LIQUEFACTION-INDUCED DAMAGE TO BUILDINGS IN 1990 LUZON EARTHQUAKE

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ABSTRACT: The Luzon Earthquake of 1990 (M_s = 7.8) caused soil liquefaction in a widespread area that in turn caused crucial damage to various structures. After presenting an overview of the geotechnical aspects of the earthquake, this paper describes damage patterns of buildings in Dagupan City where fine to silty sands liquefied extensively. Also presented are geomorphological conditions, Standard Penetration Test (SPT) results, and shear wave velocity (V_s) profiles determined by a Rayleigh wave method; and their relations to the building damage of the city. The buildings that suffered large settlement and tilting are found to be concentrated in the banks of active rivers and in the fills built on recently abandoned river channels. It is found that the currently available empirical correlations using SPT N-value can well estimate the extent of liquefaction-induced damage to buildings. The Rayleigh wave method used shows capability in determining cross sections of V_s structure in the near surface soil.

INTRODUCTION

The central part of the Luzon Island, Republic of the Philippines, was shaken on July 16, 1990, by an earthquake of magnitude of 7.8. It affected a widespread area of approximately 20,000 km^2, and caused extensive damage to reinforced concrete buildings, wooden houses, roads, embankments, natural slopes, and bridges. At least 1,600 persons were killed by this event and its aftershocks, more than 900 were missing, and over 3,400 were seriously injured.

One of the most significant features of the damage resulting from this event is the bearing failure of buildings in Dagupan City, where fine to silty sands liquefied extensively (Tokimatsu et al. 1991a). In view of the scarcity of well-documented cases of soil liquefaction involving fine to silty sands and of the resulting building damage [except that in the 1964 Niigata earthquake (Yoshimi and Tokimatsu 1977; Rollins and Seed 1990)], it seems important to characterize the damage in the 1990 event, together with their relation to soil conditions. It is also worthwhile to examine whether the currently available empirical correlations are effective in estimating liquefaction potential of fine to silty sands. The objects of this paper are to describe the liquefaction-induced damage to buildings in Dagupan City, and to discuss its relation to geomorphological conditions, SPT N-values, and shear wave velocity (V_s) profiles. Also presented is an overview of geotechnical aspects of the earthquake in other affected regions.

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Note. Discussion open until July 1, 1994. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on January 15, 1992. This paper is part of the ***Journal of Geotechnical Engineering***, Vol. 120, No. 2, February, 1994. ©ASCE, ISSN 0733-9410/94/0002-0290/$1.00 + $.15 per page. Paper No. 3301.

GEOLOGIC SETTING AND AFFECTED AREA

Fig. 1 shows the geological map of the northern Luzon. The region between the Gulf of Lingayen and Manila City is the Central Plains. Most of the northern Central Plains is an alluvium deposit formed by the Agno River that is fed by a large number of tributaries. Tracing the northeast edge of the Central Plains is a part of the Philippine Fault zone. One of its branches entering the Cordillera Central is the Digdig Fault. The July 16, 1990, earthquake occurred along the Philippine Fault zone including the Digdig Fault, with a left-lateral movement up to 6 m. Its epicenter is located near Rizal, in the northeast of Cabanatuan City, at a depth of about 25 km. The solid line shown in Fig. 1 is the trace of a 110-km surface fault resulting from this event. No strong motion record was registered for the main shock. The Modified Mercalli intensities estimated from interviews and inspections of building damage are VIII or more in Baguio City and Agoo, VIII in Cabanatuan City, and VII to VIII in Dagupan City (Midorikawa 1990). The damage associated with a combination of strong shaking and inadequate design and construction of buildings was concentrated in Baguio City and Agoo, where many multistory reinforced concrete buildings collapsed (Tokimatsu et al. 1991a). Ground problems such as soil liquefaction and slope failure, in contrast, affected various structures and infrastructures in a widespread area. These include the epicentral region and the Central Plains from

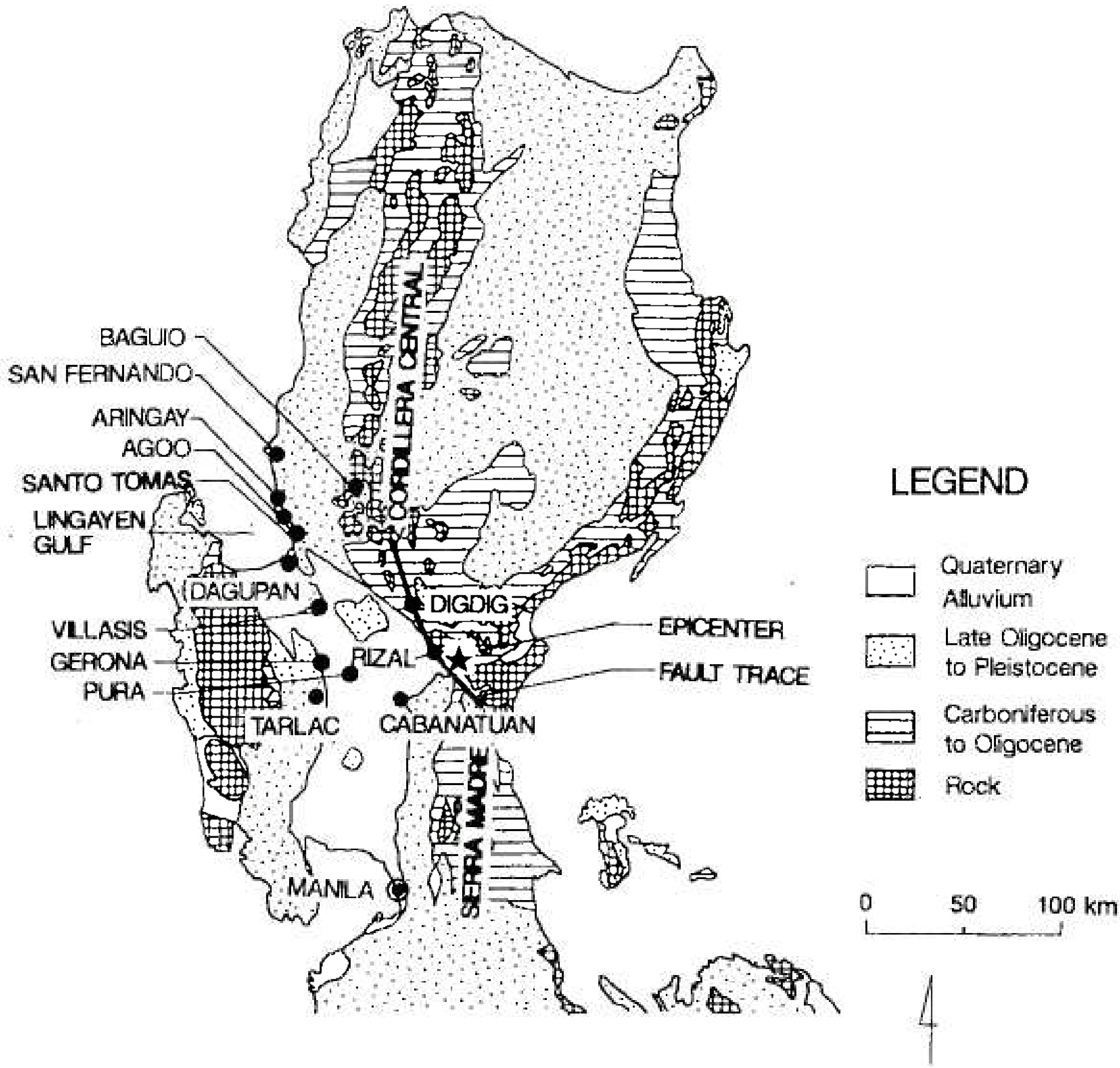

FIG. 1. Geological Map of Northern Luzon

Tarlac to the Gulf of Lingayen. A large number of slope failures triggered by the earthquake, its aftershocks, or subsequent rainfalls, shut off many roads in the mountain regions near the epicenter.

Most of the damage to wooden houses, roads, bridges, and embankments along the Gulf of Lingayen was related to soil liquefaction. A large number of reinforced concrete buildings in Dagupan City suffered large settlement, often accompanied by severe tilt, due to liquefaction-induced bearing failure. Considerable damage to wooden houses was also observed in Dagupan and many other towns such as Aringay, Agoo, and Santo Tomas. Some villages on sand bars in these towns were totally submerged below sea level, due to extensive soil liquefaction and associated ground settlement and spreading (Tokimatsu et al. 1991a). These include Alaska, Aringay; and Narvacan and Rawis, Santo Tomas. Liquefaction and associated lateral ground spreading also occurred in many alluvial deposits of sandy soils in the epicentral region and the middle of the Central Plains, causing failure of embankments, roads, and bridges. The damage to the Carmen Bridge that crosses the Agno River near Villasis, caused cutoff of the main route from Manila to the Northern Luzon. Many wooden houses in small towns such as Gerona and Pura, in the north of Tarlac, were badly damaged, and some places in these towns had been inundated for several months until a dry season came.

LIQUEFACTION-INDUCED DAMAGE TO BUILDINGS IN DAGUPAN CITY

Historical and Geological Setting

Dagupan City is located at the southeastern end of the Gulf of Lingayen and extends over an alluvium deposit formed by several rivers including the Pantal River, a tributary of the Agno River. The altitude of the city is 1–2 m, and the water table is very shallow. The city has a population of about 110,000, within 43.6 km^2. Fig. 2 shows a map of current Central

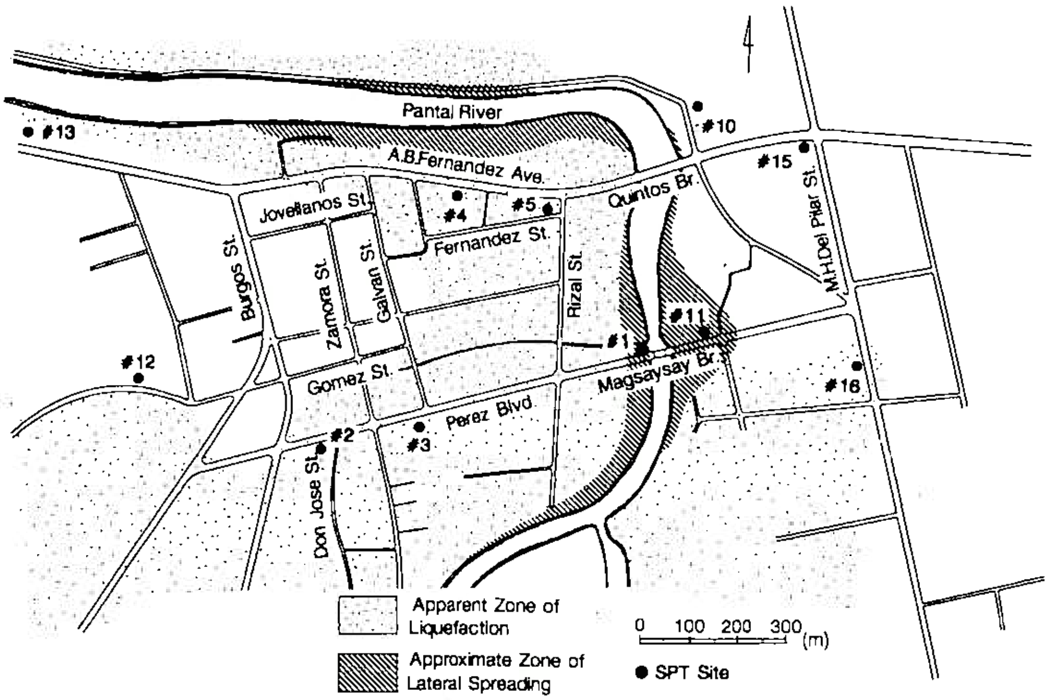

FIG. 2. Map of Dagupan Showing Zones of Liquefaction and Lateral Spreading and Locations of SPT Sites

Dagupan, which is bisected by the Pantal River. Most of the commercial buildings are concentrated in the area from Burgos Street to M. H. Del Pilar Street and from A. B. Fernandez Avenue to Perez Boulevard. A town called Bactotan was built by 1590 within the current Dagupan City, and renamed Nandaragupan ("where people meet") in the 1660s and then Dagupan in the 1720s (*Socio-economic profile*). The old downtown area in the late 1800s reportedly existed on the west of Zamora Street between A. B. Fernandez Avenue and Perez Boulevard. Much of the current downtown area on the west side of the Pantal River was either a fishpond or a swamp then. The completion of the Manila-Dagupan railroad in 1891 accelerated the development of the city (*Socio-economic profile*). As a result, the fishponds and swamps were reclaimed to create more space for commercial facilities. The commercial center extended, beyond the Quintos Bridge, to the east side of the river in 1908, and the Perez Boulevard and the Magsaysay Bridge were constructed in 1948 (Punongbayan and Torres 1990). It is interesting to note that Dagupan experienced soil liquefaction during the earthquake of March 16, 1892 (*Series on seismology* 1985). This is the only earthquake that shook Dagupan with Intensity VIII or greater in the last 100 years.

Affected Area

Fig. 2 shows the approximate zone in which field manifestation of liquefaction such as a sand boil was evident after the 1990 earthquake. The zoning was made based on quick inspection during our site visit. Also shown in the figure is the area that is considered to have slid or spread, due to liquefaction of underlying soils. Many ground fissures that took place parallel to the course of the river in this area indicated that the ground has slid and expanded toward the river.

In the liquefied area, most of the reinforced concrete buildings suffered significant damage. Much of road pavement was split open or buckled up as a result of lateral ground movement, differential ground settlement, and/or settlements of buildings. Buried utilities such as fuel storage tanks in gas stations and sewage tanks came to the ground surface due to increased buoyant force during soil liquefaction. Water and sewer pipes that broke in many places, caused vital damage to water supply and sewerage systems of the city. Liquefaction also induced bearing failure of many electric poles, resulting in power outage. The Magsaysay Bridge collapsed due to lateral ground spreading and bearing failure of the piers, whereas the Quintos Bridge was unaffected.

Almost all damage to buildings is related to foundation bearing failure associated with soil liquefaction (Fig. 3). Damage due to strong ground shaking was rarely seen in the city. This includes the collapse of the upper portion of an unreinforced old church building on Zamora Street located outside the liquefied area. The peak ground acceleration in the nonliquefied region of the city during the July 16 event was estimated to be 1.8 m/s^2 (Midorikawa 1990).

Damage Statistics of Buildings

During our site visit, we inspected almost all reinforced concrete buildings and many wooden houses in the liquefied area, together with those in the nonliquefied area. The total numbers of reinforced concrete buildings and wooden houses inspected are about 300 and 250, respectively. Fig. 4 summarizes damage statistics of 217 reinforced concrete buildings that were

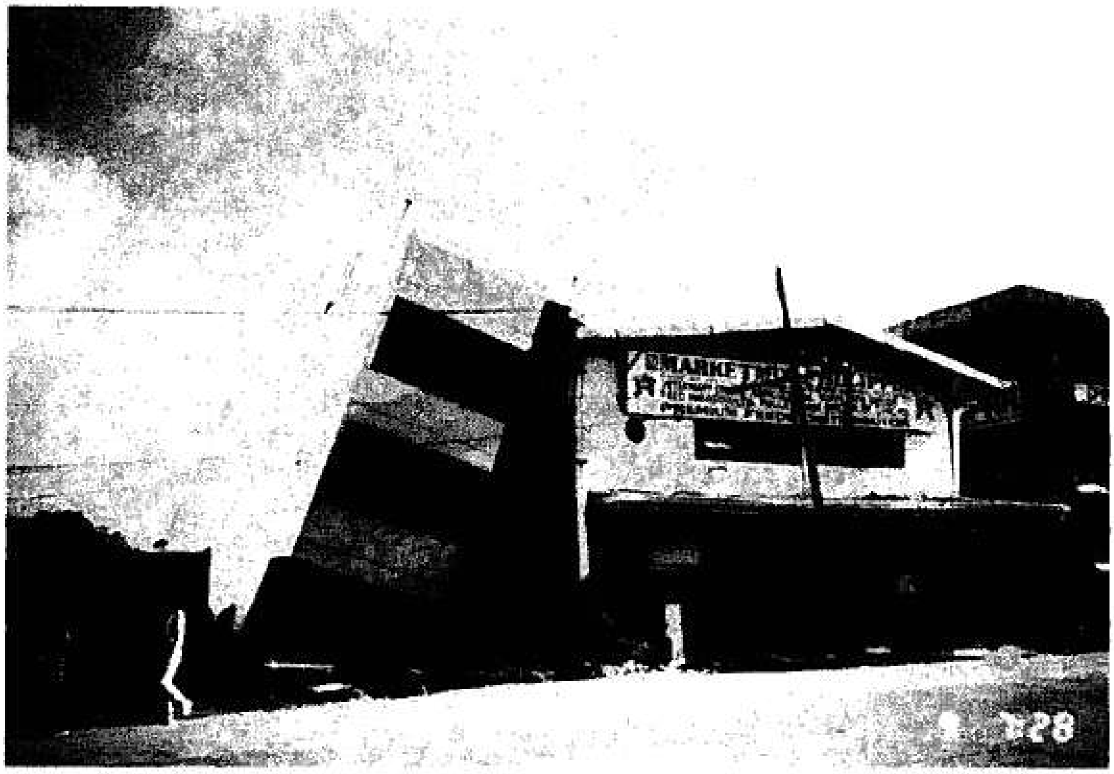

FIG. 3. Typical Bearing Failure of Building

inspected and located in the liquefied area. Most of the buildings are two- to four-storied with shallow footings [Fig. 4(*a*)]. Apparent lack of pile foundations for the buildings in the liquefied area shows that no consideration was made in the foundation design to mitigate liquefaction hazards. More than half of the buildings in the liquefied area tilted more than 1° [Fig. 4(*b*)]. The average settlement of the buildings relative to the ground surface is about 50 cm [Fig. 4(*c*)], and tends to increase with increasing number of stories [Fig. 4(*d*)].

Distribution of Damage to Buildings

Fig. 5 shows the distribution of the settlements relative to the ground surface of reinforced concrete buildings, and Fig. 6 shows the distribution of the damage to wooden houses. The damage to wooden houses is classified into one of four categories shown in the figure. Over two-thirds of the reinforced concrete buildings suffered severe damage on the west side of the Pantal river, whereas the buildings on and to the west of Galvan Street from Jovellaros Street to Gomez Street remained intact (Fig. 5). Many of the reinforced concrete buildings on Perez Boulevard, Fernandez Street, and Rizal Street between A. B. Fernandez Avenue and Perez Boulevard settled and tilted considerably, whereas those on A. B. Fernandez Avenue generally suffered smaller settlement and tilting. The damage to superstructure was, however, significant in many buildings on A. B. Fernandez Avenue. Several reinforced concrete buildings on Don Jose Street and Zamora Street south of Perez Boulevard suffered large settlement and tilting. Wooden houses in this area were also badly damaged (Fig. 6). The ground surface along Don Jose Street appeared to have settled as much as 50 cm, and had been inundated until a dry season. The liquefaction-induced damage was obvious within a limited area on the east side of the river. Only a limited number of the reinforced concrete buildings in the lateral spreading zone

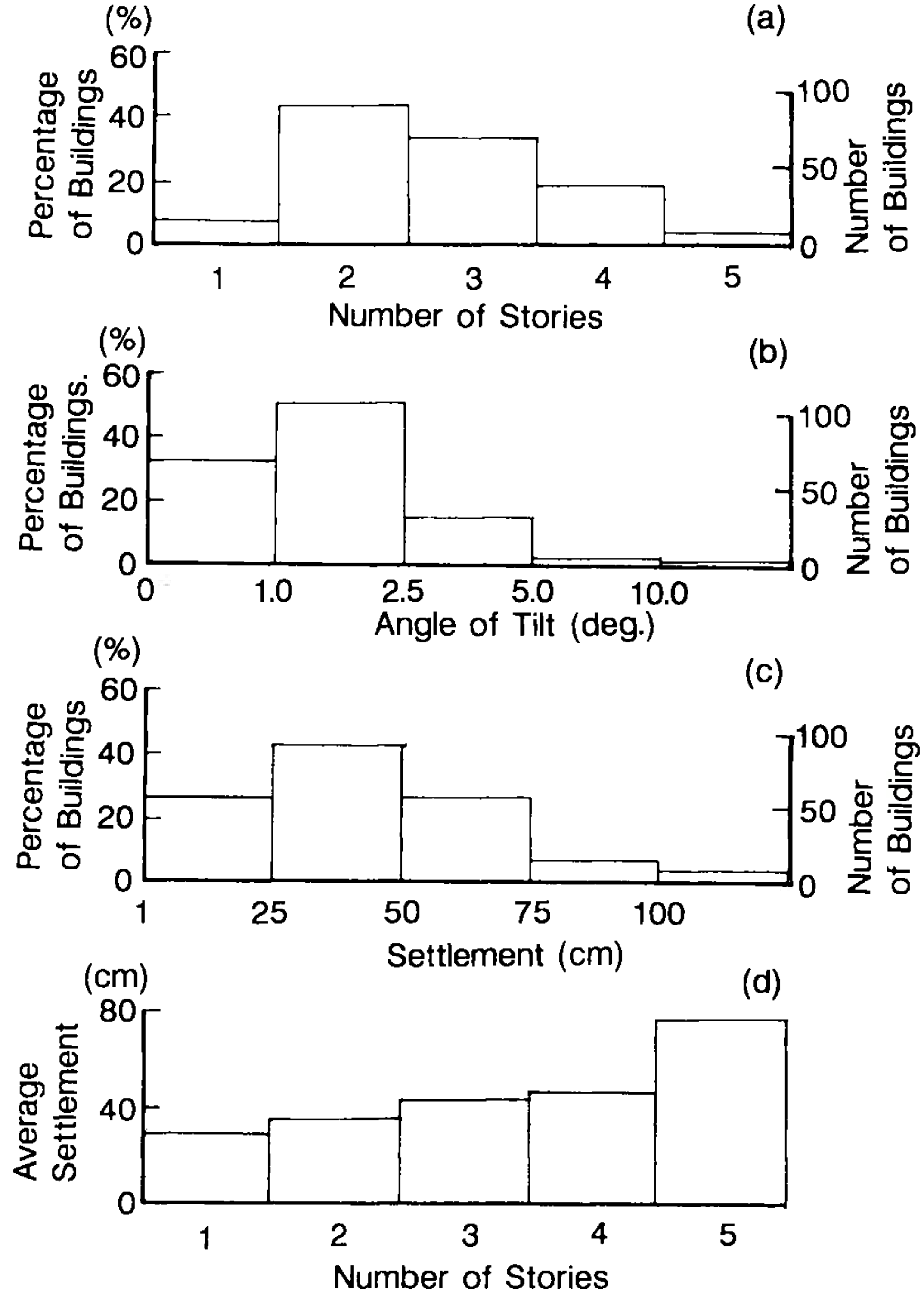

FIG. 4. Damage Statistics of Reinforced Concrete Buildings in Dagupan

suffered large settlement and tilting. The damage to buildings outside the lateral spreading zone appeared light to moderate.

Damage Patterns of Buildings

In the liquefied area, relatively new reinforced concrete buildings that have continuous foundations or mat foundations appear to have settled or tilted with their superstructures remaining intact or with little damage. However, if their floor slabs are unreinforced, they were heaved and broken due to foundation settlement. Extensive damage to superstructure was apparent in many buildings that have individual shallow footings without tie beams

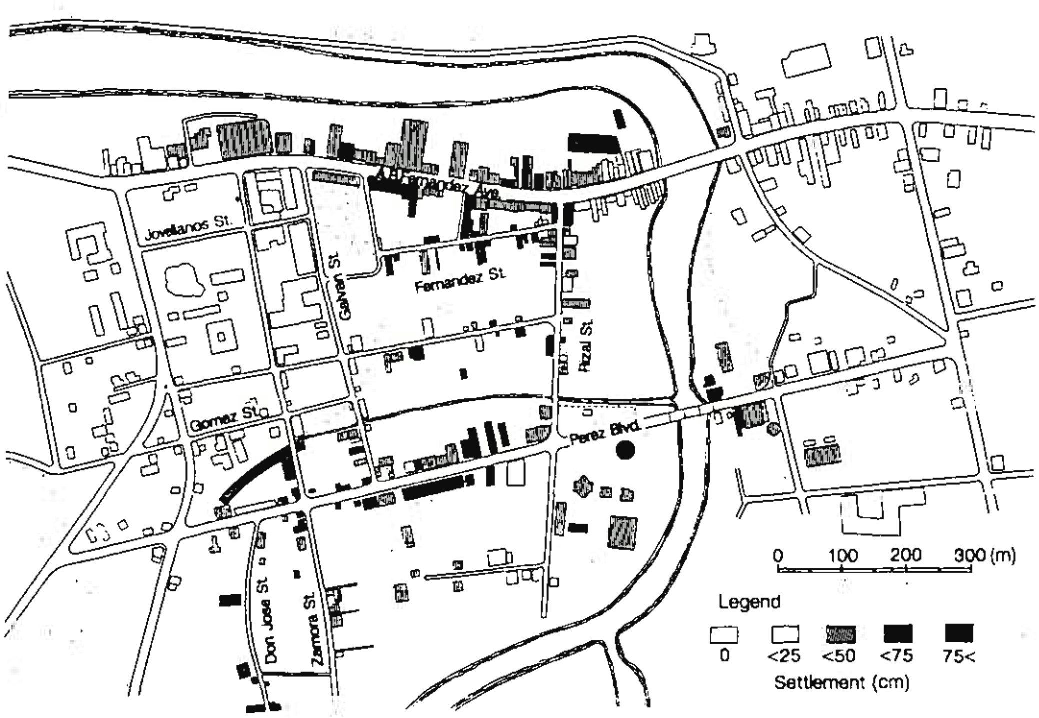

FIG. 5. Map Showing Settlement of Reinforced Concrete Buildings

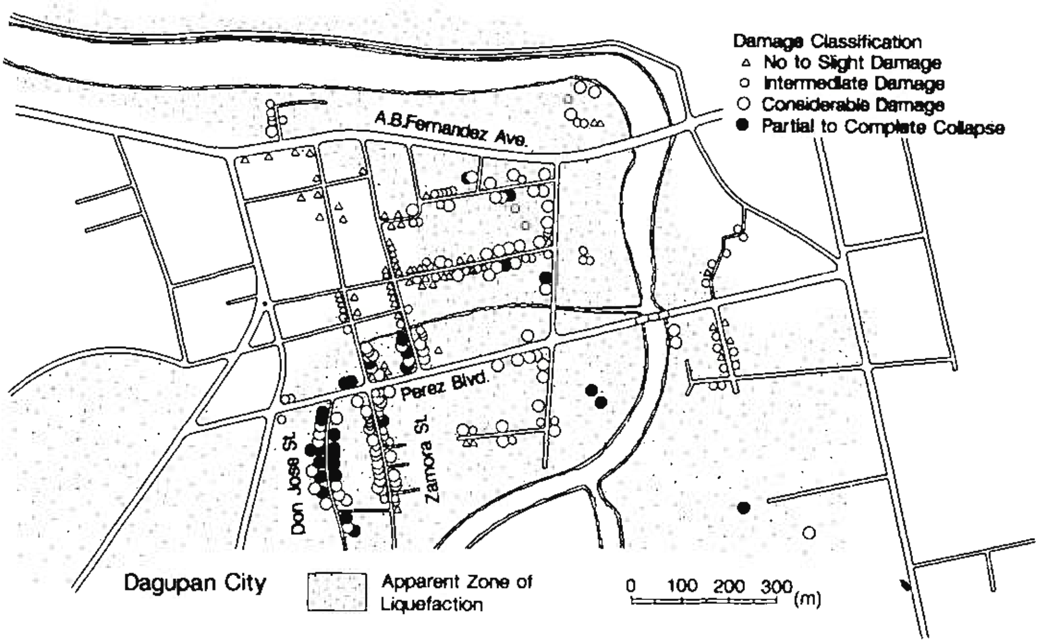

FIG. 6. Map Showing Distribution of Wooden Houses

or, if any, with beams of low rigidity. In these buildings, many exterior columns on the first story failed. Probably, the building settlement followed by the lateral ground spreading under the building, compelled the exterior column footings to move outward. The weak foundation is also the major cause of the differential settlement of buildings, which induced even the collapse of entire superstructure. These types of failure were particularly noted in many of old buildings on A. B. Fernandez Avenue. In the zone of lateral spreading, superstructures of many old buildings were extensively damaged as their weak foundation was elongated in the direction perpendicular to the course of the river.

Damage induced by structure-structure interaction was also observed in many places. Significant tilting of a massive building caused vital damage to an adjacent building (Fig. 3), and large settlements of massive buildings resulted in differential settlement and associated failure of adjacent lighter buildings. Much of obvious damage to wooden houses was associated either with large settlement, differential settlement, or differential lateral movement of their foundations. The areas around many wooden houses in the extensively liquefied zone and in the lateral spreading zone were inundated due to ground subsidence. Many wooden houses in the lateral spreading zone were also destroyed due to differential lateral movement or differential settlement of their foundations.

CORRELATION OF BUILDING DAMAGE WITH SOIL CONDITIONS

Geomorphological Conditions

Most part of the liquefied area is considered man-made sandy fills and/or alluvial deposits of sandy soil that overlie a clay layer at a depth below 10–15 m. The fills were built with fine sand after 1900 over recently abandoned river channels, fishponds, or swampy lands. As stated previously, the reclamation was initiated in the very early 1900s along A. B. Fernandez Avenue and in the late 1940s along Perez Boulevard.

Fig. 7 shows the locations of recently abandoned river channels (Punongbayan and Torres 1990), fishponds, and swamps, all of which can be detected from aerial photographs taken in between 1966 and 1974. Much of the north side of Perez Boulevard was still either swamp or fishpond then. Superimposed on the figure are the locations of the buildings that settled more than 75 cm and/or tilted more than 2.5°. Most of the buildings concentrate in the recently abandoned river channels and in the banks of the active river. Several buildings on the north side of Perez Boulevard that settled and tilted considerably are found to have been located within the old fishponds. It is also found that the extensive damage to wooden houses and ground settlement along Don Jose Street are attributed to the fact that these areas are situated on fills in recently abandoned river channels. These findings confirm that geomorphological conditions have significant effects

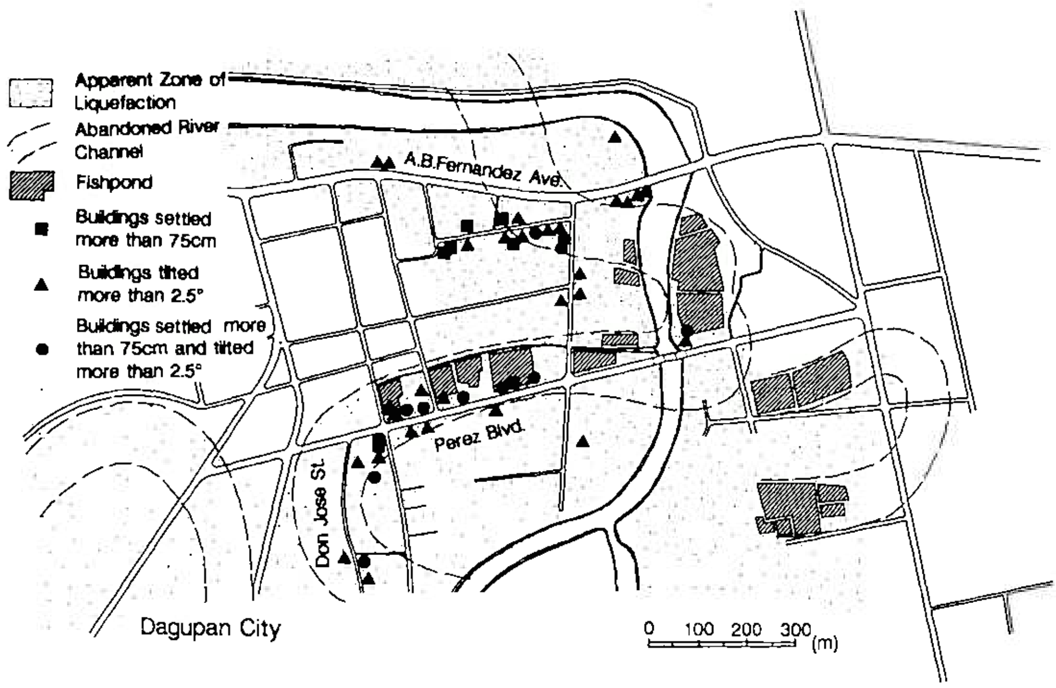

FIG. 7. Map Showing Effects of Geomorphological Conditions on Damage to Buildings

on liquefaction-induced damage during earthquakes [e.g., Youd and Bennett (1983), Youd (1991)].

Standard Penetration Resistance

The standard penetration tests (SPT) were conducted in the city by the Department of Public Works and Highways before and after the earthquake. The location of the test sites is shown in Fig. 2. The boring logs along A. B. Fernandez Avenue (sites 13, 4, 5, 10, and 15) and Perez Boulevard (sites 12, 2, 3, 1, 11, and 16) are shown in Figs. 8 and 9. Also shown in the figures are SPT *N*-values and fines contents (FC). The SPT *N*-values at site 1 were obtained before the earthquake, while those at the other sites were determined within three months after the earthquake. In the affected area, where the reinforced concrete buildings settled more than 25 cm, the surface layer to a depth of 5–10 meters is predominantly sandy soil with SPT *N*-values less than 30. Underlying the surface layer is a layer of dense sand or stiff clay. In the unaffected area, in contrast, a layer of clay and/or dense sand

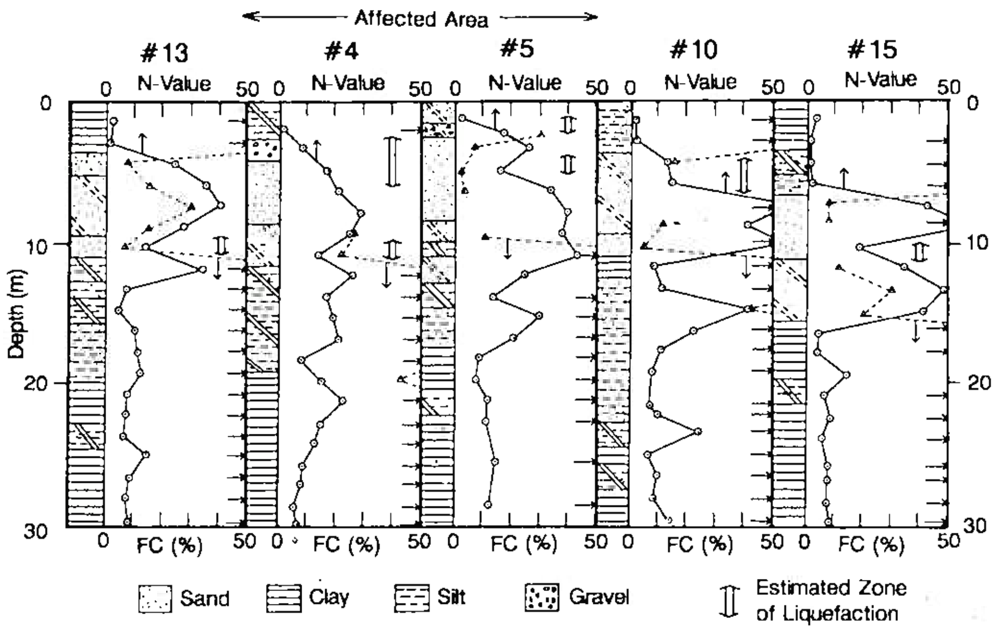

FIG. 8. Boring Logs along A. B. Fernandez Avenue

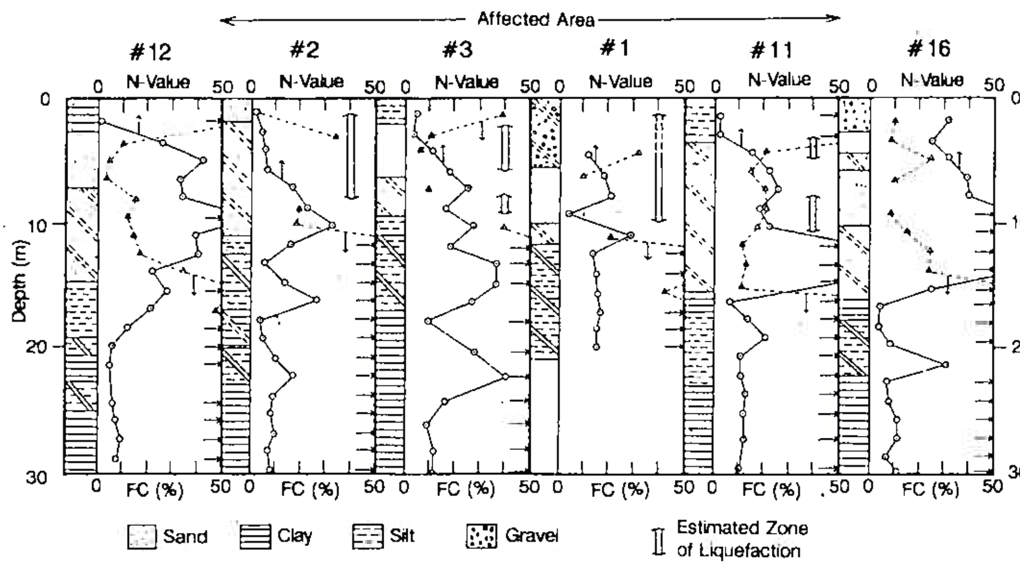

FIG. 9. Boring Logs along Perez Boulevard

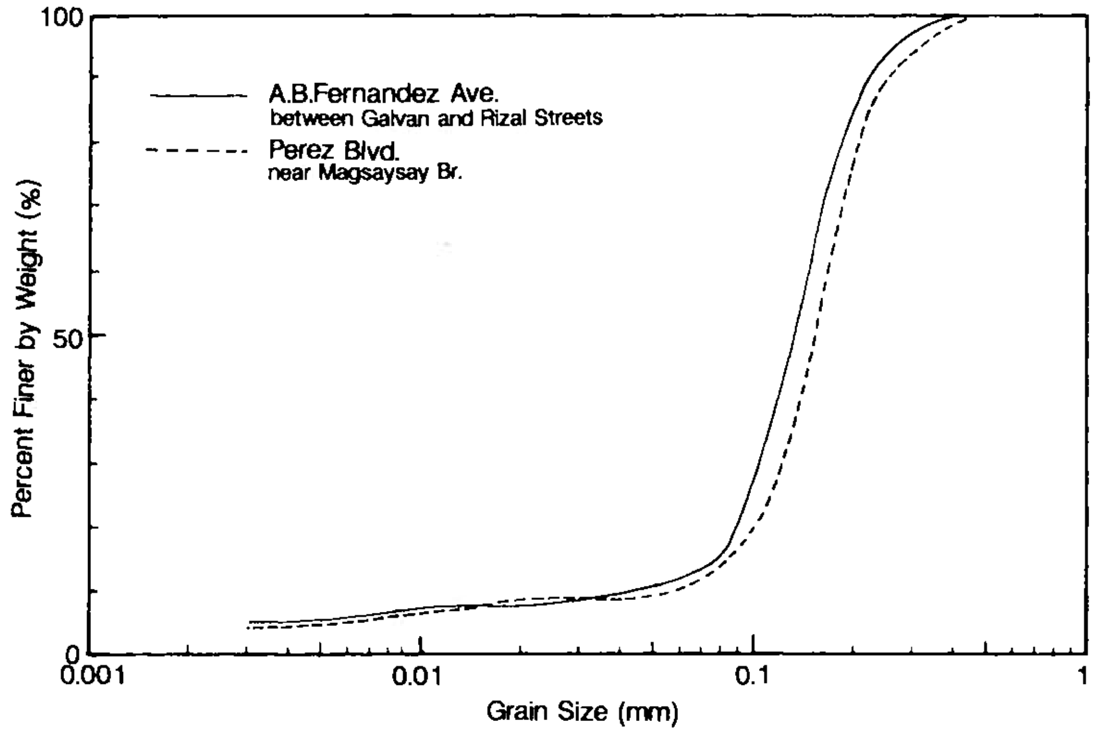

FIG. 10. Grain Size Distribution Curves of Sands

with SPT N-values greater than 30 exists from the ground surface. The grain size distribution curves for the boiled sands obtained in the city are shown in Fig. 10. The results of sieve analysis suggest that the liquefied sands are fine to silty sands having mean grain sizes of 0.1–0.2 mm. Thus, a part of the surface layer of fine to silty sands is considered to have liquefied, though their SPT N-values are somewhat higher than that generally expected for these sands [e.g., Seed et al. (1985)].

It appears common SPT practice in the Philippines to raise the doughnut hammer by means of a wire rope wrapped many times around a winch and to release it by letting out the clutch of the winch. The SPT machines used were imported from either the U.S. or Japan. On the basis of velocity measurements of the hammer at the impact for a Japanese machine, it was estimated that the aforementioned method would produce an energy efficiency somewhere between 35 and 45%, which is considerably smaller than the value recommended by Seed et al. (1985) as an international standard. A similar result has been presented by Orense et al. (1991). It seems therefore that the low energy efficiency delivered to the rods is the major cause of the unexpectedly high N-values for the liquefied sands. It is worthwhile to investigate whether the liquefaction damage to buildings might have been expected using currently available SPT-based correlation, since such a correlation is based on a limited number of well-documented cases involving building damage. Thus, the liquefaction potential for the boring logs shown in Figs. 8 and 9 was estimated using the empirical correlations proposed by Seed et al. (1985) and Tokimatsu and Yoshimi (1983), assuming that the SPT N-values measured after the earthquake do not differ significantly from those measured before the earthquake. Before using each empirical chart, correction has been made to SPT N-values to accommodate the difference in energy efficiency using the following (Seed et al. 1985):

$$N_{60} = N_p \left(\frac{ER_p}{60} \right) \qquad (1)$$

$$N_j = N_p \left(\frac{ER_p}{ER_j} \right) \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (2)$$

in which N = SPT N-value, N_{60} = corrected SPT N-value to an energy efficiency of 60%, ER = rod energy efficiency, subscripts j and p indicate the methods used in Japan and the Philippines, and $(ER_p/60) = 0.6$ and $(ER_p/ER_j) = 0.5$ were assumed. The rod length correction suggested by Seed et al. (1985) was made to SPT N-values. The cyclic stress ratios developed in the deposit at each site during the earthquake were also estimated, assuming the peak horizontal ground surface acceleration of 1.8 m/s^2. The stress ratios have been slightly adjusted to correspond to a magnitude of 7.5, by dividing them by the correction factors suggested by Seed et al. (1985).

Fig. 11 shows correlations between estimated cyclic stress ratio and SPT $(N_1)_{60}$ at critical depths of all sites. Sites known to have liquefied are plotted

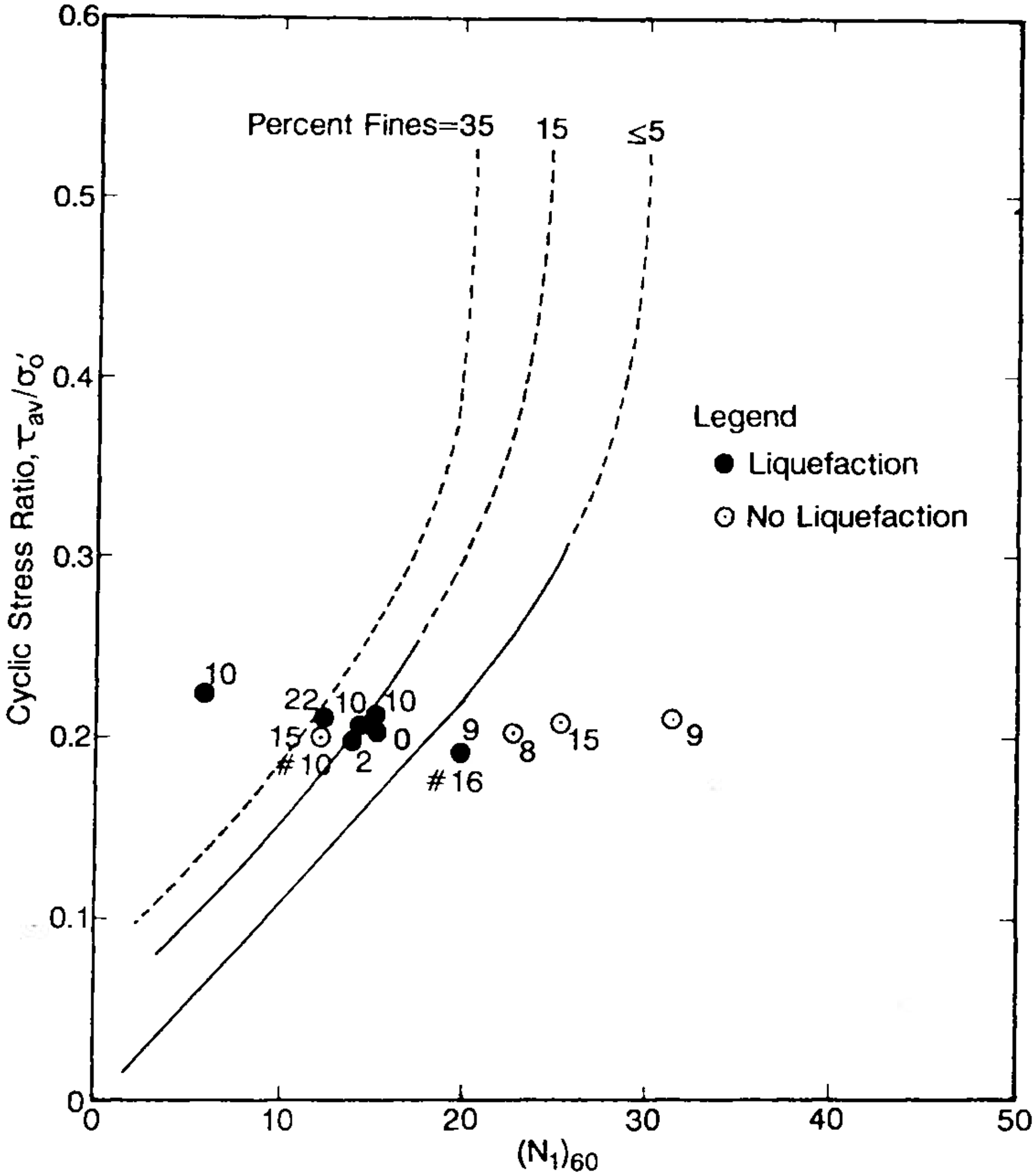

FIG. 11. Chart for Evaluation of Liquefaction Potential with Field Data from 1990 Luzon Earthquake (Numbers without # by Data Points Indicate Fines Content, and Numbers with # Indicate Site Number)

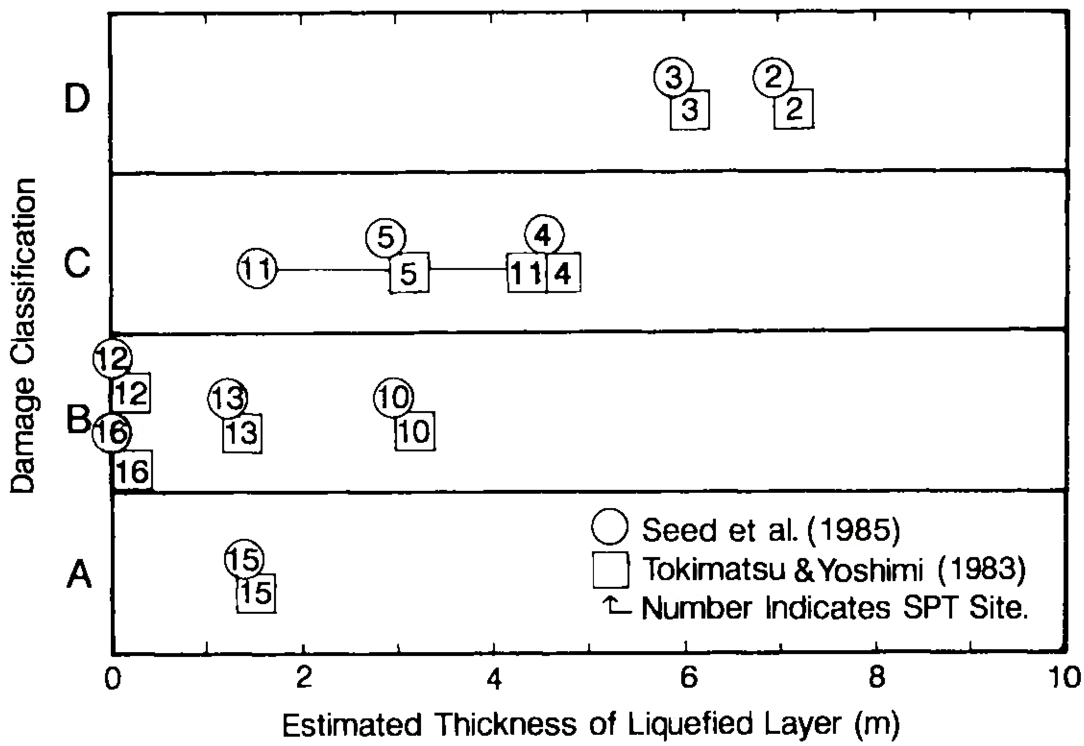

FIG. 12. Correlation of Estimated Thickness of Liquefied Layer with Damage Classification of Buildings

as solid circles, and sites that did not show any evidence of liquefaction are plotted as open circles. The number without # beside each symbol indicates fines content. Superimposed on the figure is the empirical chart developed by Seed et al. (1985), which separates liquefiable from nonliquefiable conditions. The field data are in fairly good accord with the chart, except two sites (10 and 16), which are located near the boundary separating liquefied from nonliquefied area, as shown in Fig. 2.

To investigate the effectiveness of the empirical chart in more detail, the layers that are estimated to have liquefied from at least one of the two methods, are indicated in Figs. 8 and 9. In the extensively affected area along the Perez Boulevard (sites 2, 3, and 1), almost all sand layers below the ground-water table to a depth of about 10 m are estimated to have liquefied, whereas only a part of the sand layers near the ground surface are estimated to have liquefied in the other affected area (sites 4, 5, and 11). This is consistent with the field observation that the structures along Perez Boulevard on the west side of the river settled and tilted more than the structures in the other liquefied area. The soil in the zone that experienced little or no damage (sites 10, 12, 13, 15, and 16) is estimated to have not liquefied or slightly liquefied, which also shows good agreement with the field observation. To confirm the aforementioned findings, the estimated thickness of liquefied layer at each site is plotted in Fig. 12 against the extent of damage to buildings at the site. The extent of damage is classified into one of four categories from A to D. Category A correspond to a site in the nonliquefied area with no structural damage, category B to a site near the boundary separating liquefied from nonliquefied zone with no structural damage, and categories C and D to sites in the liquefied area with considerable structural damage (average settlements of reinforced concrete buildings are less than 50 cm) and with vital structural damage (average settlements of reinforced concrete buildings are greater than 50 cm), respectively. At sites in categories A and B, the thickness of liquefied layer is less than 3 m. In contrast, at sites in categories C and D except site 11, it is larger than 3. Besides, the thickness of liquefied layer tends to increase with increasing extent of damage to buildings. Thus, the empirical correlations using SPT N-values appear to be able to estimate not only the occurrence of liquefaction but also the extent of resulting damage, with a reasonable degree of reliability.

Shear Wave Velocity

Shear wave velocity (V_s) profiles of surface soil across the city were determined by a Rayleigh wave method (Tokimatsu et al. 1992). Stokoe and Nazarian (1985) conducted similar field investigation at several sites in the Imperial Valley, using the SASW method. The principles of these methods lie in the facts that, once a Rayleigh wave dispersion curve for a site is obtained in any way, an inverse analysis of that dispersion curve results in a V_s profile of the site. Unlike the SASW method, the method used in

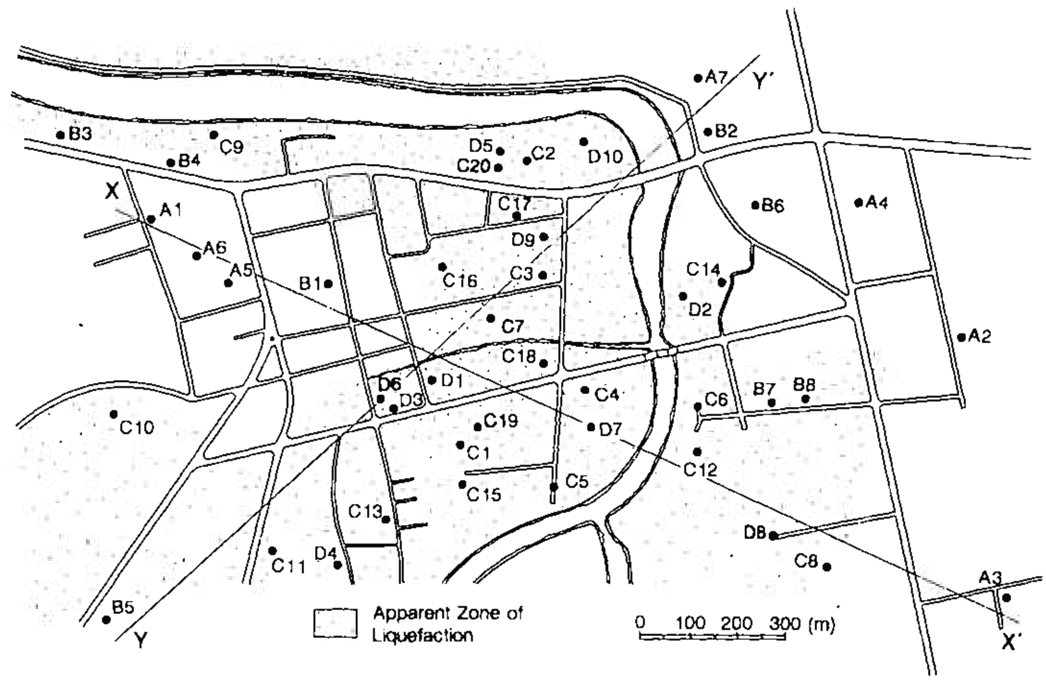

FIG. 13. Locations of Sites for Rayleigh Wave Investigation

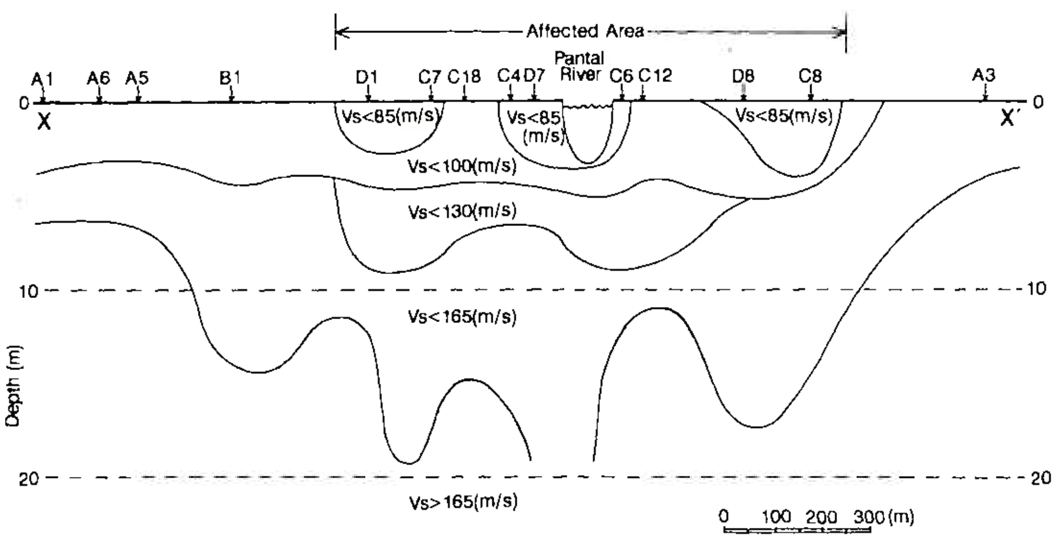

FIG. 14. Cross Section of Shear Wave Velocity Profiles along X-X' Line

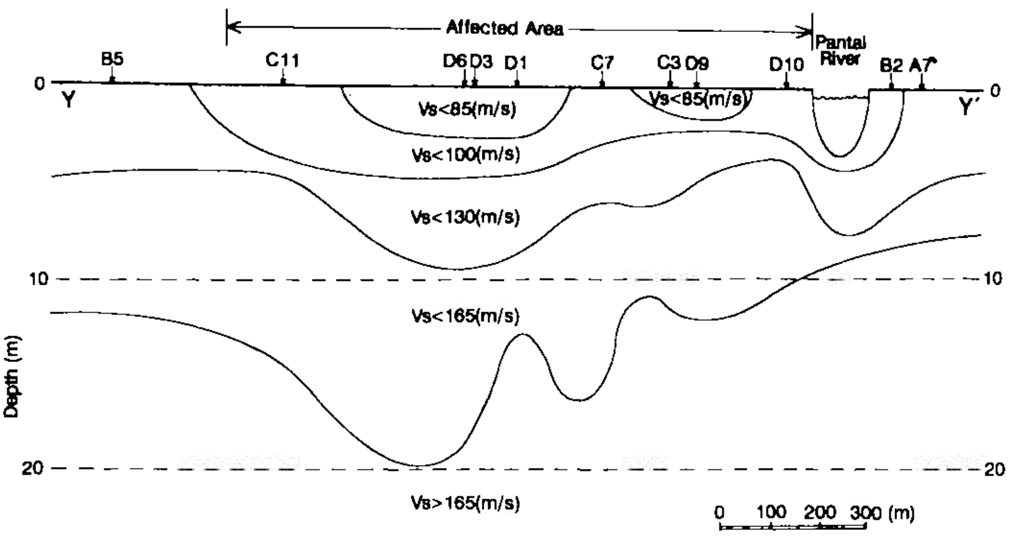

FIG. 15. Cross Section of Shear Wave Velocity Profiles along Y-Y' Line

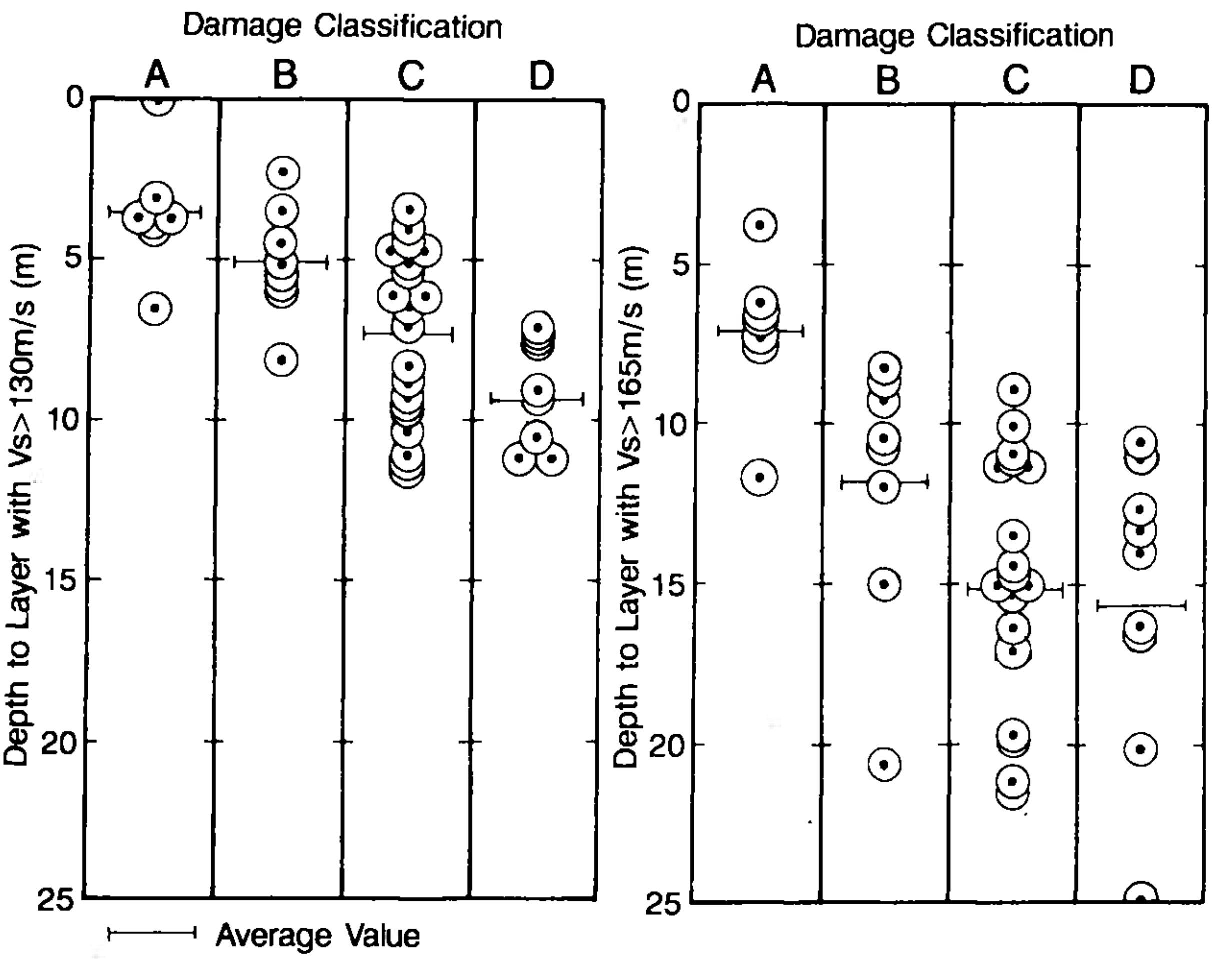

FIG. 16. Depths to Layers with V_s > 130 and 165 m/s

Dagupan mainly observes Rayleigh waves in short-period microtremors. The test procedure and test equipment used are the same as those reported by Tokimatsu et al. (1992). The test equipment is portable as it consists only of sensors, amplifiers, and a laptop computer. Besides, the test can readily be run on the ground surface by measuring microtremors with a small array of sensors. This enables one to conduct a large number of tests within a short period. The tests were conducted at 45 sites that are distributed over liquefied and nonliquefied zones as shown in Fig. 13. The extent of damage to structures at each site is classified into four categories from A to D cited previously, and the site is labeled accordingly. Figs. 14 and 15 show cross sections of V_s structure along X-X' and Y-Y' lines shown in Fig. 13. The V_s profiles in the affected area contrast well with those in the unaffected area in the following manners. The shear wave velocity of the top layer is less than 85 m/s in the severely affected area, and more than 85 m/s in the other area. Comparison of Figs. 13 and 14 with Fig. 7 further reveals that the soft surface layer with V_s less than 85 m/s occurs only near the active river and on recently abandoned river channels. In much of the affected area, the layer with V_s less than 130 m/s exists to a depth more than 5 m. In the unaffected area, in contrast, such a layer with V_s less than 130 m/s exists generally at depths smaller than about 5 m. In the affected area, a stiff layer with V_s greater than 165 m/s occurs generally at depths greater than 12 m but in the unaffected area at depths less than 8 m. A similar tendency can be seen at all test sites as summarized in Fig. 16, in which both the depths to layers with $V_s \geq 130$ and ≥ 165 m/s tend to increase as the extent of damage to buildings increases.

Comparison of Figs. 13 and 16 with Figs. 8 and 9 suggests that the layer with V_s less than 130 m/s corresponds to either the surface clay layer or the sand layer with SPT N-values less than 30. The layers with V_s greater than 165 m/s are likely to correspond to the clay layer that underlies the sand layer and/or the sand layer with SPT N-values greater than 30. The reasonably good agreement of the V_s profiles with the available information suggests that the Rayleigh wave method used shows capability in characterizing two- or three-dimensional V_s structures of near-surface soils.

Recent studies have shown that the shear wave velocity may be used as an indicator for liquefaction evaluation (Stokoe and Nazarian 1985; Stokoe et al. 1988; Tokimatsu et al. 1991b; Robertson et al. 1992). For example, Tokimatsu et al. (1991b) presented a chart separating liquefiable from nonliquefiable conditions for silty sands as shown in Fig. 17 in which the stress ratios to cause liquefaction in the triaxial test and in the field $(\sigma_d/2\sigma_c)_{tri}$ and $(\tau/\sigma'_v)_{field}$, for a magnitude 7.5 earthquake, is plotted against normalized shear wave velocity given by:

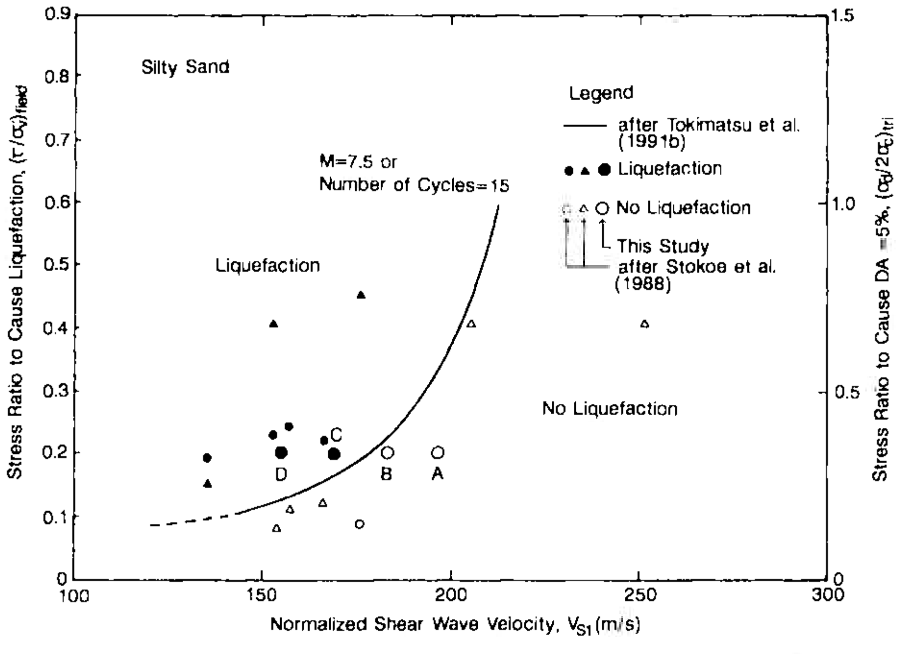

FIG. 17. Relation between Stress Ratio Causing Liquefaction and Normalized Shear Wave Velocity

$$V_{s1} = \frac{V_s}{(\sigma'_m)^n} \quad \text{(3)}$$

in which σ'_m = mean effective stress and n = 1/3.

From Figs. 13–16, representative values of shear wave velocities at a critical depth of 5 m are 140, 130, 120, and 110 m/s for categories A to D sites, respectively. Based on these values, the relation between the normalized shear wave velocity and the stress ratio corresponding to $M = 7.5$ conditions at the critical depth of four category sites were estimated, and are plotted in Fig. 17. Also shown in the figure are the data from the 1979 Imperial Valley ($M_s = 6.5$) and 1981 Westmorland ($M_s = 5.6$) earthquakes (Stokoe et al. 1988) for which correction has been also made to the stress ratio to allow for the difference in earthquake magnitude. The proposed chart is consistent with the field observation, indicating its potential usefulness, though further field verification is evidently needed.

CONCLUDING REMARKS

The effects of soil conditions on the damage patterns of buildings in Dagupan City during the Luzon Earthquake of July 16, 1990 ($M_s = 7.8$), have been extensively reviewed. The buildings that suffered large settlement and tilting have been found to be concentrated in the banks of active rivers and in fills built on recently abandoned river channels. The currently available empirical criteria have performed well in estimating not only the occurrence of liquefaction but also the extent of resulting building damage in the city, when appropriate corrections are made to allow for the low energy efficiency delivered to the rods in the Philippines. It has been also found that there is a good correlation between shear wave velocity of the surface soil and the extent of damage to buildings.

Although much of the damage to buildings in Dagupan City was similar to that observed in Niigata City during the 1964 Niigata earthquake, the following types of damage should be kept in mind: (1) Damage significantly affected by geomorphological conditions; (2) damage caused by lateral ground spreading and/or differential lateral ground displacement; and (3) damage amplified by structure-structure interaction. Further research appears to be needed for developing a method to identify an area that is vulnerable to lateral ground spreading, sliding, and significant subsidence due to soil liquefaction. For this purpose, the Rayleigh wave method appears promising as it shows capability in characterizing two- or three-dimensional V_s structure in the near surface soil.

ACKNOWLEDGMENTS

The writers wish to express their sincere appreciation to Messrs. S. Fukumoto, S. Tamura, and J. Yamashita for their assistance and cooperation during and after the field survey. The writers also wish to express their sincere thanks to Dr. R. Punongbayan, Director of Philippine Institute of Volcanology and Seismology (PHIVOLCS) for providing and showing valuable data including the aerial photographs, and Mr. Macaranas, engineer of PHIVOLCS, for his cooperation in the field survey. The Department of Public Works and Highways, Republic of the Philippines, kindly provided the results of SPT tests made in Dagupan. This research is partly supported by a research grant provided by the Japanese Ministry of Education, Science and Culture.

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APPENDIX II. NOTATIONS

The following symbols are used in this paper:

ER_j = typical rod energy efficiencies in Japan;
ER_p = typical rod energy efficiencies in Philippines;
FC = fines content;
N_{60} = corrected SPT N-value to energy efficiency of 60%;
N_j = SPT N-value obtained in Japan;

N_p = SPT N-value obtained in Philippines;
n = constant;
V_s = shear wave velocity;
V_{s1} = normalized shear wave velocity;
σ'_m = mean effective stress;
$(\sigma_d/2\sigma_c)_{tri}$ = stress ratio to cause double amplitude axial strain of 5% in triaxial test; and
$(\tau/\sigma'_v)_{field}$ = stress ratio to cause liquefaction in field.