PIPELINE DESIGN AGAINST SAND LIQUEFACTION IN ISLA DEL CARMEN, MEXICO

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ABSTRACT

The seismic design analysis for the sewage pipeline system in the city of Carmen at "Isla del Carmen", in the State of Campeche, Mexico, is presented. This city is founded on very fine loose silty sand deposits, in an area where the groundwater level is very shallow. Due to the seismic risk of this area, it was necessary to carry out cyclic triaxial tests to measure the susceptibility of soil liquefaction, and to study the stress and strain conditions of the segmented pipelines. These pipelines made of asbest-cement, varied between 60 to 180 cm in diameter. The results of these studies showed that the liquefaction of the soil deposits in that area will not occur under the effects of an earthquake of magnitude in the order of 6.4. Regarding the effects of ground strain and curvature on pipelines, the results obtained indicated that the expected displacements during an earthquake can be absorbed by the displacement capacity of the pipeline joints.

INTRODUCTION

The sewage pipeline system of Ciudad del Carmen, Mexico, represents more than 20 km of mains and more than 100 km of networks. The pipelines are made of asbest-cement and have a diameter that varies between 0.60 to 1.80 m. These pipeline system is actually under construction.

Since the groundwater level is very shallow (0.5 to 1.5 m), and a relatively thick loose fine silty sand deposit was encountered at the site, it was necessary to verify if an earthquake motion could cause liquefaction of this deposit or induce ground displacements that might yield pipeline displacements larger than those allowed by the pipeline joints.

Therefore liquefaction analysis was performed in order to investigate and evaluate the seismic risk potential of the loose silty fine sand deposit. In a first step, deterministic and probabilistic approaches were used. Further, an experimental program was carried out in order to assess the crucial parameters of soil behavior when subjected to cyclic loading. Thus, the results provided essential information in order to decide if special construction methods should be applied to alter the soil characteristics and to prevent liquefaction, or design the sewage lifelines for earthquake effects.

The analyses included the effects of ground strain and curvature on pipelines and joints subjected to movements caused primarily by wave propagation during the earthquake. Special considerations were taken to design the junction between pipes and buried structures, such as inspection-wells and pumping plants.

SITE DESCRIPTION

Ciudad del Carmen is located at the Campeche State of Mexico (91°45'W and 18°45'N). This city has a particular importance because it is an access to the petroleum activities in the Campeche Sonde, where PEMEX takes out about 65% of the total national production. The city was founded on the Isla del Carmen, near the west mouth of the Terminos Lagoon, which ends in the Gulf of Mexico. The batimetry is about 20 cm/km at this point, and the topographic contour of the city is noticeably flat, with altitudes typically less than 2 m above the sea level. Pluviometry is high, and can reach 419 mm in 24 hours, with 1800 mm at a year. The wind velocity can be as strong as 160 km/h, and the sea level can raise between some centimeters to 0.5 m during calm and storm conditions, respectively.

From a tectonic point of view, the Isla del Carmen is located near the boundary between the Yucatan Platform at the north, the Massif of Chiapas at the south, and the Macuspana Basin of the flexured Reforma-Akal Calcareous Belt at the west-northwest. The island is closely from a very complex tectonic ocean deep named the Paleocanyon of Chilam formed during the Pleistocene ocean regression, which is plenty of normal and inverse faults, angular discordances, and pronounced compressional folds. At about 50 km from the island there is a group of normal faults, caused by compressional stresses, with general alignment NW-SE. It has been assumed that there is 650 m of Quaternary sediments at this place. The tectonic characteristics on the one hand, and the influence of strong earthquakes in the Pacific Subduction zone on the other, leads to put Ciudad del Carmen into the seismic zone B of Mexico; according to the last Seismic Risk Zonation for Mexico, the maximum ground horizontal acceleration (a_{max}) is 0.09g, with a maximum ground velocity (V_{max}) of 14.8 cm/sec.

STRATIGRAPHY

Based on 17 boring performed for this investigation, the typical stratigraphy between the ground level and the explored depth of 18 m, can be described as follows.

The most shallow strata consists of 2 to 3 m thick of light brown calcareous fine to medium SAND (SP), with 3 to 12% of gravel (<2") to sand sized intact and fragmented shells, and 3 to 5% of low compressibility silt, mica (SP-SM). Sometimes, this strata presents a very high shell content and must be classified as shell layer (GP, GM). Relative density is erratic from medium to dense, and loose in few thin lenses. The unit weight was estimated to be in the range of $17.2 \leq \gamma_m \leq 19.6$ kN/m^3, as computed from moisture content (w) and gravity of solids (G_s).

Underlying the most shallow strata it is a deposit of dark greenish gray SILTY FINE SAND ($d_{50} \leq 0.15$ mm), with some thin lenses of intact and fragmented little shells (5%, d<8 mm); the silt content is 15 to 39% of ML. At someplace there are thin layers of soft dark and light greenish brown CLAY (CH) with traces of fine sand, shells and colloidal organic matter. This deposit extend up to 7.2 to 9.8 m depth, thus with a variable thickness between 3.9 to 7.3 m. Relative density is also erratic: medium to loose at the west side of the city, medium to dense at the East side, and otherwise it is at a loose to very loose state. The unit weight of this soil was assumed to be $\gamma_m = 17.9$ kN/m^3.

Below this strata and up to 14 m depth, there is a sequence of soft and very soft light greenish gray CLAY (CH) layer with lenses of soft SILTY CLAY and loose CLAYEY SAND. For a depth greater than 14 m and up to 20 m, the soil is similar in nature but its consistency is firm to hard. The unit weight was measured in the range of $1650 \leq \gamma_m \leq 1875$ kg/m^3 between 9 and 14 m, and in the range of $18.6 \leq \gamma_m \leq 20.1$ kN/m^3 below 14 m. Undrained strength was measured as $\phi_{uu} = 0°$ with $24 \leq c_{uu} \leq 42$ kN/m^2 between 9 to 14 m, and $\phi_{uu} = 0°$ and $c_{uu} \geq 136$ kN/m^2 for depth over 14 m. Fig 1 shows the stratigraphy at boring SM-9.

LIQUEFACTION ANALYSIS

Lateral spread due to soil liquefaction could be triggered when the layers slope is over 0.5 to 1% [Youd and Perkins (1978), Nyman *et al.* (1984), and Flores-Berrones and O'Rourke (1992)]. However, the topographic and geologic characteristics of Ciudad del Carmen allow to assume that this kind of failure has a very little probability to occur, because the ground slopes are really very flat. On the other hand, liquefaction of the saturated sand deposit may induce ground settlements and/or pipe flotation during earthquake, even for flat ground, and then, liquefaction susceptibility of the sand deposit must be analyzed.

Hamada *et al.* (1985) report that gas and sewage pipelines damage at Noshiro City was relatively reduced where the ground surface is almost flat and the permanent displacements were very small. These authors report also that severe damage on lifelines was observed where lateral spread take place and the permanent ground displacement became large. Kawashima *et al.* (1985) report that for main sewage pipes at Noshiro City failures were concentrated at reinforced concrete pipes joints of tongue and groove joint type, with 95% of failures for pipes with diameter between 6 and 60 cm; a 67% of pipe failures was observed for an embedment less than 1.8 m; while a 72% of joint failures was observed between 1.5 to 3 m depth.

Liquefaction susceptibility has been analyzed in regarding the following fundamental aspects of the problem: (a) the Seed procedure based on standard penetration test and for the earthquake to considered in design; a liquefaction susceptibility safety factor was computed at different depths. (b) Cyclic load undrained triaxial tests with pore pressure measurement were carried out on reconstituted and "intact" soil samples with different fines content. (c) In-Situ recognition and inspection of ancient buildings in the city as well as observation damages during past earthquakes.

The susceptibility analyses are based principally on the works of Seed and Idriss (1971, 1982), Seed (1979, 1987), Seed *et al.* (1983, 1984), Castro (1975), Tokimatsu y Yoshimi (1983, 1984) and Tokimatsu andy Seed (1987), as well as others reported in the references.

Seed and Idriss assume that for practical purposes the average shear stress (τ_{av}) due to the earthquake is about 65% of the shear stress computed at the maximum horizontal ground acceleration. Thus, it can be obtained as follows,

$$\tau_{av} \cong 0.65 \frac{\gamma_m z}{g} a_{\max} r_d \qquad (1)$$

where **g** is the acceleration of gravity, **z** is the ground depth under consideration, and $\mathbf{r_d}$ is a stress reduction factor recommended by Seed and Idriss (1971, 1982) to take into account the soil deformation during the shaking.

Data from the Geophysics Institute of UNAM as well as those of seismic risk studies performed by Guerra and Esteva (1978), Guzmán (1982) and Chávez (1987) for the Campeche Bay, lead to suppose that seismic magnitude could be in the order of 6. Moreover, a very recent seismic risk study (MDOC, 1993) allows to assume that $a_{\max}$ is 0.09g, for a return period of 100 years.

Seed *et al.* (1984) suggest to use a scale coefficient (r_m) in order to take into account the earthquake magnitude for a particular site and to allow using of charts normalized to a magnitude of 7.5. For this case $\mathbf{r_m} = 1.2$, when considering the Jáltipan-Coatzacoalcos, August 29, 1959 earthquake, which had a magnitude of 6.4 at the epicentral area.

In order to take into account several factors which affects the in situ **N** value obtained in the standard penetration test, correction factors were applied based on the works of Tokimatsu and Yoshimi (1983, 1984), Skempton (1986), and Liao and Whitman (1986). The in situ **N** value was normalized to a vertical effective soil stress of 98.1 kN/m^2; and the other factors were also considered by reducing the obtained values by a factor of 0.75. Furthermore, in order to include the fines particle influence on liquefaction, as has been pointed out by Seed *et al.* (1984) and Ishihara (1985), equivalent blows could be added as ΔN_1, which might be computed as a function of the soil fines content (**F**), as: $\Delta N_1 = 0$; $= 0.6(F-5)+1$; and $= 0.1(F-10)+4$; for $F < 5\%$, $5 \leq F \leq 10\%$, and $F \geq 10\%$, respectively. N_1 was then corrected according with

$$(N_1)_{60} = 0.75 \left(\frac{167}{69 + \gamma_m z} \right) N + (\Delta N_1) \tag{2}$$

where γ_m is the unit weight of soil (kN/m^3) and **N** is the blow count, at a depth **z** (m).

The equation leading to compute the susceptibility factor ($\mathbf{F_L}$) at a depth **z**, as it has been used for the analysis presented in this paper, can be expressed as follows

$$F_L = \frac{r_m \, \tau_{hz}}{\tau_{av}} \tag{3}$$

where (τ_{hz}) is the shear stress to induce liquefaction.

Alternatively, the probabilistic criteria proposed by Liao, Veneziano and Withman (1986), have been used in order to have an estimate of the liquefaction probability $\mathbf{P_L}$. The resulting equation for Ciudad del Carmen is as follows,

$$P_L = \frac{1}{1 + \exp\left(- \left(10.167 + 4.1933 \ln\left(0.65 \, a_{\max} \frac{\sigma_z}{\sigma_z'} \frac{r_d}{r_m} \right) - 0.24375 \, (N_1)_{60} \right) \right)} \tag{4}$$

where σ_z' and σ_z are the effective and total overburden stresses, respectively.

RESULTS

Liquefaction susceptibility and probability have been estimated for selected borings and the results are presented in fig 2. This figure shows that only in the SPE-2 boring the liquefaction susceptibility factor was less than 1.1, considering that this is a reasonable minimum value to assume that no liquefaction take place. For this case the liquefaction probability was $\mathbf{P_L}=0.39$, that is very near to the maximum acceptable value of 0.40. Alternatively, Ishihara (1985) proposed that $\Delta \mathbf{N_1} = \mathbf{6.5} \, log_{10}\mathbf{N}$, and in this case the factor $\mathbf{F_L}$ will be 1.47.

Because the exposed method can be used as a reference or for estimation purposes only, a laboratory test program was proposed and carried out in order to confirm these preliminary results. Silty sand samples recuperated by thin wall sampler, and reconstituted soil samples with different densities and fines content were tested. Cyclic load undrained triaxial tests with pore pressure measurements were performed and representative results are presented herein in table 1 and in fig 3. In order to have reliable pore pressure measurements, relatively high backpressure was applied to keep a Skempton B parameter greater than 0.98 in all cases. After pore pressure stabilization, the samples were subjected to 30 cycles of load as follows: 15 cycles with a cyclic deviatoric stress (q_C) selected to be at least twice those stress induced in the subsoil during the earthquake. For the last 15 cycles q_C was incremented up to 6 times the induced stress.

The results showed soil dilatance for samples recuperated or prepared with fines content varying from 15 to 90%. Thus, these results as well as those obtained by the empirical method based on SPT and by the probabilistic method, allow to assume that liquefaction of the loose silty sand deposit will not occur under the effects of an earthquake of magnitude of 6.4.

It is believed that the sharp angles of the sand and silty grains, together with the carbonate content of the soil deposits, are factors that contributed to these results.

EFFECTS OF GROUND MOTION

The effects of ground strain and curvature on pipelines, in addition to relative joint displacement and rotations were analyzed for Rayleigh waves for which the dispersion curve was computed as show in fig 4. Furthermore, the particle velocity at Isla del Carmen has been estimated as V_{max} = 14.8 cm/sec, and the ground acceleration as a_{max} = 88.3 cm/sec^2. As the pipelines will be embedded at a depth greater than 1.8 m, the soil-pipe interaction was taken into account.

A simplified analysis was also performed considering the propagation of a plane wave traveling in the longitudinal and transverse axes of the pipeline. In this analysis, it is assumed that the pipe has no stiffness or mass and, hence, the strain and curvature are the same as those experimented by the soil (*i.e.* an analysis without soil-pipe interaction). Thus, the axial strain of the pipe is equal to the maximum free-field ground strain, ϵ_g, due to the seismic waves and was computed as

$$\epsilon_g = \pm \frac{V_{max}}{C_L} \tag{5}$$

where V_{max} is the maximum ground velocity and C_L is the apparent longitudinal propagation velocity of the seismic waves.

The upper bound for the curvature of the pipeline corresponds to the maximum soil curvature, θ_g, and has been computed as

$$\theta_g = a_{max} / C_T^2 \tag{6}$$

where C_T is the apparent transverse propagation velocity of the seismic waves.

In order to compute the effects on the pipeline joints spaced at a distance L_p, it is assumed that the pipeline consists of rigid segments at which the middlepoints move with the soil. Thus, the maximum relative motion-rotation between two points on the ground will be entirely accommodated by movement at the joints. Hence the upper bound of the maximum joint displacement (Δ_J) was conservatively computed as

$$\Delta_J = \epsilon_g L_p = \pm \frac{V_{max} L_p}{C_L} \tag{7}$$

And the upper bound of the maximum joint rotation (θ_J) was conservatively computed as

$$\theta_J = \theta_g L_p = \frac{a_{max} L_p}{C_T^2} \tag{8}$$

On the other hand, if the relative displacements between the pipeline and the ground are taken into account, the resultant force due to the friction at the interface (t_U) during the earthquake can be computed as

$$t_U = \pi D \left(\frac{1 + K_o}{2} \right) \gamma_m H \tan \delta \tag{9}$$

where **D** is the external pipe diameter computed as 1.08 times the internal or nominal diameter of the pipe; K_0 is the horizontal earth pressure coefficient at rest which was taken as 0.63 for $\phi = 22°$; γ_m is the unit weight of the soil overlaying the pipe (18.64 kN/m^3); **H** is the backfill thickness; and δ was equal to 20° for 0.9 tan ϕ.

According with O'Rourke and El-Hmadi (1985 and 1988), the maximum length over which the relative displacement between the pipe and soil takes place (L_S) can be computed as

$$L_S = \frac{\epsilon_g E_p A_p}{t_U} \tag{10}$$

where $\mathbf{E_p}$ is the Young's modulus of the pipe (25,000 MN/m^2); and $\mathbf{A_p} = \pi \mathbf{t_p}(\mathbf{D_e} - \mathbf{t_p})$, is the transversal area of the pipe, with $\mathbf{t_p}$ and $\mathbf{D_e}$ the pipe thickness and its external diameter respectively.

Assuming that the wave propagation is harmonic, the maximum strain occurs at each 1/4 of the wave length. Thus, the pipe strain will be equal to ϵ_g, and can be computed as follows,

$$\epsilon_a = \frac{\lambda \, t_U}{4 \, E_p \, A_p} \tag{11}$$

where λ is the wave length in meters (m).

The results obtained for pipes of nominal diameter of 45 and 110 cm, when the soil-pipe interaction is taken into account for different thickness of the backfill ($\mathbf{H_{BACKFILL}}$ of 1.5, 3, 3.5, 4, 4.5 and 5 m thick), together with those obtained without soil-pipe interaction, are show in fig 5.

From fig 5, the longitudinal (or axial) strain for design of a pipeline with a nominal diameter **D**, corresponds to the intersection between the curves obtained with and without soil-pipe interaction.

For the pipelines with flexible joints, considering that the longitudinal and transverse Rayleigh waves velocity is 140 m/sec, and for a pipe length of $\mathbf{L_p}$ = 5 m, the maximum estimated displacement in the joints are $\Delta = \pm 5.3$ mm, and the rotation $\theta_J = 0.01°$.

The asbest-cement pipelines for Ciudad del Carmen consist of straight pipes class B-7.5, jointed spigot-to-spigot and separated 10 mm. The segments are jointed using a straight cylindrical pipe coupling with special rubber gasket joints.

The manufacturer had determine experimentally that for this kind of flexible pipe coupling the allowable relative displacements and rotation have the magnitudes indicated in table 2. The pipes have a longitudinal compressional strength of 50 MN/m^2, a flexure permissible strength of 25 MN/m^2 and, and a permissible internal pressure of 2.5 MN/m^2.

Regarding the values reported in table 2, the results obtained show that the expected displacements during the earthquake can be absorbed by the displacement capacity of the pipeline joints, assuming that the maximum total displacement in the joint is less or equal than 5 mm.

From the obtained results, it was concluded that the selected pipelines constitute an economical and secure solution for the sewage pipeline system of the city.

CONCLUSIONS

The main conclusions of this paper are the following:

1) The uncertainties related to the possibility of occurring soil liquefaction in the loose silty sand deposits of Ciudad del Carmen, Mexico, where a sewage pipeline system of asbest-cement has to be installed, were dissipated after carrying out a soil liquefaction analysis, and performing several cyclic undrained triaxial tests with both, undisturbed and reconstituted soil samples; these studies were complemented by a probabilistic approach to know the susceptibility of liquefaction. The experimental results as well as those obtained by the empirical method based on SPT and by the probabilistic method, allow to assume that liquefaction of the loose silty sand deposit will not occur under the effects of an earthquake of magnitude of 6.4.

2) The results of the cyclic triaxial tests showed soil dilatance for samples recuperated or prepared with fines content varying from 15 to 90%. It is believed that the sharp angles of the sand grains and the medium (15%) to high (78%) content of silt, together with the carbonate ($CaCO_3$) content of the soil deposits, are factors that effectively contributed to these results.

3) For the pipelines of asbest-cement with flexible joints, considering a backfill of 3 m and a maximum particle velocity of 14.8 cm/sec, a maximum ground acceleration of 88.3 cm/sec^2, a Rayleigh wave velocity of 140 m/sec, and a pipe length of 5 m, the estimated maximum displacement in the joints was 5.3 mm, and the maximum rotation of 0.01°. According to the manufacturer information, give in table 2, those values are well bellow the permissible ones. Therefore, the selected pipelines for the sewage system in Ciudad del Carmen might be considered, from the seismic point of view, on the safe side.

4) More refined mathematical modelling of the soil-pipe interaction could be possible if the mechanical behavior of the joints of the asbest-cement pipes is correctly taken into account. In order to do this, experimental results on the stress-strain relationship and stiffness, obtained from tests carried out on real joints subjected to similar forces or displacements as it will be under the effects of the earthquake for design, are needed. The authors expect that, with the collaboration of the manufacturer, a research program could be undertaken in this direction in the very near future by the National Water Commission of Mexico (CNA).

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Table 1. Cyclical undrained triaxial tests with pore pressure measurements and constant volume on reconstituted samples				
INDEX PROPERTY	**Initial**		**Final**	
	SM-9	**SM-3**	**SM-9**	**SM-3**
Moisture content (w) %	6.70	10.3	35.8	30.8
Dry unit weight (γ_d) kN/m^3	13.36	14.40	13.34	14.41
Wet unit weight (γ_m) kN/m^3	14.25	15.88	18.12	18.85
Saturation degree (S_r) %	18.4	33.5	98.0	99.0
Void ratio (e)	0.984	0.831	0.988	0.841
Property or parameter	**Sample: SM-9**		**Sample: SM-3**	
SUCS classification: SP (fines ML, $CaCO_3$) F	28		78	
Relative gravity of solids (G_S)	2.703		2.704	
Effective consolidation pressure (p_o') kN/m^2	58.86		58.86	
Parameter B (Skempton)	0.985		0.993	
Cyclic load was applied in 3 sets of 15 cycles each with $\frac{\tau_{cyc}}{\sigma_{vo}}$ of: 0.53, 0.76 and 1.53				

Table 2. Manufacturer permissible displacements for asbest-cement pipeline joints			
Nominal Diameter Asbest-Cement Pipe Class B-7.5 (mm)	**Relative displacement between coupled pipes**		**Permissible rotation for coupled pipes (degrees)**
	Pull-out (Tension) (mm)	**Pull-in (Compression) (mm)**	
450	$\geq$ 40	10	**2.5°**
600 to 750	$\geq$ 45	10	**2.5°**
900 to 1100	$\geq$ 50	10	**2.0°**
1500	$\geq$ 100	10	**1.5°**
1800	$\geq$ 100	10	**1.0°**

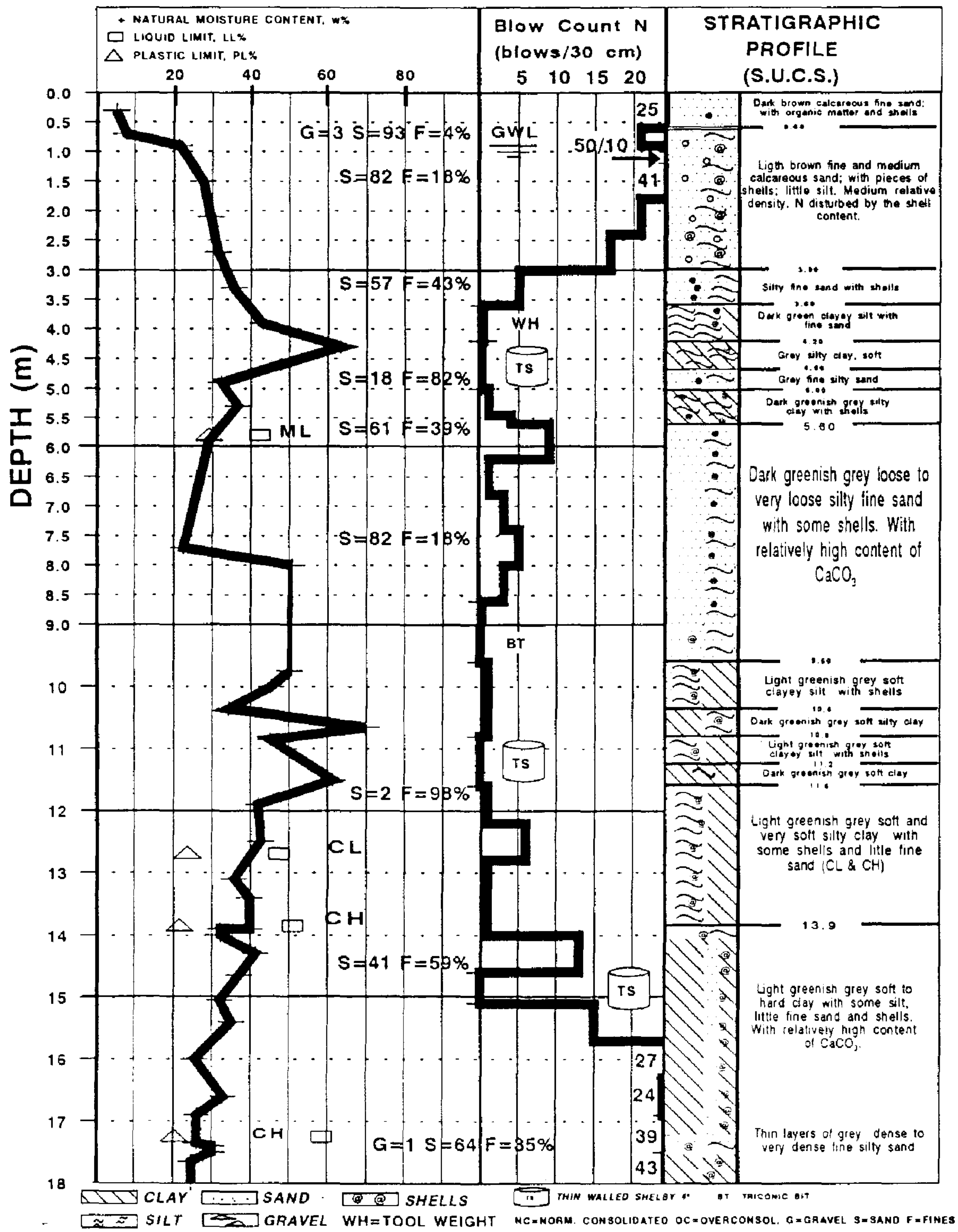

Fig. 1 Stratigraphy of the subsoil in Ciudad del Carmen

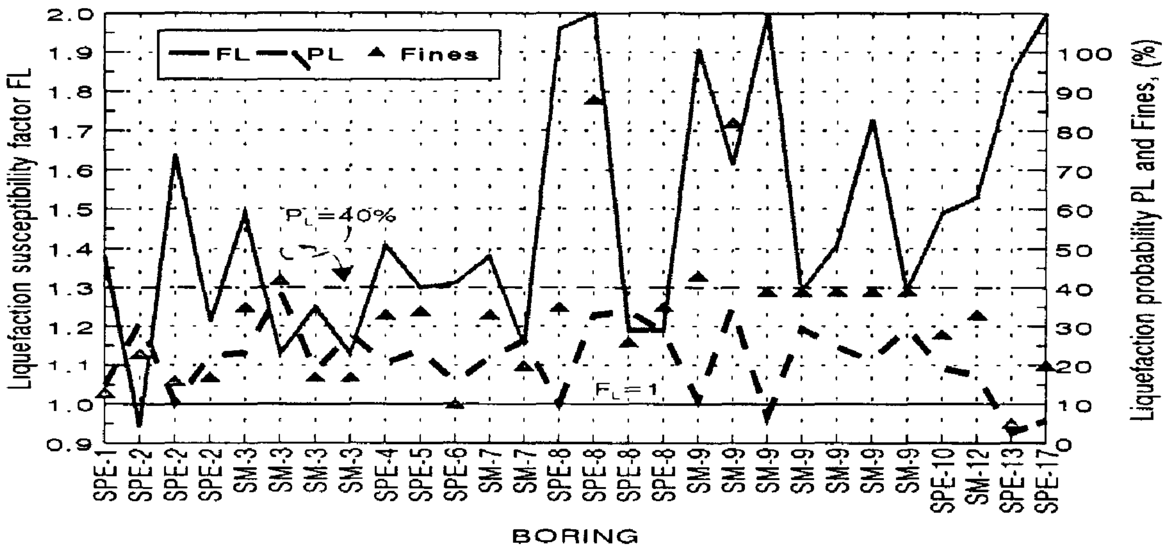

Fig. 2 Liquefaction susceptibility factor and probability computed for Ciudad del Carmen

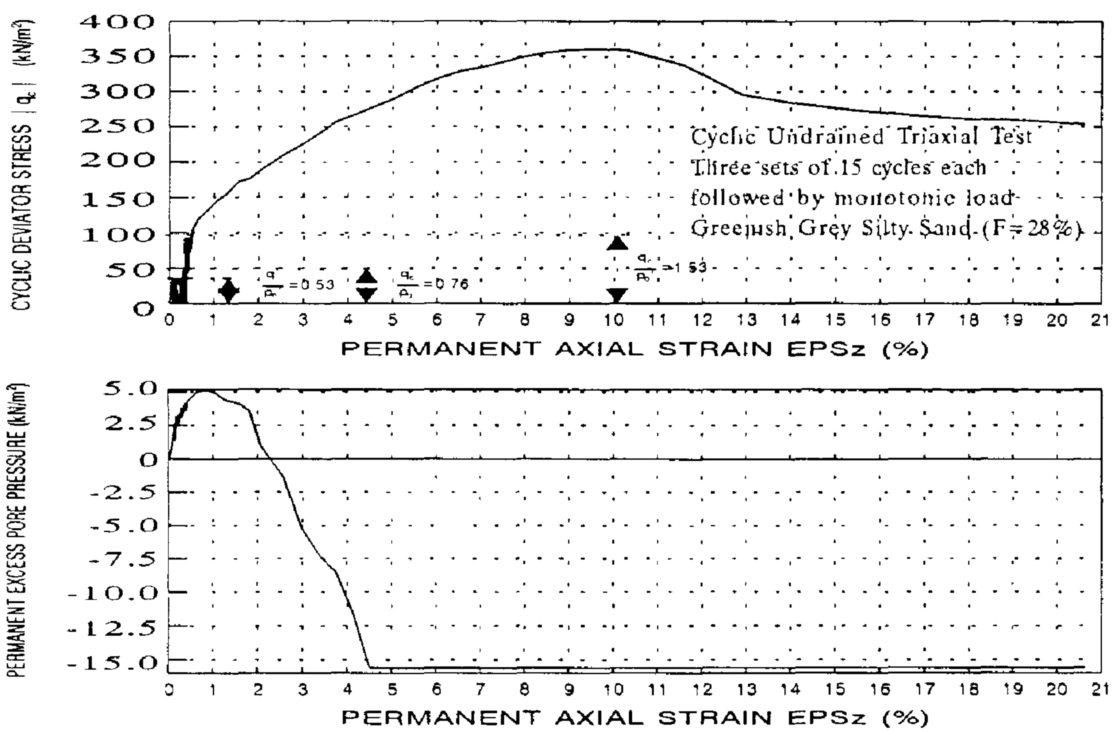

Fig. 3 Cyclic undrained triaxial tests results (F=28%)

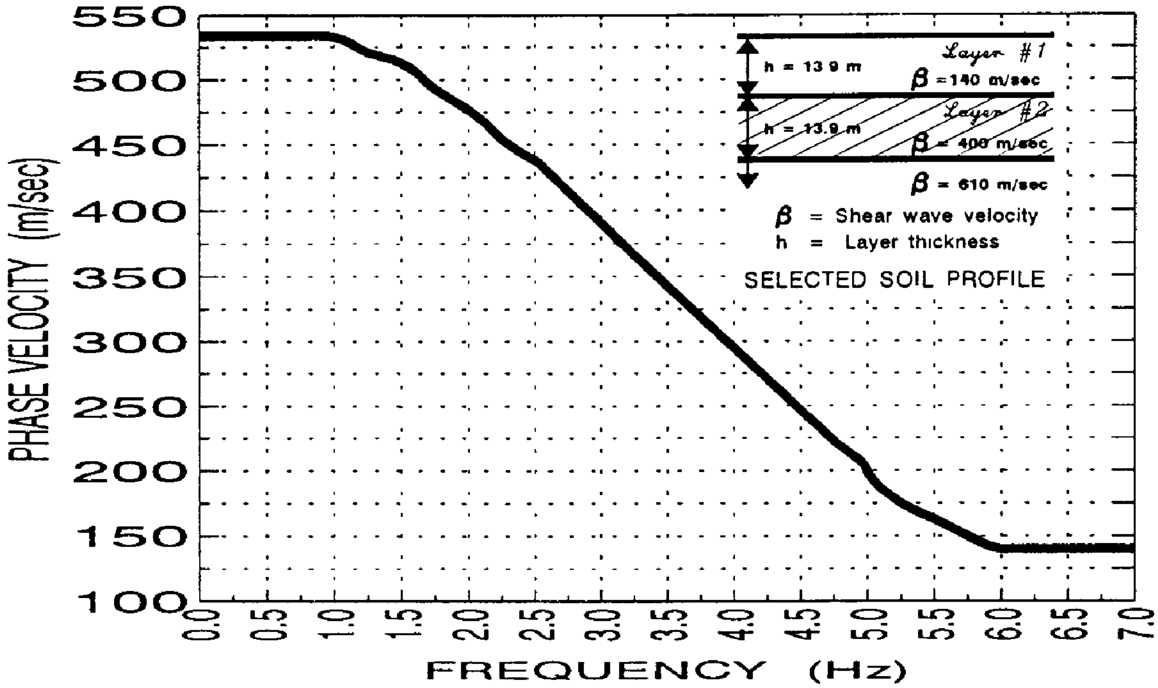

Fig. 4 Computed dispersion curve for Rayleigh waves in Ciudad del Carmen

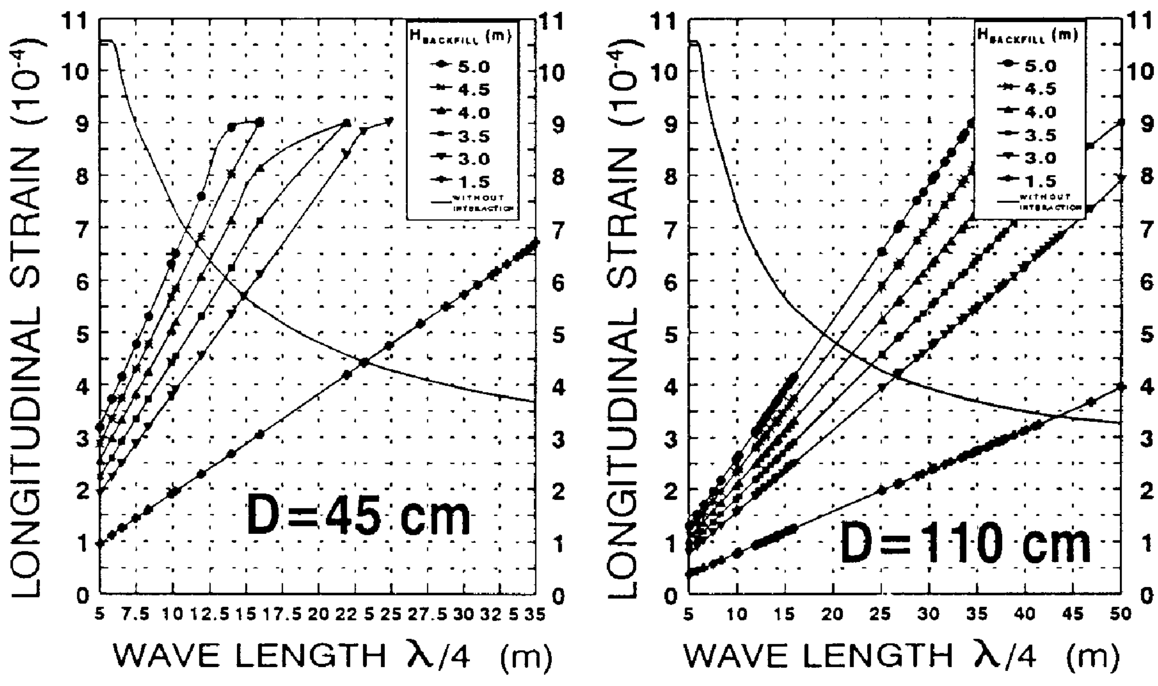

Fig. 5 Computed longitudinal strains with and without pipe-soil interaction in Ciudad del Carmen