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ABSTRACT

Seismicity in Mexico is largely influenced by subduction earthquakes that originate along much of its Pacific Coast. These events have recurrently damaged Mexico City but other less frequent earthquakes produced by other sources and mechanisms also contribute to seismic hazard there and have damaged other important cities and towns. In this paper we review, from the point of view of geotechnical engineering, the effects of three of these less frequent events: the Manzanillo earthquake of Octobrer 9, 1995, the Tehuacán Earthquake of June 6, 1999 and the Tecomán Earthquake of January 21, 2003.

INTRODUCTION

Seismicity in Mexico is determined by earthquakes having at least three distinct mechanisms: a) subduction earthquakes originating off the Mexican Pacific Coast; b) normal faulting earthquakes that occur inland, in the subducted Cocos plate; c) earthquakes that originate along faults that emanate from the Mexican Neovolcanic Axis. By far subduction earthquakes are the most frequent and are generally the most damaging but normal faulting events, with return periods of a few decades, have also produced extensive destruction in several inland cities, mainly in the eastern portion of the country.

In this paper we look at the effects of three earthquakes, two of which originated by the subduction of the Rivera Plate under the Noth American plate, close to the zone where the Cocos, the Rivera and the Pacific plates meet. These earthquakes damaged the state of Colima, mainly affecting the port of Manzanillo on October 9, 1995 and the state capital, also called Colima, on January 21, 2003. Damage distribution in these two events was very different. We address this question making reference to the characteristics of the source mechanisms and to local site effects.

The third earthquake belongs to the second category, it is a normal faulting event which took place on 15 June, 1999 and whose epicenter was located near the city of Tehuacán. The 15 June 1999 earthquake was the first large normal faulting event to be recorded by the local accelerographic network that has been operating since 1985 (Alcántara et al, 1999 and Singh et al, 1999). In this case the characteristics of the source and local site conditions were also key factors in the distribution of damages.

The general tectonic environment and the location of the epicenters of the three earthquakes discussed here are given in the map shown in Fig. 1.

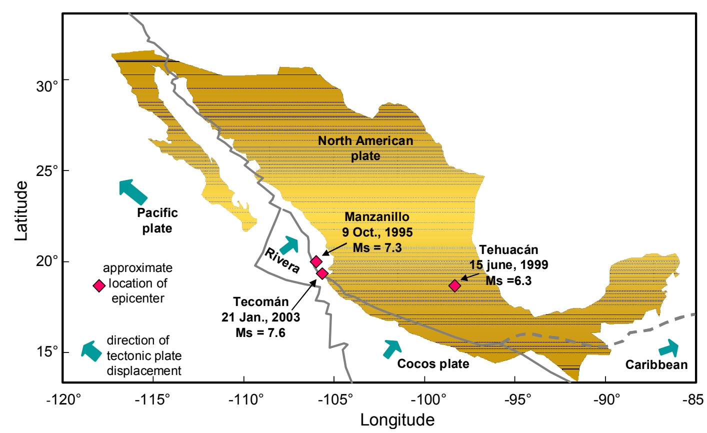

Fig. 1 General tectonic environment and approximate location of the epicenters.

MANZANILLO, 9 OCTOBER, 1995

Manzanillo, the second most important port in Mexico, located in the state of Colima, on the Pacific Coast, some 510 km north of Acapulco and about 200 km south of Puerto Vallarta was hit by a large earthquake on 9 October, 1995 (M_s=7.3; M_w=8.0), Fig. 1. Strong shaking was felt in Manzanillo, in the state of Colima, as well as in other important cities and resorts along the coastline of the Colima and Jalisco states. Seismic intensity was quite strong in some inland cities and small towns as well. The tremor was felt in Guadalajara and in Mexico City, 230 and 515 km from the epicenter, where virtually no damage occurred.

Historical records show that Manzanillo has been hit by strong tremors at least 8 times before the 1995 shock, starting in 1611.

The epicenter of the largest instrumentally recorded earthquake in Mexico (3 June, 1932, M_S=8.2) was located off Manzanillo (Singh et al, 1985). The earthquake was followed by a large aftershock on 18 June (M_s=7.8). It should be pointed out that in this earthquake rupture propagated towards the NW for a length of about 110 km, i.e. rupture followed a roughly divergent path from the coastline.

Extensive damages were observed along the coastline and in most inland towns of the Jalisco and Colima states. In what follows we focus on the damages in Manzanillo, mainly in the San Pedrito harbour facilities.

Seismological information

Several seismological studies were performed after the 1995 earthquake and are summarized by Domínguez-Rivas et al (1997); their main conclusions and findings are the following. The epicentral zone, 17 km deep, was located on the interface of the Rivera and North American plates, some 24 km south of Manzanillo, as seen in Fig. 2. Surface wave and seismic moment magnitudes were $M_S=7.3$ and $M_W=8.0$, respectively. The rupture area was roughly rectangular, 170x70 km^2 and the source, complex, was formed by four sub-events that promoted a unilateral rupture which propagated towards the NW; total duration was 60 s. These authors also state that the available seismological data suggest that the 1995 event was not a repetition of the 1932 earthquake. Consequently, the occurrence of another large earthquake along the coast of Colima and Jalisco in the next few decades can not be ruled out.

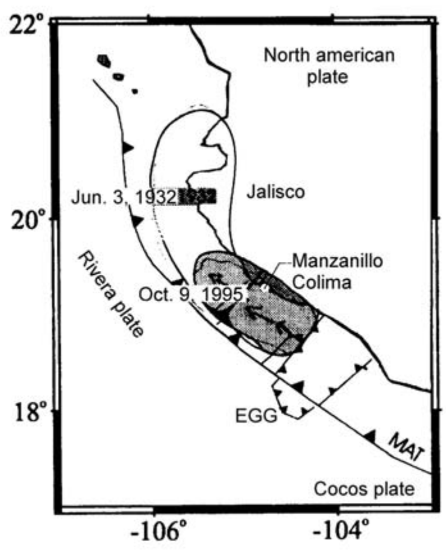

Fig.. 2 Rupture zones for the 1932 and 1995 Manzanillo earthquakes.

Accelerations produced by the earthquake were recorded in several important cities of the state of Jalisco including Puerto Vallarta, Ciudad Guzmán and Guadalajara, where maximum accelerations were 133, 50 and 24 gals. In Mexico City accelerations on hard soils or basalts was 2 gals and in the soft clay area, around 15 gals. There were no accelerographic records in Manzanillo itself or in the San Pedrito harbor. A network of several three-component accelerographs was installed in a thermoelectric power house, some 5 km from Manzanillo but only one of these was placed in the free field. It recorded a maximum horizontal acceleration of 380 gals. The duration of the intense portion of the accelerogram at this site was about 50 s; the ratio of vertical to horizontal acceleration maxima turned out to be 0.8. The apparatus was placed over dense sands of undetermined depth. Fourier and response spectra show that site effects influenced ground motion at this recording station. The graphs in Fig. 3 are the response spectra obtained there.

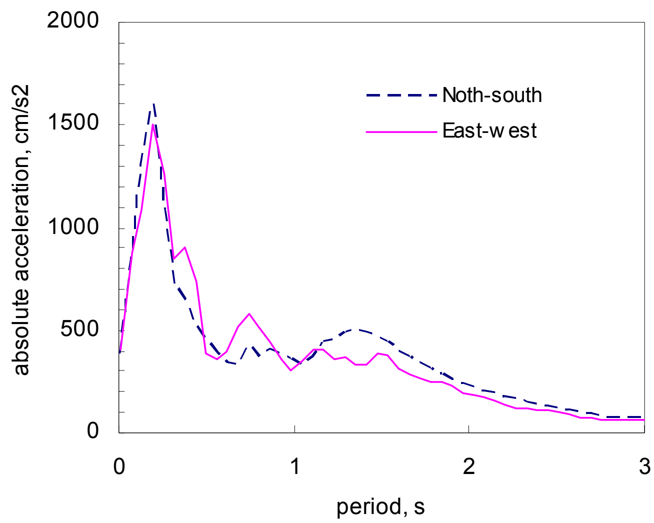

Fig.. 3 Response spectra obtained from free field acceleration records at the Manzanillo powerhouse, 9 October, 1995.

Damages in the city and port of Manzanillo

Damages in the city of Manzanillo and in the port facilities built there and in San Pedrito —the main harbor, some 3 km from the central part of the city—, were not as severe as could be expected given that the epicentral distance was about 24 km. This is partly explained by the propagation of the rupture along the subduction trench towards the NW, away from the port, together with the fact that the main energy release occurred some 100 km away. However, the earthquake produced the collapse of a nine storey building, where more than 30 people died, and of two low rise - 2 - structures. All of these were built on dense sand bars. The causes of collapse were attributed to structural design flaws and constructive defects, rather than to soil or foundation related failures. The good behavior of a thermoelectric power plant, some 3 km south of Manzanillo, confirmed the validity of current design practice for these installations. The tsunami that followed the earthquake also produced considerable damage, mainly along the coast north of Manzanillo where it produced waves of 3 to 10 m. In Manzanillo, where the height of the waves did not exceed 1m, it only contributed marginally to the observed damage.

Damage distribution in Manzanillo was influenced by the existence of granular fills covering large areas. Part of the city, the Colonia Burócratas, lies on reclaimed land that liquefied during the earthquake and produced differential settlements in a large number of houses and small rise buildings. Uncompacted granular fills formed with materials dredged from the bottom of the neighboring shallow bays and lagoons, vary from slightly more than 0.3 to about 2.5 m in depth and their standard penetration resistance, from 8 to 17 blows per foot. No structures collapsed or overturned on account of the settlements induced by liquefaction but several two and three story buildings had to be condemned. Dwellings are generally built with reinforced brick masonry and, occasionally, concrete frames. These structures are rather rigid and are generally founded on shallow masonry footings; some of them settled differentially more than 15 cm. Large total settlements were not observed on account of the relative thinness of the fills. Unfortunately, geotechnical information in the zone is scant and it is not possible to correlate observed damages to the thickness of the fills or their geotechnical characteristics.

The first dock in San Pedrito was built in 1970 and two more in 1981 and in 1988. In 1995 there was one container terminal that was commissioned in 1992. Two more such terminals as well a new dock for grains have been added in the last few years. Most harbor facilities have been built on reclaimed land, which liquefied extensively during the earthquake. Materials used for land reclamation were dredged from the neighboring lagoons but are much deeper than in urban areas, typically their depth reaches 10 to 15 m; liquefaction was observed in most of them. Settlements in yards produced the rupture of water supply and electricity lines; fissuring, cracking and dislocation of pavements were extensive; some buildings settled differentially and some others tilted, in many cases severely. Differential settlements in the yards of the Container Terminal exceeded 80 cm and lateral displacements in unconfined slopes were more than 2 m.

Structurally, the docks follow a common design for this kind of facilities: reinforced concrete slabs supported by vertical and battered piles. The docks were constructed by first driving the piles, with their tips located some 8 m below the seabed. After removing the seafloor mud, coarse gravel was then placed between the piles to form an embankment that was finished with rockfill faces. Three auxiliary embankments perpendicular to the dock were built to facilitate the construction of the main one. The seaward toe of this embankment was aligned with the outer line of piles, as seen in Fig. 4. Silty sands dredged from the lagoon were placed behind the embankment with the hydraulic fill method. These materials now constitute the subsoil that underlies the Container Terminal yards.

The piles that support the docks were subjected to strong dynamic shears, especially those having short unconfined lengths. In one of docks, nearly 50 % of the existing piles had some degree of damage; in the Container Terminal, 11 % of the piles were damaged. Movements in the rockfill embankments accounted for some of these damages, enhanced by lateral thrusts produced as the granular fills liquefied.

Structural repair work in San Pedrito finished by mid 1996 and works for improving the characteristics of the granular fills, in early 1998. Soil conditions in the Container Terminal are known with greater detail than in the city of Manzanillo and the other docks. After the October 1995 earthquake, the authors participated in a series of field and laboratory investigations aimed at assessing the liquefaction potential of the fills during future earthquakes and at putting forth methods for reducing this hazard. The main aspects and conclusions of these studies are discussed in the following paragraphs.

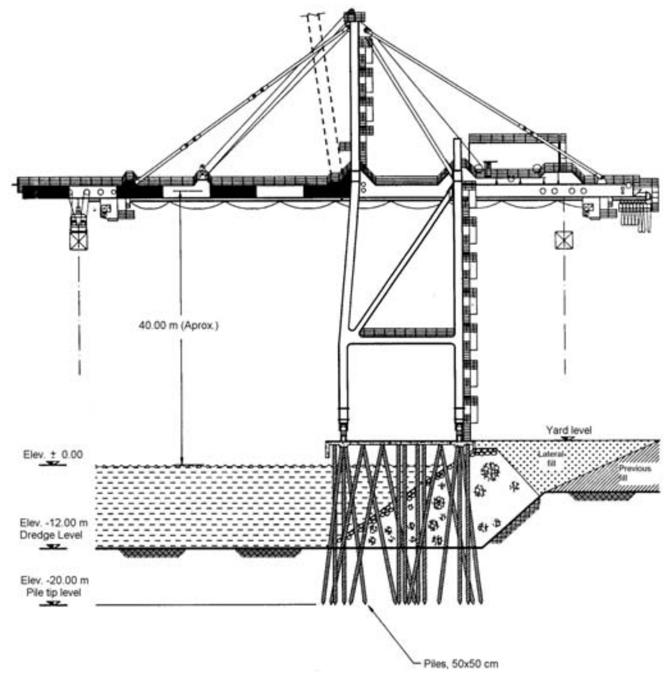

Fig. 4 Typical crane in the San Pedrito harbor facilities.

<u>Geotechnical conditions at the container terminal</u>

Rock outcrops in and around Manzanillo extend from the Cretaceous to the Pleistocene; the older formations are granitic with intrusions of igneous acidic rocks which form the hills and mountains that surround Manzanillo, producing estuaries and inlets along the coast. Alluvia —gravelly sands as well as heterogeneous accumulations of sandy silts— were deposited near the coast during the Pleistocene and the Holocene; they some times mix with *pied de mont* deposits at the foot of the hills that surround the city. Pleistocenic organic clays can be found in lagoons along the coastline, overlain by younger sands that contain quartz, volcanic basalt and variable amounts of mica and fragmented seashells. There are four such lagoons in Manzanillo; dock and harbor facilities in San Pedrito were built along the eastern margin of one of them.

The liquefaction potential of the sandy fills was estimated as part of the geotechnical studies performed before the construction of the Container Terminal, in 1990 and 1991. The analysis was carried out using the Seed and Idriss (1971) simplified approach with the results of standard penetration tests which showed that the fills would liquefy if maximum ground surface accelerations exceeded 0.15 g, the recommended value for designing the facilities at the Container Terminal, according to a seismological study performed in early 1985 (Jica, 1985). The maximum ground acceleration for design purposes was raised to 0.2 g as a result of this study, but the fills were left uncompacted.

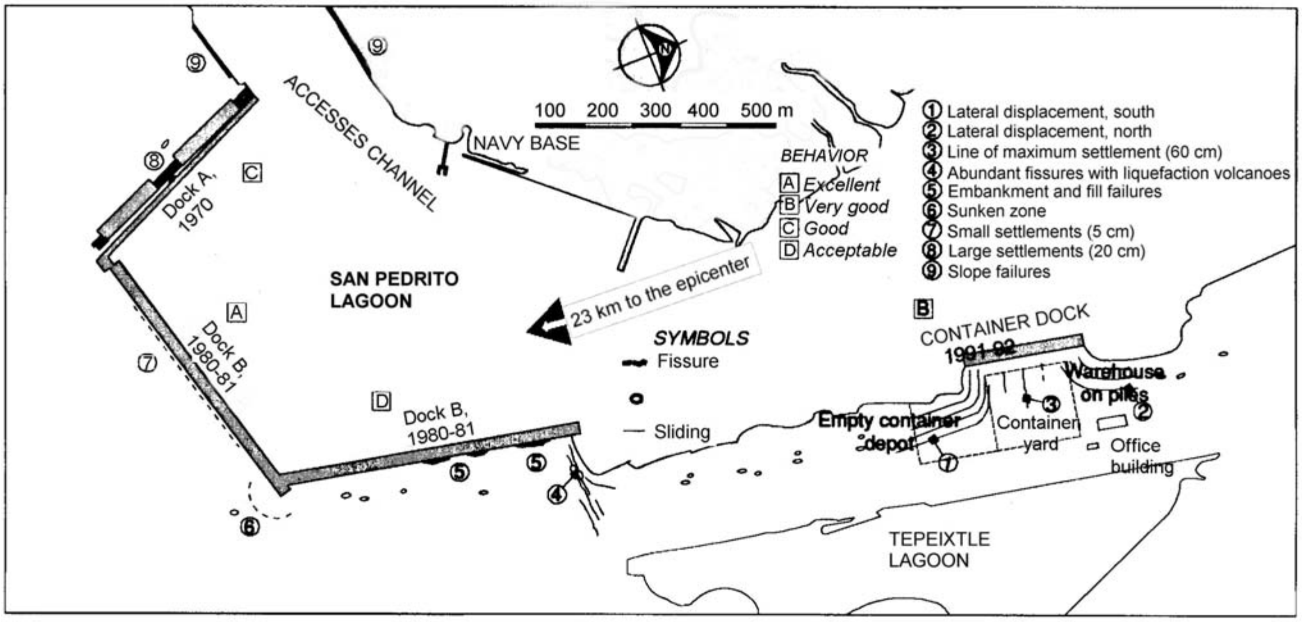

Fig. 5 General distribution of sites that suffered liquefaction in the San Pedrito harbor on 9 October, 1995.

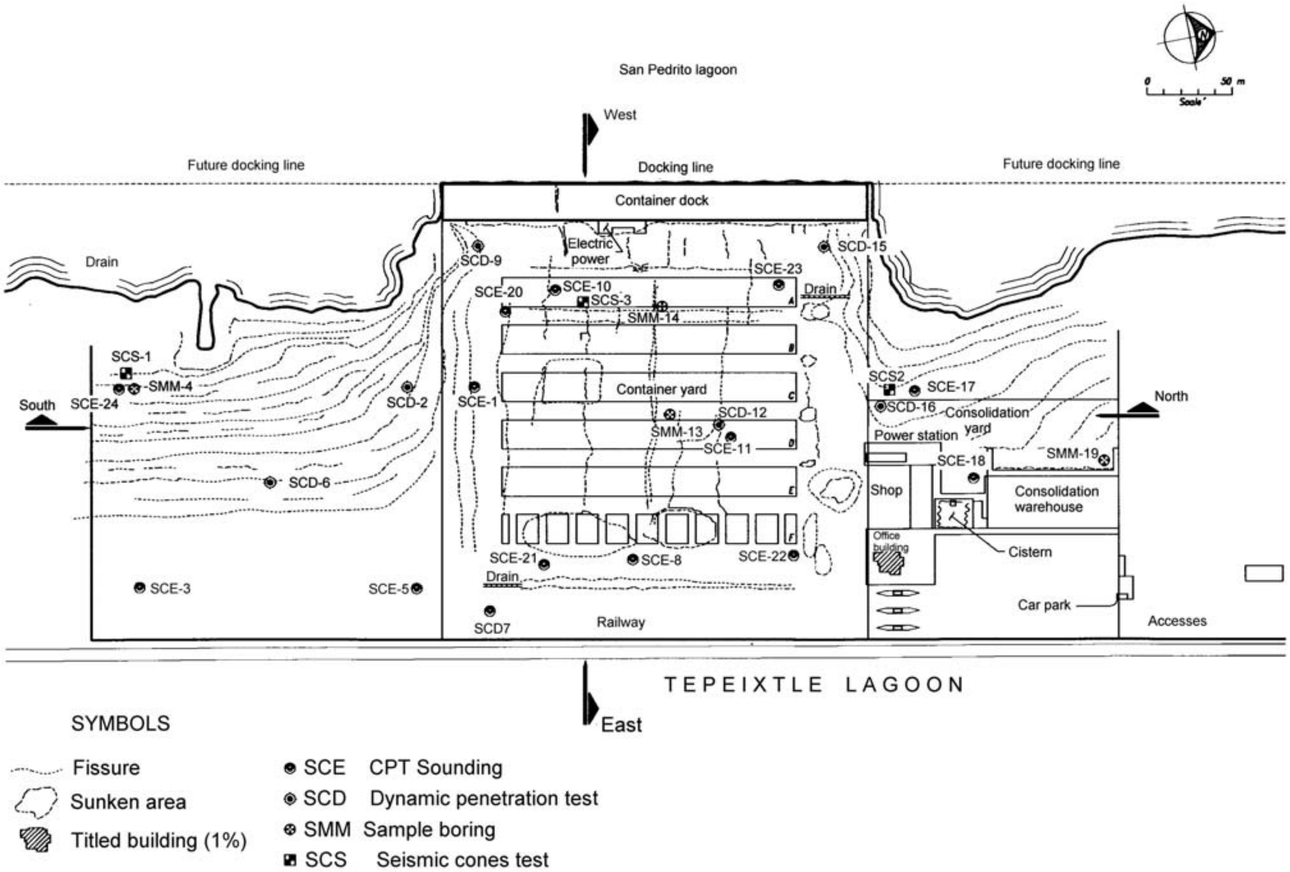

Fig. *6 Sites where liquefaction occurred during the earthquake of 9 October, 1995 in the San Pedrito container Terminal.*

The map shown in Fig. 5 indicates the distribution of the sites where liquefaction occurred in the San Pedrito Harbour during the earthquake of October 9, 1995, and Fig. 6 gives a plan of fissures and cracks observed in the yards of the Container Terminal. The pattern shown there suggests that one of the major causes for damage was the lateral displacement of unconfined slopes on both ends of the dock. In general, settlements were greater near the dock where the sandy fills were probably looser; the maximum differential settlement, between the south western and the north eastern corners, exceeded 80 cm. Settlements in the east side of the yards were smaller as the fills there are older, thinner and have been compacted by heavy traffic and by the placement and removal of containers.

Field studies after the earthquake

Field and laboratory tests were performed after the earthquake to characterize the sands that underlie the yards at the Container Terminal. Field exploration included 4 SPT tests, 7 non standard dynamic penetration tests, 8 CPT and 3 seismic cone tests. The plan given in Fig. 5 also shows the location of the points for measuring soil properties.

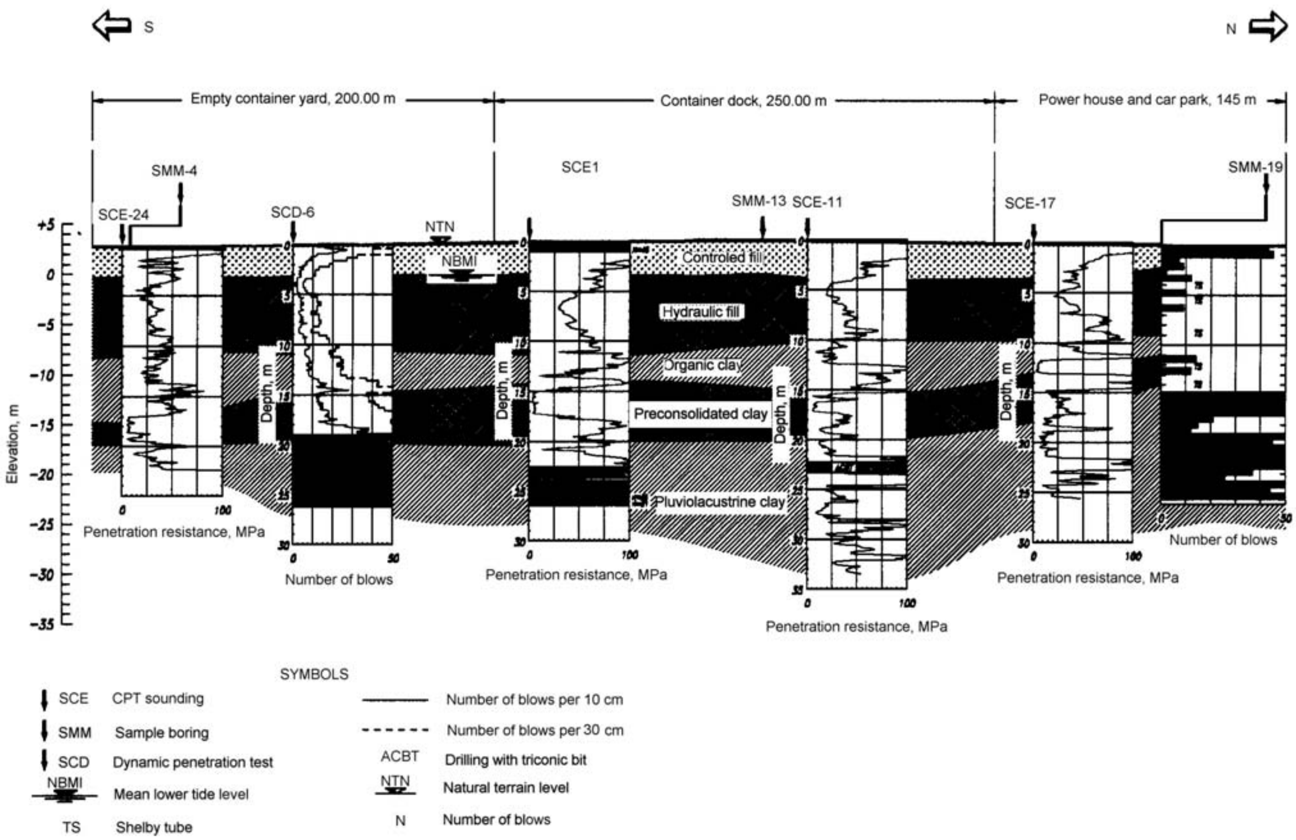

Fig. 7 Geotechnical profile along the North-South direction in the San Pedrito container terminal.

Results of typical CPT and SPT soundings along an east-west and a north-south axes can be seen in the profiles shown in Figs. 7 and 8; the former is perpendicular to the dock and the latter, parallel to it. The results of the field tests suggested that the sandy soils located bellow the upper fills and above the natural organic clays were quite loose, especially the fills in the southern part of the yards. The results obtained from these soundings led to the following stratigraphical model:

Surficial crust. These are controlled fills formed with materials taken from nearby banks and have thicknesses that vary between 2.5 and 3.5 m. Its upper layer was compacted after placement and has been subjected to further densification by heavy traffic and movement of containers within the yards. The materials there are silty sands (SP or SP-SM) in which the finer particles seldom exceed 7 %.

Hydraulic fills. These are the sandy materials that were originally deposited in the lagoon and were then dredged to form the fills; they lie below the surficial crust and reach the elevation -12 m in the container yards, -6.5 in the north side of the terminal and -8.0 m in its south end.

Lake bed muds. These are compressible, low strength, organic clays, CL-OH, that are some times mixed with fine sands. The thicknesses of these deposits vary between 1 and 6 m. These

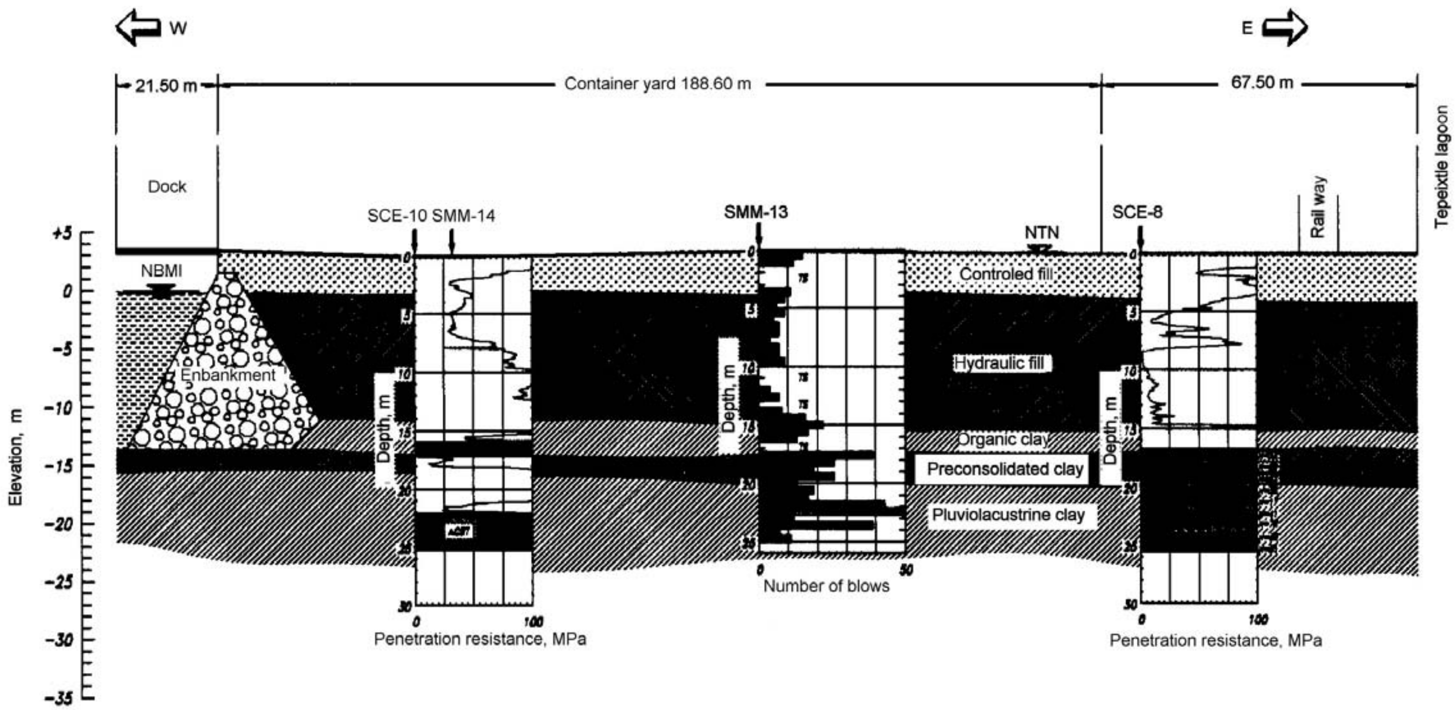

Fig. 8 Geotechnical profile along the West-east direction in the San Pedrito container terminal

materials were removed before constructing the embankment but were left in place below the area of the yards.

Preconsolidated clays. They form deposits of about 2 m thick and contain small amounts of sand. They were subjected to solar drying during a regression of the sea level, hence their preconsolidated state. Penetration resistance in these clays, CPT soundings, is about 0.7 MPa which can be taken as an index of their relatively high compressibility and low shear strength.

Fluviolacustrine soils. These soils are found at elevations that vary between -11 to -16 m and are formed by a sequence of lenses of compact sands, gravel and shell fragments within a matrix of clayey materials (10 to 15 %). Their CPT strength is high, more than 10 MPa and SPT resistance is usually higher than 50 blows.

Seismic cone tests. A seismic cone developed at the Instituto de Ingeniería, UNAM (Ovando *et al* 1995), was used to measure shear wave velocities at three locations within the yards in the Container Terminal. Results of these tests are summarized in Fig. 9 where values of shear wave velocity, V_s, have been plotted against depth. V_s in the loose granular fills averages about 150 m/s and in the soft lake bed muds, less than 100 m/s.

Liquefaction potential from the results of field tests. The results of the static and seismic CPT soundings were used to evaluate the liquefaction potential of the granular fills in the Container Terminal following some of the most commonly used criteria. The intensity of seismic motions in the area was specified from the results of the studies for assessing seismic hazard in Manzanillo which were described previously.

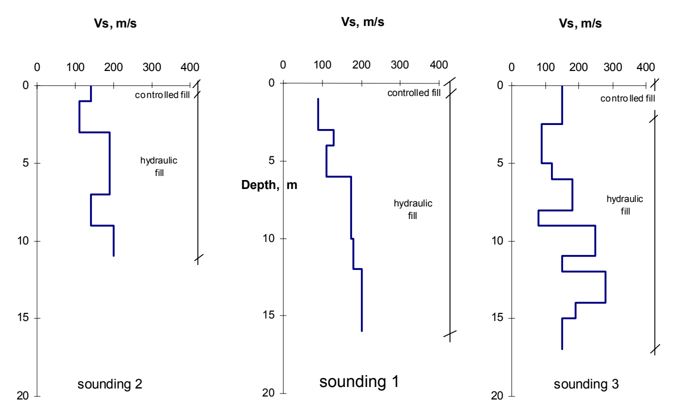

Fig. 9 Results of three seismic cone tests at the container terminal yards. For location of soundings see Fig. 6.

Liquefaction potential from static cone tests. Analyses were first made using the Seed and Idriss (1971) criterion. Results indicated that the sands at depths ranging between 4 and 10 m would liquefy again if the maximum ground acceleration exceed 0.15 g, which is the same result obtained when the liquefaction potential of these sands was assessed before the earthquake. Robertson et al (1992) put forth an empirical criterion in which the measured CPT strength, q_c, is compared with the q_c values of sites that have liquefied in the past. According to it, all the hydraulic fills that underlie the surficial crust were highly susceptible to liquefaction.

Shear wave velocity as an index of liquefaction potential. Shear wave velocities in granular media depend on the same factors that determine whether a sand will or will not liquefy and, consequently, criteria based on the values of shear wave velocity have also been put forth by some researchers (e. g., Holzer *et al*; 1988 Robertson *et al* 1992). The latter authors use an approach similar to Seed and Idriss's in that they compare the normalized cyclic stress with a normalised value of the shear wave velocity to establish whether the sand will liquefy. According to thesepredictions, the fills in the Container Terminal can hardly undergo an earthquake that induces maximum accelerations larger than 0.2 g without liquefying. In the Holzer *et al* method, established from the observation of a limited number of liquefaction cases in California, the values of V_s are compared with a measure of the seismic intensity at the site, the maximum horizontal acceleration. The results of the analyses performed using this method indicated that very loose sands, i.e. $V_s \approx 100$ m/s, would liquefy when subjected to shocks that induce ground accelerations of about 0.2 g; for those in which $V_s \approx 175$ m/s, the threshold maximum ground acceleration for the possible occurrence of liquefaction is about 0.42 g, which is 5 % larger than the recommended ground acceleration for design purposes. Such high accelerations can occur at least once in the life time of the Container Terminal, as mentioned previously.

Summarizing, the simplified methods for estimating the liquefaction potential of the fills in the Container Terminal consistently indicated that liquefaction could occur again in future earthquakes. Very loose fills lying at depths that vary between 4 and 6 m would liquefy if ground accelerations exceeded 0.2 g; denser materials would also liquefy should ground accelerations approach the values suggested for design in the seismic risk study, 0.4 g. The vulnerability of the Container Terminal to future very costly and possibly catastrophic disruptions, prompted the commissioning of a more detailed investigation into the stress-strain-pore pressure characteristics of the granular fills.

An analysis of the results of field tests led to a correlation between shear wave velocities with cone penetration resistance, as seen in Fig. 10 from which V_s can be estimated assuming that a hyperbolic function can be used to describe the pre-peak stress-strain relationship of the granular fills:

$$V_s \approx \sqrt{\frac{q_c}{N_{kh}\gamma_r \rho_s}} \tag{1}$$

where N_{kh} is a correlation coefficient; γ_r is a reference strain, determined from laboratory tests, as described later (see Fig. 16) and ρ_s is the sand's mass density. Derivation of this correlation which assumes that the stress-strain behaviour of the granular fills is hyperbolic has been presented elsewhere (Ovando and Romo, 1992).

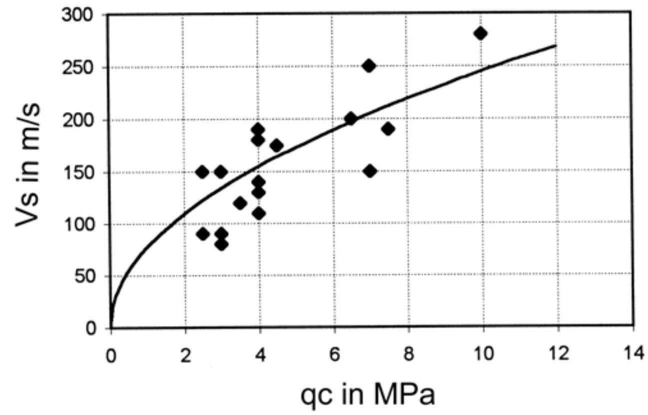

Fig. 10 Correlation between CPT strength (q_c) and shear wave velocity for the San Pedrito Harbour sandy fills.

Equation 1 reproduces the actual trend followed by the experimental data, from which:

$$V_s \approx 77.5\sqrt{q_c} \tag{2}$$

with V_s in m/s and q_c in MPa

Laboratory investigations

Samples from the fills in the Container Terminal were tested at the Instituto de Ingeniería, UNAM, using a computer-controlled triaxial cell and a resonant column. Static undrained triaxial tests performed on reconstituted samples formed by the wet tamping method were used to study their behaviour at large strains whereas resonant column testing with a torsional Drnevich type apparatus was utilized to define their stiffness-damping-strain characteristics at shear strains smaller than 0.1 %.

Materials tested. They are fine to very fine sands in which the fine particles vary between 4 and 12 % and occasionally contain small proportions of shell fragments, some of which can be relatively large (2 to 4 mm in diameter). The greater percentage of fragments is formed by quartz but basalt particles, transported from two volcanoes about 150 km west of Manzanillo, are also abundant; laminar micaceous fragments, 10 to 15 % in weight, can be seen with the naked eye. Samples retrieved from different depths and locations during the geotechnical exploration program were sieved and their average grain size distribution curve was used to select the materials for the laboratory test specimens, Fig. 8. These materials contained 8% of fine particles (liquid limit = 54.4 %, plastic limit = 32.6 %); those larger than 2 mm, mainly shell fragments, were removed. Typical grain-size distribution curves are given in Fig. 11.

Conditions for Triaxial tests. Samples (diameter = 36 mm; height = 81 mm) having a range of initial densities were consolidated isotropically or anisotropically against a backpressure of 250 kPa before shearing them in undrained conditions, in extension or compression; mean effective stresses after consolidation varied between 50 and 300 kPa. The wet tamping method was used to form the samples and saturation was achieved by circulating CO_2 and then water through them; all of the test specimens were provided with lubricated end platens. As seen in Fig. 11 the amount of fine particles passing sieve No 200 varied over a relatively wide range.

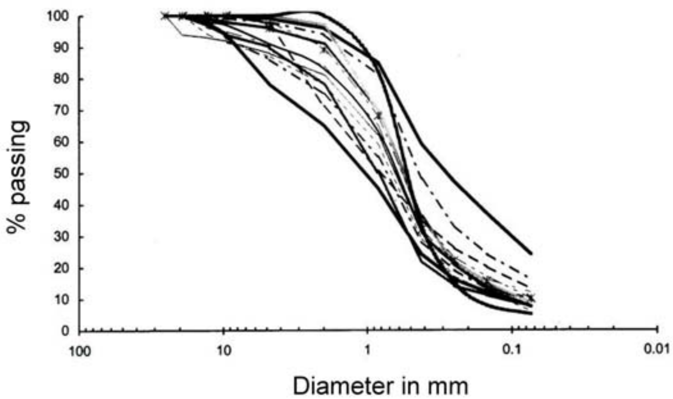

Fig. 11 Typical grain-size distribution curves obtained from the San Pedrito Harbour.

<u>Experimental results and their interpretation</u>. The intergranular voids ratio, e_g, or the intergranular specific volume, $v_g=1+e_g$ were used as parameters for analyzing the experimental results; Mitchell's (1976) expression for e_g is:

$$e_g = e + \frac{\frac{c}{G_{sc}}}{\frac{1-c}{G_{sg}} + \frac{c}{G_{sc}}} \tag{3}$$

where c is the percentage (in weight) of particles passing sieve No 200; G_{sg} is the specific gravity of the coarse materials retained in sieve No 200 (*=2.73*); G_{sc} is the specific gravity of the fine particles *(=2.84*) and e is the void ratio. Since c varies in the actual sand deposits, this expression can be used to generalize the results for different percentages of fine particles. Consequently, the results of tests on samples that did and did not have particles passing sieve No 200 can be interpreted together, assuming that the finer particles will not contribute to the material's shear strength, as shown previously in other studies (Ovando-Shelley and Pérez, 1997).

The plots shown in Fig. 12 —effective stress paths, stress-strain and pore pressure-strain curves— exemplify the type of results obtained from triaxial tests performed on a loose sample (D_r*=43 %* , v_g*=1.78*) and a dense one (D_r*=78 %*, v_g*=1.61*) both consolidated isotropically. The loose sample developed flow deformations after accumulating 1.2 % axial strain. Despite its rather low density, the dense sample was still able to generate some positive pore pressure and dilative tendencies were only apparent after it developed more than 1.0 % axial strain.

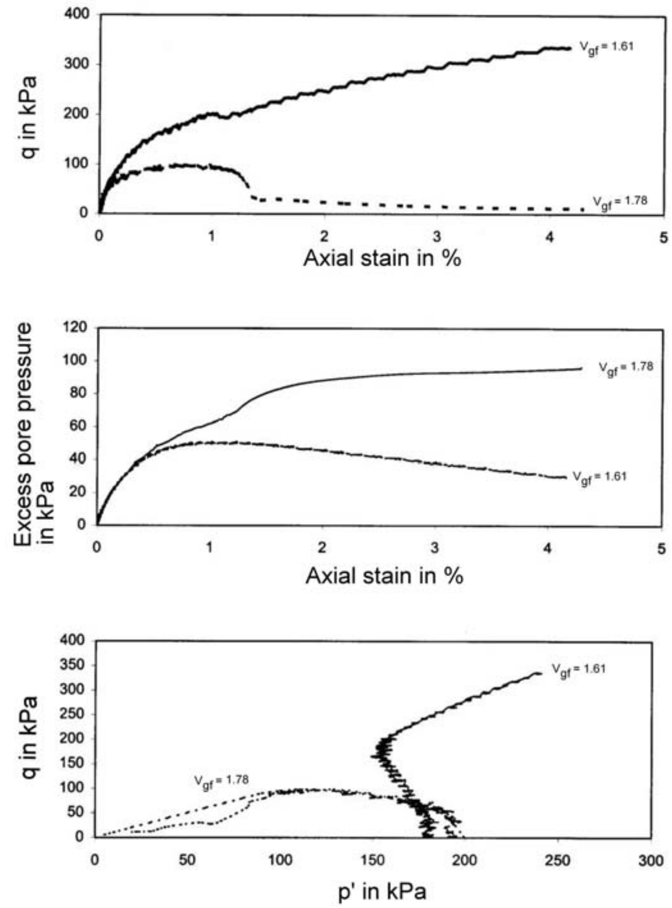

Fig. 12 Examples of test results obtained from undrained triaxial tests on reconstituted San Pedrito Sand samples.

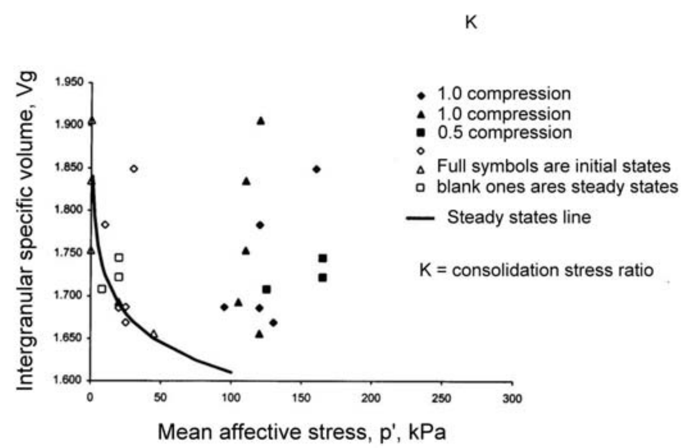

Fig.. 13 Initial and steady state lines obtained from the results of undrained triaxial tests on San Pedrito Sand samples.

Steady states were obtained from the results of all the triaxial tests and are plotted in the state diagram of Fig. 13. Been and Jefferies' (1985) state parameter, ψ, can be obtained from the steady state line in $ln(p') - v_g$ space, given a sample's density and consolidation stress. The experimental data follow the following well known expressions:

$$p' = exp\left(-\frac{v_{ss} - \Gamma}{\lambda_{ss}}\right) \tag{4}$$

$$v_{ss} = \Gamma - \lambda_{ss}\, ln(p') \tag{5}$$

where v_{ss} is the specific volume along the steady state line at an arbitrary mean effective stress, p'; Γ is the intercept of the steady state line with a reference pressure, in this case equal to 1 kPa; λ_{ss} is the slope of the steady state line. From our data: $\Gamma=1.61$ and $\lambda=0.05$.

Peak or maximum shear stress, q_{sc}, i. e. the shear stress at the onset of the structural collapse that brings about liquefaction, is a useful practical parameter for identifying the conditions —stress and density states— in which the phenomenon will appear. This stress state is normalizable by the mean effective consolidation stress, p'_0, and, if a modifying parameter β is also introduced, the consolidation stress ratio can also be included as a normalizing factor (Ovando and Pérez, 1997); $\beta = \sigma'_3/\sigma'_1$ for compression tests, $\beta=\sigma'_1/\sigma'_3$ for extension tests. The graph in Fig. 14 shows that the normalized maximum (peak) shear stresses obtained from the results of compression and extension triaxial tests, $\beta q_{sc}/p'_0$ and $\beta q_{ss}/p'_0$ respectively, plotted as functions of the state parameter, are nearly constant for ψ values larger than about 0.1which correspond to specific intergranular volumes larger than approximately 1.7. A very important practical conclusion follows from this experimental fact in that densification of the Manzanillo sands is only effective for increasing peak strengths when relative densities exceed 85%.

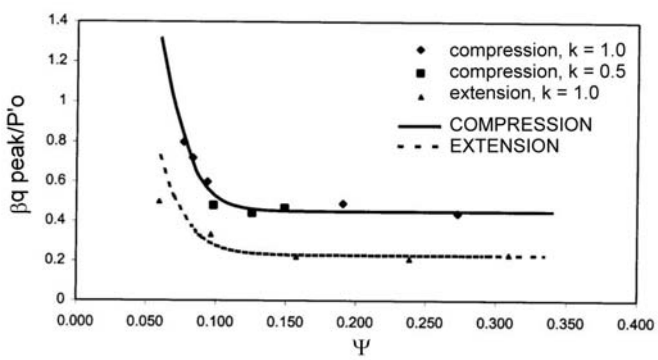

Fig. 14 Normalized peak deviator stress as a function of the state parameter.

For very loose samples ($v_g>1.7$, $\psi>0.1$) the friction angle at failure, mobilized when the material develops its residual strength turned out to be ϕ'=26° and the friction angle mobilized at the onset of liquefaction, which corresponds to the maximum shear stress, was ϕ'_{sc} = 14°. The values of ϕ' and ϕ'_{sc} increase sharply as the value of v_g diminishes below 1.7 and the value of ψ below 0.1.

Resonant column tests. Samples having a range of initial densities tested in the resonant column were subjected to isotropic consolidation stresses of various magnitudes. The results obtained from these experiments are shown in Fig. 15 which gives shear moduli as a function of shear strain and the energy dissipation characteristics of the materials tested, given in terms of equivalent viscous damping coefficients, also as a function of shear strain.

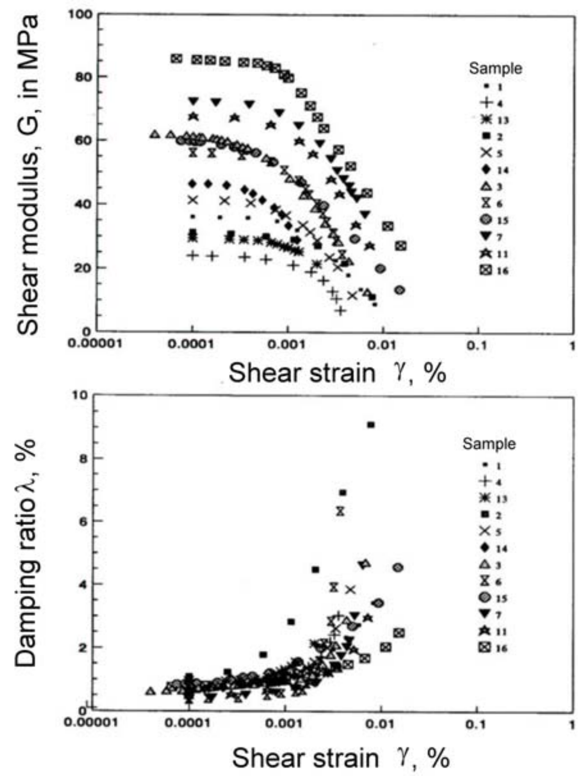

Fig. 15 Results of resonant column tests on San Pedrito sand.

Hyperbolic functions have been used extensively to model the stiffness-strain behavior of soils, including sands (see for example, Ishihara, 1982). The following function has been used successfully to model the undrained dynamic behaviour of a wide variety of soils (Romo, 1995; Romo and Ovando, 1996; Flores and Romo, 1997):

$$G = G_{max}\left[1 - H(\gamma)\right] \tag{6}$$

where G = shear modulus; G_{max}= maximum value of G (at small strains); the function $H(\gamma)$ is given by:

$$H(\gamma) = \left[\frac{(\gamma/\gamma_r)^{A'}}{1 + (\gamma/\gamma_r)^{A'}}\right]^{B'} \tag{7}$$

where γ_r is a reference strain; A' and B' are fitting parameters determined experimentally. Damping coefficients, λ, may also be expressed in terms of $H(\gamma)$, as suggested initially by Romo (1995):

$$\lambda = (\lambda_{max} - \lambda_{min})H(\gamma) + \lambda_{min} \tag{8}$$

where λ_{max} and λ_{min} are the maximum and minimum values of the damping coefficient, respectively. The small strain value of the damping coefficient, taken from Fig. 14, is $\lambda_{min} \approx 0.5$ %; over the range of strains imposed upon the samples in these tests, a fair approximation to the damping coefficient at large strains is $\lambda = 5$ %, as can be judged from that same figure.

Values of the parameters and constants involved were determined from the experimental results and were related to the state parameter ψ. A' and B' were found to be constant and equal to 1.55 and 1.0, respectively; γ_r can be determined from Fig. 16

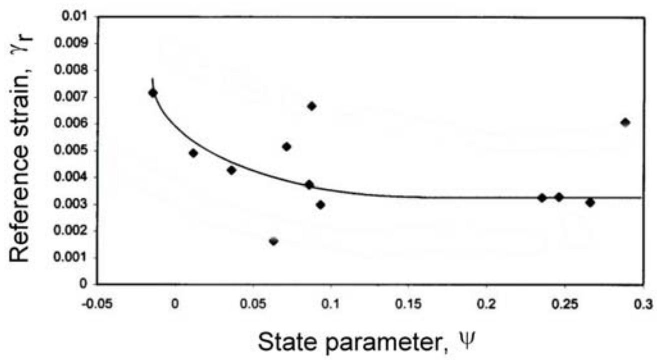

Fig.. 16 Relationship between the reference strain, γ_r, and the state parameter.

On the other hand, G_{max} can be approximated by a linear function of ψ, as seen in Fig. 17:

$$\frac{G_{max}}{(p'_0)^{3/4}(p_a)^{1/4}} = 472 - 250\psi \tag{9}$$

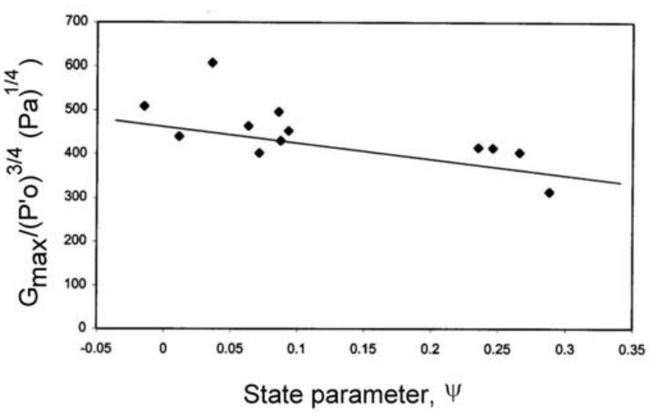

Fig. 17 Normalized initial stiffness as a function of the state parameter.

The data in Fig. 16 as well as expressions 6 to 9 allow for the derivation of a small strain, total stress model of the sandy fills that was used to evaluate, as a first approximation, the seismic response of the fills using a one dimensional wave propagation computer code (Santoyo and Segovia, 1997). For the very loose specimens the experimental parameters are all constants and the behavior becomes almost insensitive to changes of ψ, i.e. to changes in density, an observation that coincides qualitatively with the triaxial test results.

Relationships between field and laboratory tests

Relationships between data obtained from laboratory and field tests are always difficult to find or to obtain. In the case of sandy materials like the fills in the San Pedrito Harbour, assumptions must be made in order to find them. Alternatively, use can be made of the experience and suggestions of other researchers. For example, on the assumption that the field and laboratory stiffness-strain curves have similar shapes, the strain-dependent shear modulus can be approximated by using the elastic relationship:

$$G_{max} = \rho V_s^2 \tag{10}$$

substituting equation 1and 7 into 10 provides the following expression for the strain dependent shear modulus

$$G \approx \frac{q_c}{N_{kh}\gamma_r}\left[1 - \left[\frac{(\gamma/\gamma_r)^{A'}}{1+(\gamma/\gamma_r)^{A'}}\right]^{B'}\right] \tag{10}$$

with the values of A', B' and γ_r given before.

<u>CPT strength and state parameter</u>. In deriving an expression linking q_c with ψ, it is convenient to recall that CPT strength and in situ density were related by Jamiolkowski et al (1988), following a similar proposal suggested previously by Skempton (1986), based on the results of standard penetration tests. Jamiolkowsky's correlation is:

$$D_r = A - B\,log\left(\frac{q_c}{\sigma'_v}\right) \tag{11}$$

where D_r is the relative density; A and B are experimentally determined parameters that, according to Tatsuoka et al (1990), are equal to 85 and 76, respectively. With this, the following relationship between q_c and the state parameter was obtained (Sánchez, 1998):

$$q_c = \sqrt{\sigma'_v}\,exp\left(\frac{\psi - b_2}{2.3}\right) \tag{13}$$

where

$$b_2 = \left(1 + e_{g\,max}\right) + \frac{A}{100}\left(e_{g\,max} - e_{g\,min}\right) - \Gamma + \lambda_{ss}\,ln\left[\sigma'_v\,\frac{(1+2K_0)}{3}\right] \tag{14}$$

and

$$m_2 = -\frac{B}{100}\left(e_{g\,max} - e_{g\,min}\right) \tag{15}$$

e_{max} and e_{min} are the maximum and minimum voids ratios; K_0 is the effective stress ratio after consolidation; the other parameters have all been defined. Alternative expressions correlating ψ to q_c have been presented previously (e. g., Been *et al*, 1986, 1987).

Note that the factors involved in equations 10 to 14 are experimental constants, like *A* and *B*, or factors that depend on the physical characteristics of the sand mass, e_{gmax} and e_{gmin}, which are fixed, given a standard method for determining them. The values of σ'_v and K_0 reflect the influence of the effective stress; the former can generally be estimated with sufficient accuracy from simple analyses and the latter may be estimated from formulae that lead to acceptable estimates of the stress state. The CPT penetration resistance, q_c, is also related to the effective stress state in the sand, its density and its shear strength. On the other hand, Γ and λ_{ss} were obtained from the results of triaxial tests and their incorporation into these equations is justified because they both reflect conditions at the steady state where the influence of initial stress state and structure have vanished.

Initial stiffness and state parameter. The results of the resonant column tests evinced that there exists a relationship between the value of the normalized value of G_{max} and the state parameter, ψ, which is given by equation 6. It was then necessary to investigate whether an analogous expression could be established, using the results of the field tests. To this end, the elastic relationship (equation 1) was used to obtain the value of initial stiffnesses from the shear wave velocities measured with the seismic cone, normalizing it with respect to the effective vertical stress, rather than with the mean effective stress. The values of ψ were calculated using expression 11 with the penetration resistances obtained from the CPT soundings. The results given in Fig. 17 indicate that G_{max}, derived from the results of seismic cone tests, can be correlated to the state parameter.

Remedial measures

The results of the field and laboratory tests pointed out that liquefaction would very likely occur again in the fills, even for smaller earthquake intensities than those adopted for deriving the seismic design spectra in Manzanillo. Furthermore, the results of the laboratory investigation also indicated that the fills in the Container Terminal only become dilative when their relative densities are larger than 85%. In planning the densification of the fills in the Container Terminal these findings were taken into account.

Soil improvement scheme. Vibrocompaction was recommended for densifying the sandy fills in the Container Terminal. The improvement scheme required that the zones in the vicinity of the dock be compacted more intensely minding the fact that, according to the results of the field tests, the loosest sands are located down to about 11 m, on average.

Referring back to the stratigraphical conditions in the Container Terminal given in Fig. 6, it is evident that a sliding mechanism can develop at the contact of the fills and the seabed mud. This mechanism will operate during strong earthquakes even if the upper sands are densified and it was estimated, on the basis of simplified analyses, that lateral displacements of at least 50 cm along an axis perpendicular to the dock can occur for the design earthquake; lateral displacements can be at least twice this magnitude in the unconfined zones of the yards. In order to cope with this eventuality, a hard buffer zone would be formed between the embankment and the yards, to confine the sands near the dock by jet grouting a strip 200 m long, 20 m wide, and 15 m deep, parallel to the axis of the dock.

Compaction Control criteria. Since most of the field exploratory soundings were performed using the Dutch penetrometer, CPT tests were prescribed for controlling the densification works. It was then necessary to put forth a means for establishing the minimum penetration resistance that would guarantee an adequate behavior of the improved fills in the event of strong earthquake shaking.

In order to incorporate into the criterion the relevant experimental results obtained from the triaxial tests, namely that the liquefaction potential of the Manzanillo sands is unaffected by densification over a rather large range of relative densities, equation 13 can be applied to specifiy the minimum CPT strength that can safely be accepted. Liquefaction or flow deformations can be avoided if values of $\psi \leq 0$ are prescribed in that expression, to guarantee dilatant behaviour during seismic shocks. This may impose heavy, usually very costly demands on the actual field densification process that were eased admitting that the maximum acceptable value of the state parameter was 0.05. This is equal to accepting a limited amount of damage, should the design earthquake occur ($a_{max}=0.4\ g$).

Another acceptance criterion may be derived using the correlation between initial stiffness (or shear wave velocity) and the state parameter. From Fig. 17, taking $\psi = 0.05$, the minimum acceptable penetration resistance is obtained from:

$$\frac{G_{max}}{(\sigma'_v)^{3/4}(p_a)^{1/4}} \approx 600 \qquad (16)$$

Substituting the value of G_{max} given in equation 8 into the former, one obtains an expression for the minimum penetration resistance required.

Of course, one of the recommendations of the study was that the fills be compacted in order to avoid damages during future earthquakes. The recommendation was followed partially in the yards behind the container terminal (see Fig. 6) which were improved with the vibrocompaction method. Unfortunately the fills in the rest of the San Pedrito harbour were not treated in this manner.

Minimum penetration resistance profiles obtained from the above two criteria are given in Fig. 18 together with the results of typical CPT soundings performed before and after vibrocompaction, for a sample site in the Container Terminal yards. As seen in this figure, use of equation 13 leads to a more optimistic estimate of the minimum value of q_c whereas the adoption of equation 16 is more conservative. The actual penetration profiles obtained after vibrocompaction satisfy both.

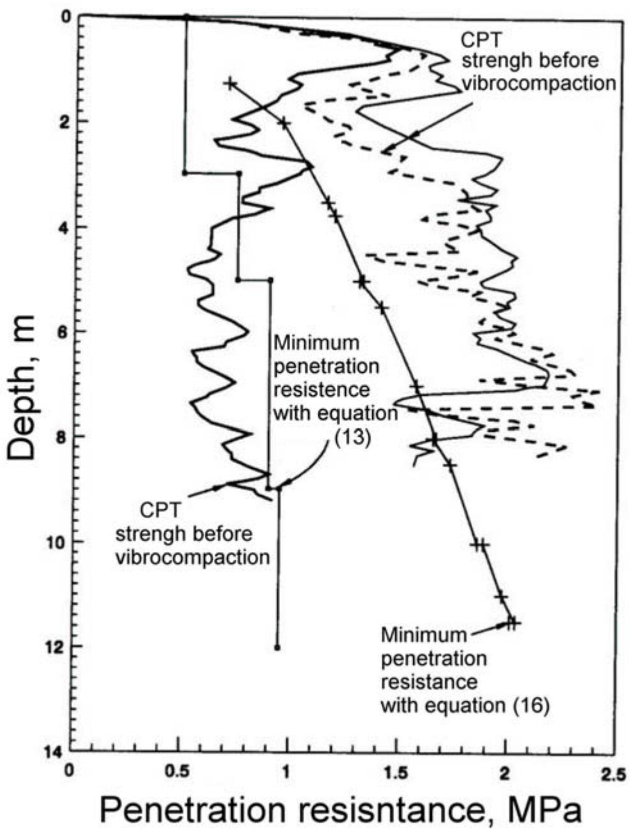

Fig. 18 Example of CPT test results before and after vibrocompaction. Acceptance criteria also shown.

Final comments

The high vulnerability of the dredged fills in San Pedrito to liquefaction, prompted the initiation of further, more detailed studies into their behaviour. The reassessment of the liquefaction susceptibility of the fills in the Container Terminal confirmed that they would liquefy again, even for earthquakes of moderate intensity. Furthermore, the laboratory studies showed that the properties of the fills would only improve significantly if their relative density exceeded 85 %. The presence of elongated, planar shell fragments and micaceous sheets within the sand mass enhances the formation of loose unstable structures, which explains that, at least partially.

The study also concluded that the fills in other harbor facilities in San Pedrito that had not been improved would in all likelihood, liquefy again should earthquakes of similar intensity occur.

TECOMÁN EARTHQUAKE OF 21 JANUARY, 2003

Between 1973 and 1995, the rupture areas of earthquakes along the Colima-Jalisco coastline had left a gap, Fig. 19. This gap broke during the 21 January, 2003, Tecomán Earthquake (Singh et al, 2003).

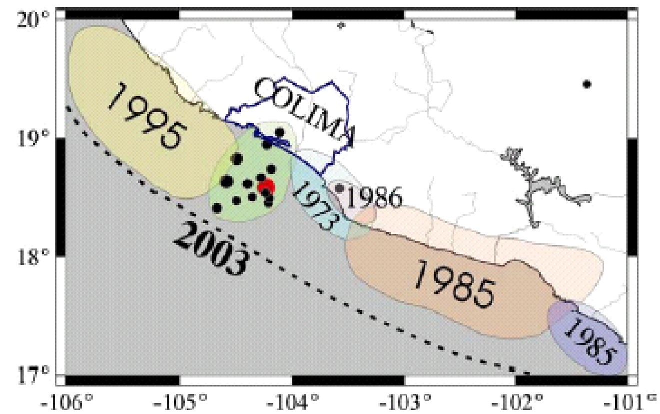

Fig. 19 Rupture areas of recent earthquakes that have affected the State of Colima. The large dot represents the epicentre of the 21 January, 2003, event; smaller dots are replicas that followed immediately after the main event (up to January 22). Source: Servicio Sismológico Nacional (Pacheco, 2003).

There were no accelerograhic measurements near the epicentral area but accelerations measured at the Manzanillo power plant, some 55 km from the epicentre, reached 323 gals. The next closest station to produce a record was located 137 km away from the epicentre. Accelerations there were 38.8 gals. Seismologists have established that the rupture propagated from the epicentre following a NW path towards the city of Colima (Singh et al, 2003). This fact is clearly illustrated by looking at the response spectra obtained from the accelerograhic records recorded at the Manzanillo power station, Figs. 20 and 21. As seen there, intensity of motions along the east-west direction was considerably larger than in the north-south direction.

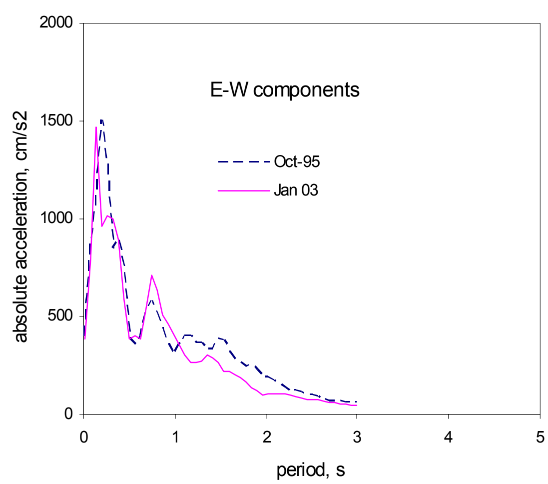

Fig. 20. Response spectra along the east-west direction obtained from accelerograms recorded at the Manzanillo Power Station during the 1995 and 2003 events.

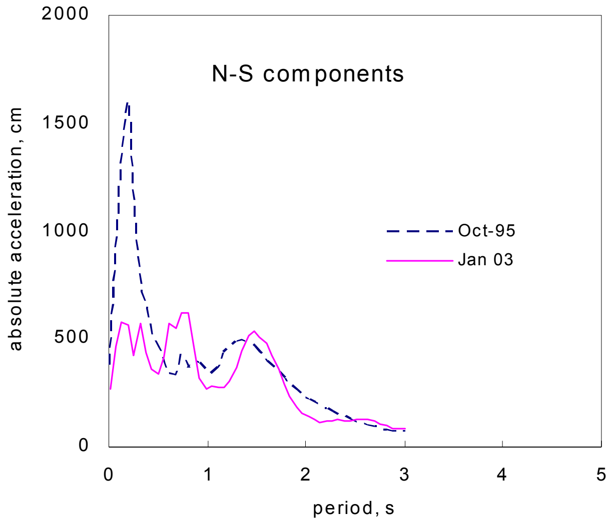

Fig. 21. Response spectra along the north-south direction obtained from accelerograms recorded at the Manzanillo Power Station during the 1995 and 2003 events.

Shortly after the earthquake, several reconnaissance teams visited Tecomán, Manzanillo, the city of Colima and other sites and towns in the state of Colima as well as in the neighbouring states of Jalisco and Michoacán (i.e. Wartman et al, 2003). The authors visited the zone a few weeks later. In what follows we summarize some of the main findings, from the point of view of geotechnical engineering.

Damages in Manzanillo

Liquefaction and lateral spread again affected Manzanillo but to a much lesser degree than in the 1995 event. In the Colonia Burócratas, there were no apparent damages to houses or other structures but it was possible to identify isolated sand blows that had no consequence. The uncompacted granular fills were evidently subjected to lower accelerations than in the previous large earthquake. The municipal pier, some 3 km south of the San Pedrito Harbour, moved seawards about 5 cm. This structure, built early in the XXth century, has displaced in a similar manner during previous earthquakes.

The harbour facilities in San Pedrito have expanded and there are two new container terminals, north and south of the terminal shown in Fig. 6, as well as a grain-handling terminal where large silos were constructed. The port's infrastructure has also been enhanced, including additional thoroughfares. The authors have not been able to confirm if the fills behind the new docks were did take place, as explained later. It is known for certain that liquefaction was also observed north of the container terminal described previously and that the vibrocompacted fills behaved well. The silos in the grain terminal were founded on cast in place piles not connected to the super structure. The space left in between was vibrocompacted compacted. Vibrocompaction increased SPT resistance from 8 to 15 blows to at least 50 (López et al, 1999). The silos did not suffer any damage nor were residual displacements reported after the earthquake.

Overall, the sandy fills in San Pedrito behaved according to the prognosis made after the 1995 earthquake.

General geological features in the city of Colima

The city of Colima lies on a plateau that dips slightly towards the southwest. It is drained by five main parallel brooks that run from the northeast to southwest. Volcanic breccias outcrop towards the northeast, the northwest is covered by sandstones containing andesitic gravels. Clastic deposits are found in the eastern portion of the city and, finally, its southern and lower part is constituted mainly by alluvial deposits which contain materials transported by the water flows. The map in Fig. 22 illustrates the surface geology and the location of the brooks. It is important to note that much of the metropolitan area (Colima and the adjacent Villa de Álvarez) is covered by materials transported from those brooks and by many other smaller affluents, some of which were covered with loose fills, as urban area expanded.

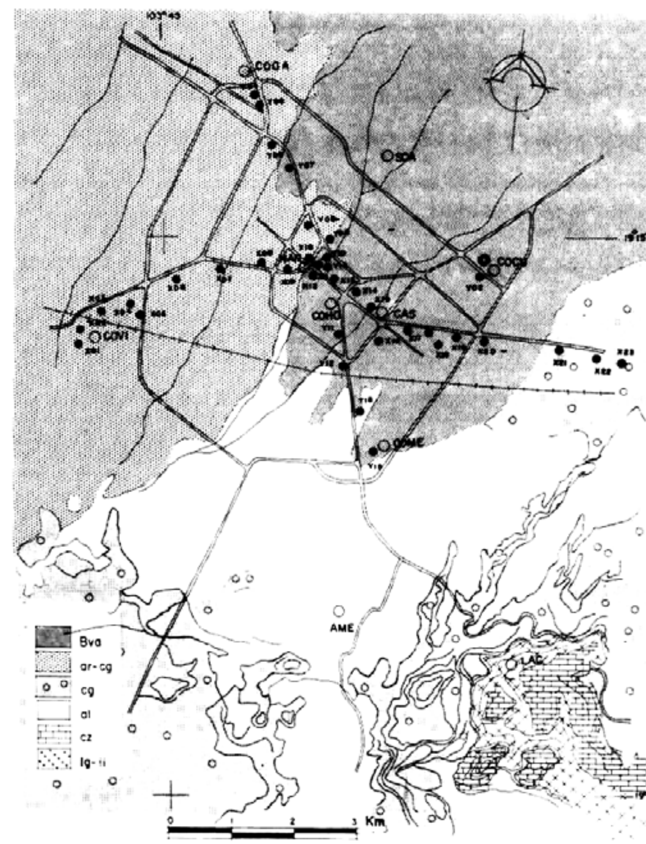

Fig. 22 Surface geology in the Colima-Villa Aldama urban area. Bva = volcanic breccia, cg = sandstone, al = alluvial deposits, cz = limestone.

From records of temporary accelerograhic networks it has been established that seismic amplification with respect to a limestone outcrop varies between 2 and 6 (Tena et al, 1997, Gutiérrez et al, 1996). In other studies it has also been shown that the Nakamura spectral quotients in the urban area show two distinct peaks: one between 0.3 and 0.7 s and the other at about 0.1 s (Lermo, 1997).

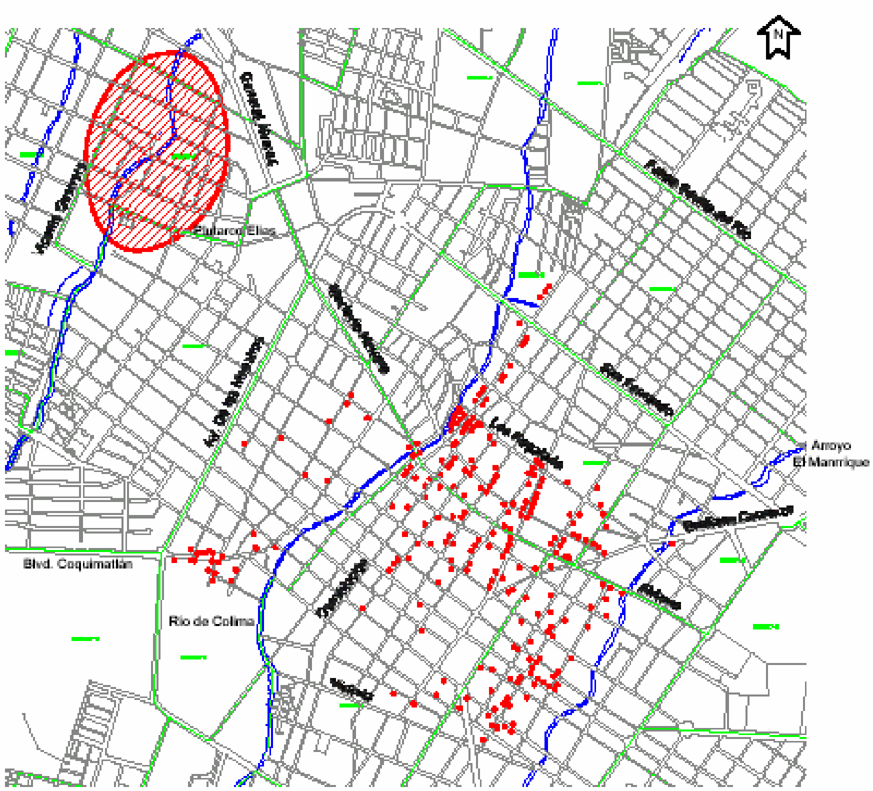

Fig. 23 Map showing the distribution of damages in the Colima-Villa Aldana urban area

Damages in the Colima-Villa de Álvarez urban area

Structural damage concentrated mainly in small rise houses. In the central part of the city old adobe dwellings collapsed but there were lots of confined brick wall houses that also suffered unrepairable damage, many of which were "engineered". These low rise buildings have short natural periods that approximate the periods at which strong amplifications were identified by Lermo (1997). The map in Fig. 23 indicates that some of the damage aligned along the waterways, where loose fills are frequently found.

Local heterogeneities in the subsoil together with deficient engineering practice also contributed to produce damage. For example, there were several cases in which new structures were founded using the abandoned foundations of older previously demolished houses. In other less frequent situations, footings were partly supported on large boulders and partly on poor uncompacted silty fills.

Liquefaction (sand boils, local differential settelements) and lateral spread were also apparent along the banks of some of the brooks. However, sand liquefaction was especially severe in Villa de Álvarez, in northwest of the urban zone within the area enclosed by a circle in Fig. 23. Sand boils and large settlements were observed throughout the area in which 450 small rise structures were condemned after suffering cracks and severe tilts due to differential settlements (Fig. 24). The zone had been previously been occupied by brick factories that mined in open pits the surficial clayey soils to make their product.

Fig. 24 Cracks and soil displaced in the Villa de Álvarez sand liquefaction event.

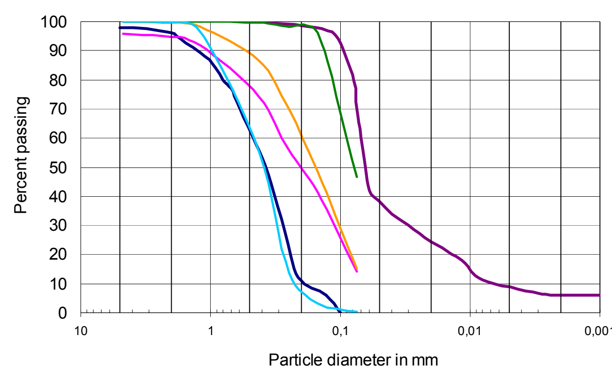

Fig. 25 Grain size distribution curves of samples from the Vila de Álvarez fills that liquefied

When the brick factories were abandoned, urban developers filled the pits with loose, uncompacted granular materials and in some cases, rubble. The graphs in Fig. 25 show typical grain size distribution curves obtained from samples of these fills.

Landslides

It has been estimated that the number of landslides may have been of the order of several thousands, generally involving small volumes of displaced material. Many of these occurred along the Armería and Remate rivers in northern Colima. Slopes also became unstable in some of the deep ravines in flanks of the large Nevado de Colima volcano and along the Atenquique Canyon. Seven highways in the states of Colima, Michoacán and Jalisco were closed temporarily on account of landslides albeit in most cases for a few days at the most. The largest landslide along a highway involved the displacement of 100,000 m^3 of material (Fig 26). Overall, according to engineers from the Ministry of Communications, slope instabilities in highways were fewer and of less importance than those that usually occur during the rainy season.

Fig. 26 Landslide in the Colioma-Tecomán highway (100,000 m^3)

Other damages

The Coahuayana bridge suffered displacements and in one of its supports local structural failure was also observed. At the time of the earthquake the bridge was being underpinned. Settlements at one of its supports, founded on concrete piles, had already been observed after the 1985 and 1995 earthquakes. Liquefaction was observed during the 2003 event. SPT soundings performed a few months before the earthquake indicate the presence of a loose sand stratum between 2 and 9.5 m with N values varying between 10 and 15. The displacements suffered by the bridge were attributed to the fact that the piles did not reach the required depth and to constructive defects, rather than to liquefaction.

The Las Trojes and the Las Piedras earth dams about 175 km from the epicentre suffered some cracking and settlement but of no serious concern in terms of their stability or of their safety.

Final comments

Site effects evidently influenced the magnitude and distribution of damage in Colima and Villa de Álvarez but further studies in this respect are necessary in the future. In the case of Villa de Álvarez, the University of Colima, together with municipal authorities are performing a detailed geotechnical investigation there. On the other hand, seismic and geotechnical microzoning studies are also being revised and updated.

TEHUACÁN EARTHQUAKE OF 15 JUNE, 1999

This earthquake mainly affected the states of Puebla and Oaxaca. The city of Puebla, located some 110 km south east of Mexico City, Fig. 27, is the fourth largest in Mexico, with 1,500,000 inhabitants it In what follows we succinctly make reference to the relationship between geotechnical conditions and response spectra obtained at three different sites in the city of Puebla during the event.

This was a normal faulting earthquake. Events of this type have return periods of a few decades (they are much less frequent than subduction seisms) and have historically been the most damaging in the city. In 1973, for example, one of these produced extensive destruction in the city. The approximate location of its focal mechanism is indicated in Fig 28.

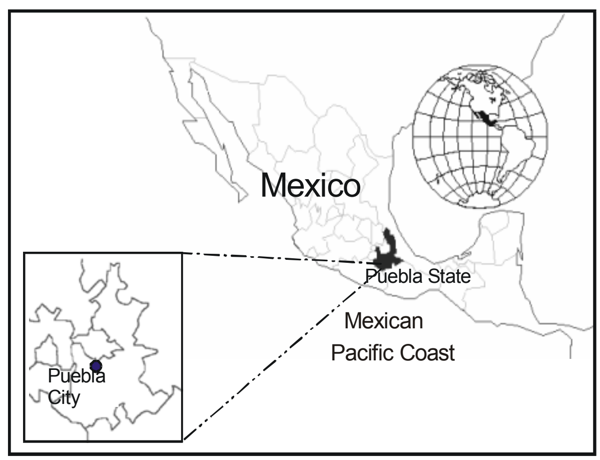

Fig. 27 location of the city of Puebla

The largest and the most important damages in structures were recorded in over 500 historical monuments. In the city of Puebla and its environs, some 125 km from the epicentre, many churches and conventual cloisters underwent different degrees of cracking, fissuring, tilting, loss of plumb and, in a few extreme cases produced the collapse of belfries, towers, domes and vaults. Damage in the city concentrated in the central core where important architectonic monuments exist, replicating a pattern observed previously in the 1973 event. Notwithstanding the fact that previous earthquakes had affected many of the structures that failed or were damaged by the 1999 seism, it is significant that damage again concentrated in and around the same part of Puebla.

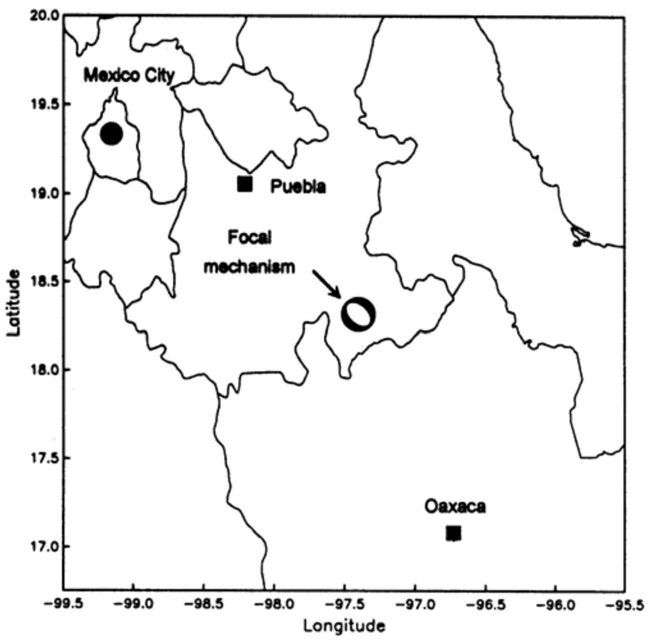

Fig. 28 Location of the focal mechanism of the 15 June, 1999 earthquake.

Geotechnical conditions

There are nine accelerographic stations in the city of Puebla (see Fig. 29), installed in sites having different geotechnical conditions. Accelerations recorded in three of the stations, PBPP, PHPU and SRPU, are used to illustrate, by means of acceleration response spectra, site effects during this earthquake.

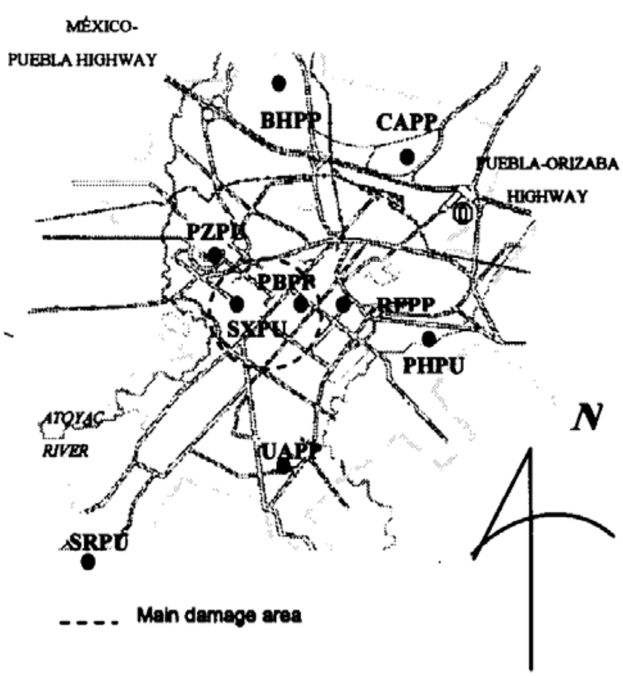

Fig. 29 Accelerograhic network in the city of Puebla.

The first station, PBPP, is located in the central part of the city where old masonry structures suffered intense structural damage. The graph in Fig. 30 shows a shear velocity profile obtained from a down-hole sounding and a seismic cone test down to a depth of 14 m. Silty clays and sands form a fill in the upper 3.5 m. Rather large shear wave velocities were measured next, some times exceeding 700 m/s, within a layer of good quality travertine (hydrothermal) rock, occasionally with small size cavities. Lacustrine clays were then found, down to the base of the sounding at 14 m. The soils beneath this depth are alluvial silty clays and sands interspersed with more layers of travertine or calcareous tuffs. This sequence extends to about 50 m where the base rock, limestone, is found.

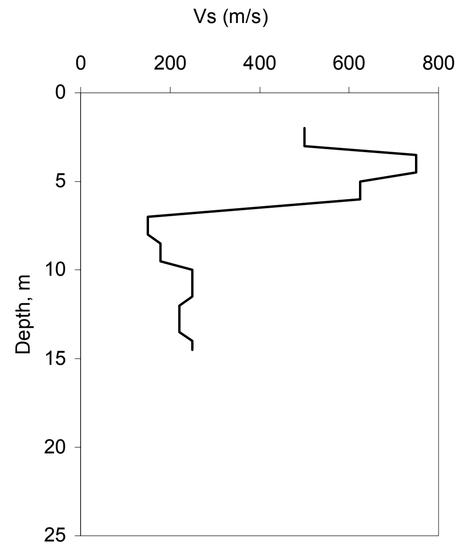

Fig. 30 Shear wave velocity profiles at the PBPP station in Puebla

The second station is PHPU, located in the southeast of the city on a sequence of volcanic deposits. The upper strata down to about 7.0 m contain lenses of clayey materials but medium dense to dense silts and silty sands interspersed with coarser materials prevail in the rest of the stratigraphic column, down to 27.0 m. This stratigraphic sequence can be readily identified in the shear wave velocity profile given in Fig. 31, obtained from a down-hole test. Measurements in this test were performed simultaneously with two sensors separated 90 cm.

The last station, SRPU, lies south of the city. Plastic clays were found between 2.0 and 5.0 m, followed by sequence of volcanic sands, silty sands and coarser materials down to the bottom of the sounding (27.0 m). Shear wave velocities are given in Fig. 32.

Response spectra and site effects

The maximum horizontal acceleration, 279 cm/s^2 was recorded at the PHPU station, along the north-south component, showing a strong directivity effect with respect to the source. The maximum acceleration along an east-west axis (nearly perpendicular to the direction of the source), 217 cm/s^2, was measured at the SRPU station. The ratio between maximum vertical and horizontal accelerations varied between 0.2 and 0.5. These quotients are much higher than those found in records obtained from the more frequent, more distant and locally less intense subduction earthquakes that also affect the city.

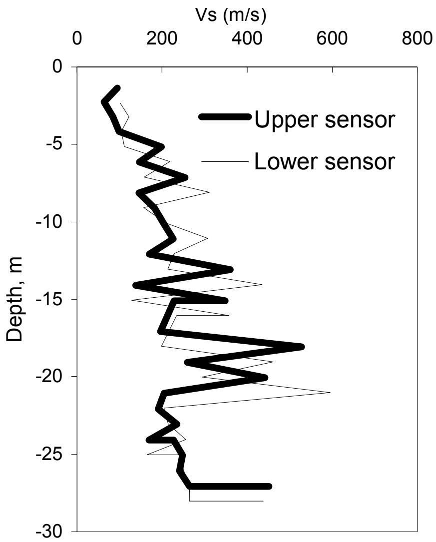

Fig. 31 Shear wave velocity profile at the PHPU station in Puebla.

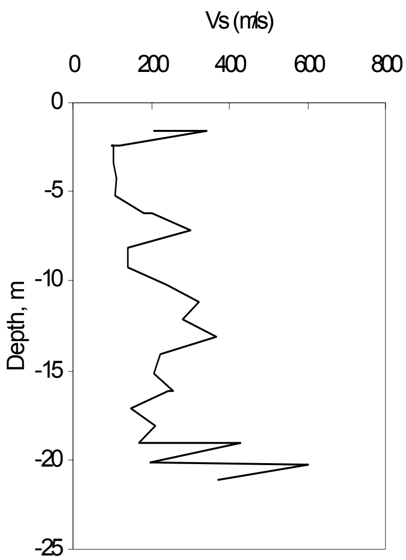

Fig. 32 Shear wave velocity profile at the SRPU station in Puebla.

Site effects at these three sites can be noted by looking at response spectra for 5% damping. The spectra derived from the north-south components of acceleration, Fig. 33, show that relatively large spectral ordinates are found at the PHPU in a period range of 0.2 to 1.2 s whereas the largest ordinates in the SRPU are found between 0.35 and 0.7 s. The spectrum from the PBPP station shows two very well defined peaks at 0.45 and 1.0 s, thus reflecting the stratigraphic conditions at that site, i.e., the presence of a very rigid layer of travertine rock overlying soft lacustrine clays (see Fig. 30). The response spectra obtained from the east-west components of acceleration measured at the PHPU and the SRPU stations display very notable differences with respect to the spectra obtained along the other horizontal direction, Fig. 34. Finally, the vertical component spectra display large ordinates between 0.2 to 0.5 s, as indicated in Fig. 35.

Comparing the spectral ordinates at each site with the corresponding maximum ground acceleration, the largest spectral amplifications are found at the PBPP station, for the horizontal components and for the vertical components, these maxima were found at the SRPU station.

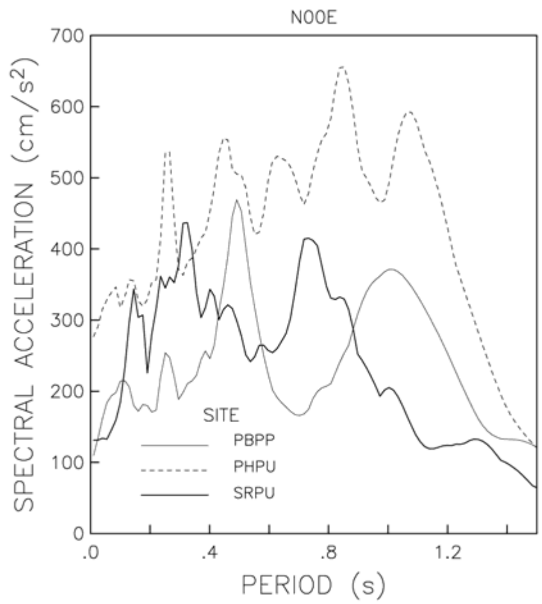

Fig. 33 Response spectra along the north-south direction.

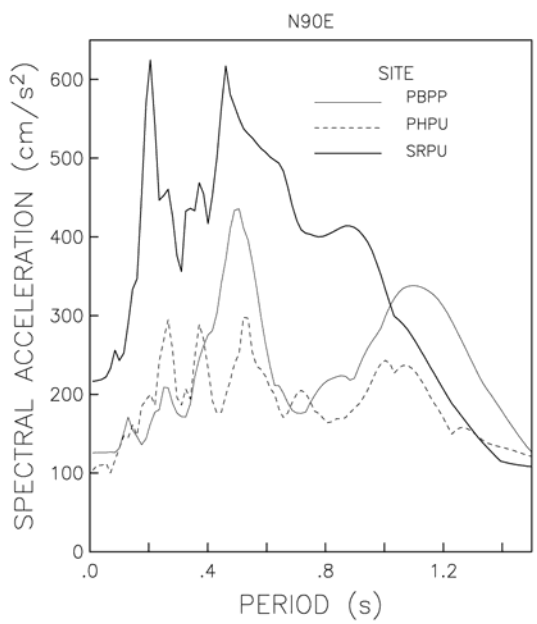

Fig. 34 Response spectra along the east-west direction.

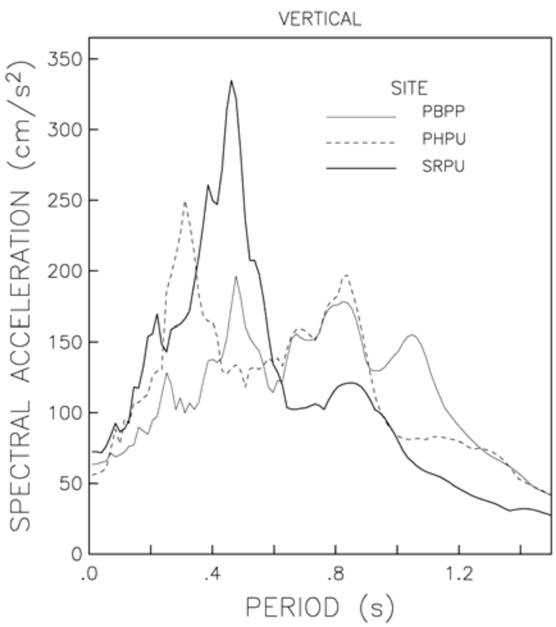

Fig. 35 Response spectra for the vertical component.

It is clear, from the information presented in Figs. 34 to 35, that site effects are extremely important in the city of Puebla. The shear wave velocity profiles also revealed important differences in the stratigraphy and the mechanical properties at the three sites where this information was available. The intensity of seismic motions varied strongly along the two directions analysed, a fact that suggests that topography and morphology in Puebla may also play an important role in the characteristics of seismic motions there

A general feature of the seismic movements in Puebla during the event under consideration was that seismic energy concentrated in the 0.3 to 1.0 s period range which is higher than the ones associated to motions produced by the more common subduction earthquakes in Mexico. Accordingly, the relatively stiff soils in the three sites studied here amplified the incoming motions. Pending further research into the matter, it ought to be expected that dynamic amplification during subduction earthquakes will not be as high.

Site effects can be best appraised if a reference site is available where no such effects are present, i.e. a “hard rock” site. There is no such reference in the city of Puebla. Still, a qualitative assessment of site effects can be carried out by a looking at the strong motions recorded at a rock outcrop in Oaxaca, station OXLC, 270 km away from Puebla and 150 km away from the epicentre. That is roughly the same as the epicentral distance from Puebla.

The response spectrum obtained from the north-south acceleration component recorded at the OXLC station is compared with the response spectra of Puebla’s PBPP and BHPP stations. Geotechnical soundings are not available for the latter site but the interpretation of electrical resistivity studies performed nearby suggest that the outcropping basalts are about 6.0 m thick and are underlain by alluvial deposits of undetermined depth. As seen in Fig 36, spectral ordinates at the PBPP station in central Puebla are about 4 times larger than those of the BHPP spectrum for periods ranging from 0.45 to 1.0 s and about 14 to 11 times larger than the ordinates of the OXLC spectrum over the same period range.

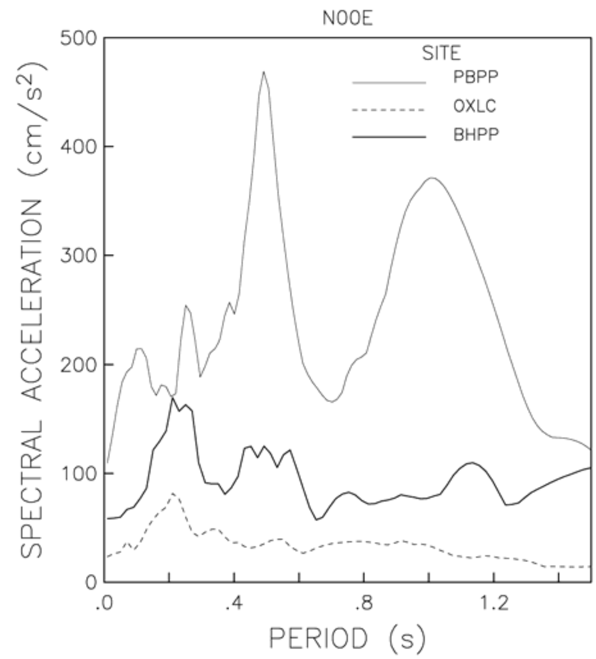

Fig. 36. Response Spectra in PBPP, BHPP and OXLC sites

It is significant that earthquake damage in the city of Puebla has always concentrated in and around its historic core. According to the present knowledge about its geotechnical and stratigraphical conditions, the greater damage zone coincides approximately with the parts of town where the travertine rocks are found, like the PBPP station (Auvinet, 1976; Jiménez, 1997; Asomoza et al, 1998). The 15 June 1999 event was not exceptional in this respect despite the fact that, as shown previously, motions were more intense in other parts of the city, as in the PHPP and SRPU stations.

It is evident, from looking at the response spectra for the PBPP station, that rigid, short period structures were the most vulnerable to this event. It is not surprising then that many churches and many architectonic monuments having natural periods at around 0.45 s, corresponding to the second peak observed in the PBPP spectra were damaged. In the case of ancient structures, however, it must also be mentioned that in all probability, some of them had hidden structural distress from previous quakes. It can also be argued that the vertical component oscillations, rich in high frequencies, were also rather large. The coupled effect of both vertical and horizontal vibrations certainly contributed to increase the level of damage, as it is the case for locations near deep source events.

<u>Final comments</u>

Acceleration histories recorded on 15 June 1999 at three stations showed that site effects in the city of Puebla have a major influence on local intensities. More effort is needed to understand the relationship between seismic intensity, geotechnical conditions and damage distribution. To this end, more and deeper shear wave velocity profiles should be performed in the future, as well as more research into the dynamic properties of Puebla´s subsoil.

Given the geotechnical conditions of the three sites studied here, it was suggested that larger dynamic amplifications are to be expected during normal faulting earthquakes than during subduction events. Frequency contents of both types of events bear an enormous influence on local amplifications but more accelerographic data are certainly required to confirm this statement

CONCLUSIONS

The three case histories described here illustrate that the nature and quality of the geotechnical lessons that can be learnt from the observation of damages after an earthquake depend very much on the type, amount, and quality of the available data. In the case of the 1995 Manzanillo earthquake, geotechnical data allowed for the assessment of the liquefaction potential of the sand fills in the San Pedrito Harbour with a degree of detail that could not be achieved in the other two cases. In the case of the Puebla 1999 event the authors were able to collect acceleration records as well as detailed shear wave velocity profiles at three sites in the city and were also able to evaluate site related effects in those sites. On the other hand, data of all types in connection to the evaluation of damage that resulted from the Tecomán 2003 earthquake was scant. Consequently, the analyses were mainly of the qualitative type.

REFERENCES

Alcántara L, González-Pomposo G, Posada-Sánchez A E, Almora D & Torres M [1997]. “Seismic site effects observed in Puebla City, México”, *Proc. 29th General Assembly of International Association of Seismology and Physics of the Earth´s Interior* (IASPEI), Thessaloniki, Greece

Alcántara L, González-Pomposo G, Almora D, Posada-Sánchez E, Torres M, Vázquez R, Macías M, Aguilar A, Jiménez J, Pérez C, Sandoval H [1999]. *“The Tehuacán, Puebla earthquake of June 15 1999 (M =6.7), recorded at the Puebla City Accelerographic Network*”, Internal report RACP-II/BUAP-04, Instituto de Ingeniería, UNAM, Mexico City.

Asomoza, G., Vera, A. and Reyes, L. [1998]. “Zonificación geotécnica para el área urbana del valle de Puebla”, *Proc., XIX Reunión Nacional de Mecánica de Suelos*, Mexican Society for Soil Mechanics, Puebla, Mexico, pp. 67-74.

Auvinet G. [1976]. “El subsuelo de la ciudad de Puebla”, *Proc., VIII Reunión Nacional de Mecánica de Suelos,* Mexican Society for Soil Mechanic, Guanajuato, Mexico, vol 2.

Been, K. and Jefferies, M. G. [1985]. “A state parameter for sands”, Geotechnique, vol. 35, pp. 299-112

Been, K., Crooks, J. H. A., Becker, D. E. and Jefferies, M. G. [1986]. “The cone penetration test in sands: part I, state parameter interpretation”, Geotechnique, vol.36, No. 2, pp. 239-249

Been, K., Jefferies, M. G., Crooks, J. H. A. and Rothenburg, L. [1987]. “The cone penetration test in sands. Part II, general inference of state”, Geotechnique, 37, No. 3, pp. 285-299

Domínguez-Rivas, J., Singh, S. K. and Sánchez-Sesma F. J. [1997]. “Sismología”, 1 in: *El Macrosismo de Manzanillo del 9 de Octubre de 1995*, (A. Tena, ed.) Colima, Mexico: Universidad de Colima, Sociedad Mexicana de Ingeniería Sísmica, pp. 11-26.

Flores, O. and Romo, M. P. [1997]. “*Comportamiento de los jales de la unidad minera El Herrero”,* Instituto de Ingeniería, UNAM, internal report, Mexico,.

Gutiérrez, C. K. Masaki, J. Lermo y J. Cuenca [1996]: “*Microzonificación Sísmica de la Ciudad de Colima*”, Serie Cuadernos de Investigación CENAPRED No. 33, Mexico.

Holzer, T. L., Youd, L. and Hanks, T. C. [1988], “Dynamics of liquefaction during the Superstition Hill earthquake (M=6.5) of November 14, 1987”, *Post presentation, ASCE Geotechnical Engineering Division Specialty Conference, Earthquake Engineering and Soil Dynamics*, vol II, Park City, Utah.

Ishihara, K. [1982]. “Evaluation of soil properties for use in earthquake response analysis”, *Proc. Int. Symposium on Numerical Models in Geomechanics*, Zurich, (R. Dungar, G. N. Pande and J. A. Studer, eds.), pp. 237-259.

Jamiolkowski, M., Ghionna, V. N., Lancellotta, R. and Pasqualini, E. [1985]. “New correlations of penetration tests for design practice”, *Proceedings of Penetration Testing, Internation Symposium on Penetration Testing*, Orlando, Florida, vol. 1, pp. 263-296

Jica [1985]. “*Study on the development project of the port of Manzanillo in the United Mexican States*”, Interim report, Japan International Cooperation Agency.

Jiménez, G. [1997]. “*Microzonificación sísmica de la ciudad de Puebla*”, MsC Thesis, División de Estudios de Posgrado, Facultad de Ingeniería, UNAM.

Lermo, F. J. [1997]. “Microzonación”, in: *El Macrosismo de Manzanillo del 9 de Octubre de 1995*, (A. Tena, ed.) Colima, Mexico: Universidad de Colima, Sociedad Mexicana de Ingeniería Sísmica, pp. 45-84.

López, R. Guiarte, F. Po Lam, G. and Shao, L. [1999]. “Compsite ground reinforcement foundation system for large capacity grain domes under static and seismic loadings”, *Proc. XI Panamerican Conf. on Soil Mechanics and Foundation*

Engineering, Foz de Iguazú, Brazil, vol. 3, pp. 1459-1463.

Mitchell, J. K. [1976]. "*Fundamentals of Soil Behavior"*. John Wiley and Sons, New York.

Ovando E y Romo M P [1992], "Estimación de la velocidad de propagación de ondas S en la arcilla de la ciudad de México con ensayes de cono", Sismodinámica, vol 2, pp. 107-123.

Ovando-Shelley, E., Contrera, R. and Sámano, A. [1995]. "Seismic cone test in Mexico City", *Proc. Xth Panamerican Conference on Soil Mechanics and Geotechnical Engineering*, Guadalajara, Mexico, vol. 1, pp. 340-352.

Ovando-Shelley, E. and Pérez, B. E. [1997]. "Undrained behavior of clayey sands in load controlled triaxial tests", Geotechnique, vol. 47, No. 1, pp. 91-111.

Ovando, E., Vieitez, L., and Alemán, J. D. [1997]: "Geotecnia", in: *El Macrosismo de Manzanillo del 9 de Octubre de 1995*, (A. Tena, ed.) Colima, Mexico: Universidad de Colima, Sociedad Mexicana de Ingeniería Sísmica, pp. 82-133.

Pacheco, J. [2003], "*Reporte del Sismo de Colima de enero 2003*", Report of the Servicio Sismológico Nacional, Instituto de Geofísica, UNAM, Mexico.

Robertson, P. K., Woeller, D. J. and Finn, W. D. L. [1992]. "Seismic cone penetration test for evaluating liquefaction potential under seismic loading", Canadian Geotechnical Journal, No. 24, pp. 686-695

Robertson, P. K., [1996]: "Soil liquefaction and its evaluation based on SPT and CPT", *Proceedings*, *Workshop on Evaluation of Liquefaction Resistance*, Salt Lake City, Utah, January 4-6, NCEER Technical Publication.

Romo, M. P. [1995]. "Clay behavior, ground response and soil-structure interaction studies in Mexico City", State of the art paper. *Proc. Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, University of Missouri-Rolla. (S. Prakash, ed.), vol. II, pp. 1039-1051.

Romo M P y Ovando E [1996]. "Modelling the dynamic behavior of Mexican clays", *Proc., IX Int. Conf. on Earthquake Engineering*, Acapulco, México.

Sánchez Leal Freddy (1998). "*Marco unificado para el análisis del comportamiento no drenado de arenas en cámara triaxial"* MSc. Thesis, División de Estudios de Posgrado, Facultad de Ingeniería, UNAM, Mexico, 1998.

Santoyo, E. and Segovia, J. A. [1997]. "*Informe geotécnico para la elaboración y fundamento de la rehabilitación del muelle y patio de contenedores de Manzanill",*. Technical report presented to Administración Portuaria Integral de Manzanillo, Mexico.

Seed, H. B. and Idriss, I. M. [1971]. "Simplified procedure for evaluating soil liquefaction potential", Journal Soil Mechanics and Foundations Div, ASCE, vol. 97, No. SM9, pp. 1249-1273.

Singh, S. K., Ponce L. and Nishenko, S. P. [1985]. "The great Jalisco, Mexico, earthquake of 1932", Bulletin of the Seismological Society of America, vol. 75, No. 5, pp. 1301-1313.

Singh S K, Ordaz M, Pacheco J, Quaas R, Alcántara L, Alcocer S, Gutiérrez C, Meli R and Ovando E [1999]. "A Preliminary Report on the Tehuacán, México Earthquake of June 15, 1999 (M_w=7.0)", Seismological Research Letters, vol 70, No. 5, pp. 489-503.

Singh S K, Pacheco, J., Alcántara, L., Ordaz, M., , Quaas, R., Alcocer, S., Gutiérrez, C., [1999]. "A Preliminary Report on the Tecomán, México Earthquake of 22 January 2003 (M_w=7.4) and its effects", Seismological Research Letters.

Skempton, A. [1986]. "Standard penetration test procedure and effects of overburden, relative density, particle size and over consolidation", Geotechnique, vol. 36, pp. 1301-1313.

Tatsuoka, F., Zhou, S, Sato, T. and Shibuya, S. [1990]. "*Evaluation method of liquefaction potential and its application",*. Report on seismic hazards on the ground in urban areas, Ministry of Education of Japan.

Tena, A. [1997], "Edificios" in: *El Macrosismo de Manzanillo del 9 de Octubre de 1995*, (A. Tena, ed.) Colima, Mexico: Universidad de Colima, Sociedad Mexicana de Ingeniería Sísmica, pp. 268-324.

Wartman, J., Rodriguez-Marek, A., Keefer, D., Deaton, S., Repetto, P., Macari, E. J., Navarro, C., Tejeda J., C., Silva C. y Armando, J., [2003]: "*Preliminary Reconnaissance Report on the Geotechnical Engineering Aspects of the January 21, 2003 Colima, Mexico Earthquake*", Report of the National Science Foundation, Geotechnical Earthquake Engineering Server, http://geoinfo.usc.edu/gees/