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Earthquake-Induced Liquefaction Near Lake Amatitlan, Guatemala

Prepared by H. Bolton Seed, I. Arango, C. K. Chan, A. Gomez-Masso and R. Grant Ascoli

University of California

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Introduction

One of the effects of the February 4, 1976 Guatemala earthquake was the extensive liquefaction which occurred at the settlement of La Playa on the north-east shore of Lake Amatitlan. The settlement is located on a deltaic deposit near the mouth of a small river, Rio Villalobos, and consists mainly of a resort area with several newly constructed brick and cement-block houses surrounded by grass lawns and trees. An overall view of the area is shown in Fig. 1. The general area behind the shore line is quite flat with the ground sloping gently up at about 2° away from the lake. The ground some distance from the beach is devoted to agriculture. The area of extensive liquefaction extended about 600 ft behind the nearest shore line and is marked by the shaded zone in Fig. 2. S second zone, where evidence of liquefaction was readily apparent but its effects were less severe, extended about 1200 ft behind the shore line (see Fig. 2).

Within the heavy damage zone there was subsidence and flooding of beach areas, (see Fig. 2), severe ground cracking with cracks ranging in size up to several meters (Fig. 3), severe damage to houses where they were located in areas of extensive ground cracking, as well as numerous sand boils. The ground cracking was generally more severe near the water-front with crack widths decreasing progressively in size as the ground rose gradually in elevation towards the inland edge of the liquefied area.

FIG. 1 OVERALL VIEW OF LIQUEFACTION ZONE AT LA PLAYA

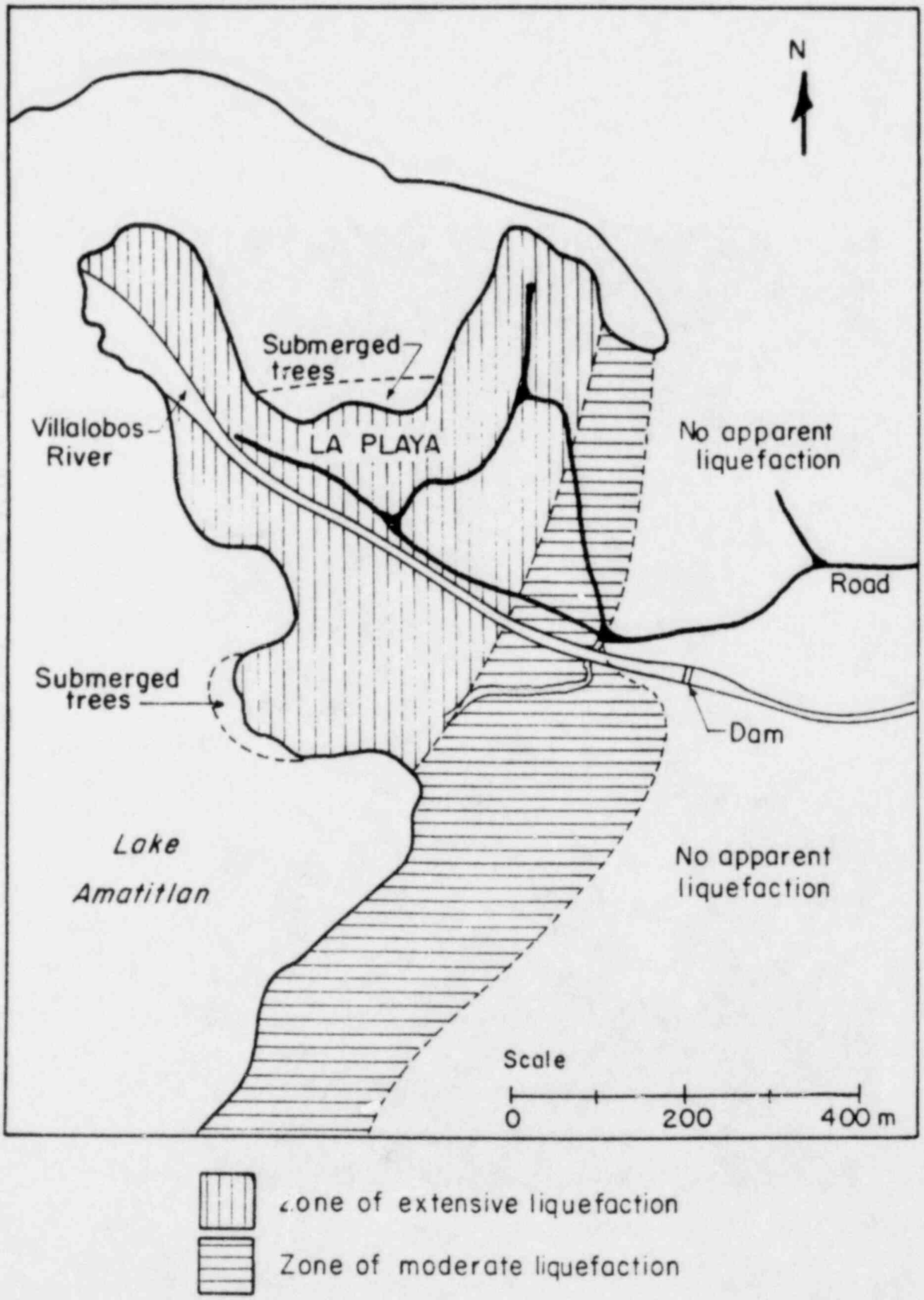

FIG. 2 ZONES OF LIQUEFACTION AT LA PLAYA

FIG. 3 GROUND CRACKING IN ZONE OF EXTENSIVE LIQUEFACTION

Of the 32 houses at La Playa, 29 were destroyed or damaged by differential lateral displacement, generally as a consequence of lateral spreading and subsidence. Fortunately most of the houses at La Playa were mainly vacation houses and they were largely unoccupied at the time of the earthquake.

An interesting feature of the area was the expulsion, together with sand, of occasional pieces of pumice, ranging from small gravel to cobble sizes in some of the mounds formed around the sand boils. The pumice particles were of very light weight so that they were readily carried up with the liquefied sand through the vents and cracks through which water moved upwards to the ground surface. Areas of sand boils in the zone of moderate liquefaction are shown in Fig. 4.

In contrast to the extensive damage due to liquefaction in the La Playa area, damage to houses in nearby towns was slight. Even a cement-brick house located near an area of intense sand boils and land spreading at La Playa was virtually undamaged (Krinitzky and Bonis, 1976).

In view of the scarcity of well documented cases of soil liquefaction it was considered important to determine the soil conditions within and immediately adjacent to the liquefied zone in order to supplement the available empirical data base used for liquefaction evaluations of other sites. At the same time it was considered useful to investigate whether the liquefaction that occurred at La Playa might have been anticipated through the use of currently-used analyses and laboratory test procedures. The following pages present the results of such studies.

The Guatemala Earthquake of Feb. 4, 1976

The February 4, 1976 Guatemala earthquake occurred on the Motagua Fault and has been assigned a Richter Magnitude of 7.5. Its epicenter (see Fig. 5) was located about 170 kms north-east from Lake Amatitlan, but surface ruptures

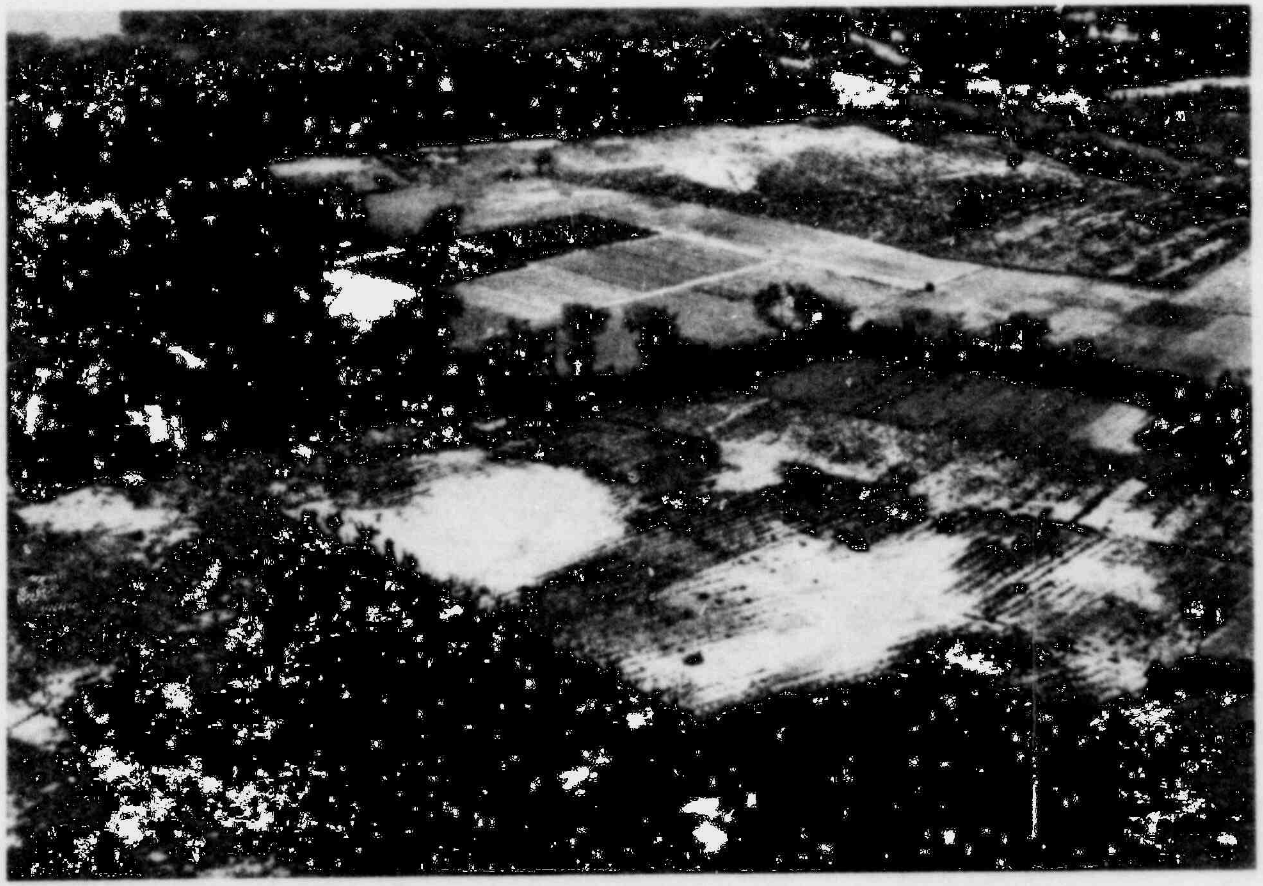

FIG. 4 SAND BOILS AT GROUND SURFACE IN ZONE OF MODERATE LIQUEFACTION

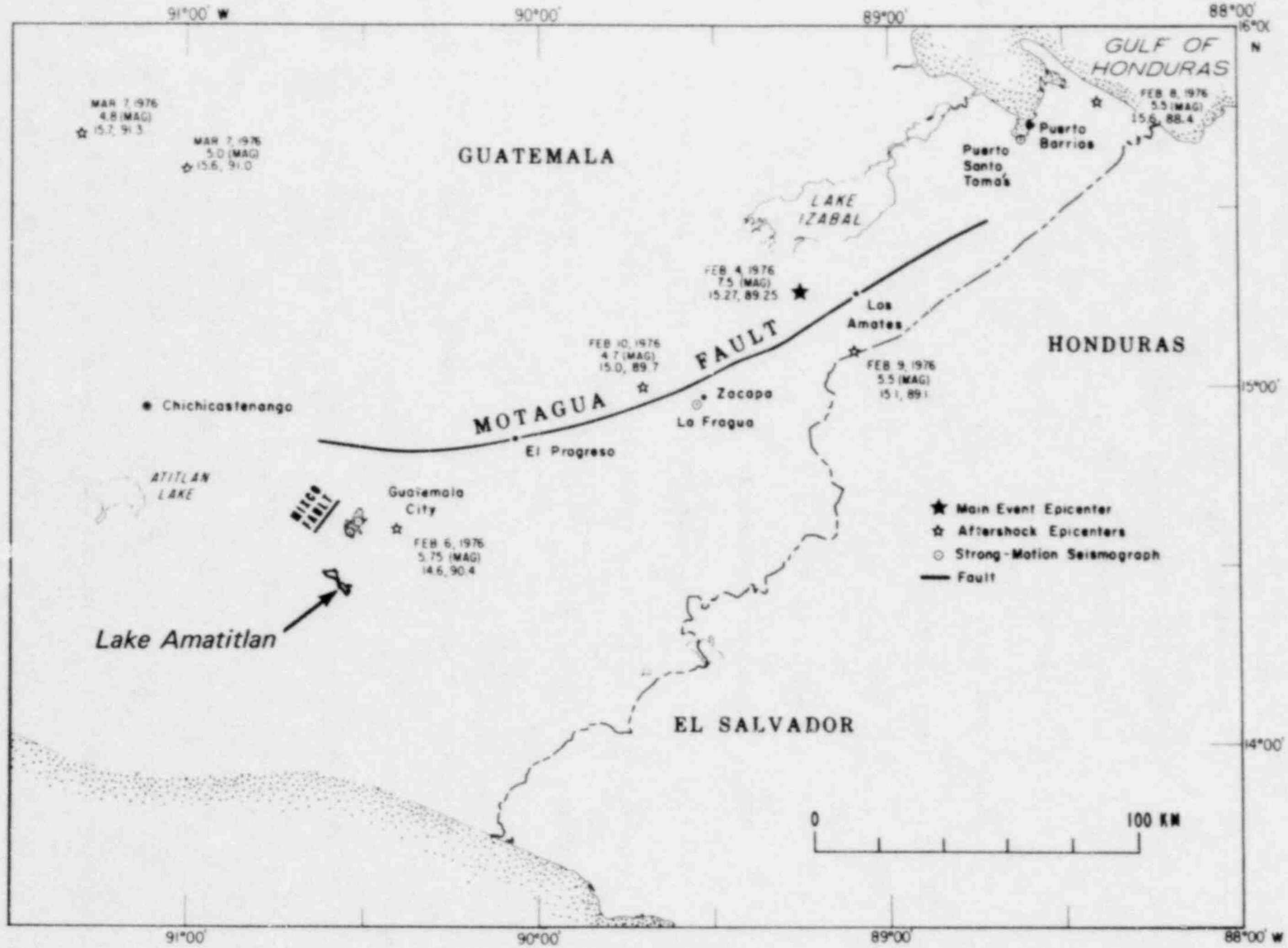

FIG. 5 LOCATION OF LAKE AMATITLAN IN RELATION TO MOTAGUA FAULT

(after Espinosa)

along the fault were mapped over a length of at least 240 kms and the western end of the major east-west fault break was only about 40 kms (25 miles) due north of the lake. In addition some short length north-south offset faults were mapped in and around Guatemala City about 20 kms (12 miles) north of the site. The distribution of Modified Mercalli intensities in the strong motion area of the earthquake, as assigned by the U. S. Geological Survey (Espinosa, 1976) is shown in Fig. 6. Lake Amatitlan is located in Intensity Zone VI but very close to the isoseismal for Intensity VII. Based on the correlation of intensities with peak ground acceleration developed by Trifunac and Brady (1976), this would indicate a peak ground acceleration in the vicinity of Lake Amatitlan of the order of 0.12g.

Data on the damage to adobe houses during the earthquake has also been assembled by the U. S. Geological Survey (Espinosa, 1976) and presented in the form of a damage distribution map shown in Fig. 7. The Amatitlan area lies near the boundary between Zone 0 (0 to 25% damage) and Zone 1 (25 to 50% damage). Observations of housing damage at La Playa would seem to confirm this assessment and suggest that the peak ground accelerations were nearer the Trifunac and Brady mean value of 0.12g rather than a significantly higher value which might be warranted by probability considerations within the available intensity vs. peak acceleration data base.

Peak ground accelerations generated by the Magnitude 7.5 earthquake at Lake Amatitlan can also be assessed using the correlations between peak acceleration, earthquake magnitude, distance from causative fault and local soil conditions proposed by Schnabel and Seed, 1973 and Seed et al., 1976. The correlations would indicate a peak acceleration in rock near Lake Amatitlan of the order of 0.21g, but for the softer soil conditions near the lake this value would be expected to be reduced to about 0.16g. A similar value is indicated by the correlations presented by Idriss, 1979.

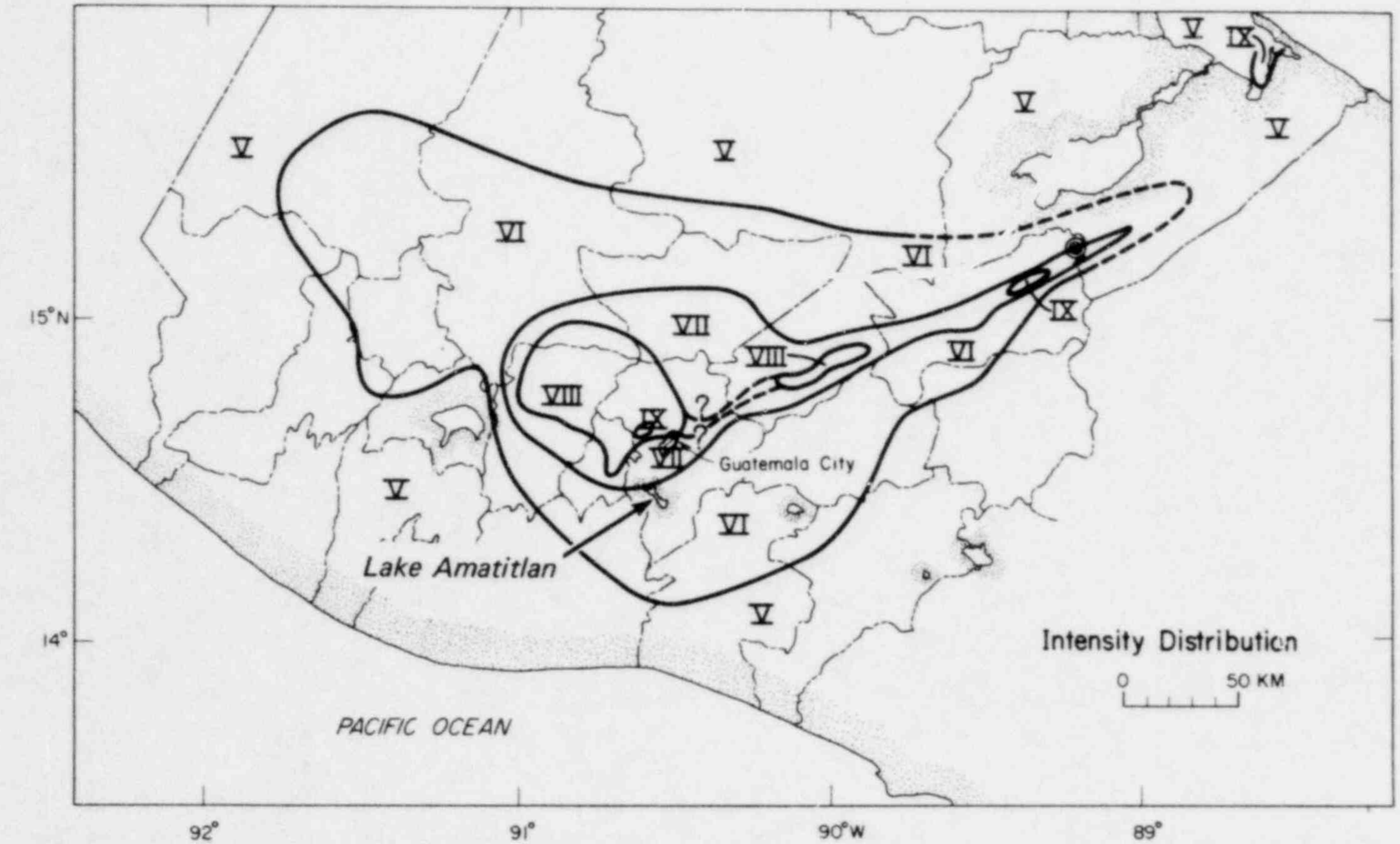

FIG. 6 DISTRIBUTION OF MODIFIED MERCALLI INTENSITIES FROM 1976 GUATEMALA EARTHQUAKE

(after Espinosa)

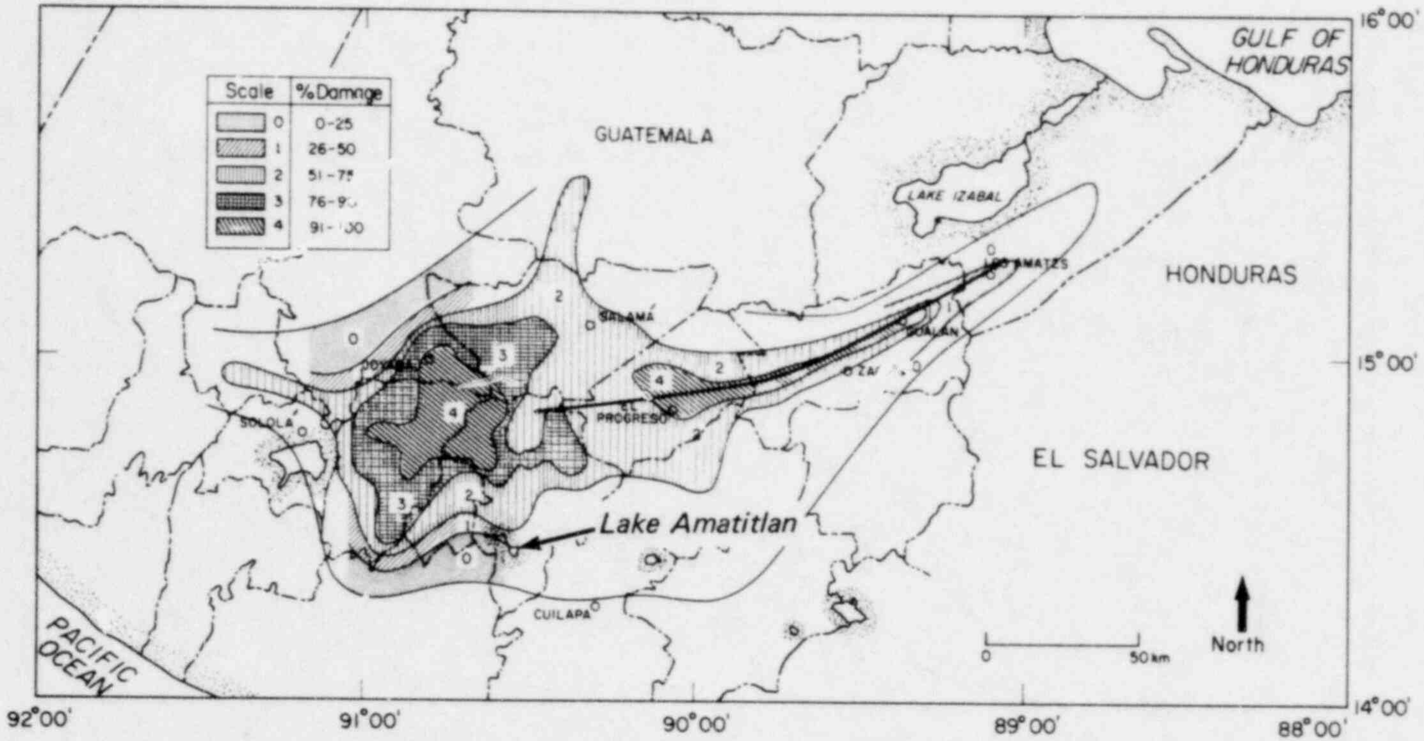

FIG. 7 DISTRIBUTION OF DAMAGE TO ADOBE HOUSES IN 1976 GUATEMALA EARTHQUAKE

(after Espinosa)

Unfortunately there were no records of the ground motions induced by the earthquake near Lake Amatitlan. However seismoscope records were obtained in Guatemala City (about 25 kms from the causative fault) and an interpretation of these by the U. S. Geological Survey (Espinosa, 1976) indicates that the peak ground acceleration in this area was probably of the order of 0.25g. At the increased distance (40 kms) to Lake Amatitlan, the use of attenuation laws (see for example, Fig. 8) would indicate a peak acceleration in the vicinity of Lake Amatitlan of the order of 0.15g.

The lack of damage to adobe houses in the vicinity of the lake would seem to indicate that the actual peak acceleration in this area was nearer the lower bound of the values discussed above, but since the range indicated by the different procedures is comparatively low, it seems reasonable to conclude that the peak acceleration developed near the lake was probably in the range of about 0.12 to 0.15g and the liquefaction behavior can be assessed based on this probable range of values.

Soil Conditions in Liquefaction Area

The soil conditions in the area where liquefaction occurred (and did not occur) were investigated by four borings, each made to a depth of about 70 ft. The borings were located as follows:

(1) One boring (No. 1) in the midst of the highly liquefied zone

(2) One boring (No. 2) just outside the boundary of the liquefied zone

(3) Two borings in the non-liquefied zone (Nos. 3 and 4).

The locations of the borings are shown in Fig. 9. It would clearly have been desirable to have boring data in the liquefied zone before the earthquake. However it was reasoned that by making borings as described above it should be possible to establish the soil conditions representative of a marginal liquefaction condition for the ground motions resulting from the 1976 earthquake.

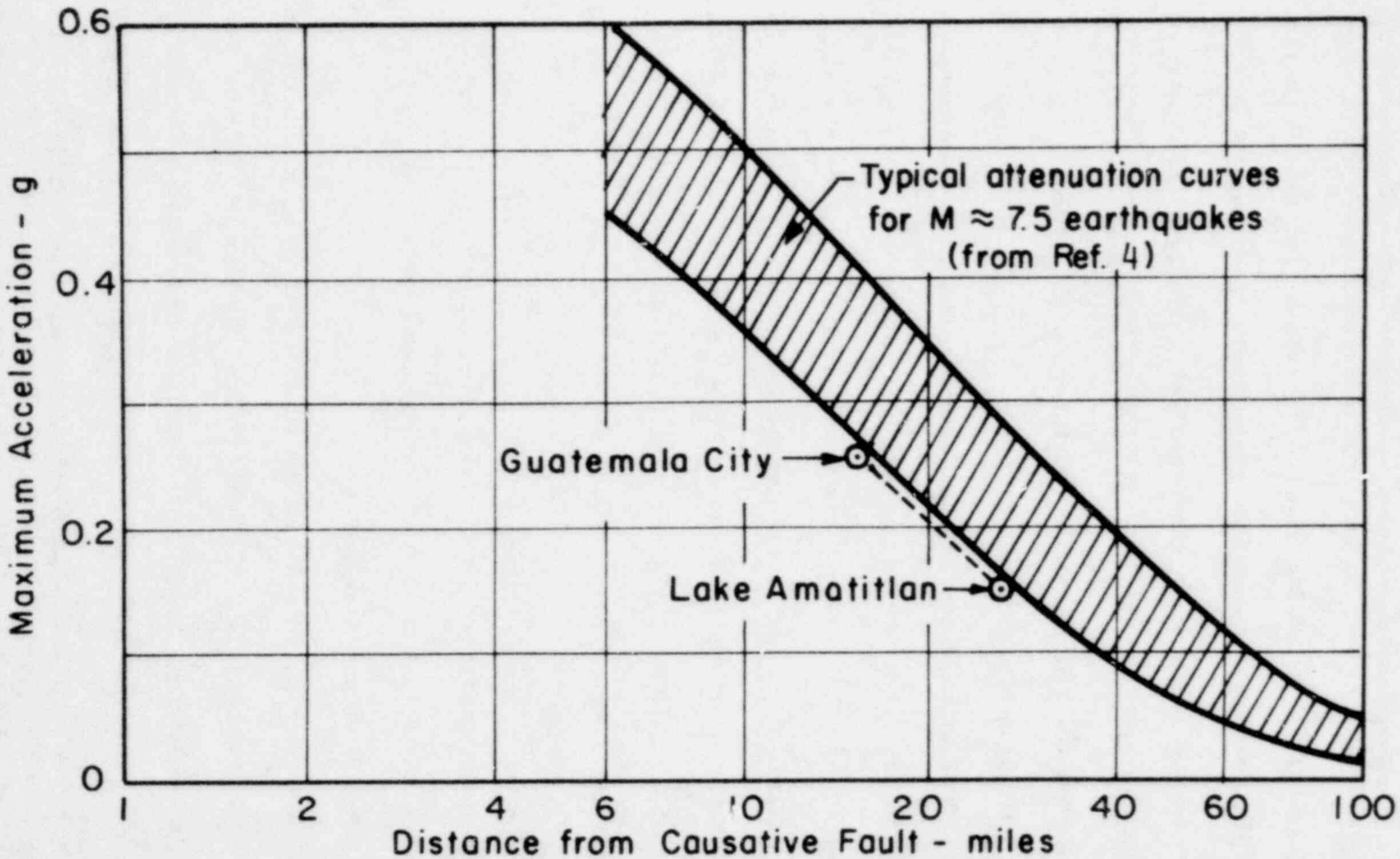

FIG. 8 USE OF ATTENUATION LAWS TO ESTIMATE PEAK ACCELERATION AT LAKE AMATITLAN

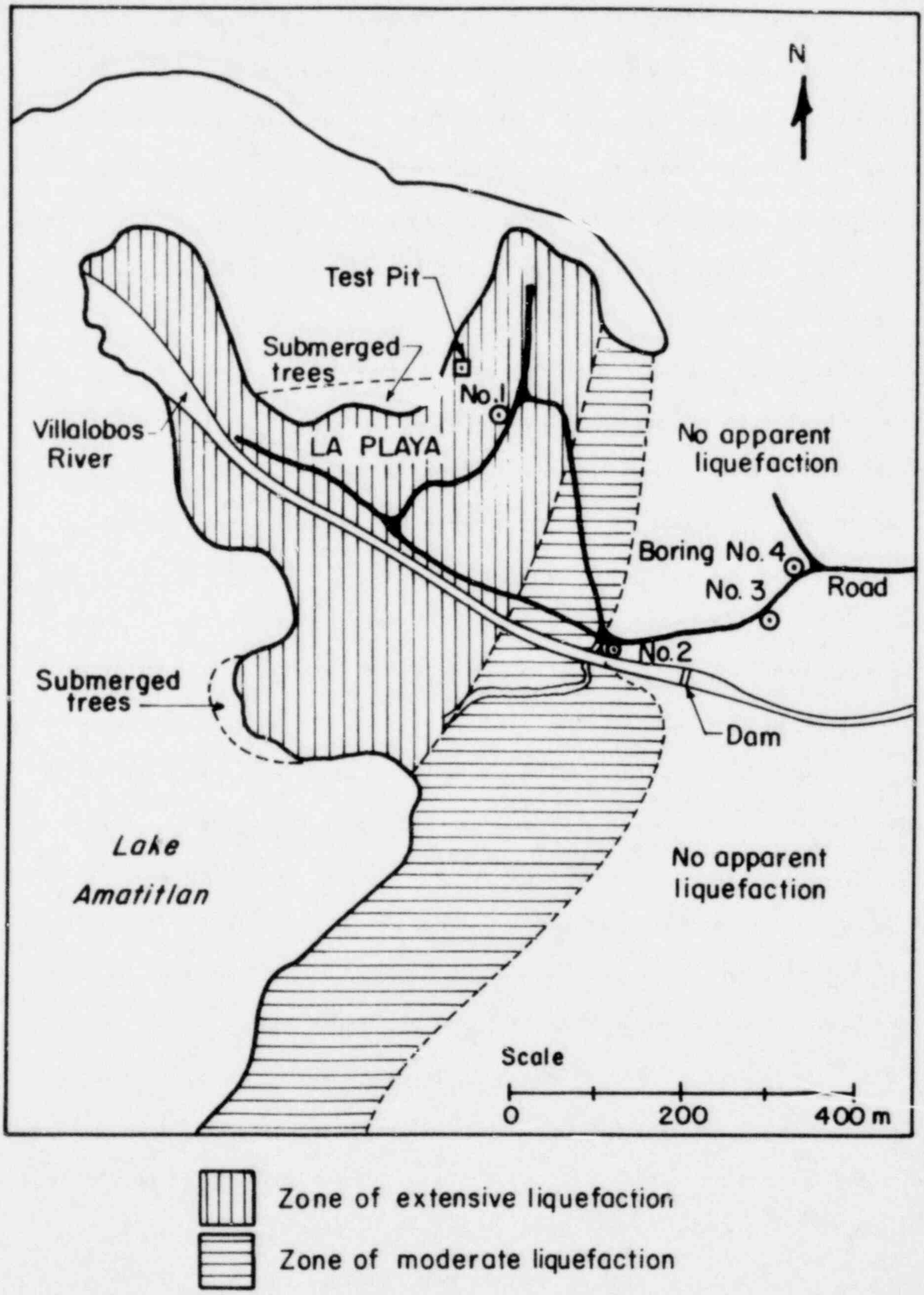

FIG. 9 BORING AND TEST PIT LOCATIONS

In each boring standard penetration tests were performed at intervals of 5 ft and 'undisturbed' samples were extracted in thin-wall seamless steel tubes for cyclic load testing in the laboratory. Standard penetration resistance values were determined using a rope and pulley system (2 turns of the rope around the pulley) to raise the 140 lb weight and measuring the number of blows per foot penetration at the bottom of a drill hole whose sides were supported by drilling mud.

In addition two test pits were dug to a depth of about 5 ft on the level ground about 40 ft from the beach. These showed the upper 4 ft of soil to be pumice sand underlain by a thin layer of dense fibrous peats and organic silt. The soil exposed on the walls of the pits was a stratified brown fine sand with some silt. One pit was dug across four cracks with a total width of 25 cms; the ground surface showed several centimeters of vertical subsidence towards the lake. The same vertical differential movement could be seen in the walls of the pit and increased with depth. The second pit was dug across a 10 cm wide crack showing no vertical differential subsidence but the crack was partially filled with sand which had been forced up from below and had intersected the horizontal stratifications. Two field density tests in the pumice sand which constituted the upper four feet showed that it had a wet density of 55.5 to 61.5 lb/cu ft, a water content of about 56 percent and a dry density of 35.5 to 39.4 lb/cu ft. These tests were taken in the moist soil above the water table.

The soil profile to a depth of about 70 ft is shown by the logs of the borings in Fig. 10, which also show values of the standard penetration resistance. Based on these results, a schematic soil profile along a section extending from the shore line to a distance about 1600 ft inland is shown in Fig. 11.

It may be seen that the entire area is covered by a surficial layer of brown pumice sand varying in depth from about 5 to 10 ft underlain in the zone of liquefaction by medium coarse sand containing some pumice fragments and in

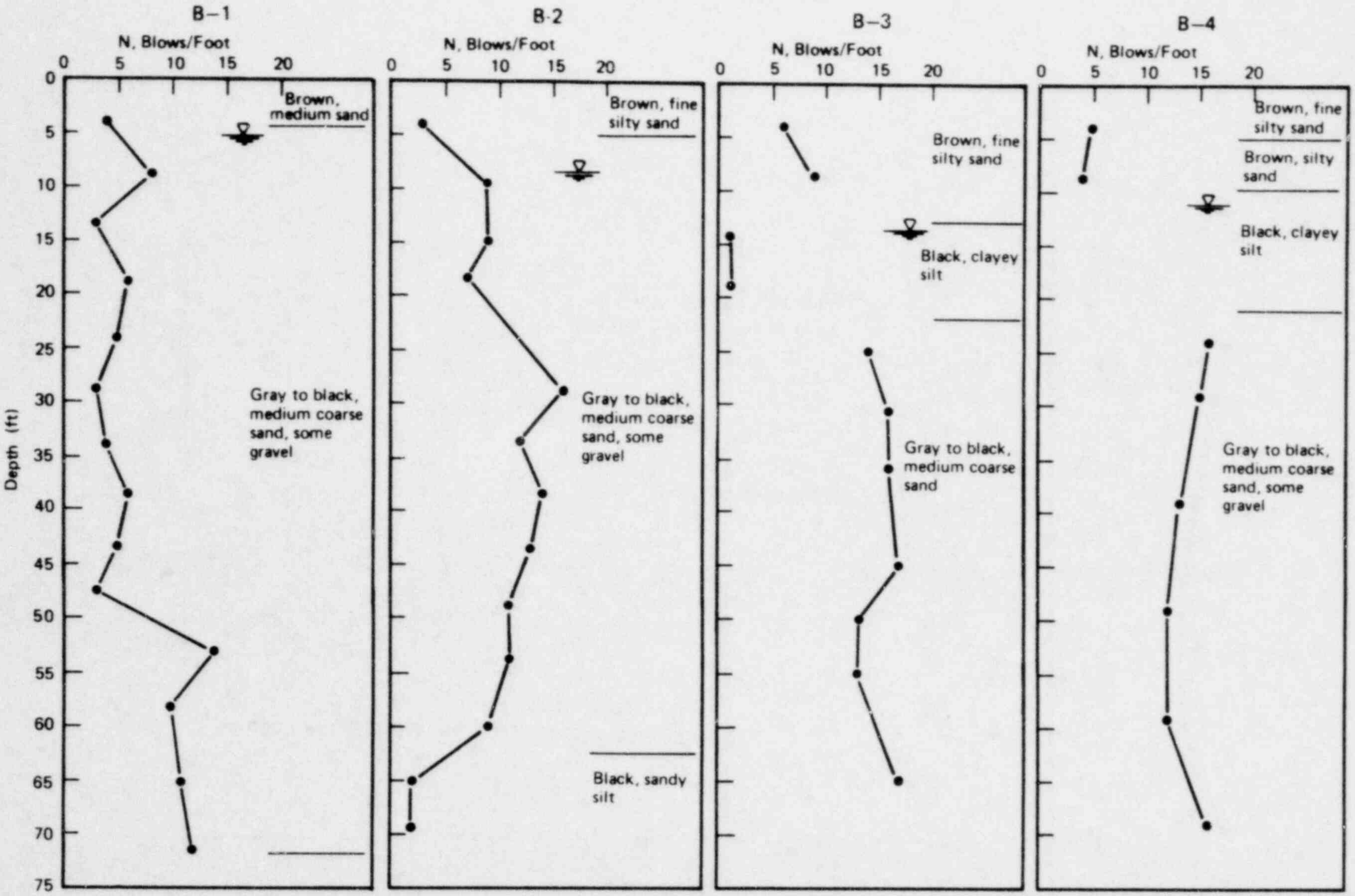

FIG. 10 BORING LOGS AT LA PLAYA SITE

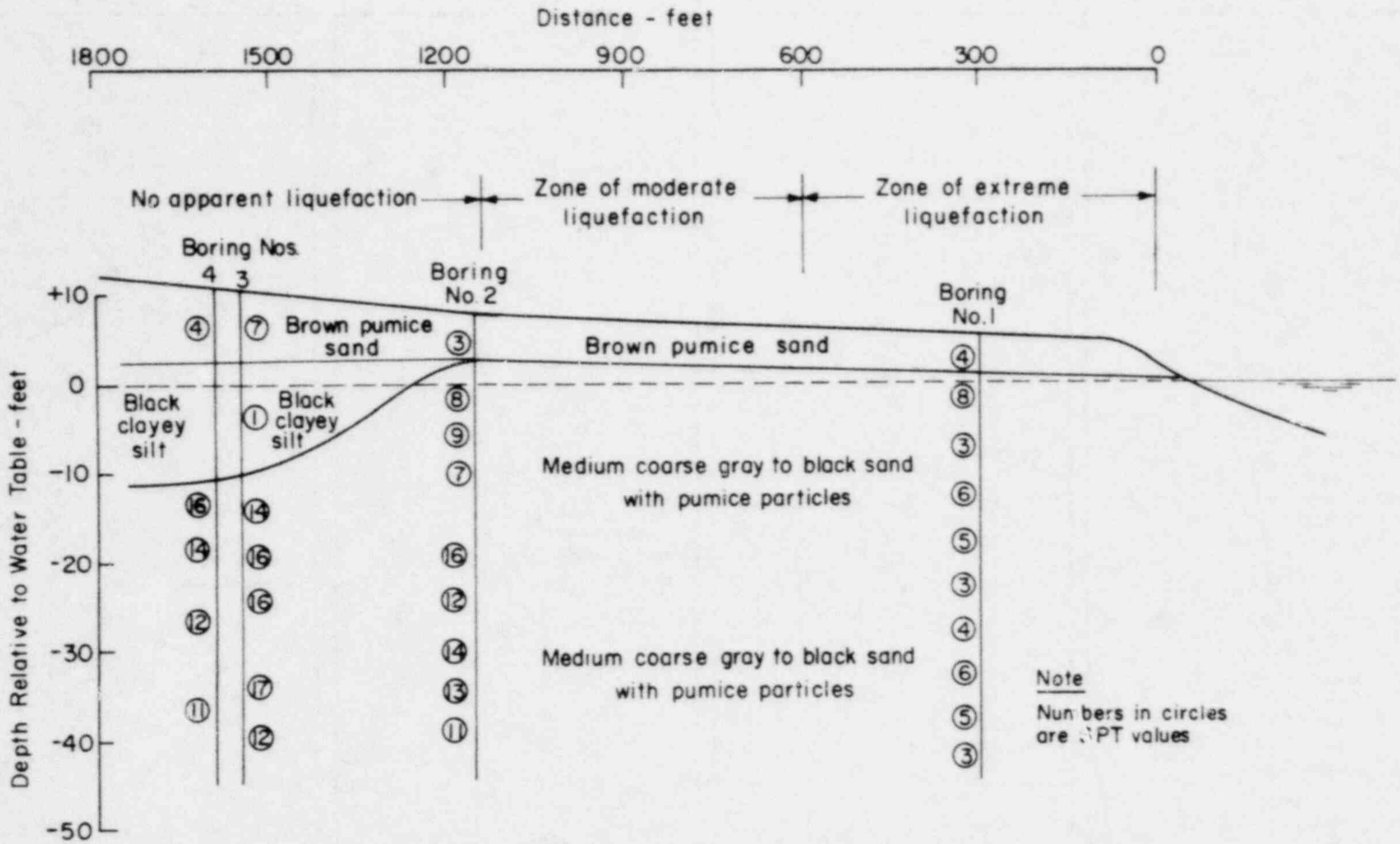

FIG. 11 SCHEMATIC SOIL PROFILE THROUGH LA PLAYA AREA

the non-liquefied zone by an intermediate layer of black clayey silt above the same type of sand with pumice particles. The water table varies in depth from about 4 to 5 ft near the shore line to about 12 ft depth in the area where no liquefaction was observed. It is interesting to note that the very light-weight pumice sand at all boring locations was located above the water table and thus was not itself the zone in which liquefaction occurred. The source of liquefaction was presumably in the saturated sand containing pumice particles below depths of 6 to 10 ft. In the heavily liquefied zone this material had a penetration resistance value, N, of only about 5 to a depth of 50 ft but presumably this value is representative of the liquefied and re-stablized sand. There is no way of establishing the penetration resistance of this material before the earthquake.

For this reason the remaining borings were made in the non-liquefied zone, one very close to the boundary (Boring No. 2) and two well behind the liquefaction zone. With increasing distance from the shore line the average penetration resistance increased and the depth of the water table increased, both factors which would tend to reduce the possibility of liquefaction of the sand deposit.

An examination of the soil profile in Fig. 11 indicates that the conditions at Boring No. 2 are probably representative of the limiting conditions at which liquefaction would just occur or just not occur for the ground motions and soil conditions in the La Playa area and this profile has therefore been used for a more detailed study. The general soil characteristics are shown more clearly in Figs. 12, 13 and 14. The latter two figures show the grain size distribution curves for 9 samples taken from depths between 30 and 45 ft in Borings No. 3 and 4. Similar soil types were found below the water table in Borings 1 and 2 where liquefaction occurred. It may be seen that the sand ranges from fine to coarse, though most of the samples are medium coarse sand. However

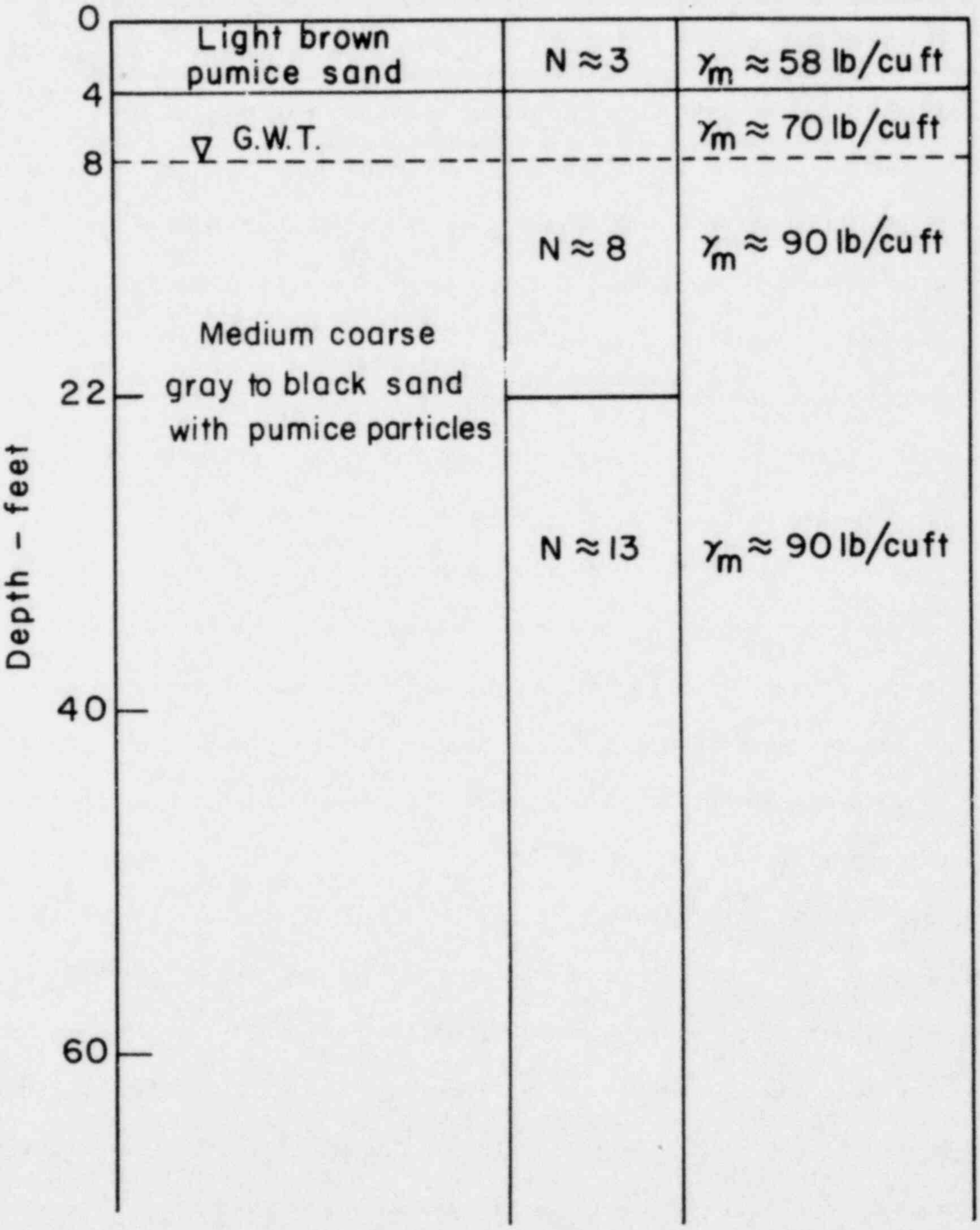

FIG. 12 GENERAL SOIL CHARACTERISTICS AT BOUNDARY OF LIQUEFACTION ZONE

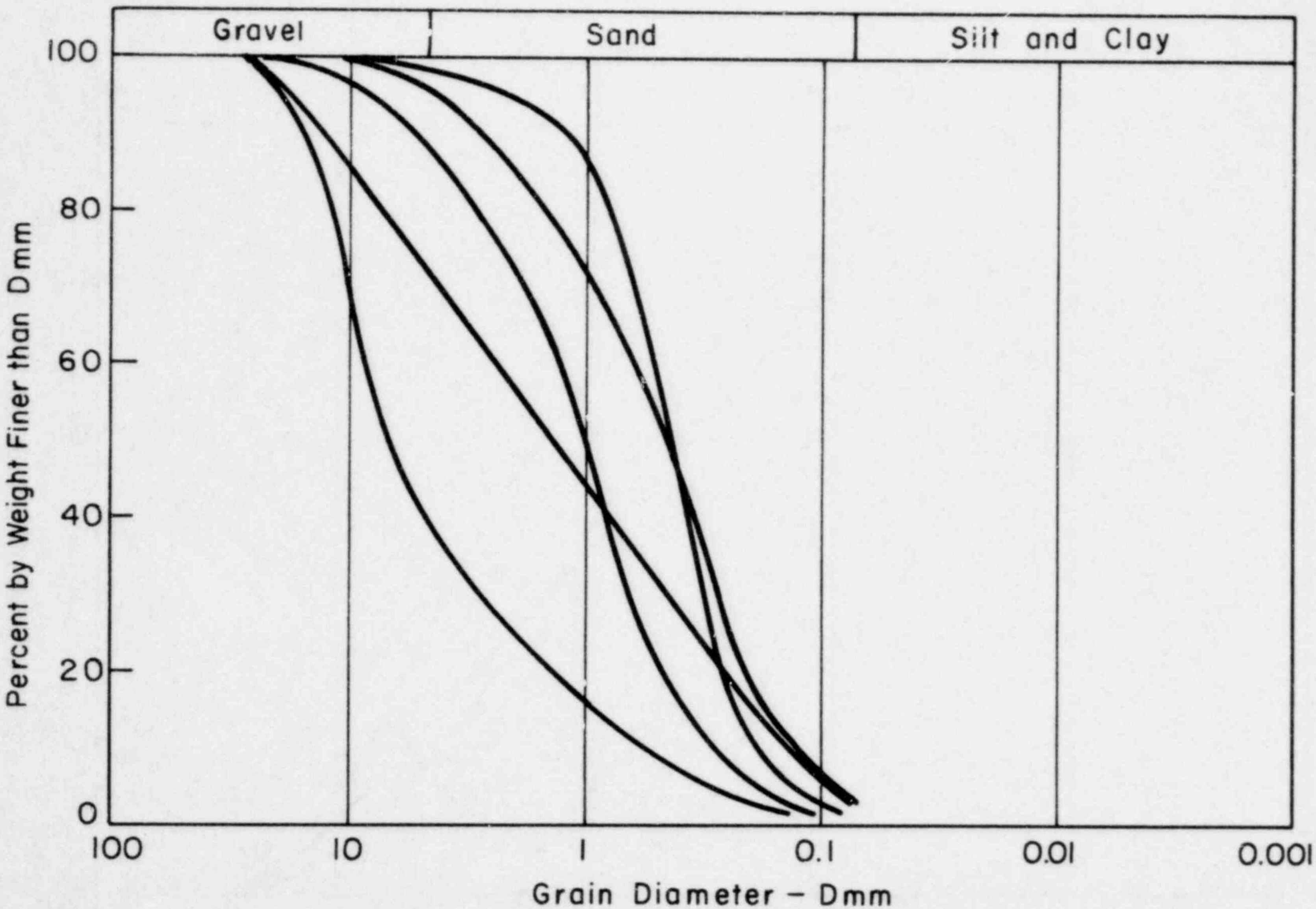

FIG. 13 GRAIN SIZE DISTRIBUTION CURVES FOR SAMPLES OF SAND REPRESENTATIVE OF LIQUEFIED MATERIAL

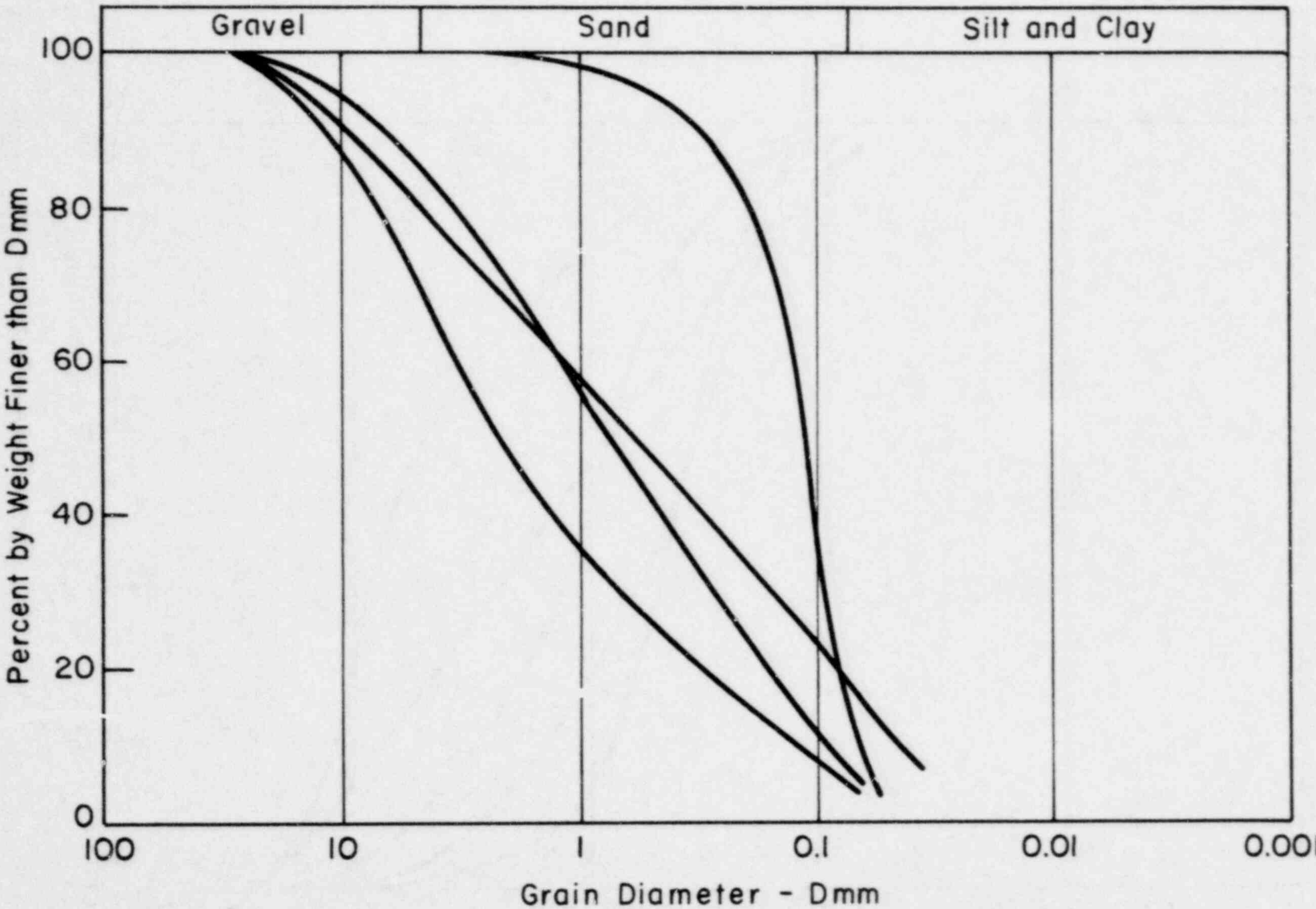

FIG. 14 GRAIN SIZE DISTRIBUTION CURVES FOR SAMPLES OF SAND REPRESENTATIVE OF LIQUEFIED MATERIAL

because of the presence of pumice particles ranging in size from coarse sand to gravel, the sand samples tested had the following characteristics:

Average dry density of sand	≃	58 pfc
Average specific gravity of solid particles	≃	2.1
Average water content of saturated sand	≃	60%
Average density of saturated sand	≃	92.5 pcf

For the generalized soil profile shown in Fig. 12, the saturated density was simplified to a rounded value of 90 pcf.

Liquefaction Analysis Based on Empirical Data

One of the methods currently used to assess the liquefaction potential of a sand deposit is a procedure based on an empirical correlation between the cyclic stress ratio induced by the earthquake and the standard penetration resistance corrected to an effective overburden pressure of 1 ton/sq ft (Seed, 1979). The proposed empirical curves, based on analyses of other sites, which separate areas where liquefaction has occurred in previous earthquakes from those which have not are shown in Fig. 15. Correction factors for determining the standard penetration resistance N_1 at an overburden pressure of 1 ton/sq ft are shown in Fig. 16. The data for the Lake Amatitlan site may be used to check the position of the curve for $M = 7\frac{1}{2}$ earthquakes on Fig. 15.

For borderline conditions of liquefaction and no liquefaction at La Playa the soil conditions are shown in Fig. 12. From this it may be seen that liquefaction is most likely to have developed in the lower blow count zone, $N \simeq 8$ extending from about 8 to 22 ft below the ground surface.

At this depth, computations of the effective overburden pressure show

$$\sigma_o' \simeq 4 \times 58 + 4 \times 70 + 7(90 - 62.4)$$
$$\simeq 704 \text{ psf}$$

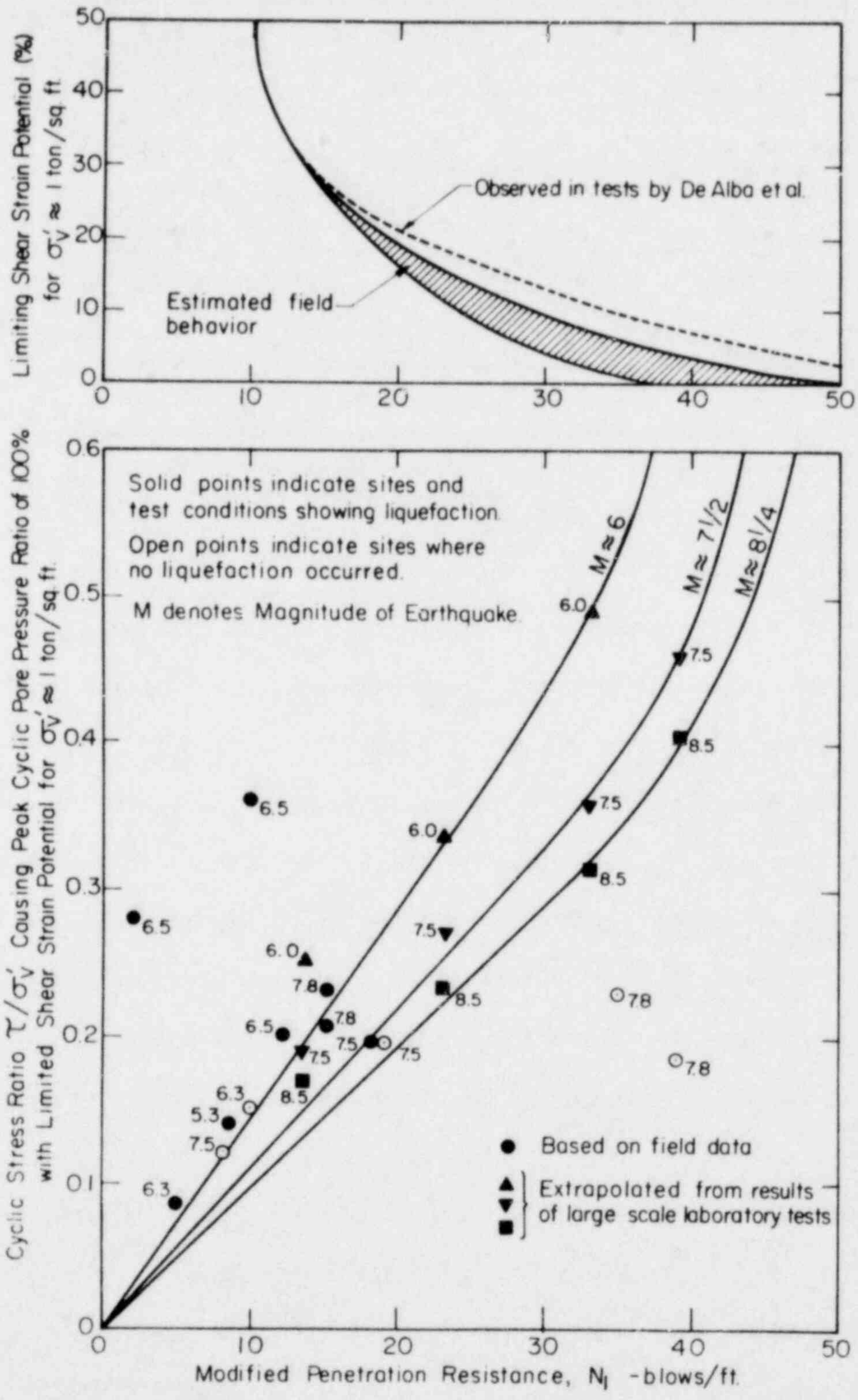

FIG. 15 CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE (SUPPLEMENTED BY DATA FROM LARGE SCALE TESTS)

(after Seed, 1979)

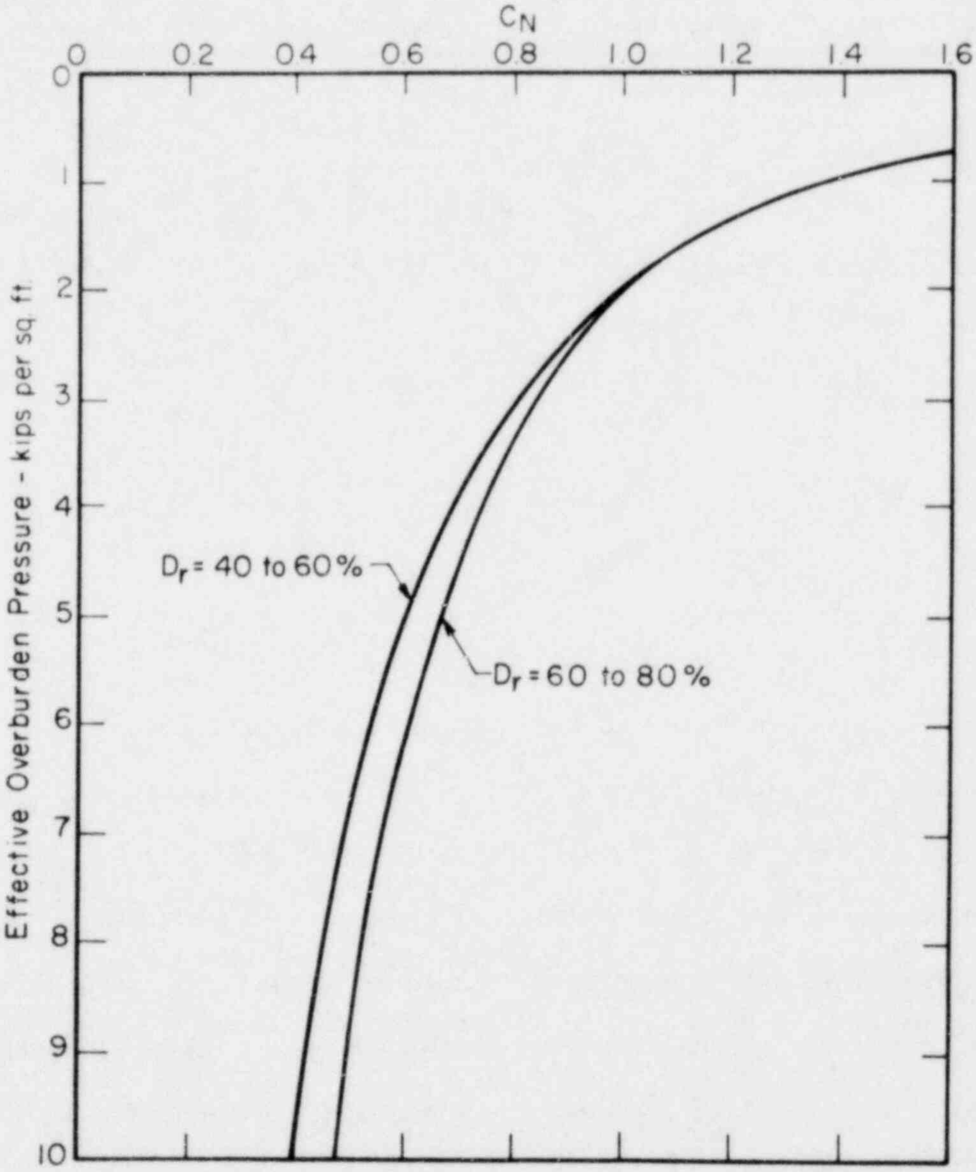

FIG. 16 RECOMMENDED CURVES FOR DETERMINATION OF C_N
BASED ON AVERAGES FOR W.E.S. TESTS

The value $N_1 = C_N \times N$ where C_N is a function of the effective overburden pressure. For $\sigma_o' = 704$ psf, the value of $C_N \simeq 1.6$ and thus

$$N_1 \simeq 1.6 \times 8 \simeq 13$$

The cyclic stress ratio induced by the earthquake may be estimated from the equation (Seed and Idriss, 1971).

$$\frac{\tau_{av}}{\sigma_o'} \simeq 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

where τ_{av} is the average horizontal shear stress induced by the earthquake,

r_d at a depth of 15 is about 0.95,

and $\sigma_o \simeq 4 \times 58 + 4 \times 70 + 7 \times 90 \simeq 1150$ psf

Thus $$\frac{\tau_{av}}{\sigma_o'} \simeq 0.65 \times \frac{1150}{704} \times 0.95 \times \frac{a_{max}}{g}$$

$$\simeq 1.0 \frac{a_{max}}{g}$$

Since a_{max} was estimated to be in the range of 0.12 to 0.15g, the probable value of $\frac{\tau_{av}}{\sigma_o'}$ is about 0.12 to 0.15. These values are plotted against the corresponding value of N_1 in Fig. 17, together with the boundary curve for $M = 7\frac{1}{2}$ earthquakes taken from Fig. 15. It may be seen that the data for the La Playa area are in excellent agreement with the proposed empirical curve. It is readily apparent that small changes in the values of unit weights used in these computations could influence the positions of the plotted points but they would not change significantly. Thus the data obtained from the field study would seem to provide further corroboration of the position of the empirical curve for predicting liquefaction potential in Magnitude $7\frac{1}{2}$ earthquakes.

Laboratory Investigation

In addition to the empirical approach discussed above, liquefaction potential is sometimes evaluated by comparing the earthquake-induced stress

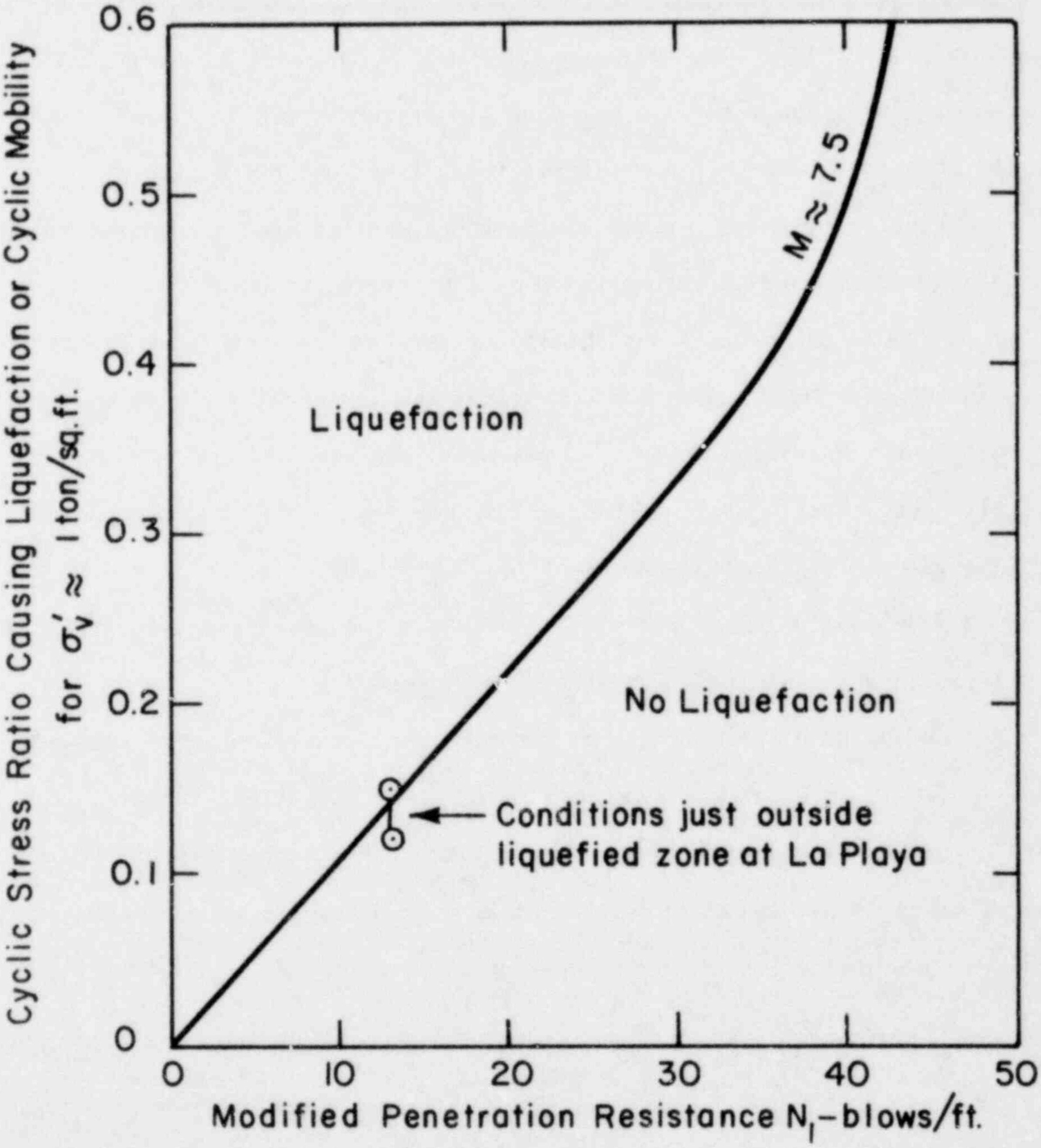

FIG. 17 EMPIRICAL RELATION BETWEEN STRESS RATIOS CAUSING LIQUEFACTION AND MODIFIED PENETRATION RESISTANCE

(after Seed, 1979)

ratio with that determined to cause liquefaction or cyclic mobility in laboratory tests on undisturbed samples.

During the course of the present investigation, 8 cyclic triaxial compression tests were performed on samples taken from depths ranging from 30 to 45 ft in borings Nos. 3 and 4. In these zones the standard penetration resistance was about 13 to 16, corresponding to an N_1 value of about 18, so these samples are likely to show somewhat higher resistance to liquefaction than the soil in the depth range of 8 ft to 20 ft in Boring No. 2.

The results of these tests, showing the number of cycles required to produce a residual pore poressure ratio of 100% are plotted in Fig. 18. The scatter is due to the natural variability of the samples, both in characteristics and density but it does not appear to show any significant relationship to either. A reasonable average curve has therefore been drawn through the entire set of data. Beyond the point in the test where the pore pressure reached a value of 100%, large deformations developed rapidly so the data in Fig. 18 may be considered representative of the stress conditions in triaxial compression tests producing a liquefied condition.

For a magnitude 7.5 earthquake, the number of equivalent uniform stress cycles is likely to be of the order of 15 and the field stress ratio τ_{av}/σ_o' likely to cause liquefaction is about 0.57 times the stress ratio in cyclic triaxial tests (Seed, 1979). Thus from this test data, the stress ratio required to cause liquefaction would apparently be about:

$$\left(\frac{\tau_{av}}{\sigma_o'}\right)_\ell \simeq 0.57 \times 0.32 \simeq 0.185.$$

The earthquake-induced stress ratio at a depth of 35 ft may be computed as described previously and shown to have the following values:

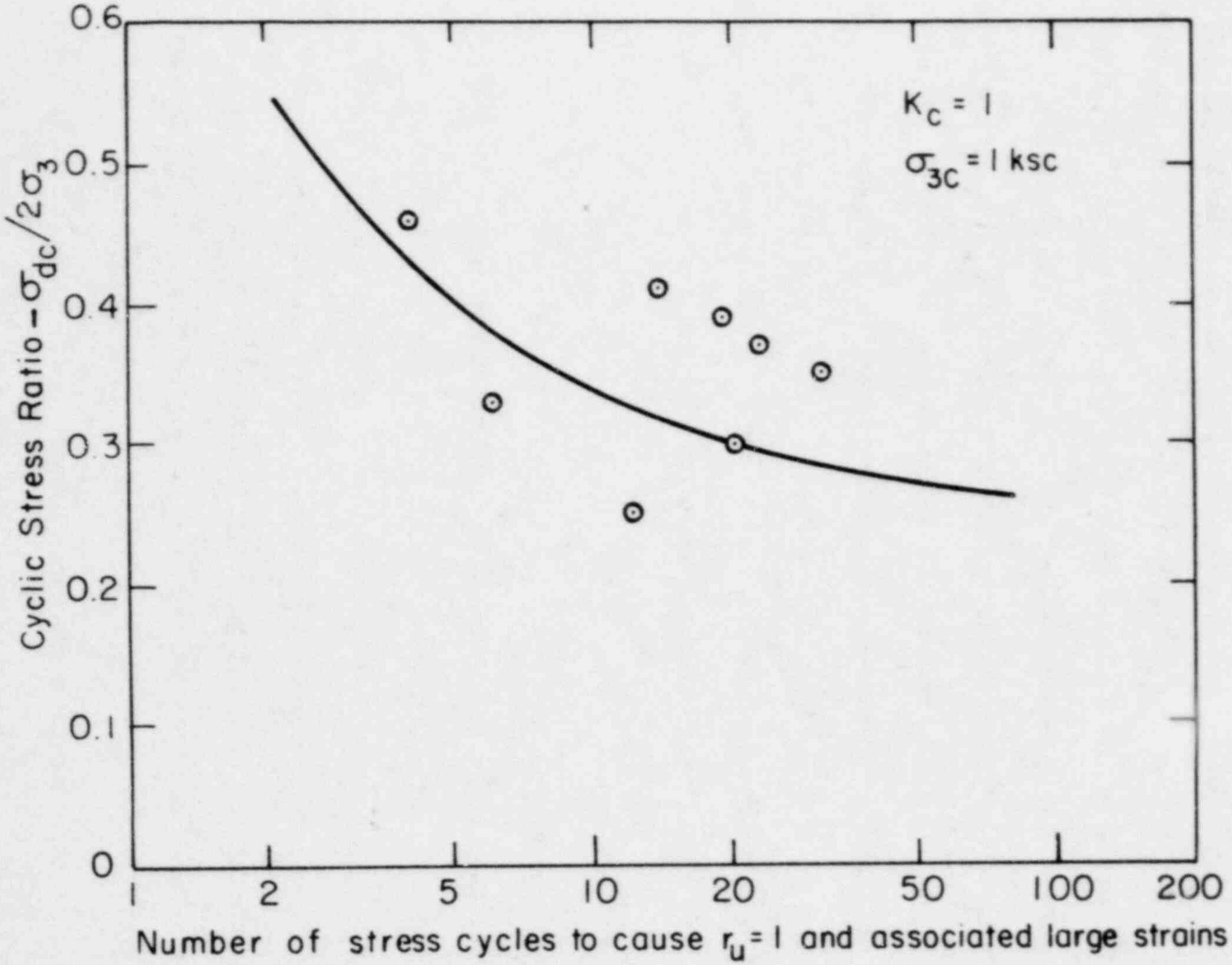

FIG. 18 RESULTS OF CYCLIC LOADING TRIAXIAL COMPRESSION TESTS ON UNDISTURBED SAMPLES

$$\text{For } a_{max} \simeq 0.12g, \ \frac{\tau_{av}}{\sigma_o'} \simeq 0.125$$

$$a_{max} \simeq 0.15g, \ \frac{\tau_{av}}{\sigma_o'} \simeq 0.16.$$

This would indicate a factor of safety ranging between about 1.15 and 1.5 which seems to be in accord with the observed absence of liquefaction in this area. It should be noted that this result may well be the effect of compensating errors. The 'undisturbed' samples were probably densified to some extent during the sampling and handling process. At the same time they probably lost some resistance which they had acquired due to the prior stress history (sustained loading plus prior seismic loading). As a result the measured strength may have been not too different from the in-situ strength but this would not necessarily be the case for all such studies, especially those involving very dense soils.

Analysis of Highly Liquefied Zone

Since it is impossible to determine the pre-earthquake properties of the sand in the zone of extensive liquefaction near the shore line, it is not possible to make a detailed analysis of its potential behavior either by empirical methods or by analytical-experimental methods. However some indication of its liquefaction potential may be obtained if it is assumed that the liquefaction characteristics of the sand in this zone were no better than those of the sand in boring No. 2, just outside the zone of liquefaction. Conditions near the shore line would be expected to be more conducive to liquefaction due to the shallower depth of the water table in this zone.

In the zone of extensive liquefaction the water table was at an average depth of about 5 ft below the ground surface. At a depth of 15 ft, the effective overburden pressure would then be only

$$\sigma_o' \simeq 4 \times 58 + 1 \times 70 + 10(90 - 62.4) \approx 578 \text{ psf}$$

while the total overburden pressure would be

$$\sigma_o \simeq 4 \times 58 + 1 \times 70 + 10 \times 90 \approx 1202$$

Thus the stress ratio induced by the earthquake would be about

$$\frac{\tau_{av}}{\sigma_o'} \simeq 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

$$\simeq 0.65 \times \frac{a_{max}}{g} \times \frac{1202}{578} \times 0.95$$

$$\simeq 1.28 \cdot \frac{a_{max}}{g} .$$

For values of a_{max} ranging from 0.12g to 0.15g this leads to values of cyclic stress ratios developed of about 0.155 to 0.19.

If the sand at a depth of 15 ft had the same penetration resistance as the sand in Boring No. 2, then the cyclic stress ratio required to induce liquefaction would be of the order of 0.12 to 0.15, thus indicating a factor of safety against liquefaction in the near-shore area of only about 0.63 to 0.97. Under these conditions the extensive liquefaction which actually developed should not be considered surprising. In fact if the density or penetration resistance of the sand in this zone were also somewhat lower than that further away from the shore (near Borings 2, 3 and 4) the situation would be even more unfavorable.

An alternative approach to assessing the resistance to liquefaction of the sand in the near shore area is to use the laboratory test results in conjunction with the penetration test data to assess the stress ratio causing liquefaction for the sand in Boring No. 2. Up to values of N_1 of about 30 or 35, the resistance to liquefaction of a sand seems to be essentially directly proportion to N_1 (see Fig. 15). Thus since the stress ratio causing liquefaction in the laboratory tests corresponds roughly to sand for which $N_1 \approx 18$ while the value of N_1 at a depth of 15 ft in Boring No. 2 is about 13, the laboratory test data would indicate a liquefaction resistance for this sand corresponding to a cyclic stress ratio of

$$\left(\frac{\tau_{av}}{\sigma_o'}\right)_\ell \approx 0.185 \times \frac{13}{18} \approx 0.135$$

Again it may be seen that this value, if it applied to the sand near the shore line, would be much less than the earthquake-induced stress ratios estimated to range between 0.155 and 0.19. In fact it would indicate a factor of safety in the range of 0.71 to 0.87. Again if the sand nearer the shore line were of somewhat poorer characteristics, the situation would be even less favorable than that indicated by the above figures.

On the basis of these evaluations, it should not be considered surprising that liquefaction near the shore line was so extensive and dramatic in its effects.

Effects of Pumiceous Nature of Sand

The low unit weight of the pumiceous sands at the La Playa site would appear to have been a significant factor in producing the liquefaction that occurred. The primary reason for this is the very low effective stresses produced by this sand at significant depths below the water table.

Consider for example, the conditions that would have developed at La Playa if the sand had been a typical quartz sand with the same resistance to liquefaction as the actual sand containing pumice; that is, requiring an induced stress ratio of 0.12 to 0.15 to induce liquefaction.

For the quartz sand, the unit weight of the moist sand might have been about 105 lb/cu ft and the total unit weight about 115 lb/cu ft. For a water table at a depth of 5 ft, as existed in the highly liquefied zone, the effective pressure at a depth of 15 ft would have been

$$\sigma_o' \simeq 5 \times 105 + 10\ (115 - 62.4)$$

$$\simeq 1051 \text{ psf}$$

while the total overburden pressure would be

$$\sigma_o \simeq 5 \times 105 + 10 \times 115$$

$$\simeq 1675$$

Thus the stress ratio induced by the earthquake would be about

$$\frac{\tau_{av}}{\sigma_o'} \simeq 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

$$\simeq 0.65 \times \frac{a_{max}}{g} \cdot \frac{1675}{1051} \cdot 0.95$$

$$\simeq 0.98 \cdot \frac{a_{max}}{g}$$

For values of a_{max} ranging from 0.12g to 0.15g this leads to values of cyclic stress ratios developed of about 0.12 to 0.15, and corresponding computed factors of safety against liquefaction ranging from 0.83 to 1.3.

Thus depending on the acceleration developed there is a very good chance that a normal type of quartz sand would not have liquefied at all, in contrast to the much lower factors of safety indicated for the pumiceous sand (0.63 to 0.97) and the evident high degree of liquefaction of the area.

It may be concluded that very light weight saturated cohesionless soils are much more vulnerable to earthquake-induced liquefaction than typical quartz sands and should be treated with special caution in seismically active regions.

Conclusions

The preceding pages present the results of a field and laboratory investigation of the extensive area of liquefaction which occurred at La Playa on the shore of Lake Amatitlan in the Guatemala earthquake of 1976. The investigation leads to the following conclusions:

1. The soil in which liquefaction occurred was a layer of sand containing particles of pumice which occurred between depths of about 5 to 70 ft or more below the ground surface. It was covered by a surficial layer of light-weight pumice sand and because of the pumice particles in the liquefied layer, its total unit weight had the relatively low value of about 90 lb/cu ft.
2. In spite of the fact that the sand is somewhat lighter in weight than sand deposits which have liquefied in other earthquakes, its liquefaction characteristics are apparently influenced by the same factors as other sand deposits and its overall behavior is consistent with that exhibited by other sands.
3. The penetration resistance of the sand at the boundary between liquefied and non-liquefied zones is in good accord with previously developed empirical correlations between liquefaction potential and the standardized penetration resistance (N_1) at which liquefaction can just be expected to occur.
4. The behavior of the sand in the liquefied and non-liquefied zones was consistent with experimental-analytical predictions of liquefaction potential based on the results of cyclic loading triaxial compression tests performed on undisturbed samples to evaluate the liquefaction characteristics of the sand and conventional procedures used in conjunction with this type of test data to evaluate liquefaction potential.

5. The high degree of liquefaction at the La Playa site was probably due in large measure to the light-weight nature of the pumiceous sands. A typical quartz sand at the same site with the same cyclic load characteristics as the sand containing pumice might well have remained stable in spite of the earthquake shaking. Consequently light weight cohesionless soils should be treated with special caution with regard to their liquefaction potential in seismically active regions.

6. The results of the investigation provide an extremely useful case history in which field data on soil characteristics in an earthquake-liquefied zone and a non-liquefied zone can be correlated with field performance, thereby supplementing the limited number of available case studies of this type which can be used as a basis for predicting probable behavior at other sites. The results also tend to corroborate currently-used procedures for evaluating liquefaction potential although in the case of the laboratory test data, this clearly depends on the degree to which the in-situ properties of the soil are represented by the 'undisturbed' samples extracted from the deposit.

In spite of the inevitable limitations of any study of this type it is hoped that the results of the investigation, used with appropriate judgment, will contribute to the available data base relating to earthquake-induced liquefaction and thereby to an improved predictive capability of this type of behavior in seismically active regions.

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An analysis of data gathered from a field survey, including bore hole cores, and from laboratory measurement of soil particles, penetration resistance and other parameters, leads to the development of a soil profile and the determination of conditions which caused the soil liquefaction resulting from the earthquake motion. Field data are very consistent with experimental-analytical theory. Boundary conditions between liquefaction and non-liquefaction zones are identified and correlated with physical conditions of partical size, water content, etc.

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