETWEEN STRATA INTERFACES

Hence: Interface $(\Delta\varepsilon_{n-1} = \Delta\varepsilon_n)$

Surface $\dfrac{V_1}{C_1}$

1 - 2 $\dfrac{V_1}{C_1}\,e^{-r_1 d_1} = \dfrac{V_2}{C_2}$

2 - 3 $\dfrac{V_2}{C_2}\,e^{-r_2 d_2} = \dfrac{V_3}{C_3}$

3 - 4 $\dfrac{V_3}{C_3}\,e^{-r_3 d_3} = \dfrac{V_4}{C_4}$

. . . .

. . . .

$(n-1)-n$ $$\frac{V_{n-1}}{C_{n-1}} e^{r_{n-1}d_n} = \frac{V_n}{C_n}$$

From the expression above mentioned we calculate the maximum strain at the interface (n) of two strata

$$\Delta\varepsilon_{xn} = \frac{V_n}{C_n} e^{-r_n d_n}$$

and substituting above the relations for each strata interface we obtain

$$\Delta\varepsilon_{xn} = \frac{V_1}{C_1} \cdot e^{-r_1 d_1} . e^{-r_2 d_2} . e^{-r_3 r_3} ... e^{-r_n d_n}$$

or

$$\Delta\varepsilon_{xn} = \left(\frac{V_1}{C_1}\right) e^{-\sum_1^n r_i d_i} \qquad (16)$$

The equation (16) gives the configuration of the strain with depth at the interface of the strata satisfying the compatibility of deformation. Here V_1 and C_1 represent the orbital velocity and celerity of the wave at the ground surface, respectively.

Notice, from equation (16) if the soil is considered isotropic with depth, then $r_n = r$, equal constant and $\Sigma d_i = z$, and equation (16) takes the value of equation (7) for the maximum soil strains in the isotropic soil mass.

The pressure distribution with depth may be obtained multiplying the value of the strain equation (16) by the dynamic soil modulus Ref (11) $(1/M_{xz})$, therefore we obtain

$$(\Delta p_x)_n = \pm \left(\frac{2\rho}{1-\nu} C_s^2\right)_{n-1} \cdot \frac{V_1}{C_1} e^{-\sum_1^n (r_i d_i)} \qquad (18)$$

with equation (18) we calculate the pressure in each stratum.

1) at the bottom of the stratum, (n-1)

$$(\Delta p_x)_{n-1} = \left(\frac{2\rho}{1-\nu} C_s^2\right)_{n-1} \cdot \frac{V_1}{C_1} e^{-\sum_1^{(n-1)} (r_i d_i)} \tag{19}$$

2) at the top to of the stratum, (n)

$$(\Delta \bar{p}_x)_n = \left(\frac{2\rho}{1-\nu} C_s^2\right)_n \cdot \frac{V_1}{C_1} e^{-\sum_1^{(n-1)} (r_i d_i)} \tag{20}$$

from above mentioned equations we find the maximum orbital velocity with depth

$$(v_x)_n = v_1 e^{-\sum_1^{n} (r_i d_i)} \tag{21}$$

and the accelerations

$$(a_x)_n = a_{x1} e^{-\sum_1^{n} (r_i d_i)} \tag{22}$$

The effective stresses may be computed knowing the "Seismic Pore Water Pressure" created by the horizontal component of the surface wave in every soil strata. In the particular case of a water satured soil we may use the following maximum value(17)

$$\Delta w_{cz} = \frac{1}{1+\beta_{cx}} \Delta p_{xz} \tag{23}$$

in which

$\beta_{cx} = \left(\frac{M_e}{M_c}\right)_z$

M_{ez} Dynamic strain modulus for vertical expansion or stress relief.

M_{cz}, Dynamic strain modulus for horizontal compression

The value of β_{cx} is a function of the stress level induced by the seismic wave in relation to the soil strength(3). For low ratios the order of magnitud of $\beta_{cx} \cong 0.95$ and for high ratios it has the tendency to reach a value on the order of (0.5). However, for usual stress levels in practice we may consider $\beta_{cx} \cong 0.85$ in the saturated Mexico City silty clay (7,11)

Therefore, the effective stress reads approximately as follows:

$$\sigma_{xz} = \frac{\beta_{cx}}{1+\beta_{cx}} \cdot p_{xz} \cong 0.46\, p_{xz} \qquad (24)$$

To illustrate the method herein presented, a sample of computation will be given to show the configurations of the action induced by the horizontal component of the plane surface seismic wave.

The dynamic parameters and subsoil stratigraphy, where carefully determined for a site to the south of Mexico City center. The calculation may be easily followed in Table 1, and the results are reported in Fig 2 in graphical form

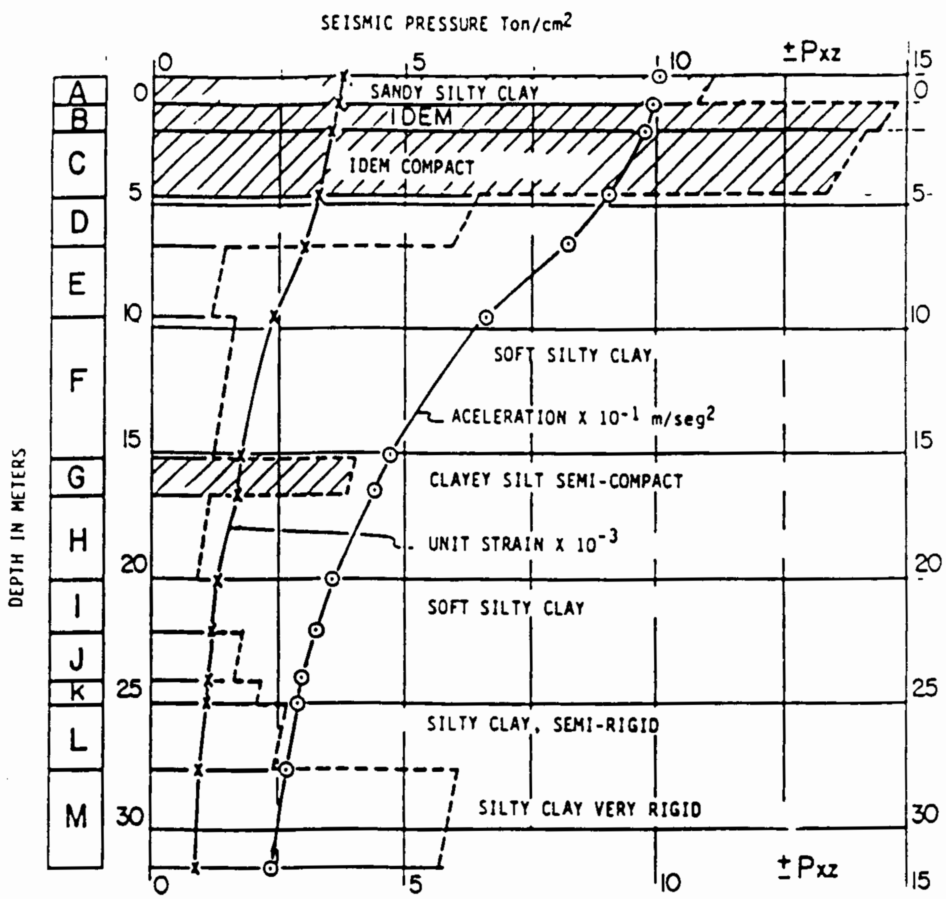


FIG 2. SEISMIC HORIZONTAL PRESSURES AND ACCELERATIONS FOR A SURFACE WAVE. GROUND SURFACE ACCELERATION 100 cm/sec^2

TABLE 1. SEISMIC PRESSURES CALCULATION

SOIL	ESTRATA	PARAMETERS μ	ρ	ν	C_s	d	$(a_v)_n$	$1/H_n$	r	$r\cdot d$	$\Sigma r\cdot d$	$e^{-\Sigma r\cdot d}$	$\Delta\epsilon_n$	Δp_n	a_c	ℓ_{cx}	$\Delta\sigma_n$
		T/m^2	$T\cdot s^2/m^4$	-	m/s	m	-	T/m^2	1/m $\times10^{-2}$	- $\times10^{-2}$	$\times10^{-3}$	-	$\times10^{-3}$	T/m^2	m/s^2	-	T/m^2
CLAYEY SILTY SAND	A	1140	0.136	0.25	91.66	1.10	0.85	3040	2.78	3.06	3.05	1.00 0.970	3.65 3.54	11.10 10.76	1.0 0.97	0.85	11.10 10.76
IDEM	B PIAS V	1560	0.144	0.25	104.08	1.00	0.85	4160	2.44	2.44	5.49	0.947	3.46	14.73 14.38	0.95	0.85	14.16 14.36
IDEM	C	1536	0.141	0.25	104.37	2.40	0.85	4096	2.44	5.86	13.35	0.893	3.26	14.16 11.35	0.89	0.85	6.51 6.13
CLAYEY SILT	D	636	0.131	0.35	69.68	2.10	0.89	1957	3.82	8.02	19.37	0.824	3.01	6.38 5.89	0.82	0.90	3.02 2.79
SILTY CLAY	E	144	0.115	0.35	35.39	2.90	0.89	443	7.52	21.81	41.18	0.662	2.42	1.33 1.07	0.66	0.90	0.63 0.507
IDEM	F	210	0.114	0.35	42.92	5.60	0.89	646	6.20	35.17	76.35	0.466	1.70	1.56 1.10	0.47	0.90	0.739 0.521
CLAYEY SILT	G	864	0.137	0.25	79.40	1.40	0.85	2304	3.20	4.48	80.83	0.446	1.63	3.94 3.77	0.45	0.90	1.87 1.78
SILTY CLAY	H	198	0.111	0.35	42.23	3.50	0.89	609	6.30	22.06	102.89	0.357	1.30	0.99 0.79	0.36	0.80	0.44 0.351
IDEM	I	318	0.125	0.36	50.94	2.00	0.89	978	5.28	10.55	113.44	0.322	1.18	1.28 1.15	0.32	0.90	0.602 0.545
IDEM	J	406	0.110	0.35	66.47	2.00	0.89	1495	4.00	8.00	121.44	0.297	1.09	1.76 1.63	0.30	0.90	0.857 0.794
CLAYEY SILT	K	648	0.114	0.35	75.39	1.00	0.89	1994	3.53	3.53	124.97	0.287	1.05	2.17 2.09	0.29	0.95	1.057 1.018
SILTY CLAY	L	816	0.120	0.35	82.46	2.70	0.89	2511	3.23	8.72	133.68	0.263	0.961	2.64 2.42	0.26	0.94	1.286 1.174
IDEM	M	2040	0.135	0.35	122.93	3.80	0.89	6277	2.16	8.21	141.91	0.242	0.884	6.05 5.57	0.24	0.95	2.938 2.704

FIRM SOIL

PERIOD T=2.10 sec, CIRCULAR FRECUENCY P_s = 2.99 rad/sec, SURFACE ACELERATION 1m/sec²

SURFACE ORBITAL VELOCITY V_1 = 0.334 m/sec, SURFACE STRAIN 3.65 x 10^{-3}

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