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Geotechnical Lessons Learned from Mexico and Other Recent Earthquakes

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Geotechnical Lessons Learned from Mexico and Other Recent Earthquakes

(State of the Art Paper)

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SYNOPSIS: This paper describes the most relevant effects of the September 1985 earthquakes on Mexico City. It discusses the results of recent exploration and research on regional geology, local site conditions and site response analyses, basic soil properties, and observation of actual behavior of soil-foundation-structure systems. The paper shows the impact of these investigations upon new building code requirements, on general design practice and on the specialist's perception of earthquake behavior of Mexico City's subsoil. Recent earthquakes recorded in and around Mexico City have contributed insight into some of the unresolved questions left after the experience of 1985. Damaging earthquakes recorded in other cities in the near past are brought into the discussion. The latter events furnish - by means of the examples of San Salvador in 1986, Armenia in 1988 and San Francisco in 1989, - a more general view of the influence of local subsoil conditions on site response.

INTRODUCTION

Most structures of interest in earthquake engineering rest where there are important site effects: cities on valleys, cities on hill tops, bridges, dams, all are subjected to ground motions significantly modified by of deformable soils or by topography. The present paper centers on the most spectacular case of site effects during recent times —that of Mexico City on 19 September 1985,— ponders on the applicability of conclusions derived from this example to other sites underlain by soft soil and casts a cursory look at topographic site effects. In interpreting the phenomena observed in 1985 in Mexico City attention is given to theoretical models and to observations made in this city during later, weaker earthquakes.

MEXICO CITY SUBSOIL

The Valley of Mexico used to be open towards the South. About 600,000 yr ago the Chichinautzin Range began to rise through the formation of numerous volcanos which plugged the southern outlet. The valley became closed. A lake was formed and gradually filled with materials from the surrounding volcanic activity. During interglacial ages the lake dried out at least partially.

This subsoil was fairly well known before 1985 owing chiefly to the work of Marsal and Mazari (1959). Much was known about the materials' dynamic properties. Yet, the information was insufficient to build a dynamic mathematical model of the valley. Accordingly, extensive exploration and laboratory testing were undertaken since 1987. Conclusions from these as well as from some of the earlier studies can be found in several recent papers (Mooser and Montiel, 1989; Marsal, 1987; Romo and Seed, 1987; etc).

What we now know about this subsoil is presented succinctly in figs 1-9. The map in fig 1 shows the microzoning adopted in the 1987 Building Code for the Federal District (Departmento del

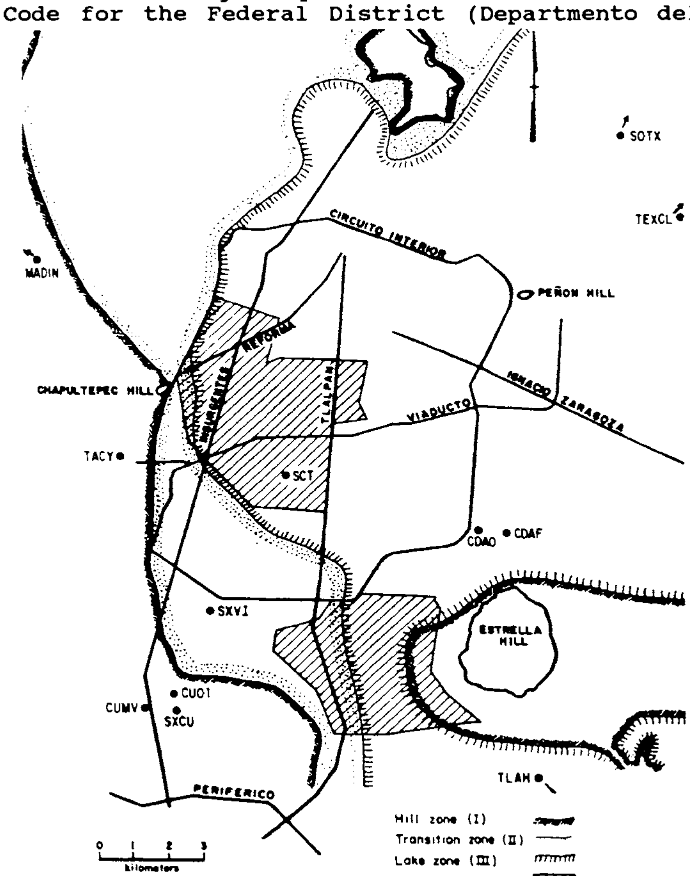

Fig 1. Microzoning adopted in Mexico City's 1987 Building Code. Circles indicate the sites where the 1985 earthquakes were recorded

Distrito Federal, 1987). Three zones are recognized: I rock and firm ground; II transition; III lake bed. Zone I comprises a few outcrops, the hilly areas and what has been termed firm ground, *i e* with no deformable clay deposits. In zone II, a transition where shaking has not been exceptionally violent, there are deformable clay deposits, their total thickness ranging between 0 and 20 m. In zone III the total thickness of deformable clay in the old Texcoco lake basin varies between 20 m at the old town center and 40 m at the virgin lake bed whereas in the Xochimilco-Chalco region it can exceed 100 m.

A simplified contour enclosing the areas where shaking was especially intense in 1985 as inferred from the damage distribution, shows that these fall within zones II and III (Iglesias, 1987). For seismic design, a subzone containing these areas, zone IV, is included within zone III and a small portion of zone II.

In zone I one typically finds up to a few hundred meters of compact, mostly granular soil or volcanic tuff followed by firm aglomerates including some lava flows and, at depths reaching more than 2000 m (fig 2) the Balsas limestones or Upper Cretaceous Formation (about 60 million years old). The firm aglomerates consist of vulcanites, first of the Cocos Formation (18 million years old) and under them the Farallón Formation, resting directly on the Balsas limestones. In zone II, on top of the harder soils there are a few meters of very deformable clay with water contents of up to 200 percent and often, on top of this, a harder crust (frequently an archaeological fill) reaching a few meters in thickness. A typical boring

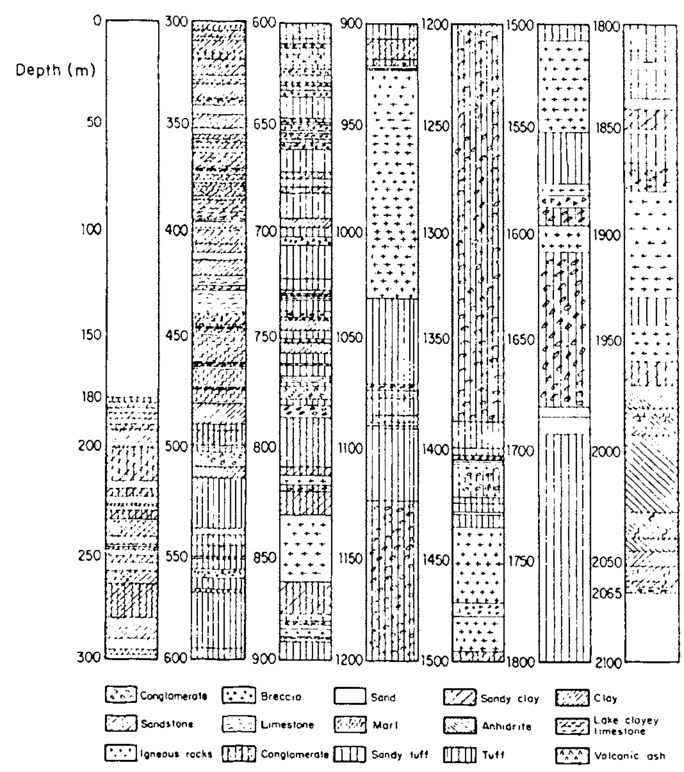

Fig 2. Profile showing the strata that underlie the compressibe clay deposits in Mexico City

in zone III (fig 3) will find a hard crust; then a first clay formation with water contents exceeding 400% and sometimes reaching 600% interspersed with thin seams of ash, sand and volcanic glass down to perhaps 30 m from the surface; next the "first hard layer," an aglomerate having 1-3 m in thickness and very variable hardness, often difficult to penetrate with concrete piles unless there has been previous perforation; under this there is a second clay formation, similar to that in the transition zone; at about 50 m from the surface one meets the second hard layer; from there on down, things are much as under zone II, save

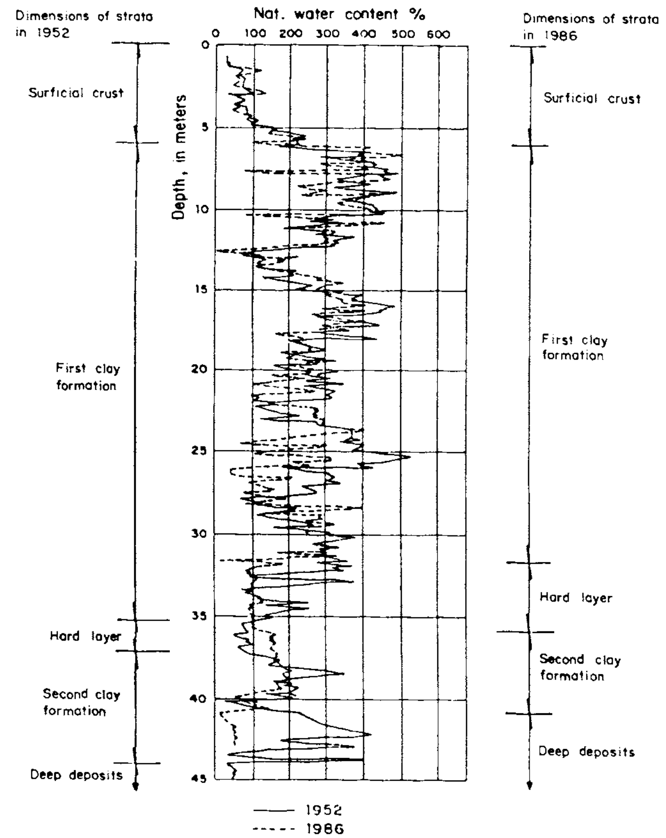

Fig 3. Natural water contents at a site in zone III (data after Marsal and Mazari, 1957 and Jaime, 1987)

that the surface of the Upper Cretaceous can reach depths of at least 3000 m. In the South of the city the first hard layer is usually absent and, for a given water content, the clay is weaker. The data given in fig 3 were taken from borings performed on the same site at different dates, 34 years apart. Note the differences in water contents and strata thicknesses (regional subsidence) that have been brought about by the extensive exploitation of aquifers in and around Mexico City.

Shear wave velocities in the soft clay have been obtained through field and laboratory tests. Typical average values (75 m/s in the upper formation and 110 m/s in the lower one) are low compared with those obtained on less plastic clays (fig 4). Somewhat smaller values are found in the South of the city. Shear wave velocities

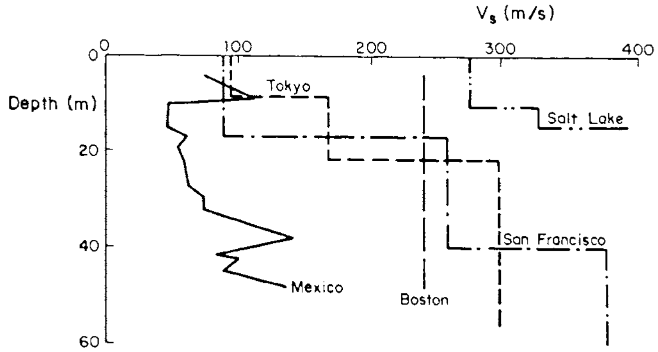

Fig 4. Shear wave velocity profiles in Mexico City and other locations (after Whitman, 1986)

in other formations are also shown in figs 5 and 6.

Cyclic undrained triaxial tests on clay samples give a nearly linear behavior up to stresses approaching the ultimate, followed by a decay. This is reflected in the dependence of the secant shear modulus and the damping ratio on shear strain (fig 7). In soils having large plasticity indices, degradation of normalized shear moduli, G/G_{max} (G_{max} = value of G at small strains) in fig 8, is smaller, at any given value of shear strain, than in low plasticity materials (Romo *et al*, 1989b). In this respect, Mexico City clays obey the same tendency as other soils (Dobry and Vucetic, 1987). Hence, their behavior cannot be thought of as extraordinary.

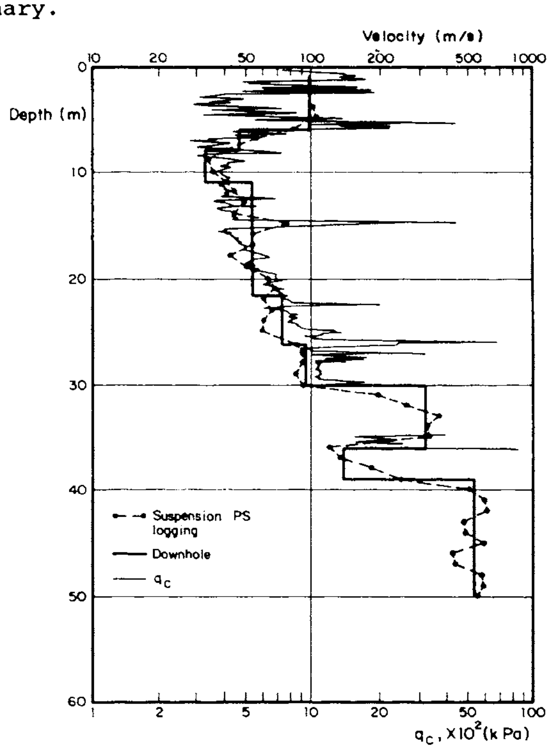

Fig 5. Shear wave velocity and cone penetration resistance. Preloaded Texcoco Lake, zone III (after Jaime and Romo, 1988)

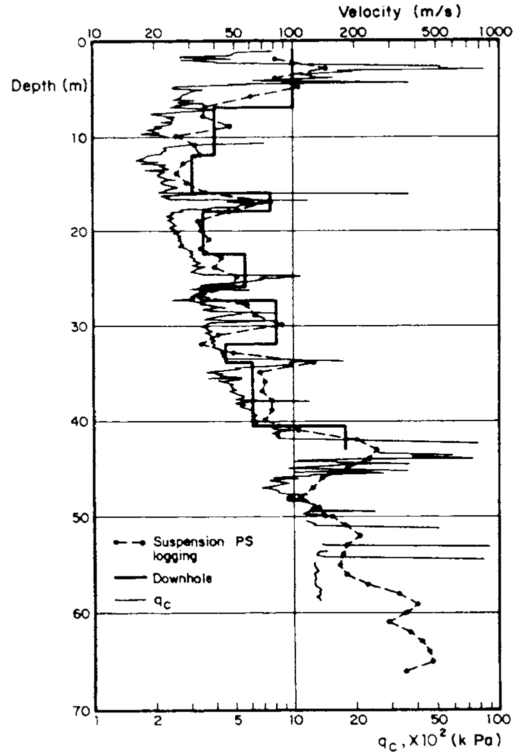

Fig 6. Shear wave velocity and cone penetration resistance. Virgin Texcoco Lake, zone III (after Jaime and Romo, 1988)

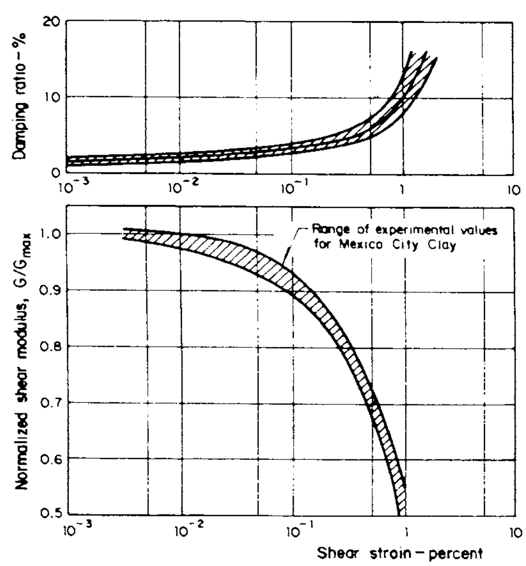

Fig 7. Damping ratio and normalized shear modulus versus strain for Mexico City clay (after Seed *et al*, 1988)

In successive load and unload cycles the clay can be treated a Masing solid but does not fit well the Ramberg-Osgood relation. Little permanent deformation accumulates in these cycles provided the maximum stress applied is not too close to the resistance. Recent experimental results support the idea that there exists a threshold shear strain, typically 3%, below which the cyclic stress-strain behavior is essentially elastic (fig 9). Furthermore, Mexico City clay, when tested under dynamic conditions, can exceed its static strength by as much as 20% (Romo *et al*, 1989b). Rate effects may account for this, according to data from CU triaxial tests (Alberro, 1979). The soil is markedly thixotropic, reducing its strength to the order of 1/30 upon remolding. Laboratory data show that after a short time it regains most of its original strength (Jaime, 1987; Legorreta, 1988).

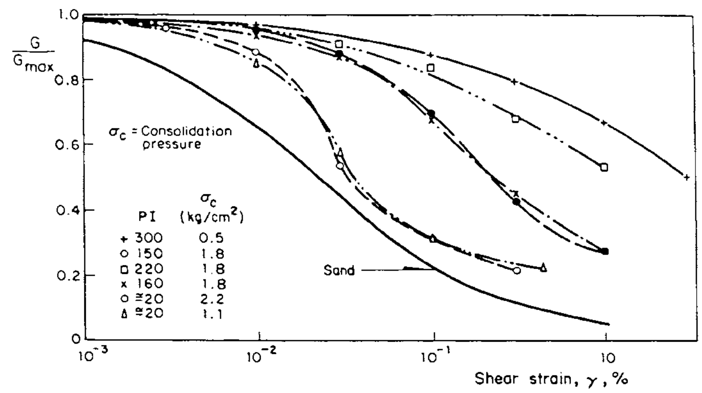

Fig 8. Normalized shear moduli versus strain for soils having different plasticities (after Romo *et al*, 1989)

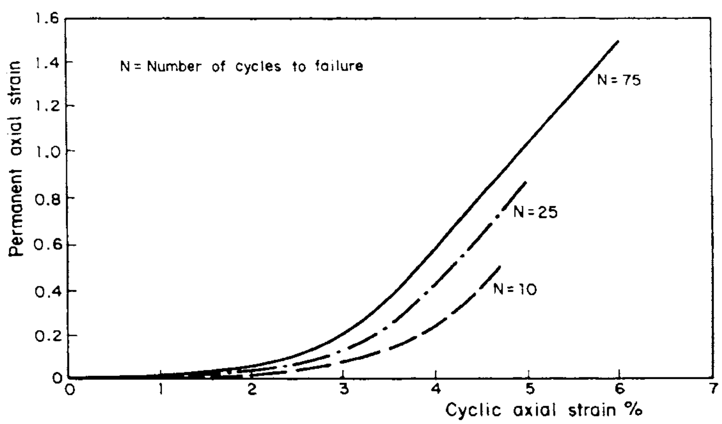

Fig 9. Permanent strain accumulation during cyclic triaxial tests on Mexico City clay (after Romo *et al*, 1989)

Different teams (Kobayashi *et al*, 1986; Lermo *et al*, 1988; Rodríguez, 1986) determined the Fourier spectra of ground vibrations caused by ambient disturbances ("microtremors") at several sites within the city. It is not possible at present to interpret the seismic implications of the amplitudes in these spectra, for the nature of ambient disturbances and the paths of the waves they generate differ from those of earthquakes, and the intensity of the disturbances varies in an unknown manner with location and time of day. Still, there is little doubt that the period at which the maximum ordinate occurs in these spectra coincides essentially with what may be termed the prevailing period at the site, that is, the period at which the peak spectral acceleration occurs during earthquakes sufficiently small so as not to cause widespread nonlinear behavior of the soil. Values found for these prevailing periods are shown in fig 10.

The Upper Cretaceous is traversed by three systems of faults under and around the valley (fig 11). One such system runs approximately NW to SE; distributed over five nearly vertical faults along a few hundred kilometers, this system could respond to a total vertical offset of around 1500 m; it may constitute one end of a graben. The other two systems —Tláloc-Apan and Santa Catarina— run at about 45° with the directions of the cardinal points. They have in all likelihood been caused by shearing stresses and ensuing strike-slip, induced by the EW component of relative motion between the oceanic Cocos Plate and the North American Plate. This component has changed in rate and sign over the 25-million-year history of the Cocos Plate.

Many faults in the base rock continue in the overlying conglomerate and some partly penetrate the lake deposits. None, however, traverse these deposits completely. The most recent movement in any of the faults took place at least 120,000 yr ago.

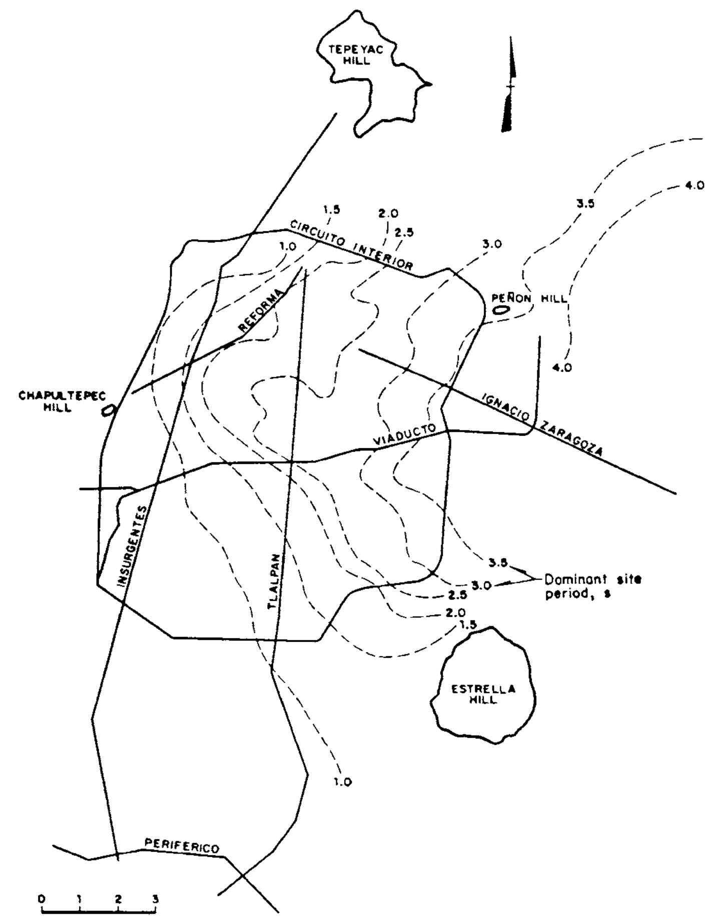

Fig 10. Dominant period contours in Mexico City (after Lermo *et al*, 1988)

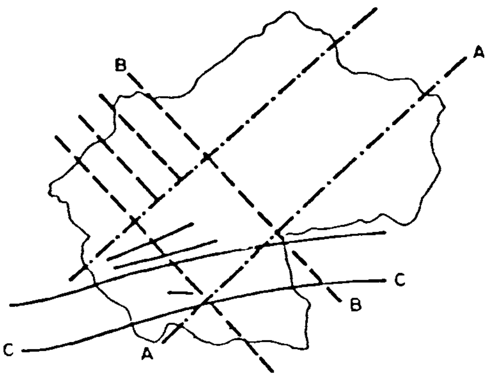

Fig 11. The Valley of Mexico and its fault systems (after Mooser and Montiel, 1990)

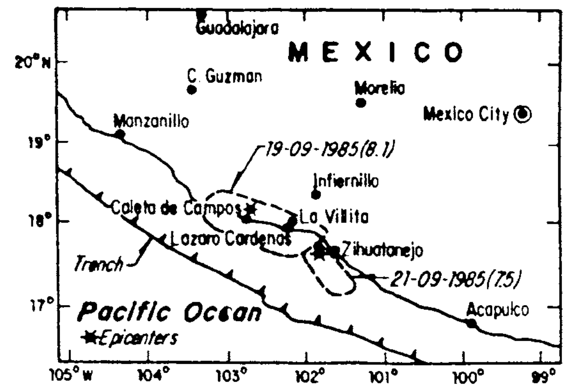

Fig 12. Epicenter for the September, 1985 earthquakes and areas of rupture in the subduction zone (after Meli and Ávila 1989)

GROUND MOTIONS IN 1985

At 7:19 local (Central Standard) time of September 19, 1985, an earthquake of magnitude M = 8.1* originated at a rupture area of about 50 x 170 km under the boundary between the states of Michoacán and Guerrero, marked by the outlet of the Balsas River into the Pacific (fig 12). The depth of the rupture area was about 16 km. The cause was the subduction of the Cocos Plate under the North American Plate.

Accelerographs were being deployed near the coast, mostly in the neighborhood of what was until then the Michoacán Gap and in that of the Guerrero Gap, as well as between the coast and Mexico City; 17 of these instruments produced records. In the Valley of Mexico, records were obtained from ten instruments (fig 1). The most striking feature of the records is the gigantic amplification of earth waves in the Valley of Mexico. Fig 13 displays the peak ground acceleration as a function of distance from the epicenter. Even at University City (CU in fig 1), on what is classified as firm ground for code purposes, there is a several fold amplification. The effect is spectacular on the soft clay.

This picture stands in contrast with the one of the Chilean earthquake of March 3 in the same year. This too was a subduction earthquake with magnitude comparable to that of the Michoacán event. In Chile the peak accelerations in the epicentral region were two to three times greater than in Mexico and the decline with distance was steady and considerably more pronounced (Anderson *et al*, 1986), as may be seen in fig 13.

The E-W componenets of representative accelerograms of the Michoacán earthquake are depicted in fig 14.

* All magnitudes quoted in this paper are surface wave magnitudes, M_s.

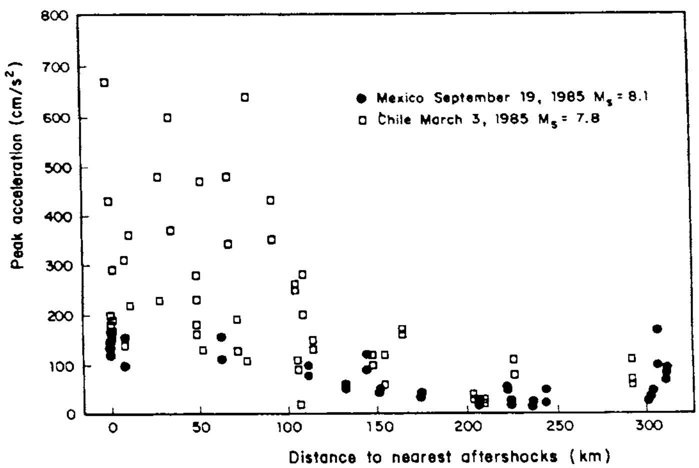

Fig 13. Peak ground accelerations as a function of distance for the 1985 Mexico and Chile earthquakes (after Anderson *et al*, 1986)

Local amplification was very selective in terms of frequency. This is evident when comparing Fourier spectral amplitudes. At University City, for a frequency of 0.5 hz the amplification is 10 relative to the value at a rocky site near the epicenter. At one station within the valley (SCT) the amplification at the same frequency is 10 relative to the corresponding Fourier spectral amplitude at University City and about 75 relative to coastal sites at equal epicentral distance (Singh *et al*, 1988). Fourier amplitude ratios observed in and around Mexico City between 0.2 and 0.5 hz are surprising since such high values are not expected from predictions derived from scaling laws of earthquake source spectra like the ω^{-2} model (Aki, 1972) or from Gusev's model (Gusev, 1983).

The amplifications, although still enormous, are not so pronounced when comparing response spectra for a finite damping ratio. Fig 15 displays the normalized acceleration response spectra with 5 percent damping for the E-W components of ground motion at several stations in the valley. The highest spectral ordinate found occurs at a period of 2.0 s in the EW component of the SCT record (fig 14). It is 1.5 times greater than for the NS component. The maximum and minimum values at this station occur in the N60°E and S30°E, respectively; they are 1.17 g and 0.4 g (Ovando and Romo, 1990).

In general, smaller response spectral accelerations were found as one moved away from the part of the city where the prevailing ground period was 2 s.

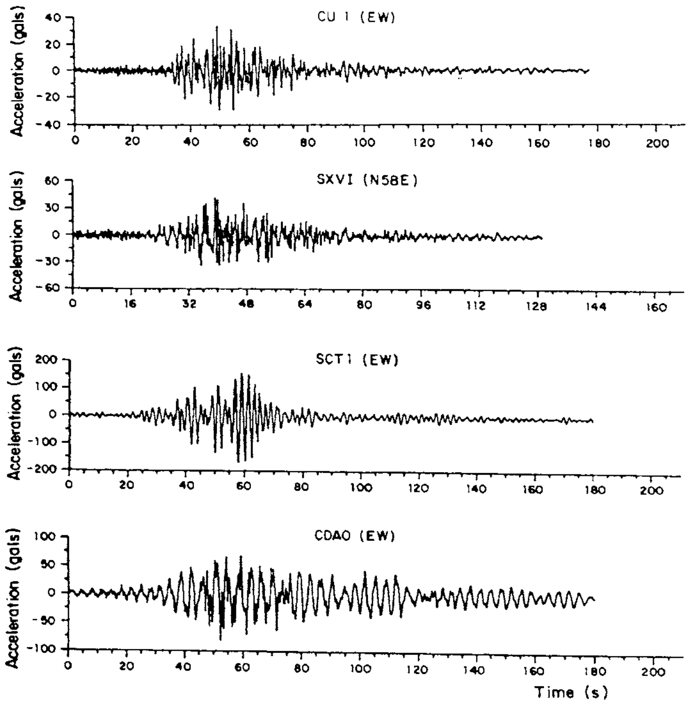

Fig 14. E-W acceleration components recorded in Mexico City. CU lies on volcanic basalt, SXVI is in the transition zone, SCT and CDAO are located in the old lake bed

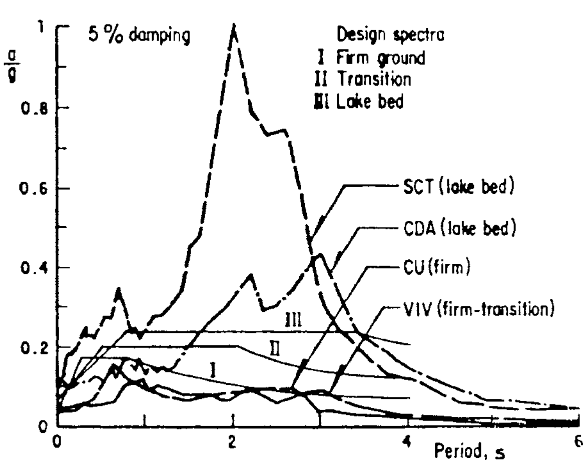

Fig 15. Normalized acceleration spectra for 5 % damping at several stations in the 1985 Mexico earthquake calculated from E-W components (after Meli and Ávila, 1989)

Analysis of near-field, far-field and teleseismic records shows that the earthquake of September 19, 1985 was anomalous in frequency contents relative to other Mexican subduction earthquakes (Houston and Kanamori, 1986; Singh *et al*, 1989). In the near field it was defective in the 0.1 to 0.5 hz range; yet the waves reaching base rock under Mexico City were rich in energy around 0.2 to 0.5 hz (Campillo *et al*, 1989a, b). The anomaly can be attributed to a source effect wtihout invoking any special path or depth of energy release effects (Singh *et al*, 1989).

The other salient characteristic of this earthquake was its exceptionally long duration. At some sites in the city the motion was perceptible for almost 3 min. The reason for this is yet unclear; however, lateral heterogeneities surely contributed substantially.

Damage Extent and Distribution

Although this may overstrain the concept of intensity, we may say that in the Modified Mercalli scale the intensity was IX in zone IV, going down to VI in zone I. This was the most serious natural disaster in the country's history, both as regards material losses (estimated at 14 billion of 1985 dollars including direct and indirect economic losses) and in what concerns loss of life (between 5000 and 20,000 victims, say 10 000).

An extensive survey of damage undergone by buildings up to four stories tall due to the earthquakes of September 19 (M = 8.1) and September 20 (M = 7.6) and aftershocks was conducted by Iglesias *et al* (1987). A damage index served for the purpose and this was combined with the structural capacity computed using a very simplified but calibrated method, to give an approximation to the base shear coefficients that must have acted (since cumulative damage was not explicitly considered, we may refer to those results as "effective base shear coefficients"), which allowed drawing the kind of isoseismals shown in fig 16. This supplied the basis for defining the contour of zone IV. In general terms this zone coincides with the area where one would expect surface waves to be significant owing to horizontal irregularities, such as the vicinity of rock outcrops and of the old lake shores.

The isoseismals disclose the existence of a low-damage area within zone III. This area essentially coincides with the old town center. Here stood heavy pre-Columbian and colonial edifices which, over the last 600 years, have induced the consolidation of the soft clays (at one location the archaeological fill reaches a thickness of 20 m). During the present century the consolidation process has been dramatically enhanced by substantial drops in piezometric levels due to extensive exploitation of the underlying aquifers (*e g*, Marsal and Mazari, 1957; Carrillo, 1969; Mazari *et al*, 1985). It is then not surprising that the underlying, heavily consolidated clay should have shaken less than the surrounding soil. Changes in the clay dynamic properties brought about by increasing effective stresses also contributed to reduce damage over this area, coupled with the fact that many colonial buildings, stout and rigid, must have been less susceptible to suffer from this earthquake, given its frequency content.

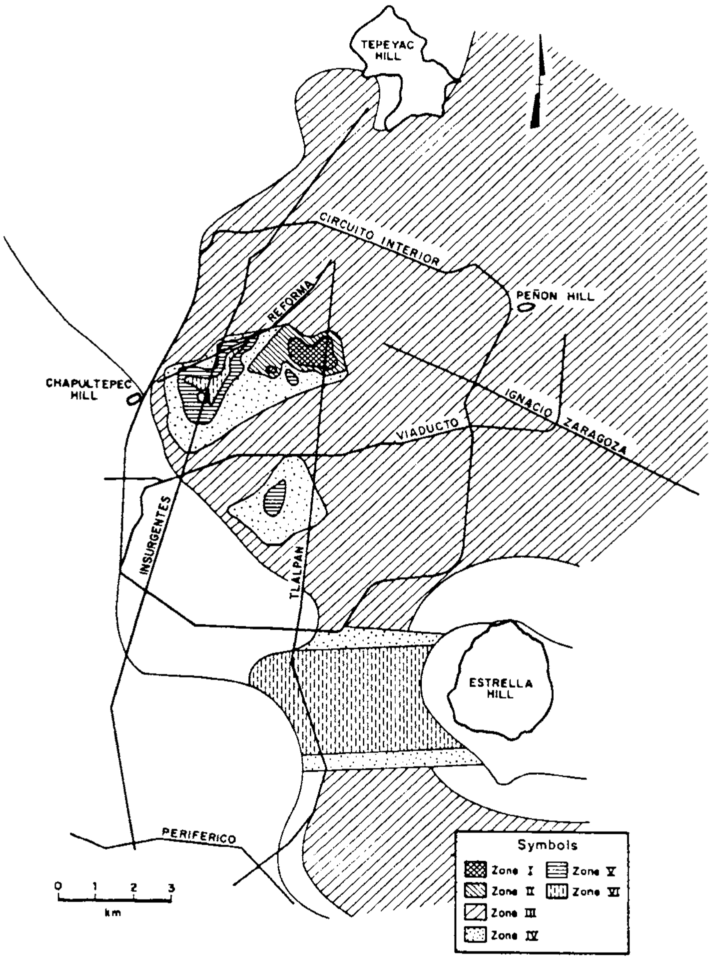

Fig 16. Isoseismals derived from a damage index obtained by Iglesias (1987)

Nature of the Damage

Several structural features stand out as correlated with the maximum percentage of collapses and of buildings with severe damage (many of which had to be demolished months or a few years after the earthquake) (Meli and Ávila, 1989).

Foremost among these features signals out buildings 7-15 stories tall. Measurements of the fundamental period of vibration of a number of buildings in Mexico City indicate that for small oscillations this period, in seconds, is about 0.12 times the number of stories (Kobayashi *et al*, 1986) for buildings between 5 and 25 stories on soft ground. Coefficient 0.12 is higher than the one found in most other countries in seismic areas. In part this can be ascribed to design practices, in part to soil-structure interaction, which should be expected to be pronounced on Mexico City's soft clay, and in a small part to the lower moduli of elasticity of concrete made with the volcanic aggregates that prevail in the valley. The initial periods of the most vulnerable 7-15 story buildings must have been about 0.8-1.8 s, frankly shorter that the 2 s prevailing period in the area of maximum damage (figs 14 and 15), but must have lengthened appreciably as the amplitude of responses increased during the main earthquake.

A clear picture of the effect of building height stands out when considering "spectra" for nonlinear behavior —in the sense of plots of maximum response acceleration of nonlinear single-degree systems as a function of initial natural period.— Fig 17 shows acceleration spectra for 5 percent damping and different forms of elasto-plastic behavior for a ductility factor of 4. Although the structures of interest are not single-degree elastoplastic systems, the major effect of nonlinearity is apparent in these curves. It may be surprising that according to these curves, the most vulnerable systems have natural periods between zero and, say, 1.8 s, not only between 0.8 and 1.8 s. The low actual vulnerability for periods smaller than 0.8 s must be ascribed to the phenomenon of "overstrength" (as called by V V Bertero) —the fact that the capacity of some structures is markedly underestimated by conventional methods of analysis.— This is particularly true of short buildings whose lateral forces are mostly taken by masonry bearing walls, even when unreinforced. Indeed, there was a large percentage of 2-4 story school buildings not protected by such walls, that were seriously damaged in 1985. A healthy change in analysis and design practice would recognize the high accelerations to be expected in the short-period range and at the same time evaluate realistically the capacity of short buildings having properly distributed and detailed bearing walls.

The second most important contributor to vulnerability was torsion. A strikingly large percentage of corner buildings and of other structures having noticeable asymmetries either collapsed or were seriously damaged. This has led to a revision of design criteria and an awakening of the conscience that there can be torsions due to asymmetric behavior in the nonlinear range even in buildings that behave symmetrically in the linear range.

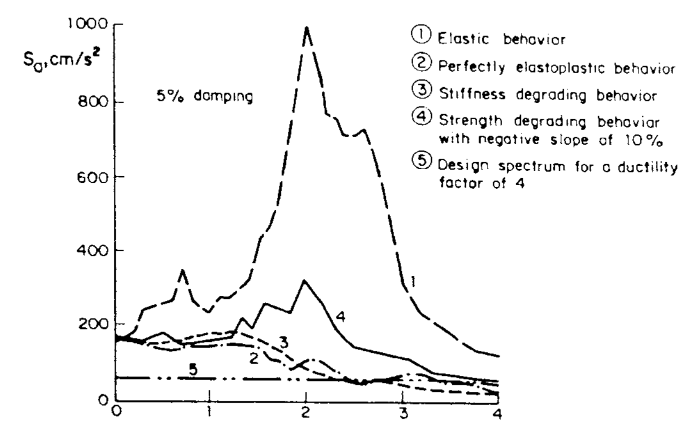

Fig 17. Inelastic acceleration spectra of the SCT-W record for a ductility factor of 4 (after Meli and Ávila, 1989)

The third most conspicuous cause was the use of improperly designed waffle slabs. Mexican building codes prior to 1985 were defective in requirements concerning the amount of slab steel to go through or close to columns; also in development lengths required in column longitudinal reinforcement and in evaluation of stiffness under lateral loading.

Several other contributors were also identified: insufficient development lengths of girder reinforcement at intersections with columns, especially in the upper stories; weak first stories; overload, especially of the upper floors, and sundry defects or omissions in design, construction or maintenance.

Age had little to do with earthquake damage for structures erected during the present century, although buildings erected after the 1976 code performed in general somewhat better than those built between 1900 and 1975 (Meli *et al*, 1986). Earlier structures, notably the colonial buildings, were practically spared.

Only in 13 % of the cases of collapse or severe damage was inadequate behavior of foundations detected. The two salient causes of foundation trouble were excessive and often uneven settlement of surface or partially compensated foundations, and insufficient capacity of friction piles. The former cause could in all cases be traced to noncompliance with established criteria: if a surface foundation or a partially compensated one had been properly designed in accordance with the code requirements in force between 1957 and 1985, then no foundation difficulties were to be expected (Mendoza and Auvinet, 1988). Among the most conspicuous cases of failures at least two stand out, in which improperly designed and distributed friction piles caused the overturning of slender buildings. There were also cases in which buildings settled massively (in a particular case more than one meter) without the structure suffering any apparent damage. The actual mechanism leading to this kind of failure is not fully known but energy dissipated due to overall nonlinear deformations in the surrounding soil may have triggered it, as noted previously for Bucarest by Sandi (1989). Friction pile behavior was alarming nevertheless and led to designing an experimental project in the field. Pending conclusions therefrom, a clause was introduced in the 1987 building code, to the effect that friction pile capacity be computed as

$$C_f = A_L f F_R \qquad (1)$$

where C_f is the frictional capacity, f the lateral adhesion and F_R a load factor. F_R, which is smaller than one, is meant to account for a possible degradation of lateral resistance due to cyclic loading.

In verifying the adequacy of design, the capacity is to be compared with the sum of the static load acting on a pile, Q_s, and the maximum axial load increment due to seismic overturning moments, ΔQ_d:

$$Q_s + \Delta Q_d < C_f \qquad (2)$$

Q_s and ΔQ_d are affected by load factors greater than unity.

The field study (Jaime *et al*, 1988) consisted in monitoring the behavior of piles under static and alternating vertical forces. It was found that the adhesion between pile and surrounding clay could be taken equal to the cohesion, c, provided that C_f was not significantly smaller than $Q_s+\Delta Q_d$. Conservatively one could say that the design adhesion could be taken as 0.7c. This is equivalent to adopting eq 1 under the (perhaps too conservative) assumption that ΔQ_d may reach twice its computed value, due to structural overstrength. Despite these results which prompted a suggestion for a code modification, the behavior of individual piles, pile groups and pile-raft systems during earthquakes is not yet sufficiently understood. Further field tests and monitoring of actual soil-foundation systems should therefore be contemplated in future research projects.

Seven thousand leaks were detected in the city's water supply system. The sewerage, however, was almost unharmed. The difference in behavior bespeaks of rigid joints in the supply system *vs* flexible joints in the drainage system. What damage there was can only be explained in terms of surface waves. The distribution of this damage is, throughout the city, in very broad lines in accord with that of damage in buildings.

Another clear manifestation of surface waves is the testimony of numerous trustworthy witnesses: they saw (Rayleigh) waves in the street pavement at areas where the most severe damages occurred in buildings. The amplitude of these waves was consistent with the recorded vertical ground motion, about 8 cm. Some waves seemed to have "frozen" and apparently were visible for some weeks after the earthquake.

One phenomenon not well understood is the cracking of the soil and, at some sites, near the airport for example, the development of grabens in the soft clay. These cracks, however, could have existed before the earthquake, in which case their formation should be associated with the subsidence of the periphery of the old lake bed, where the basal topography changes abruptly (Auvinet, 1981), with the desication of the upper crust of the clay layers (Carrillo, 1948) or with hydraulic fracturing (Alberro *et al*, 1990).

Sudden differential settlements were observed in the outskirts of the city. It was suggested that liquefaction of the sandy soils which separate some 30 to 35 m the two main clay satrata might have caused them. However, a laboratory study into the behavior of these soils under cyclic loading showed that they are highly dilatant (Ovando *et al*, 1988). The causes of these settlements are yet unknown.

In contrast, widespread liquefaction of loose deltaic and beach sand deposits causing foundation failures and severe structural damage was reported in the epicentral region (Santoyo and Gutiérrez, 1987; Marsal, 1987). Damage was especially severe at Lázaro Cárdenas, some 60 km from the epicenter, where lateral spreading induced by the liquefaction of young deltaic sands disrupted many roads and highways. Sand boils, ground cracking and strong distortions in railways were also observed. Most foundation failures within a large neighboring steel mill can be attributed to sand liquefaction as well. Liquefaction had been identified as a hazard in a field and laboratory study carried out well before the 1985 earthquake (Jaime *et al*, 1979).

The subway system suffered minor damage requiring no more than realignement of the tracks. Some analyses of soil-tunnel interaction (Romo, 1988) indicate that one should not expect significant increase in the amplitude of ground vibration in the neighborhood of the subway tunnels. The apparent correlation of the severity of damage and closeness to subway lines must be attributed to the fact that the lines run under important thoroughfares on whose sides lie many medium height —and hence vulnerable— structures.

The electric power system suffered little. Power was interrupted in some quarters for a few hours or days. The good performance of porcelain insulators and other components is probably related to the concentration of seismic energy in the range of long periods of vibration.

Long-distance telephone communication was interrupted for months due to failure of a single set of switchboards. This evinced the importance of providing redundancy in life lines.

Two major earth dams are located near the epicentral area: La Villita and El Infiernillo, 60 and 146 m tall, 60 and 68 km away from the epicenter, respectively. Even though the overall seismic behavior of both dams has been good, longitudinal cracks, probably resulting from differential settlements at the core-backfill interfaces, were noted after the 1985 earthquakes. El Infierrnillo has suffered vertical settlements of 1.65 m (0.2 % of its height) since it was completed in 1965, out of which 12.7 cm were recorded immediately after the 1985 main events; 66% of the settlement has concentrated in the upper 60 m of the dam (Romo and Villarraga, 1989). La Villita underwent a maximum acceleration of 0.7 g tranversely. The maximum vertical settlement produced by the main shock and the ensuing aftershocks was 31.8 cm whereas horizontal displacements of about 15 cm were recorded. Linear relationships between the observed settlements and horizontal movements and the logarithm of measured crest accelerations were disclosed by González-Valencia (1986) for $M > 5.0$ earthquakes that occurred during the 1975-1985 period. Careful inspections carried out in La Villita also revealed that longitudinal cracking affected only the road base that runs along the crest.

OTHER EARTHQUAKES IN THE VALLEY OF MEXICO

There is historic evidence of strong earthquakes in the valley dating back at least to the late fifteenth century (García *et al*, 1988). Ever since the mid 16th century the Spanish settlers were concerned with the effects of earthquakes on the buildings that replaced the massive Aztec temples. Cervantes de Salazar (1550), for example, gives the first known "recipe" for earthquake resistant design of buildings in the Americas: to limit their height to no more than four stories. In several instances this sort of empirism proved to be of little avail since past earthquakes caused the partial collapse of some colonial buildings. Such was the case in 1845 and 1858, for example. Information about colonial structures is useful in seismicity assessment but not in deriving lessons about the nature of local ground motion or its effects on structures, owing to the absence of quantitative data and to the changes in the extent of urbanization and in architectural styles.

The earthquake of July 28, 1957 was the first Mexican macroseism to affect a substantial number of modern buildings covering a wide range of heights and structural solutions. This was a subduction earthquake of $M = 7.7$ originating East of Acapulco. That area could rupture again when the Guerrero gap closes to the West of the port, jointly producing an earthquake of magnitude 8.2 or 8.3 (Singh *et al*, 1982). The damage pattern was similar to that in 1985; the areas of maximum intensity overlapped considerably although they did not exactly coincide (fig 18). Differences were surely due to changes in the extent of urbanization and in the frequency contents of the two events, although the earlier one seems to have been nearly as anomalous as that of 1985 (Singh, 1987). The total economic losses were of the order of one tenth of those in 1985. The number of victims was about 60 in Mexico City. From the interstory drifts recorded in the 43-story Latino Americana Tower and from its dynamic characteristics it was concluded that at that site the peak ground acceleration must have been about 0.06 g. One intriguing difference in damage patterns in the two earthquakes: it was mostly beams in 1957 and mostly columns in 1985 that failed. This must have been due in part to changes in design criteria introduced in the Emergency Regulations of 1957, which called for a more conservative design of beams against diagonal tension and in many instances a less conservative design of columns under eccentric compression. In part also to the longer duration of the 1985 disturbance —about twice that in 1957,— for this meant greater vulnerability of structural members whose strength degrades under successive cycles, which is the case of tied columns but not of beams failing in tension under flexure (practically no helically reinforced columns failed in either earthquake).

The first accelerograms in the Valley of Mexico were obtained in the early 60s. As we shall see, these served to calibrate analytical calculations of site effects, which had been initiated much earlier. However, there were no stations common to both the earlier earthquakes and that of 1985. There is little, therefore, that can be concluded from comparing their records. Things are different since early 1988,

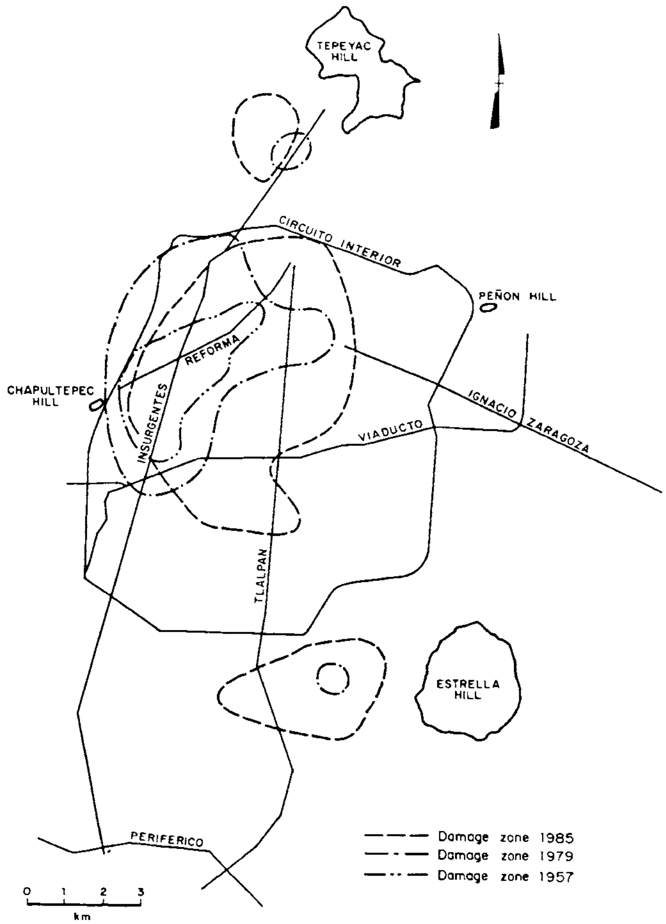

Fig 18. Damage zones in Mexico City for three large earthquakes (after Iglesias *et al*, 1988)

when most of the 80 instruments that now constitute the valley network (Espinosa *et al*, 1988; Otero *et al*, 1988) have been operating.

By comparing the Fourier spectra of these records and those of 1985 we conclude that in general the transfer functions between pairs of stations are, for distant earthquakes, mostly independent of the detailed focal mechanism, focal coordinates and magnitude (Singh *et al*, 1988a), excluding widespread nonlinear soil behavior. In this context we understand by *transfer function* from station A to station B the ratio of the smoothed Fourier amplitude spectrum at B to that at A as a function of period. This result allows reconstruction of the Fourier spectra of 1985 and, by resorting to random vibration theory, also of the response spectra. Fig 19 shows some comparisons of transfer functions (Singh *et al*, 1988a). Only two significant counterexamples have been found. One concerns station CDAO, where the actual acceleration response spectrum for 5 % damping is shifted toward longer periods and has smaller ordinates than the one inferred from the earthquake of February 8, 1988. We conclude that at CDAO, nonlinear soil behavior took place, probably within a clay layer some 8 to 10 m deep where the soil is particularly weak and has water contents in excess of 250 % (Ovando *et al*,

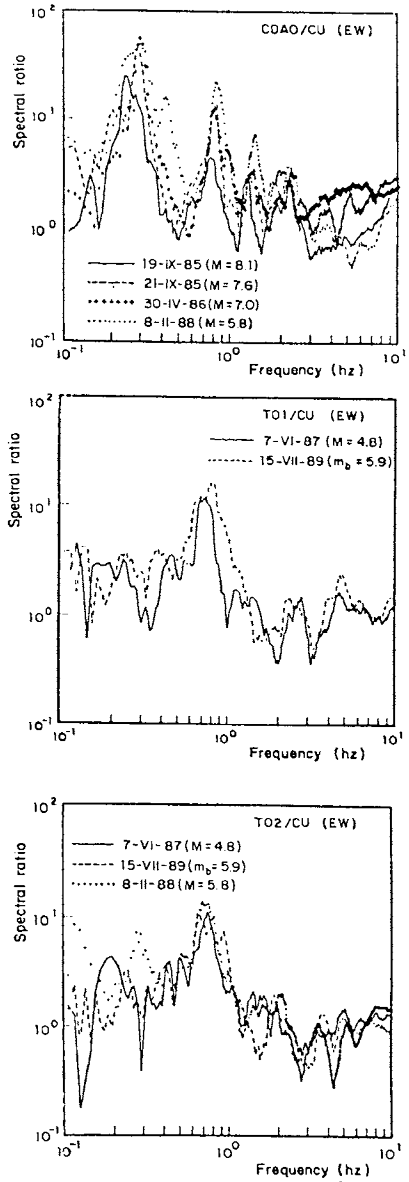

Fig 19. Transfer fucntion of different stations with respect to a site on firm ground (CU), (after Singh *et al*, 1989a)

1989). Similar results should be expected at sites having comparable soil properties. The second exceptional site lies on the lake bed near the Estrella Hill, roughly in the direction of the nearby transition zone (fig 1). Between the hill and the transition zone there lies the ancient gorge that used to connect the two sub-basins of the Valley of Mexico, associated with lake Texcoco to the North and Lakes Xochimilco and Chalco to the South. Here records seem to point to an important local azimuthal dependence (Ordaz *et al*, 1989).

The peak response spectral accelerations for 5 percent damping in the NS or EW direction —whichever was larger— reconstructed for the 1985 earthquake from the response spectra of the earthquakes recorded between February 8, 1988 and April 25, 1989 are given in fig 20. The values at some sites approximately coincide with results of one-dimensional analysis; at others they are greater, more than twice as great at a few sites.

Design is partly governed by nonsubduction earthquakes. Those originating in the subducted Cocos and Rivera Plates and in the North American Plate away from the valley are expected to have qualitatively similar effects as the subduction earthquakes, with some shift in energy content toward shorter periods, mostly because of the shorter focal distances. The Acambay earthquake of 1912 is an outstanding example of this type of phenomenon. Its magnitude was 7.0 and it originated at one edge of a graben about 70 km Northwest of Mexico City. Local earthquakes excite much higher frequencies. Their maximum known magnitude has been 5.5 and there are no signs of much stronger earthquakes since the fifteenth century. Local earthquake effects have been limited to cracking of masonry walls in one- to three-story houses and each such disturbance has been perceptible in a small area (Rosenblueth *et al*, 1988). Site effects too are very different for local than for distant earthquakes.

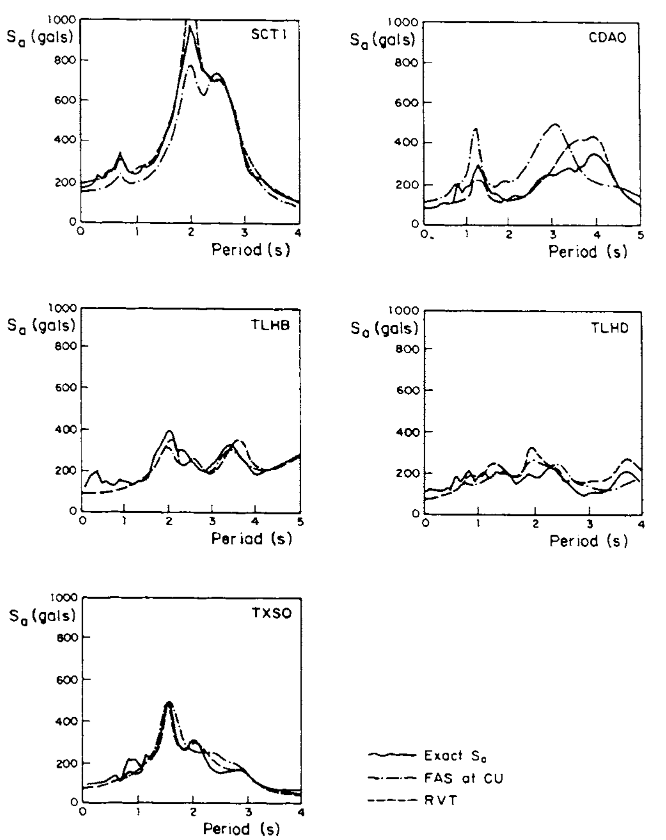

Fig 20. Comparison of exact S_a (continuous line) S_a computed from Fourier amplitude spectra at CU (dotted line) and from random vibration theory (dashed line) (after Ordaz *et al*, 1988)

ANALYSIS OF SITE EFFECTS

Aside from empirical information about the manner in which the Valley of Mexico affects incoming waves, it is important to have analytical results. The first known attempt at calculating site effects in the valley (Rosenblueth, 1952) worked in the time domain. It idealized the subsoil as a series of horizontal strata of linear behavior with internal viscous damping, resting on a homogeneous half-space at a depth of 40 m. The disturbance was idealized as upward traveling SH waves. The model was thus one-dimensional. Computed spectra were substantially correct (although this could not be ascertained until records became available) but did not transcend into engineering practice and the approach was then too time consuming. Years later a frequency-domain method was developed to deal again with the linear one-dimensional model (Herrera and Rosenblueth, 1965). Response spectra computed on this basis using as model input records obtained on firm ground were compared with spectra computed from records at some stations on the soft soil (Herrera *et al*, 1965). The agreement was excellent (see fig 21, for example). On the one hand this success and on the other the lack of practical means for analyzing more realistic models postponed attempts to progress in the analysis of site effects in the valley. Nevertheless, a number of analytical tools were developed. Thomson (1950) and Haskell (1953) were able to deal efficiently with a model that was still one dimensional but allowed consideration of oblique incident waves.

The first comparisons of the 1985-earthquake response spectra with analytical results seemed to favor the one-dimensional linear model with incoming SH waves. Yet several considerations indicated the desirability for exploring the use of more elaborate models. First there was the evidence of important surface waves as discussed above. Results of two-dimensional valleys of smaller dimensions disclosed the relevance of waves other than the SH waves in the simple model (Bard and Bouchon, 1985). Approximate two-dimensional analysis of a portion of the Valley of Mexico (Sánchez-Sesma *et al*, 1988) pointed in the same direction. Then one could not ignore that buildings in some parts of the city had experienced more severe damage, implying a more violent motion, than those where instruments had been installed and where to some extent the one-dimensional analysis gave satisfactory results. In addition, the directional effect left little doubt as to the need to improve the model. Indeed, the peak ground acceleration in the NS direction at University City was slightly greater than in the EW direction, and yet at SCT the peak ground acceleration in the EW direction was 2.5 times greater than in the NS direction. One could argue that the smoothed calculated spectrum using the one-dimensional model coincided reasonably well with the average of the EW and NS spectra at SCT (fig 22) but the discrepancy of 2.5 to 1 could not be washed away. Finally, working in the time domain, even though one-dimensional analyses were capable of reproducing the duration of the observed movements, the large, nearly sinusoidal oscillations with the appearance of beating that were observed in most records on soft soil long after the portion of strong shaking had subsided

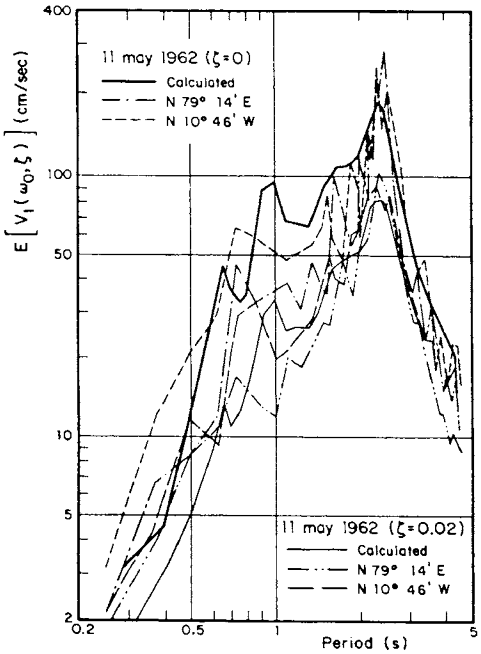

Fig 21. Early attempts to predict ground motion in Mexico City (after Herrera *et al*, 1985)

were not properly reproduced (fig 14). This feature of ground motion was not apparent in moderately damped response spectra because the maximum responses were generally attained near the end of the strong-motion segment of the record. For the same reason the coda is unimportant for the design of structures whose strength does not appreciable degrade under repeated loading, but it can be decisive for degrading structures, such as those whose capacity is conditioned by the strength of steel members in compression, by that of unconfined concrete, unreinforced masonry and, in some cases, by reinforced masonry as well, or by bond between reinforcing bars and concrete. For such structures the information furnished by response spectra is insufficient, and hence so are results of one-dimensional site-effect analyses.

In short, one-dimensional models apply throughout a large portion of the Valley of Mexico provided that we use the results for designing nondegrading structures sensitive only to horizontal displacements of the ground. Such models are inadequate for all types of structure in the rest of the valley and, for degrading structures and for those sensitive to ground rotations or to soil strain, they are inadequate in the entire valley.

For large earthquakes the double integration of vertical accelerograms gives an interesting pattern of the histories of vertical ground displacements. Their shapes as functions of time appropriately shifted very little from seismic station to seismic station (fig 23). The maximum such displacement was about 5 cm in 1985. The apparent horizontal wavespeed wihin the valley of Mexico was close to 1.5 km/s (Campillo *et al*, 1989b).

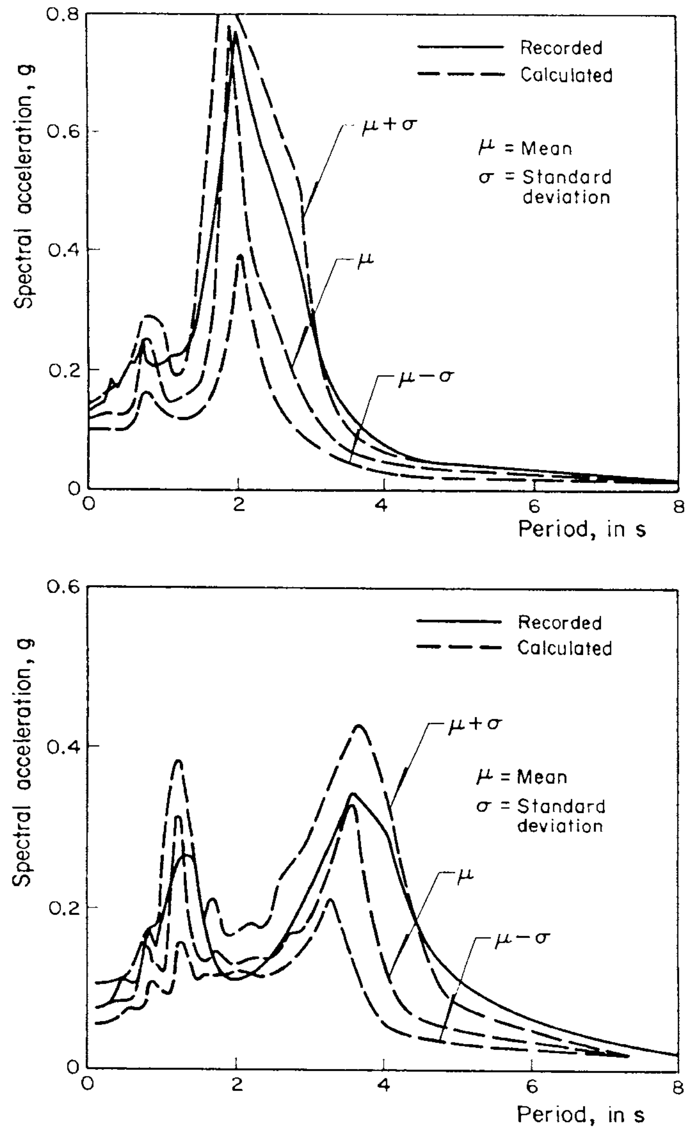

Fig 22. Calculated smoothed response spectra versus average spectra obtained from actual E-W acceleration records in 1985. Top: SCT; bottom: CDAO (after Romo *et al*, 1988)

We can only surmise that these must be the surface manifestations of Rayleigh or Lg waves. Since their prevailing period is about 10 s, they are associated with important ground rotations about horizontal axes and with significant distortion of water mains, both for water supply and for drainage. The rotations can be decisive for the response of tall slender structures such as chimneys. These waves can probably be adequately modeled using well-developed analytical solutions although there is some room for doubt concerning gravity effects on these motions. Apparently no studies exist of such effects in stratified soils.

A three-dimensional model of the valley using finite elements sufficiently small to ensure trustworthy results down to 0.5 s periods would require on the order of 3 x 10^9 elements. Its analysis is beyond the scope of present supercomputers.

One could reasonably hope for a number of two-dimensional models using less than five million finite elements each. This type of analysis should be attempted. Meantime various very efficient methods have been and are being developed which can deal with some two-dimensional models in about one-hundredth of the computer time demanded by other extant approaches (Velázquez and Sánchez-Sesma, 1987; Sánchez-Sesma *et al*, 1988). These have already produced synthetic accelerograms with the right shape and size of the coda. On such bases it is likely that the pattern of peak spectral accelerations displayed in fig 22 as well as other relevant features of ground motions in the valley will soon be approximately accounted for, with substantial improvements over the results of one dimensional analysis.

Nonlinear soil behavior has long been dealt with in one-dimensional analyses (Schnabel *et al*, 1972). The usual approach, based on the work of Seed and Idriss (1969), uses equivalent linear soil systems whose stiffnesses and damping ratios are found through iteration so that they correspond to the hysteretic curves at 65 percent of the maximum angular deformation. Recent comparisons of this method *vs* virtually exact step-by-step nonlinear analyses of the response of Mexico City clay indicate that this form of equivalent linearization will furnish good approximations provided one uses sufficiently small damping ratios for low-amplitude strains (Pérez-Rocha, 1990). This implies drifting away from the Ramberg-Osgood skeleton stress-strain curves and adopting a more nearly linear relation at low strains. The conclusion is apparently valid for all clays having high plasticity indeces, not only for those in Mexico City.

In most instances with which there has been recent experience the dimensions of the valley have been modest. Therefore interpretation of the seismic effects and of the records obtained can be done without imposing such a formidable numerical chore as for Mexico City. The number of finite elements required by two-dimensional analyses has been manageable.

Topographic Effects

The relationship between topographic effects and damage in Mexico City is still unclear but evidence elsewhere tends to indicate that orographic irregularities do have an influence on it. Observations made after the 1909 earthquake of Lambsec, France (Angot, 1909), the 1980 Italian earthquakes of Friuli (Brambati *et al*, 1980) and Iripina (Siro, 1982), and the great Chilean earthquake of 1985 (Celebi and Hanks, 1986) show that structures built on hilltops suffered more damage than those located at the base of the hills. The experiments of, *inter alia*, Tucker *et al* (1984) have confirmed that superficial topography can play an important role on seismic amplification.

Nevertheless, the influence of topographic irregularities on seismic response is far from

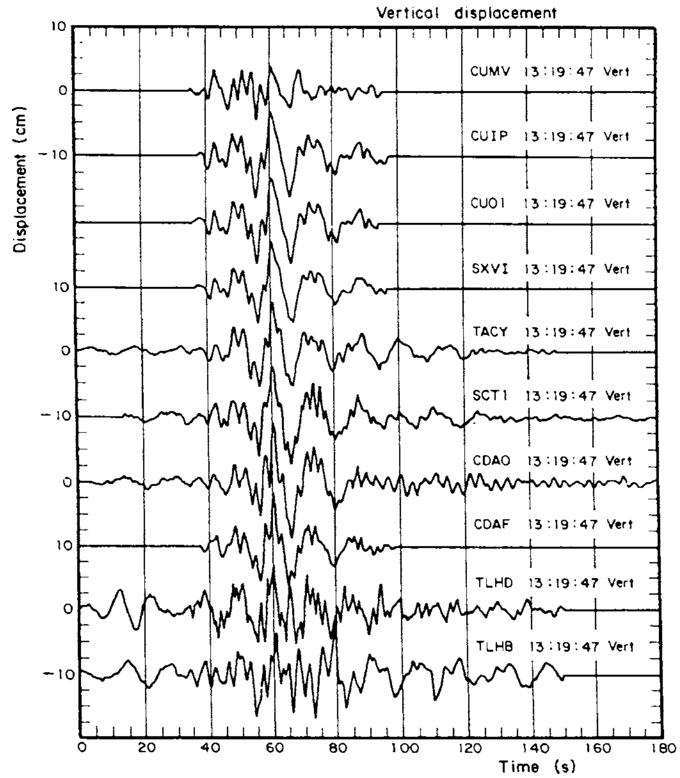

Fig 23. Vertical displacements in Mexico City during the September 19, 1985. For location of sites see fig 1 (after Campillo *et al*, 1986)

being fully understood. Work on this subject is in progress. A variety of numerical techniques are now available for studying the effect of a topographic irregularity resting on a homogeneous elastic semispace: finite differences, finite elements, integral equation methods, boundary element methods and discrete wave number menthods. In connection with the seismic response of the Valley of Mexico, approximate anlytical tools that solve the problem of diffracting P, SV and Rayleigh waves due to surficial irregularities have been put forth (Sánchez-Sesma *et al*, 1982; Bravo and Sánchez-Sesma, 1989). Recently, Kawase and Aki (1990) have applied a two-dimensional model for explaining the damage pattern observed in Whittier Narrows, California, after the earthquake of October 1, 1987.

OTHER RECENT EARTHQUAKES

The San Salvador Earthquake of October 10, 1986

San Salvador has been periodically struck by shallow earthquakes that originate along a structural depression coincident with much of the active volcanic chain extending from Guatemala. Volcanic eruptions have sometimes accompanied earthquakes, but by far the majority of the significant ones have had a tectonic origin (Faccioli *et al*, 1989). Observational evidence after the main shock indicates that the earthquake of October 10, 1986 originated along a fault subparallel to the Central American volcanic chain (White *et al*, 1987). According to Faccioli *et al* (1989), this earthquake had a left-lateral strike-slip mechanism with a preferred rupture plane oriented N32°E. They also contend that the reported magnitude of M = 5.4 is not consistent with the severity of damage. Eleven strong motion accelerographs located at epicentral distances ranging from 1 to 6 km had been installed at the time of the earthquake. Ground level accelerations recorded in six of them varied between 0.32 and 0.72 g. Representative accelerograms and response spectra are shown in Figs 24 and 25 (Shakal *et al*, 1987).

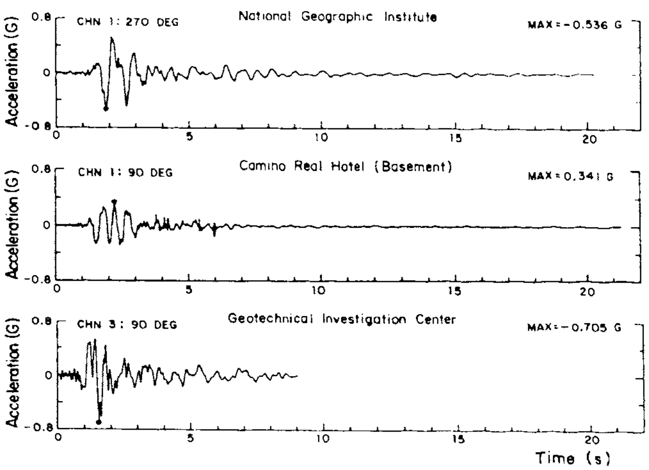

Fig 24. Accelerations recorded during the 1987 San Salvador earthquake (after Shakal *et al*, 1987)

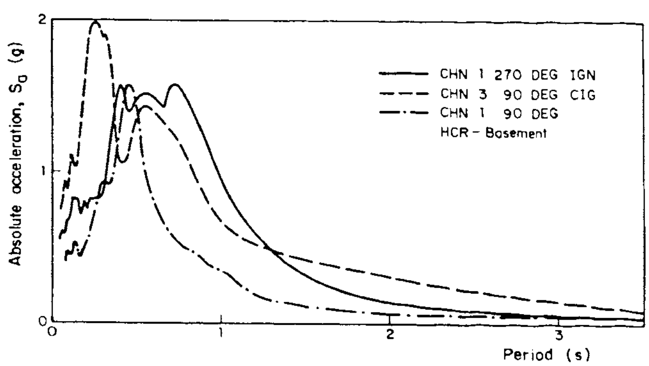

Fig 25. Response spectra for the 1987 San Salvador earthquake (data taken from Shakal *et al*, 1987)

The geological and geotechnical environment in the San Salvador area has been thoroughly described in a study for the seismic microzoning of the city which was carried out after the 1986 earthquake (Faccioli *et al*, 1989). The geologic aspects and geotechnical effects are discussed, respectively, by Rymer (1987) and Chieruzzi (1987). The paragraphs that follow are based on their work.

Cracks and fissures opened in the ground near the epicentral zone but none could be ascribed to new faulting. In San Salvador and environs the stratigraphy is formed by alternating volcanic and volcaniclastic rock sequences. A large part of the city is underlain by two pyroclastic and epiclastic tuff units. The older one, the toba café (brown tuff), may be as thick as 25 m whereas the young unit, the tierra blanca (white tuff), varies in thickness from 25 m in central San Salvador to 50 m near Lake Ilopango. Both are poorly consolidated, and steep stream and river banks have formed through them. The main water table below the city is about 50 to 150 m deep although local perched water tables can also be found.

Most of the damage concentrated in a relatively flat plateau between the San Salvador Volcano and Mt San Jacinto (fig 26). Local geotechnical and geologic conditions played an important role in the amount and distribution of damage. Foremost among these are the conditions of the two tuff units described above. In San Salvador, landslides were frequent in slopes along steep natural ravines cutting the tuffs. At the time of the slides, many dwellings constructed near the edges of the gorges were completely destroyed; others, at the base of the slopes were also damaged. In modified slopes, some cases of failures in earth retaining structures too were reported. Landslides also disrupted highways along cuts made mainly into the tierra blanca unit. Soil falls and slides, slumps and rapid soil flows were noted over an area in and around San Salvador of at least 200 km^2. No liquefaction was detected in San Salvador —the tuff units are partly saturated or dry during most of the year— nor along Lake Ilopango where

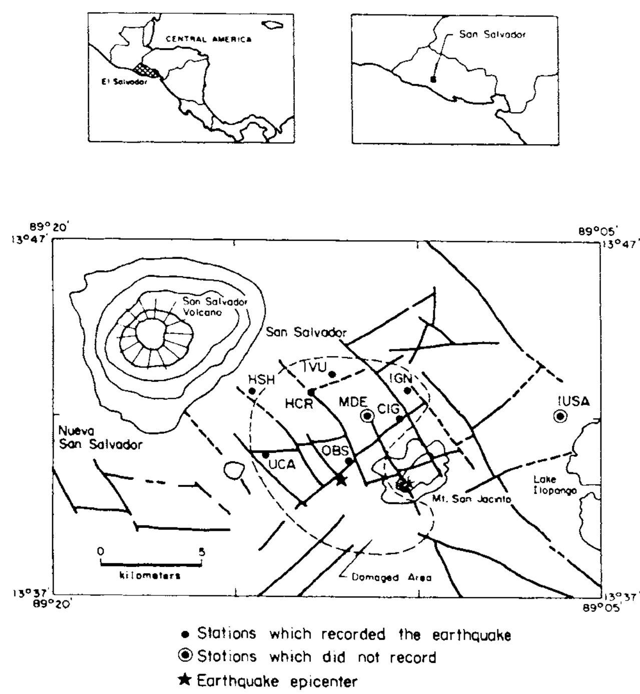

Fig 26. Damage zone, epicenter and strong motion recording stations for the San Salvador 1987 earthquake (after Shakal *et al*, 1987)

it occurred in previous earthquakes. Some artificially filled gorges and ravines did suffer from differential compaction due to the earthquake. As a result, structural damage in buildings ensued.

Even though no foundation failures have been reported in the literature known to the authors, such failures cannot be ruled out. The adequacy of general design and building practice, however, has been questioned (Lara, 1987). Building codes have, until very recently, been based on those drafted in other countries. Following the earthquake an emergency building code was published and a thorough microzoning study for the San Salvador area performed (Faccioli *et al*, 1989).

The Armenia Earthquake of 7 December 1988

The earthquake of December 7, 1988 caused extensive damage throughout the Soviet Republic of Armenia. Cities like Spitak, Leninakan, Kirovakan, Stepanovakan and Kalinino having large concentrations of people, businesses and industries were among the most severely struck. Human casualties amounted to more than 25 000 and the whole economic activity within the affected area was seriously impaired weeks and months after the earthquake (Velkov, 1989; EQE, 1989). The epicenter of the main event ($M = 6.9$) was located some 75 km Northwest of Yerevan, the Armenian capital. Accelerometers had not been deployed within the zone of major damage; accelerograms recorded in Yerevan yielded a maximum acceleration of 0.07 g. From seismoscope records taken in Leninakan and from indirect evidence, peak accelerations in the epicentral region have been estimated as ranging between 0.5 and almost 1.0 g.

Field investigation teams that visited Armenia after the disaster coincide in signaling the collapse of modern engineered structures as the main contributor to the enormous toll in human life. Some of the most spectacular failures occurred in buildings having precast concrete frames and shear walls. Other structures having essentially unreinforced stone bearing walls also performed badly (Wyllie, 1989). Soviet engineers did consider seismic forces in the design but these were grossly underestimated simply because an event of such a magnitude had not been expected. Poorly defined regions in seimic microzoning maps also contributed to underestimations of the seismic design forces.

Other factors too influenced the magnitude of the disaster. Among them stands the centralization of Soviet building design at government institutions, so standard designs are repeatedly constructed. Modifications to account for seismic forces are done by local or regional authorities. Sound earthquake resistant design must consider seismic forces in the original conception of a building. Poor construction quality was also made evident among the debris of many a collapsed building but the actual influence of this factor on damage has not been assessed.

Correlations between local soil conditions and damage can not be easily established owing to the lack of strong ground motion records in the most severely hit areas. Nonetheless, Borcherdt *et al* (1989) point out that damage percentages for longer period structures (5, 9, 12 stories) show that they underwent greater damage in Leninakan (epicentral distance ≅ 32 km) than in Kirovakan (epicentral distance ≅ 25 km) whereas shorter period buildings (up to 4 stories) suffered similarly in both cities. In the former, which lies on an alluvial valley, destruction tended to concentrate in filled areas along rivers. Sediments (clays, sands and tuff) extending in certain locations down to 200 or 300 m amplified incoming seismic motions, according to theoretical analyses using one-dimensional wave propagation models (Papageorgiou *et al*, 1989, Borcherdt *et al*, 1989). According to these studies, amplification must have peaked at periods near 1 and 2.5 s; spectral maxima at these periods were probably 25 times larger in Leninakan than on rock sites of comparable epicentral distance. Further, the duration of motion could have trebled and displacements increased by a factor of eight. The results of these analyses also highlighted the limitations of one-dimensional models, suggesting that in this case, given the nature of the underground structure, two- and three-dimensional models are required to match theoretical predictions with actual observations.

The Loma Prieta Earthquake of October 17, 1989

On October 17, 1989 an earthquake ($M = 7.1$) originated in the San Andreas Fault near Loma Prieta in the Santa Cruz mountains, some 90 km south of San Francisco, in California (fig 27). The earthquake and the thousands of aftershocks that followed —300 of these having $M \geq 2.5$ and 40 having $M \geq 4.0$— filled a gap along the San Andreas Fault which had been identified a few years before as having a high probability of rupturing within this century (Benuska, 1990). Information released by the USGS (1989) some weeks after the earthquake allows for the following description; the fault rupture zone, which was the most severely damaged, extends for 25 km along the San Andreas Fault. The main rupture occurred at a depth of 17.7 km and is believed to have involved approximately equal amounts of right-lateral and reverse slip. The fault plane strikes N50°W and dips approximately 70° to the southwest. Relative plate motion during the earthquake was 1.9 m along the strike of the fault plane and 1.3 m of reverse lift (geodetic data suggest a broad uplift of about 0.45 m). Surface extensional cracks involving right and left lateral slip were observed along the rupture zone in an area of 4.8 by 8.0 km. Some of these cracks damaged or destroyed homes and disrupted highways.

An extensive net of accelerographs yielded intrumental records of strong ground motion at more than 100 sites within 190 km from the epicenter. Recorded maximum ground accelerations on rock sites were compared with those predicted, as a function of epicentral distance, using Joyner and Boore's attenuation law (USGS, 1989), as shown in fig 28. Despite the observed scatter, predicted values tend to agree with observations even though a slight tendency for underestimating them could also be argued for. Maximum accelerations on young alluvium sites and on bay mud show the effect of local

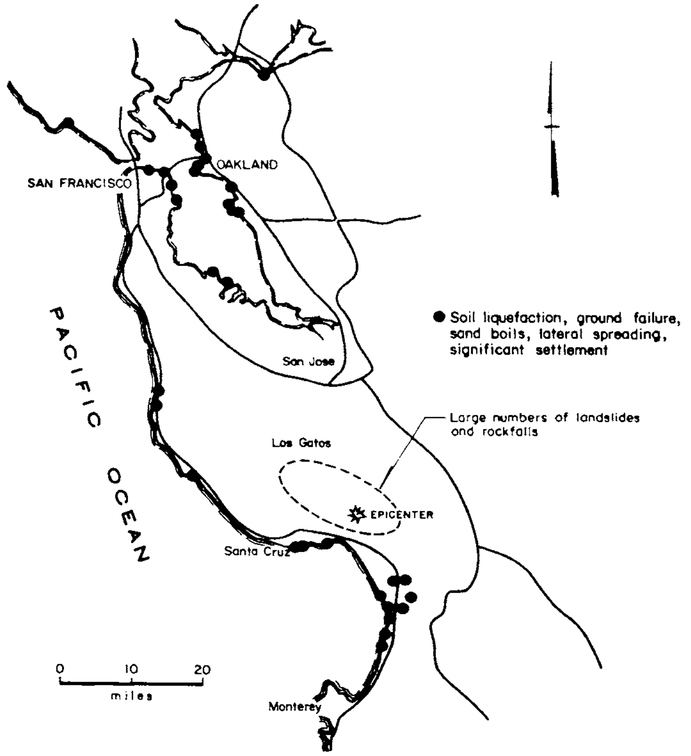

Fig 27. Epicenter and damage map for the Loma Prieta earth- quake (data taken from USGS, 1989)

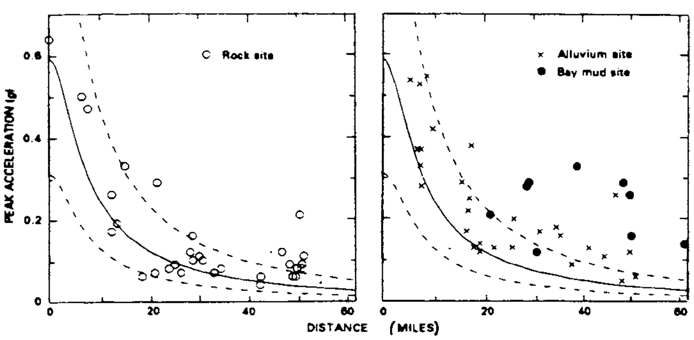

Fig 28. Maximum ground acceleration as a function of epicentral distance. Loma Prieta earthquake (after USGS, 1989)

amplification. Predicted values on these sites are, in general, considerably lower than the actual ones. A clear illustration of the influence of local site conditions during this earthquake is provided by the acceleration time histories recorded at one site on bedrock and at four sites underlain by unconsolidated deposits (fig 29). Deep clay deposits surrounding the San Francisco Bay amplified ground motions considerably. A preliminary assessment of amplification effects showed that in stiff soil sites maximum horizontal acceleration was about 0.1 g whereas in locations having deep clay deposits it was 2.5 times larger (Seed *et al*, 1989). Most of the structural damage in San Francisco and central Oakland occurred on deep clay sites, including the site of the collapsed Cypress St viaduct. About 7% of the bridges in the highway system in the Alameda, San Benito, San francisco, Santa Clara and Santa Cruz Counties suffered some degree of damage. Older bridge structures founded on poor ground and designed with what we would now call substandard methods were the most severely hit. Strong motion instruments located within buildings and in the free field provided invaluable data for studying soil-structure interaction case histories.

Landslides, thousands of them, were observed near the epicentral region in the Santa Cruz Mountains. This region receives large amounts of rainfall and is regularly subject to slope instability problems during the typically humid winter months but prior to the earthquake it had undegone two dry years, according to a USGS report (1989). The most common landslides induced by the earthquake were shallow rock falls and soil and rock slides involving small volumes of displaced materials. Scarps and cracks were left by deeper seated slow moving blocks which, in many places, formed rotational slumps that in some cases moved a few inches. Crib walls and other retaining structures near the epicenter were also damaged (Benuska, 1990). The analysis of at least some of these failures would surely render valuable information as to the actual mechanisms that triggered them and would also provide means for assessing the validity of available analytical tools.

Sand liquefaction affected many areas. The most

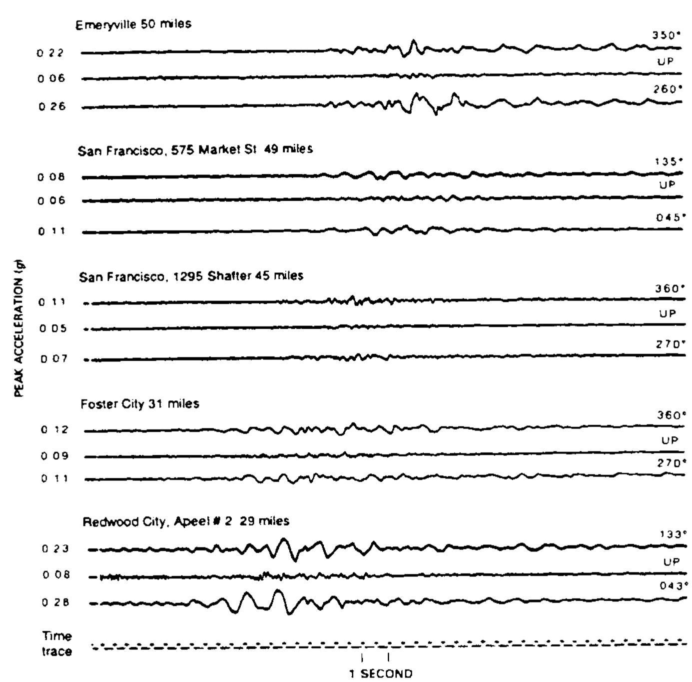

Fig 29. Accelerograms showing the effect of local soil conditions. The 1295 Shafter site lies on bedrock and the other ones on unconsolidated deposits (after USGS, 1989)

severely hit were filled areas along the margins of San Francisco and Oakland. In the Marina District in northern San Francisco, liquefaction caused extensive damage. Loosely packed hydraulic fills liquefied —as expected by using well known simple empirical methods (O'Rourke *et al*, 1990)— and resulted in sand boils, lateral spreading and foundation bearing failures (Seed *et al*, 1989). Because of partial loss of bearing capacity, many structures settled differentially. Liquefaction was also observed on the eastern shore of the San Francisco Bay. In their survey, the USGS (1989) reported that in Oakland, damage to the airport runways also resulted from liquefaction. Lateral spreading of wharves and and compaction of fills were also attributed to sand liquefaction at the Oakland harbor facilities. Liquefaction-related differential settlements and ground cracks also affected the Bay Bridge toll plaza and a naval air station. Whereas most of the cases of liquefaction identified along the coastline in the San Francisco Bay occurred in artificial fills, in the Monterrey Bay Area it was natural geologically young (late Holocene), unconsolidated fluvial deposits that liquefied. The deposits along the San Lorenzo, Pajaro and Salinas Rivers as well as spits, bars and tidal channels along the coast suffered widespread liquefaction. Among other facilities, liquefaction disrupted flood control levees, pipelines, bridge abutments and piers, homes, roads and irrigation ditches (UGSS, 1989; O'Rourke *et al*, 1990). In wharf structures affected by liquefaction, batter piles were markedly less able to sustain loads induced by spreading displacements (Benuska, 1990). Many of the areas that liquefied during the Loma Prieta earthquake had already experienced the same phenomenon in the 1906 San Francisco earthquake, which confirms that earthquake induced liquefaction can affect successively the same deposits (EERI, 1989; USGS, 1989; Benuska, 1990). Properly compacted artificial fills, even those located close to the epicenter performed well and were not affected by liquefaction (Benuska, 1990)

Many earth and rock fill dams lie within the most affected areas. Their overwhelming majority performed well during the earthquake. Seed *et al* (1989) report in some dams the appearance of minor longitudinal cracking along the crests and in some portions of the dam faces. However, Austrian dam, a 54 m high embankment that impounds Lake Elsman, experienced more severe damage: a transverse fissure appeared in the right abutment and disrupted the concrete-lined spillway.

CONCLUSIONS

From the geotechnical viewpont our cursory look at recent experience with eathquakes highlights the following site effects due to the presence of deformalble soils.

1 Amplification of long period vibrations,
2 Generation of surface waves by lateral irregularities
3 Lengthening of ground motion duration
4 Curtailing of large oscillations owing to nonlinear behavior
5 Liquefaction of some noncohesive soils

The first three effects occur even in the range of linear soil behavior, and hence for all earthquakes, including the very small ones. The first effect has been spectacular at sites of pronounced contrast between the properties of surficial soils and those of deeper ones during practically all recent events, more markedly so in Mexico City, where the contrast is highest. The meaning of "long periods" depends on the prevailing ground periods of vibration. While these may be of the order of several tenths of a second in the cities hardest struck by the Armenia earthquake, they reach 2 or even 5 s in Mexico City.

On valleys having horizontally stratified sediments and whose horizontal dimensions are several times the thickness of the sediments, as in Mexico City, one might expect a one-dimensional model to be a good predictor of site effects. Early success along these lines, added to the appeal of the model's simplicity reinforced that notion and hence postponed a more critical examination of the problem. Even in these favorable circumstances the one-dimensional model has limitations. On those areas of a valley where surface waves happen to be of minor importance, this model is adequate for predicting the response spectra of translational components of motion for, say, 5 % damping. On other areas the model has been found to underestimate response spectral ordinates by up to 50 %, and at no site does it adequately predict the lengthening of duration. The latter can be decisive for structures that deteriorate under repeated loading even if it is unimportant for hysteretic structures for whose design knowledge of response spectra usually suffices. The simple model also fails to furnish bases for designing lifelines, or very tall slender structures whose behavior is significantly influenced by surface waves. Still, the model correctly predicts the prevailing period of the ground and often its first harmonic. These periods can also be determined empirically from microtremor records.

There is the need for two- and three-dimensional models that could be solved with available means. Supercomputers open up this possibility for not too extensive valleys and in whose sediments shear wave velocities are not too small. Barring these restrictions there is hope for hybrid models. Gravity effects on Rayleigh waves are still a moot question.

In today's absence of satisfactory numerical or analytical methods, the analysis of a valley's response may in some cases rest on the use of empirical transfer functions, defined as the ratios between smoothed Fourier amplitudes at any given two sites, as functions of period. In the Valley of Mexico these functions are generally almost constant for distant earthquakes. This allows estimating spectra of major earthquakes on soft ground from records of smaller ones. Only two exceptions have been identified. One concerns a station whose motion is apparently quite sensitive to azimuth, due to its closeness to a pronounced lateral discontinuity. The other suggests nonlinear soil behavior for a particularly intense earthquake in a stratum where the clay is especially weak. This does not mean that nonlinear soil behavior only occurred at this station. Lacking information from other recent strong earthquakes it constitutes the only station in which the phenomenom has been poisitively identified but, given the geotechnical similarity of other sites within the valley, the chances are that non-linear soil behavior did occur in some areas during the 1985 events. As a consequence of the near invariability of transfer functions in this valley, damage patterns for strong distant earthquakes overlap considerably. Possible extrapolations of this approach to other valleys should proceed cautiously, as Mexico City clay is exceptionally linear in its stress-strain behavior up to rather high strains.

Even if in special cases like Mexico City the hypothesis of overall linear soil behavior is adequate for predicting free-field movements, save the exceptions noted earlier, the same supposition may not hold for soil-structure interaction analyses. This is still an open question. Very few bulding codes explicitly take into account soil-structure interacion effects, usually by means of corrections that depend on the relationship between the structure's fundamental period and the soil's dominant one. This relationship changes when the structure exceeds its elastic range of behavior. This aspect deserves further attention. Results of step-by-step methods that include nonlinear structural behavior suggest that the problem could possibly be approximated substituting the deformable soil by a rigid semispace, and introducing springs and dashpots between the soil and the foundation. Additional studies are needed in which the contribution of friction piles is realistically introduced into soil-structure interaction analyses. Accelerations recorded during the Loma Prieta earthquake consistently yielded smaller accelerations at the bases of buildings than in the free field confirming the kinematic aspect of soil-structure interaction.

In what concerns liquefaction there seem to be no new lessons, merely a verification of well-established knowledge. Empirical approaches based on SPT soundings for assessing the liquefaction potential of loose sand deposits were proven to be satisfactory in the Marina district after the Loma Prieta earthquake. Additional confirmation of these approaches as well as of others, based on other types of exploration tools like CPT borings, would be of great help to the profession. Analytical methods for predicting sand liquefaction must also be tested. In liquefaction-prone areas, the experience of the Loma Prieta earthquake in the Marina District should be reviewed carefully.

Damage to bridges, bridge abutments and retaining structures was noted in Mexico, San Salvador, Armenia and Loma Prieta. Barring liquefaction, the influence of other factors like out-of-phase movements at the bases of piles and supports should also be further investigated.

Relative movement of geologic faults also caused direct damage to homes and vital lines. The latter should be designed to account for them. Automatic isolation systems of damaged sectors as well as alarm devices can conceivably be developed.

Effects of topography can be significant. They consist in the magnification or attenuation of vibrations as a function of frequency. Models are being developed for analyzing these effects, but there is still a need for their being more realistic.

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