NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

LIQUEFACTION POTENTIAL FOR NEW YORK STATE: A Preliminary Report on Sites in Manhattan and Buffalo

by

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Technical Report NCEER-87-0009

August 31, 1987

This research was conducted at the State University of New York at Buffalo and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NCEER-87-0009	2.	3. Recipient's Accession No. PB88 163704/AS

4. Title and Subtitle	5. Report Date
Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo	August 31, 1987
	6.
7. Author(s) M. Budhu, V. Vijayakumar, R.F. Giese, L. Baumgras	8. Performing Organization Rept. No.
9. Performing Organization Name and Address National Center for Earthquake Engineering Research State University of New York at Buffalo Red Jacket Quadrangle Buffalo, NY 14261	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) ECE-86-07591 86-1022 (G)
12. Sponsoring Organization Name and Address Same as box 9.	13. Type of Report & Period Covered Technical Report 14.

15. Supplementary Notes

This research was conducted at the State University of New York at Buffalo and was partially supported by the National Science Foundation.

16. Abstract (Limit: 200 words)

The chances of a major earthquake occurring in New York State in the near future is low. However, historically New York State has experienced and no doubt will continue to experience moderate earthquakes with a maximum Modified Mercalli intensity of VI. At these intensities, catastrophic damage due to structural causes is not likely to occur for non-masonry buildings, but ground failure due to soil liquefaction is possible. Liquefaction can cause substantial damage and losses as demonstrated by the Alaska earthquake of 1964 where 60% of the estimated $500 million total loss was related directly to ground failure. For this project, nearly 6,000 borehole logs from the two selected sites (upper Manhattan and downtown Buffalo) were collected, stored in a computer database and analyzed to determine the liquefaction potential of the soils lying within 50' of the ground surface. The procedures which were used for this analysis rely heavily on standard penetration test numbers and the ground water elevation in the hole. Liquefiable soils are typically loose water saturated cohesionless materials. It appears from this study that, for the design earthquake described above, areas which lie adjacent to bodies of water (the Harlem River in Manhattan and Lake Erie in Buffalo) are liable to liquefy.

17. Document

b. Identifiers/Open-Ended Terms

LIQUEFACTION
NEW YOUR STATE - SEISMICITY
LIQUEFACTION POTENTIAL

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release unlimited	unclassified	46
	20. Security Class (This Page) unclassified	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

LIQUEFACTION POTENTIAL FOR NEW YORK STATE:

A PRELIMINARY REPORT ON SITES IN MANHATTAN AND BUFFALO

by

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August 31, 1987

Technical Report NCEER-87-0009

NCEER Contract Number 86-1022

NSF Master Contract Number ECE 86-07591

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ABSTRACT

The chances of a major earthquake occurring in New York State in the near future is low. However, historically New York State has experienced and no doubt will continue to experience moderate earthquakes with a maximum Modified Mercalli intensity of VI. At these intensities, catastrophic damage due to structural causes is not likely to occur for non-masonry buildings, but ground failure due to soil liquefaction is possible. Liquefaction can cause substantial damage and losses as demonstrated by the Alaska earthquake of 1964 where 60% of the estimated $500 million total loss was related directly to ground failure.

In this report, liquefaction potential maps for two urban areas - a region in upper Manhattan, New York City, and a portion of downtown Buffalo, New York - are presented. These maps show the probability of liquefaction for ground shaking induced by an earthquake of magnitude 7.5 and a peak ground acceleration of 0.15 *g*. Such maps are invaluable for earthquake hazard mitigation projects, especially in questions of land use regulation and in preliminary emergency response planning.

For this project, nearly 6,000 borehole logs from the two selected sites were collected, stored in a computer data base and analyzed to determine the liquefaction potential of the soils lying within 50' of the ground surface. The procedures which were used for this analysis rely heavily on standard penetration test numbers and the ground water elevation in the hole. Liquefiable soils are typically loose water saturated cohesionless materials.

It appears from this study that, for the design earthquake described above, areas which lie adjacent to bodies of water (the Harlem River in Manhattan and Lake Erie in Buffalo) are liable to liquefy.

Although the procedures used, in particular the standard penetration test, are open to question, the results of this study clearly indicate those areas in Manhattan and Buffalo where there exists a real possibility for liquefaction. Based on this study, suitable emergency response programs should be instituted.

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SECTION 1

INTRODUCTION

1.1 General

The probability of a major earthquake occurring in or near New York State is small. However, historical records indicate that parts of the State have experienced, and likely will continue to experience, moderate earthquakes with Modified Mercali intensities of VI or less. With an earthquake of this approximate magnitude, it is unlikely that catastrophic damage due to structural failures will be common, but ground failure resulting from liquefaction is probable - given the proper soil conditions and appropriate ground accelerations. Examination of historical earthquakes has dramatically demonstrated the damage that can result from liquefaction. A typical example is the 1964 Alaska earthquake where 60% of the estimated $500,000,000 damage was directly related to liquefaction.

The integrity of a structure, a transportation system, or a lifeline utility located in an area affected by an earthquake is challenged by two factors: the ability of the structure itself to withstand the seismic waves and the resistance of the environment about the structure to ground failure. It is clear that structures which are adequately designed to survive earthquakes can readily be damaged or destroyed if they rest on or within soils that liquefy. Thus, in designing earthquake resistant structures, it is also necessary to consider the geological and geotechnical characteristics of the site upon or in which the structure is to be situated. Little effort has been expended in identifying regions of the Eastern United States which are susceptible to this type of earthquake damage.

Clearly, there is more interest in earthquakes in those states lying to the west of the Rocky Mountains; there are many more earthquakes there than in the states lying to the east. But this difference, while true, tends to give the eastern US a false sense of security. The two regions of the United States are underlain by very different geological provinces. The eastern continental US is underlain by a cooler and relatively denser plate than is found in the west. This difference allows seismic waves from earthquakes to travel greater distances for a given attenuation in the east than in the west. Hence, a moderate to large earthquake in the east affects a much larger area than an equivalent earthquake in California. As the area which is shaken by an earthquake increases, the possibility of ground failure also increases.

In the western United States, the relation between geology and earthquakes is well established. The sites of major earthquakes are associated with fractures in the crust and the movement along these fractures is ultimately driven by the movements of the major plates. This simplifies the search for areas of potential ground failure. In contrast, earthquakes in the eastern United States are rarely directly related to known faults. The existence of fractures at depth is becoming clearer. But there is much about these deep and old fractures that is unknown and unpredictable. The uncertainty, therefore, puts much larger regions in the east at risk. In order to reduce this uncertainty somewhat, it is essential that those regions which have the potential for ground failure be identified. Once identified, it should be possible to design structural codes so that the damage due to the next large earthquake can be minimized.

1.2 Scope of Study

Three tasks were identified for the first year of this project. These were-

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Task 1. Collect subsurface geotechnical data from previous field investigations of two restricted sites. These were chosen to represent two types of environment in New York State - a large and a medium sized urban region. The sites which were investigated were in upper Manhattan (approximately 6 mi.2), New York City and the central part of Buffalo (approximately 11 mi.2).

Task 2. Develop liquefaction potential maps for these two regions, identifying those areas which have a high, moderate, or low probability of liquefaction for an assumed acceleration. The evaluation of the liquefaction potential for this initial study is based on data selected from the information assembled during Task 1.

Task 3. Design and implement a database system capable of storing the large quantities of geotechnical and geological data generated by Task 1. This database will ultimately provide the information necessary to accomplish Task 2.

SECTION 2
LIQUEFACTION EVALUATION

2.1 Liquefaction

Liquefaction is a phenomenon by which saturated soils - essentially cohesionless soils - are temporarily transformed into a liquefied state. In the process, the soil undergoes transient loss of strength which commonly allows ground displacement or ground failure to occur. Liquefaction in the context of this report implies liquefaction induced ground failure, because the assessment of liquefaction adopted for this project, was derived from observed ground failure data.

Three basic types of ground failure are associated with liquefaction.

1. flow failures - soil materials flowing rapidly downslope in a liquefied state.
2. lateral spreading - limited displacement of surface soil layers down mild slopes.
3. loss of bearing strength - bearing capacity failure of foundations because of weakening of underlying or adjacent soil material.

Liquefaction can also cause transient horizontal oscillations as the ground surface layers are decoupled from more rigid material underlying the liquefied layer. Such decoupled vibrations can cause severe damage to buildings, pipelines, and other structures.

Of specific interest to this proposal is the liquefaction of soils under stress conditions induced by an earthquake, either close by or at some distance. Past experience has shown that seven factors are important in determining the potential for a particular soil to liquefy. These are 1) age of the soil, 2) depth to groundwater, 3) the grain sizes present in the soil and their relative proportions, 4) density, 5) origin of the soil, 6) thickness of the soil, and 7) the acceleration experienced by the soil.

Age: Weak, cohesionless soils are typically young. With time, two factors act to increase the strength of a typical soil: compaction changes the void ratio, and various chemical processes act to cement the soil grains together. A rule of thumb is that deposits older than Late Pleistocene are unlikely to liquefy except under the most vigorous conditions, while Late Holocene deposits are most likely to liquefy (Youd and Hoose, 1977).

Groundwater: Ground water acts to reduce the bearing strength of a soil by decreasing the effective stress. Hence, the higher the ground water table, the more extensive will be this effect.

Grain Size: The type of soil most likely to liquefy is one which has a predominance of sand sized particles, with little in the way of fines or coarser grains; a good, clean sand. Silty sands and gravels are also susceptible to liquefaction under very severe cyclic loadings.

Density: Liquefaction occurs principally in loose, saturated, cohesionless soils. Such a soil will densify when subjected to cyclic loading. This tendency to densify results in an increase in pore water pressure if the intergranular pores are filled with water. When the pore water pressure becomes equal to the total mean stress, the soil loses its strength and liquefies. If the soil is dense to begin with, there will be a smaller opportunity for liquefaction.

Origin: Soils which are deposited by fluvial processes are deposited rapidly and have little chance to achieve a dense packing of grains. These will liquefy easily. On the other hand, glacially deposited sediments, particularly those over which the glacier has passed, are generally rather dense already, and are less likely to liquefy.

Thickness: To create substantial damage during liquefaction, the soil should be thick (for horizontal strata) so that the dimensional changes which occur will be substantial.

Ground Accelerations: The ability of a soil to withstand the accelerations due to an earthquake depends on the magnitude of the acceleration and on the duration of the shaking. These can only be estimated, in general, by looking at records of historical earthquakes.

2.2 Evaluation of Liquefaction Potential

Over the past decade or two, efforts have been made to establish a scientific basis for the prediction of liquefaction under earthquake conditions. From this work, two different approaches have developed: one based on essentially geological characteristics of the soils, and the other relies on geotechnical measurements of the soil.

2.2.1 Geological Criteria

Youd and Perkins (1978) developed a method for the construction of liquefaction potential maps. In essence, this is done by combining information from two different types of maps for the same region: a map showing the liquefaction opportunity, and a map showing the liquefaction susceptibility. The former is derived from the seismicity of the area; the latter is determined from the geologic age of soils and the depth to ground water. In theory, the greater the seismic activity, the younger the soil, and the closer the ground water is to the surface, the greater will be the liquefaction potential.

As a general rule, young detrital sediments (or soils in the engineers terminology) are weaker than older equivalent sediments. For example, sand deposited by a river or ocean currents initially lacks cohesion. With time, the strength of the sand is increased by a number of processes: compaction due to burial, the precipitation of mineral phases from intra-particle fluids, and the slow dissolution and reprecipitation of the material of the grains themselves. Typically, sediments which are most susceptible to liquefaction are late Holocene (1000 yrs or less), with early Holocene having moderately liquefiable, and late Pleistocene having a low probability of liquefaction. With the variety of processes which can take place in geological materials, it is to be expected that exceptions will be found.

Similarly, because water saturation is necessary for liquefaction, depth to the water table is important. A second factor enters here; the increase in the strength of a soil as it is buried deeper. For these reasons, the most easily liquefiable soils will be those which have the ground water lying within 10 ft. of the surface. As the depth to the water table increases, the likelihood of liquefaction decreases.

These criteria have the advantage that they are easily applied to a variety of geological terrains, and they require data which is normally already at hand from routine geotechnical borings. A weakness is the obvious one; they are strictly qualitative. To date, several regions, mostly in California, have been investigated and liquefaction potential maps for them have been published. These are-

Davis County, Utah	Anderson *et al.* (1982)
Los Angeles, CA	Tinsley *et al.* (1985)
San Diego, CA	Power *et al.* (1982) Idriss *et al.* (1982)
San Mateo County, CA	Youd and Perkins (1985)
San Fernando Valley, CA	Youd *et al.* (1978)
San Francisco, CA	Roth and Kavazanjian (1984)

2.2.2 Criteria Based on Standard Penetration Test (SPT) Data

The standard penetration test, though widely used, is acknowledged to be an extremely crude procedure. The name is a misnomer because there is nothing really standard about the way the test is performed, and the necessity of converting from one set of conditions to another is fraught with uncertainty, even though there are conversion factors available in the literature.

In the United States, there has been an attempt to codify the conditions under which the test is performed, and these are described by ASTM D1587-67. The existence of an ASTM standard has not in the slightest prevented the continued use of a proliferation of different tests in the field. According to the standards, the test consists of driving an 18" split tube (2" outside diameter with a wall thickness of 5/8") into a soil by repeated blows from a 140 lb hammer falling onto an anvil attached to the top drill rod, from a distance of 30". The number of hammer blows for each 6" of penetration by the tube is recorded. The standard penetration number, N, is the number of blows required to penetrate the last 12" (of the total of 18"). This N-value is sensitive to a number of factors which can vary from site to site and even within the same site: the overburden pressure, lateral soil pressure, and the density of the soil. It is now a common practice to correct the N-value for these variables to yield a presumably unbiased number, $(N)_{60}$, which is equivalent to a hammer energy ratio of 60% (Seed *et al.*, 1984).

Many variations of the SPT test have been used, particularly in the past. Variations occur principally in the size of the split tube, the drill rods, the driving technique, the weight and type of hammer, and the height of the drop. Studies were conducted (for example, Lowe and Zaccheo, 1975) to correct the N-values for these non-standard tests to the ASTM D1587-67. We will refer to this standard in this report. Lowe and Zaccheo (1975) proposed a relationship which accounts for variations in hammer weight, drop height, and the diameters of the split tube. Based on an extended series of field tests, they showed that there is a linear relationship between hammer ratio, R_s, and the N-value. The hammer ratio is defined as:

$$R_s = \frac{D_0^3 - D_i^3}{144WH} \tag{1}$$

where D_o and D_i are the external and internal diameters of the split tube (in inches)
W is the weight of the hammer (in lbs)
H is the drop height (in inches)

This equation does not consider the dynamics of the hammer. It appears from the data reported by Lowe and Zaccheo (1975) that for cohesionless soils at relative densities below 50% (which is appropriate for liquefaction studies) the following conversion is sufficiently accurate for practical purposes.

$$N_e = 4050 N R_s^{5/7} \tag{2}$$

where N and N_e are the actual and corrected standard values respectively.

2.2.3 Liquefaction Potential

Seed *et al.* (1984) proposed a method of evaluating the liquefaction potential of soils on the basis of field observations. They showed that for sites in Pan America, Japan, and China where soils were observed to liquefy during earthquakes, the liquefaction potential of clean sands and non-plastic silt is related to the $(N)_{60}$ value.

Their procedure for evaluating the liquefaction potential is:

1. Establish the soil conditions and choose a design earthquake.
2. Compute the average shear stress ratio τ_{av}/σ_o (where τ_{av} is the average shear stress and σ_o is the effective overburden stress) for the maximum acceleration (a_{max}) of the design earthquake from the relationship-
$$\tau_{av}/\sigma_o = 0.65(a_{max}/g)(\sigma_o'/\sigma_o)r_d$$

(3)

where g is the acceleration due to gravity, σ_o^t is the total overburden stress, and r_d is a stress reduction coefficient which varies from a value of 1.0 at the surface to about 0.9 at approximately 35' depth.

3. Correct the N_e value to N_{60} using the equation-

$$N_{60} = C_n N_e$$

(4)

where C_n is a correction factor equal to $\sigma_o^{-1/2}$ (Liao and Whitman, 1985).

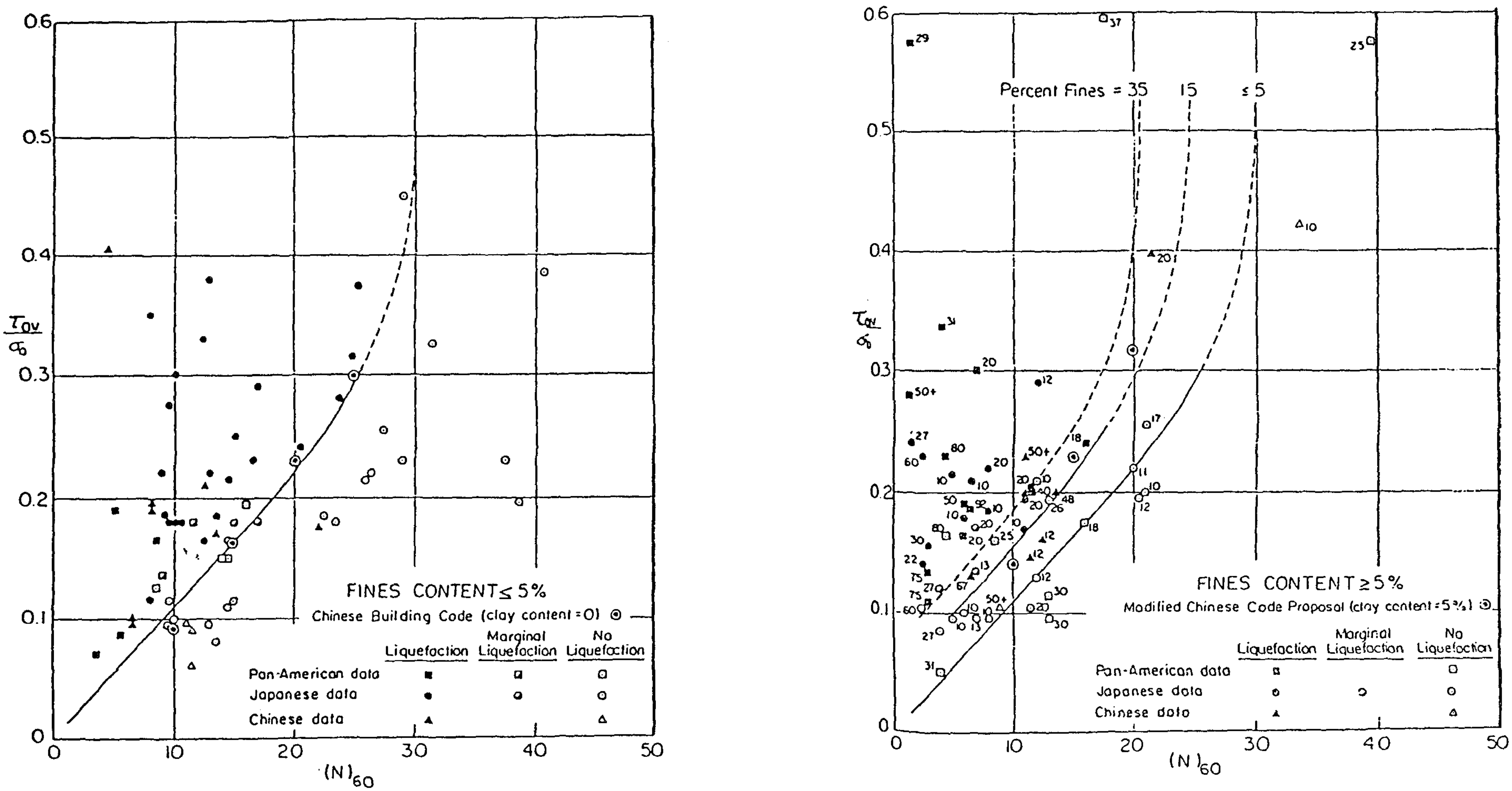
4. The presence of fine particles in a sandy soil strongly influences the liquefaction potential. The possibility of liquefaction for a given fines content, N_{60}, and τ_{av}/σ_o is determined according to scheme shown in Fig. 1. If the coordinates (N_{60}, τ_{av}/σ_o) lie above the appropriate fines-content curve, the soil will liquefy under an earthquake of magnitude 7.5. Correction factors have been suggested by Seed *et al.* (1983) to yield curves for earthquakes with magnitudes other than 7.5.

The procedure does not account for the duration of shaking during the earthquake.

2.2.4 Liquefaction Probability

Liao *et al.* (1987) proposed a probabilistic method to evaluate the liquefaction potential using a regression analysis of SPT data from 278 sites where liquefaction was observed to occur. They worked with two models: one, termed the **CSR** (for cyclic stress ratio) model (Fig. 2), is similar to that of Seed *et al.* (1984); the other, termed the **source model**, makes use of earthquake load parameters, viz magnitude and epicentral distance. The liquefaction demarcation curve of Seed *et al.* (1984) corresponds approximately to a 50% probability for the occurrence of liquefaction as formulated by the CSR model.

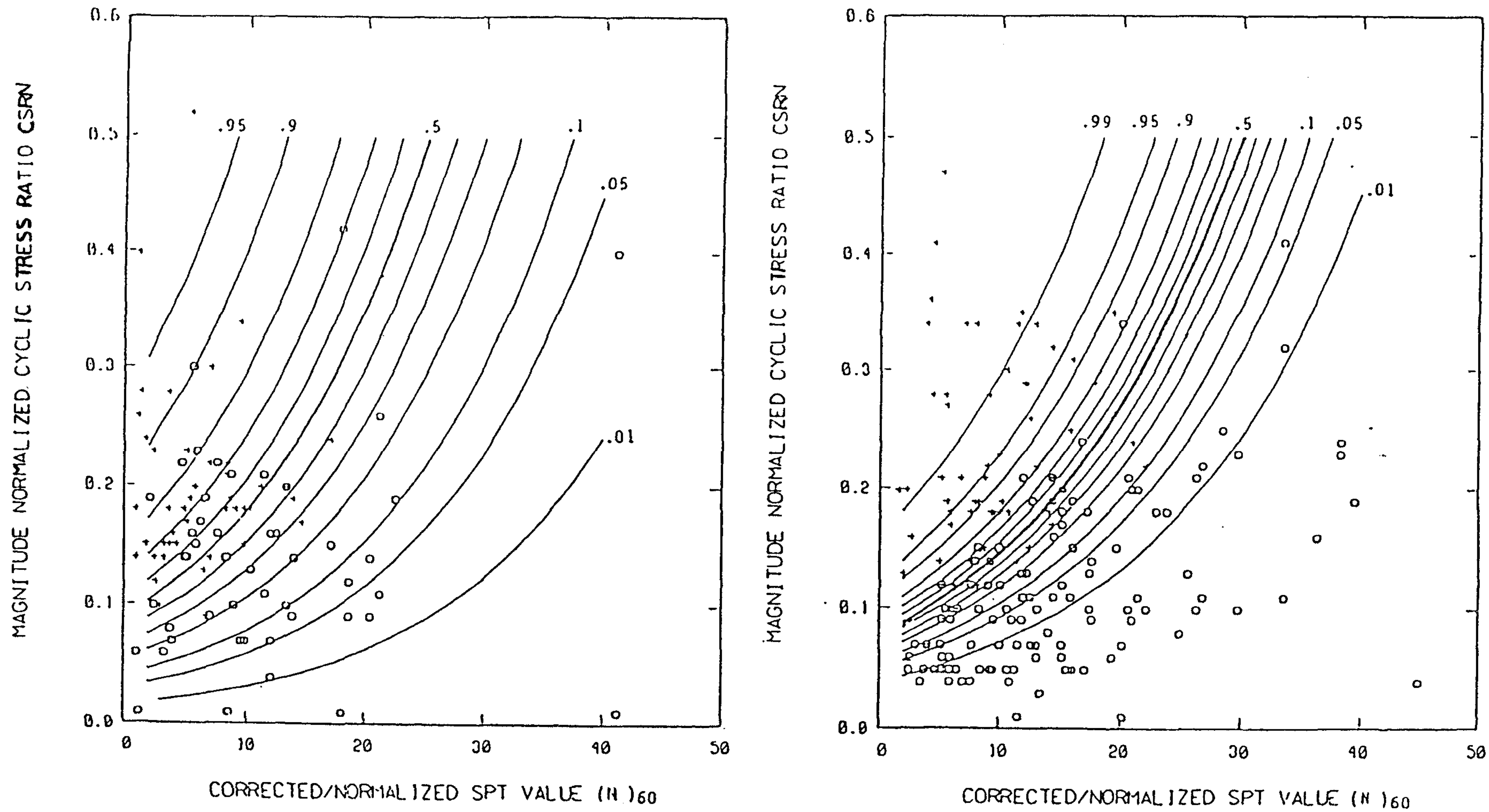
The major advantage of the work of Liao *et al.* is that the liquefaction potential of soils can be cast into probabilistic terms. Thus, one can construct a classification scheme where, for example, the letter H (for high) used by Youd and Perkins (1978) can be associated with a specific probability for liquefaction; one that is greater than 50%.



a) clean sands

b) silty sands

Figure 1. $(N)_{60}$ values leading to liquefaction for clean sands and silty sands. See text for explanation. (after Seed *et al.*, 1984)



a) silty sands

b) clean sands

Figure 2. Probability of liquefaction determined from the $(N)_{60}$ values using the magnitude normalized CSR model. See text for explanation. (after Liao *et al.*, 1987)

SECTION 3
REGIONAL SEISMICITY

The eastern United States is an intraplate region with normally a moderate level of seismic activity. Historically there have been very large earthquakes, but these have been sporadic and few in number. Nonetheless, the severity of these big earthquakes and the fact that the energy is dispersed over very large areas, makes the subject of seismic activity an important one.

Western New York experiences the arrival of seismic waves from many sources. For the purposes of the following discussion, we will distinguish between "local sources" (within the New York State and southern Canada) and "distant sources" (anywhere else in the eastern United States).

3.1 Local Seismicity

Examination of the seismic activity in New York State and adjacent areas during the early 1970s (Sbar and Sykes, 1977) showed that, at least for this period, central New York, much of the Adirondacks and neighboring areas of Pennsylvania were essentially aseismic. The active areas were restricted to four zones: a zone from Boston to central New Hampshire, a zone from Kirkland Lake (Ontario) to northern New York, a cluster of activity centered on the towns of Attica and Dale in western New York, and a region northeast of Quebec City (La Malbaie). Of the two regions in New York State, the Adirondack zone is more active presently than is the area around Attica. Historically, this may not have been true.

From a study of fault plane solutions, hydrofracturing, strain relief stress measurements, and deformation of recent origin, Sbar and Sykes (1977) postulated the existence of different stress domains in the eastern United States. One region extends roughly from Hudson Bay in the north to the edge of the Mississippi embayment in the south. It may extend as far east as Vermont and the westward limit may continue until the Rocky Mountains. Within this stress domain, the dominant orientation is ENE, and the stress is horizontal and compressive. There are indications that the stress is uniform from the eastern border of New York into Ohio and southern Ontario.

Of particular interest to this proposal is the seismic activity located in western New York region. The major event was at Attica on August 12, 1929. This had an intensity of VIII (MM) (Coffman and Von Hake, 1973; Smith, 1962), but there has been a tendency to lower the original estimates to VII (Fox and Spikes, 1977) or to a magnitude of 5.2 (Sykes, 1978). Nonetheless, the event was felt over an area of 130,000 km^2 (Fletcher and Sykes, 1977). There have been two more recent earthquakes, January 1, 1966 and June 13, 1967, each with an intensity of VI. A study of these by Herrmann (1978) indicated that the depths were shallow, on the order of 2 to 3 km, suggesting that the 1929 event was also shallow. The suggestion was made by Herrmann that the shallow depth would explain the relatively large amount of destruction given the relatively weak nature of the earthquake. Similar observations were made in connection with the New Madrid earthquakes of 1811 and 1812 (Herrmann, 1978).

The earthquake activity at Attica is closely related to the Clarendon-Linden fault zone. This is shown by the close spatial relation between the epicenters of the three Attica earthquakes mentioned above, the other seismic activity in western New York (Sbar and Sykes, 1977), and the seismic activity associated with hydraulic mining (Fletcher and Sykes, 1977). The Clarendon-Linden fault likely extends northward under Lake Ontario forming the topographic feature called the Scotch Bonnet Rise (Hutchinson *et al.*, 1979). This gives a total length of approximately 150 km. A possible further extension to the Bancroft area of Ontario is suggested by gravity data, making the total possible length of the fault about 250 km. This feature is clearly a major fracture in the Precambrian crust.

The present knowledge of earthquake activity for New York State is limited (Mitronovas and Nottis, 1984). The scientific community is not in a position to predict the next major earthquake in New York State or adjacent regions. Neither the tectonic structures nor the encompassing mechanisms have been identified for these areas that can be definitively associated with earthquake occurrences (Barstow and Pomeroy, 1984).

The region we selected for study (see Section 4) is sufficiently small that the seismicity within it is difficult to evaluate given the current state of knowledge for earthquakes in and near New York State. However, the existing earthquake records for New York State, which goes to the past 300 years, is probably the best source for evaluating the seismicity of the area. Veneziano and Van Dyck (1984) and Mitronovas and Nottis (1984) in their reports on the seismicity of New York have compiled a record of earthquake events, and an earthquake catalogue for New York State. In addition, they indicate the number of earthquakes, the total energy released for New York State, and the average recurrence times (ART) for earthquakes of various magnitudes.

Earthquakes in Pan-America, the western United States, Japan, and China are presently used as the basis for evaluation of earthquake hazards. It is essential to be aware of the characteristics of possible earthquake hazards in the eastern United States compared to the well documented western ones in order to realistically interpret, judge and apply the various evaluation techniques. Hays (1984) has compiled such a comparison, the results of which can be summarized as:

- The peak ground acceleration for the east can be higher. Ground motion in the east has a tendency to attenuate slowly away from epicenter creating potential to damage tall buildings far away (500 miles) from the epicenter where damage need not be widespread.
- Except for the 1811 - 1812 New Madrid earthquakes, no earthquakes are tied to surface expression of faulting in the east.
- The recurrence period for major earthquakes is 150 years for California but is between 700 and 1000 years in New Madrid seismic zone.
- The attenuation rate for the seismic energy in the east is much slower than in the west. This could result in damage to a much larger area in the east.
- Soil and rock columns in the east are capable of amplifying ground motion for selected frequency ranges which could lead up to a two unit increase of Modified Mercalli Intensity rating relative to rock.
- For major earthquakes, the aftershock sequence may last several years for the east in contrast to a few months in the west

The W 125th Street fault in Manhattan could be disregarded as a possible source of ground shaking according to the conclusions of both Hays (1984) and Mitronovas and Nottis (1984).

3.2 Distant Seismicity

Major historical earthquakes in the eastern United States generally are spatially restricted. Events with intensities greater than or equal to VIII have occurred in Quebec (La Malbaie), the zone from northern New York to Kirkland Lake, Boston, New Madrid, Charleston, Attica and Massena, and a few other, apparently isolated events. Looking at this distribution (Acharya, 1980), Western New York is rather centrally located, with important seismic activity to the northeast and to the southwest. Even though the distances are large, the efficient transfer of energy from the epicenters over long distances in the eastern United States makes these large earthquakes of extreme importance to the estimation of risk for the population centers of New York.

SECTION 4
STUDY AREAS

For this initial survey of the liquefaction potential in New York State, two areas were chosen. The criteria used for the selection were: 1) that the areas have a substantial population density, 2) they should have geologically young sediments, 3) a ground water table close to the surface, and 4) a history of geotechnical exploration from which to draw the information necessary for the liquefaction analysis. The reasons for choosing these criteria will be made clear in subsequent sections. Two areas were identified: one in the upper part of Manhattan, New York City and the other in the waterfront sections of Buffalo.

4.1 Manhattan

The geology of southeastern New York is among the most complicated in the state. Structural features of this region include unconformities and disconformities; anticlinal, synclinal, monoclinal, isoclinal, open, closed, and overturned folds; and normal, reverse, and thrust faults. These many types of structural features have been caused by isostatic changes in elevation, formation of mountain chains and other metamorphic events, igneous intrusions, and erosion.

The stratigraphic sequence found in Manhattan includes rocks of the oldest known type, the Grenville formation, to unconsolidated glacial deposits of Pleistocene age. Manhattan Schist is a pelitic Schist of Middle to Upper Ordovician age and is the most abundant type of rock found in Manhattan. Inwood Limestone of Cambrian to Lower Ordovician age underlies the Manhattan Schist and is found in a belt which runs roughly between Clinton and Second Avenues, south of 34th Street. Gneissic rocks of the Precambrian Fordham Series are found between the Inwood Limestone and the East River.

All of the rock formations on Manhattan have undergone folding. Most of the folding occurred during three major periods of geologic disturbance in Precambrian, post-Ordovician, and post-Devonian times. Compressional forces with primarily northwest to southeast orientations were generated during each disturbance; these forces produced a series of northwest to southeast trending synclinal and anticlinal folds.

Faulting is commonly associated with regional folding. Two major faults in Manhattan are located north of Central Park, and have a general northwest to southeast direction of strike. The larger of these two faults, the 125th Street Fault, runs from the south end of 125th Street, through the northern tip of Central Park, to the north end of Ward's Island. The smaller of the two faults runs roughly parallel to the 125th Street Fault in the Harlem River basin.

4.2 Buffalo

The western part of New York State lies on the edge of the Appalachian Basin. The surface rocks are Paleozoic limestones, dolomites, with abundant shales, and some locally important sandstones. The sedimentary rocks do not form a continuous Paleozoic sequence (the Lower Ordovician is missing, for example), but all the sedimentary rocks are conformable, and there has been little tectonic deformation. There is a general regional dip of about 40 feet per mile to the south.

North of Lake Ontario, the crystalline basement rocks outcrop. These are metagabbros, granites, marbles, and various mafic rocks. This suite presumably underlies the sedimentary sequence in western New York, and the gravity anomalies in the region have been related to the distribution of these different density materials (Hodge *et al.*, 1982). Bedrock in the study area typically consists of Hamilton Group limestones and ranges in depth from 0 to 60 feet (City of Buffalo, Division of Planning, map: "City of Buffalo Rock Elevations").

Unconsolidated deposits within the study area are highly variable due to the complex stratigraphy of surficial deposits and alteration of natural soils by extensive excavation, dredging, and fill operations. Naturally occurring soils are heterogeneous, but the stratigraphy of unconsolidated deposits can be generally characterized as follows: compact to dense clayey glacial till overlain by red-brown very soft to medium stiff varved clay, overlain by a grey-brown stiff varved silt/clay. Gradational variations are present throughout the entire area and some units are not ubiquitous due to diverse glaciation patterns. Well to poorly sorted sand, and/or gravel seams and lenses are interbedded among units at many locations. Rather thick deposits of loose fine sands exist at locations in the vicinities of the Buffalo River and Lake Erie shorelines indicating the presence of buried alluvial/glaciofluvial channels and ancient beach deposits. The groundwater table is shallow, generally at depths a few feet below grade.

Construction sites are common in the area. Soils at many of these sites have been disturbed by construction of inland waterways to facilitate shipping commerce in the mid to late 1800's. A network of inland waterways, canals, and slips were built in the areas presently known as the Black Rock and Buffalo harbors. Filling of these waterways began in the early 1900's as alternate transportation systems were implemented. Landfilling was also performed in order to make development of swampy areas possible. In addition to filling of inland areas, the original Lake Erie shoreline has been extended 600 - 800 feet west of its natural position (Goldberg-Zoino Associates, 1984). The waterfront edge is currently being extended toward the Lake by diked disposal area projects undertaken by the U.S. Army Corps of Engineers, Buffalo District.

4.3 Geotechnical Data

The data which were used for the liquefaction analysis came from two types of sources: drilling done by governmental agencies and geotechnical investigations done by private engineering companies at the request of local builders. In general, the data for Manhattan came primarily from the public sector while the Buffalo study drew largely from the data collected by private engineering companies.

4.3.1 Upper Manhattan

A total of 3,308 borehole logs covering the period from 1940 to 1986 were collected mainly from the following two agencies:

- Department of General Services, New York City
- New York City Housing Authority

Approximately ninety five percent of the borehole logs contained the soil descriptions, data on ground water level, and N values. A typical borehole log is shown in Fig. 3.

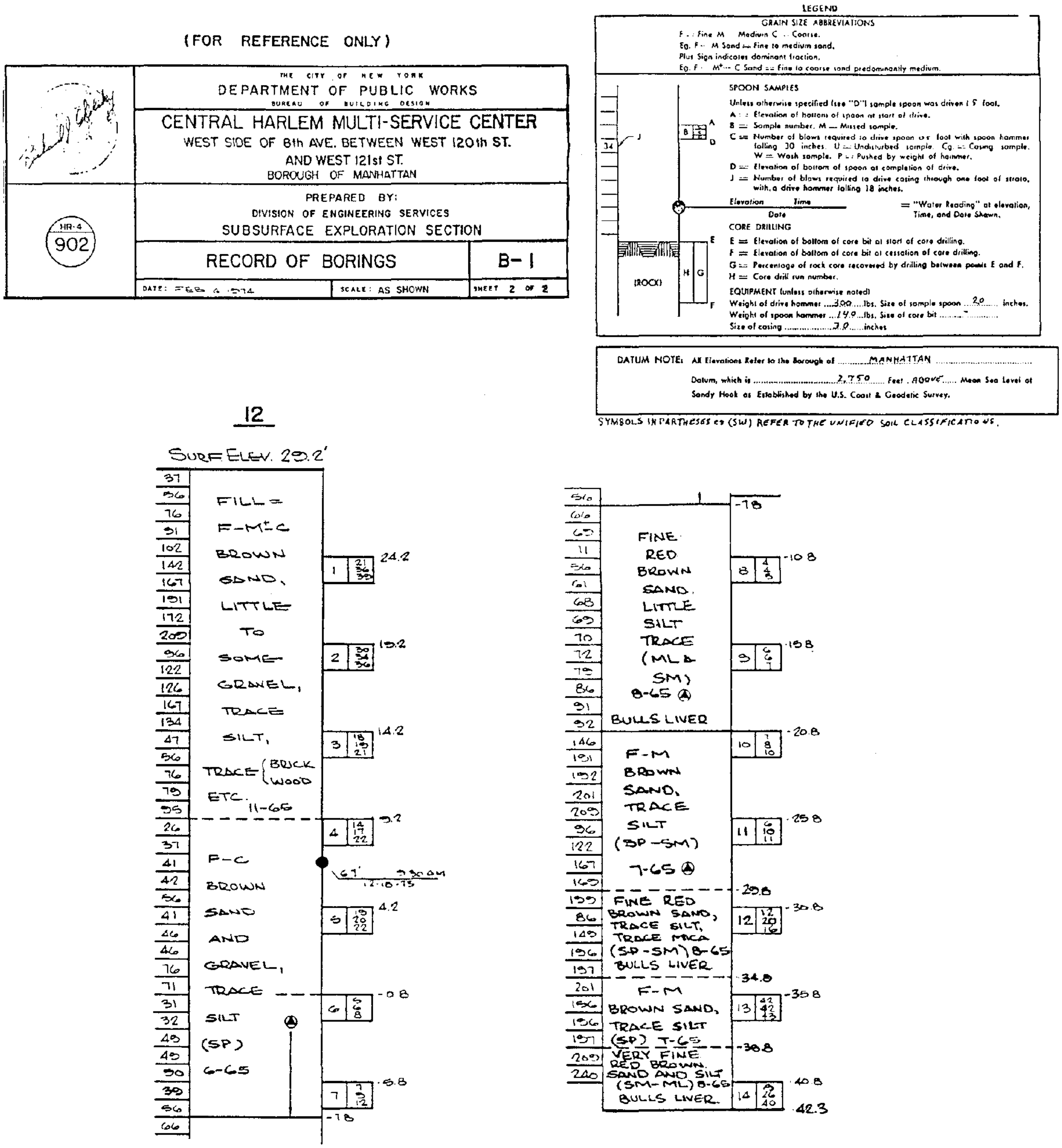


Figure 3. A typical borehole log description for a site in upper Manhattan. The legend in the upper right of the figure provides an explanation of the data contained in the description.

The ground water level at this particular site is very close to the ground surface. Thus, in nearly all the boreholes drilled at this site, casings were used to support the sides of the hole.

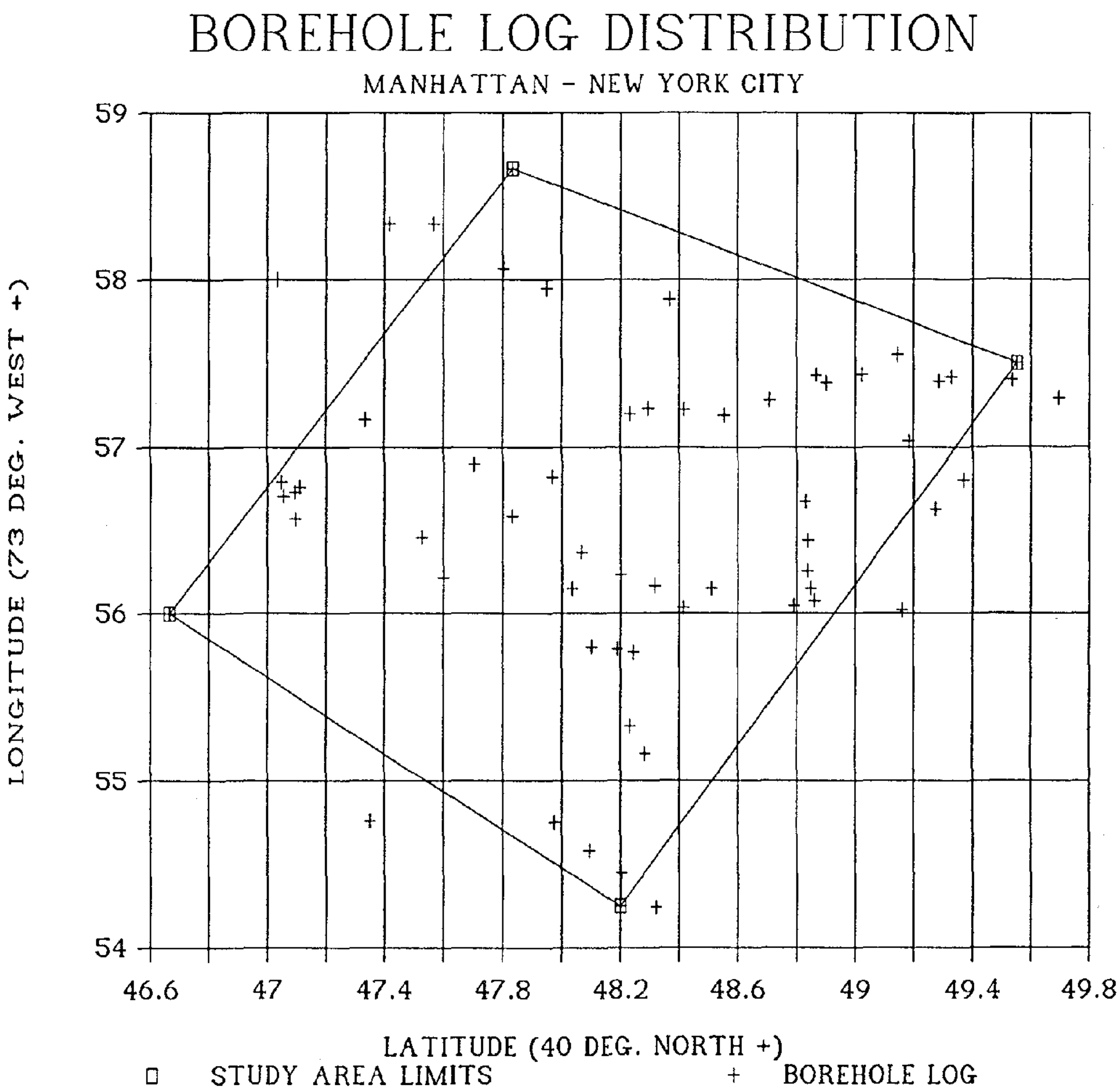
One complication appeared during the evaluation of the geotechnical data collected for parts of Manhattan. This concerns those areas, typically along the Hudson and East Rivers, where new land was created by adding fill to the river banks. The material in these reclaimed sections can extend to depths of as much as 20'. The geotechnical data for these areas are very suspect, and, even if they are of good quality, it is still difficult to make reliable estimates of the liquefaction potential because, at present, there is a complete lack of understanding and practical experience with the behavior of this type of material under earthquake conditions.

A review of the borehole logs revealed that about 60% of the near surface materials (within depths of less than 50') can be classified as silty-sand and sandy-silt, both of which are potentially liquefiable.

The quality of the data in over 75% of the boreholes appeared to be exceptionally good. Most of the N values, however, were obtained from non-standard tests and had to be corrected as described in Section 2.2.2. Fig. 4 shows the distribution of the sites of the boreholes in the study region.

4.3.2 Buffalo

The data collected for the Buffalo study area consists of approximately 2,500 borehole logs from about 90 sites. An extensive preliminary literature search was performed using the National Center for Earthquake Engineering Research (NCEER) Information Services Office and by use of the on-line computer database GEOREF. Historical records documenting local effects of past regional seismic events were reviewed at the Buffalo Historical Society and State University of New York at Buffalo libraries. Extensive data were also collected with the assistance of Empire Soils Investigations Inc., an engineering/consulting firm located in Blasdell, New York. Empire Soils Investigations, Inc. project files for the years 1963-1987 were reviewed for relevant geotechnical data. Past Empire clients owning the files of interest were contacted for permission to incorporate these data into the study.



Approximate scale:

Latitude 1 minute = 6000.0 ft.
Longitude 1 minute = 4594.8 ft.

Figure 4. A map of upper Manhattan showing the location of the borehole logs used in the determination of the liquefaction potential of the area. Only the latitude and longitude of the boreholes are shown; the outline of Manhattan is not indicated.

The following agencies have also contributed to the data base:

- United States Army Corps of Engineers
- United States Naval Reserve
- New York State Department of Environmental Conservation Division of Solid and Hazardous Waste
- New York State Department of Transportation
- City of Buffalo Department of Public Works, Engineering Division
- City of Buffalo Sewer Authority
- City of Buffalo Board of Education
- City of Buffalo Planning Board, Department of Community Development

SECTION 5

LIQUEFACTION MAPPING

5.1 Mapping Methodology

The selected study areas lie within an isoseismal area. A realistic design earthquake magnitude cannot be reliably extractable from whatever is known so far of the seismicity of New York State. The liquefaction evaluation procedures used in this project do not involve design magnitude of earthquakes. The maximum surface acceleration (a_{max}) values are used in the compilation. Therefore, the design earthquake magnitude can be taken as any arbitrary value, since its influence is taken as uniform for the region.

For the region, a 7.5 magnitude (M) will be assumed so that Seed's criteria plots can be applied. The maximum surface acceleration (a_{max}) is the critical factor and is related to the M value. Empirical relationships were established (Seed *et al.*, 1983) based on actual field liquefaction records between maximum ground acceleration (a_{max}), epicentral distance (R) and earthquake magnitude (M). However, such relationships cannot be applied to the study region with both M and R unpredictable and with no applicable field liquefaction data available.

The contribution by Algermissen *et al.* (1982) deals with the peak ground acceleration at sites underlain by rock in New York City. Since the present study area coincides with that of Algermissen *et al.*, their plots of maximum ground acceleration as a function of exposure period for the New York City area are directly applicable.

From their relations, a_{max} has a 90% non exceedence value of approximately 0.18 g over a 50 year period. Earthquakes of (MMI) intensity VI and greater are possible in New York State and these could generate maximum surface accelerations up to 0.15 g (McCann *et al.*, 1980). Therefore, a value of a_{max} = 0.15 g is not unrealistic in evaluating the liquefaction potential of the areas described in this study.

The following methodology was adopted for deriving the liquefaction potential maps for the two selected sites.

- Each borehole log was inspected and those containing N values less than 15 at depths of less than 50' were selected for detailed evaluation.

- A latitude-longitude grid system was used to reference each site.
- The relevant data - N values, hammer characteristics, ground water elevation, depth of the borehole, thickness of the soil layers, soil classification (including the fines content), and geographic location - were entered into a standard spread sheet program. An example is shown in Appendices 1A and 1B.
- The lowest N value for each layer was selected and then corrected to the equivalent standard N_e value using equation (2). The N_{60} value was evaluated using equation (4) (see Section 2.2.2).
- A maximum acceleration, $a_{max} = 0.15\ g$, was selected for both the Manhattan and Buffalo sites based on earthquake records compiled by Algermissen *et al.* (1982) and McCann *et al.* (1980).
- The probability of liquefaction was obtained by using the critical stress ratio method of Liao *et al.* (1987) as described in Section 2.2.4. An average bulk unit weight of 110 lbs/ft.^{3} was assumed.

5.2 Liquefaction Maps

The maps were produced for an earthquake of magnitude 7.5 and a peak ground acceleration 0.15 *g*. As mentioned in Section 2.2.3, suitable scaling factors can be applied to yield the liquefaction potential for different magnitudes.

Each of the two sites investigated presented special problems. The results of our preliminary analysis are presented below.

5.2.1 Upper Manhattan

Fig. 5 shows the liquefaction potential map generated for upper Manhattan using the procedure described in Sections 2.2.3 and 5.1. In view of the great uncertainties involved in the estimation of the liquefaction potential, the final calculated values were grouped into three categories and these are the zones shown in Figure 5. The categories are arbitrary but they were selected to be consistent with much of the previous work in liquefaction potential mapping.

H	high; probability of liquefaction > 50%
M	moderate; probability of liquefaction 10 - 50%
L	low; probability of liquefaction < 10%

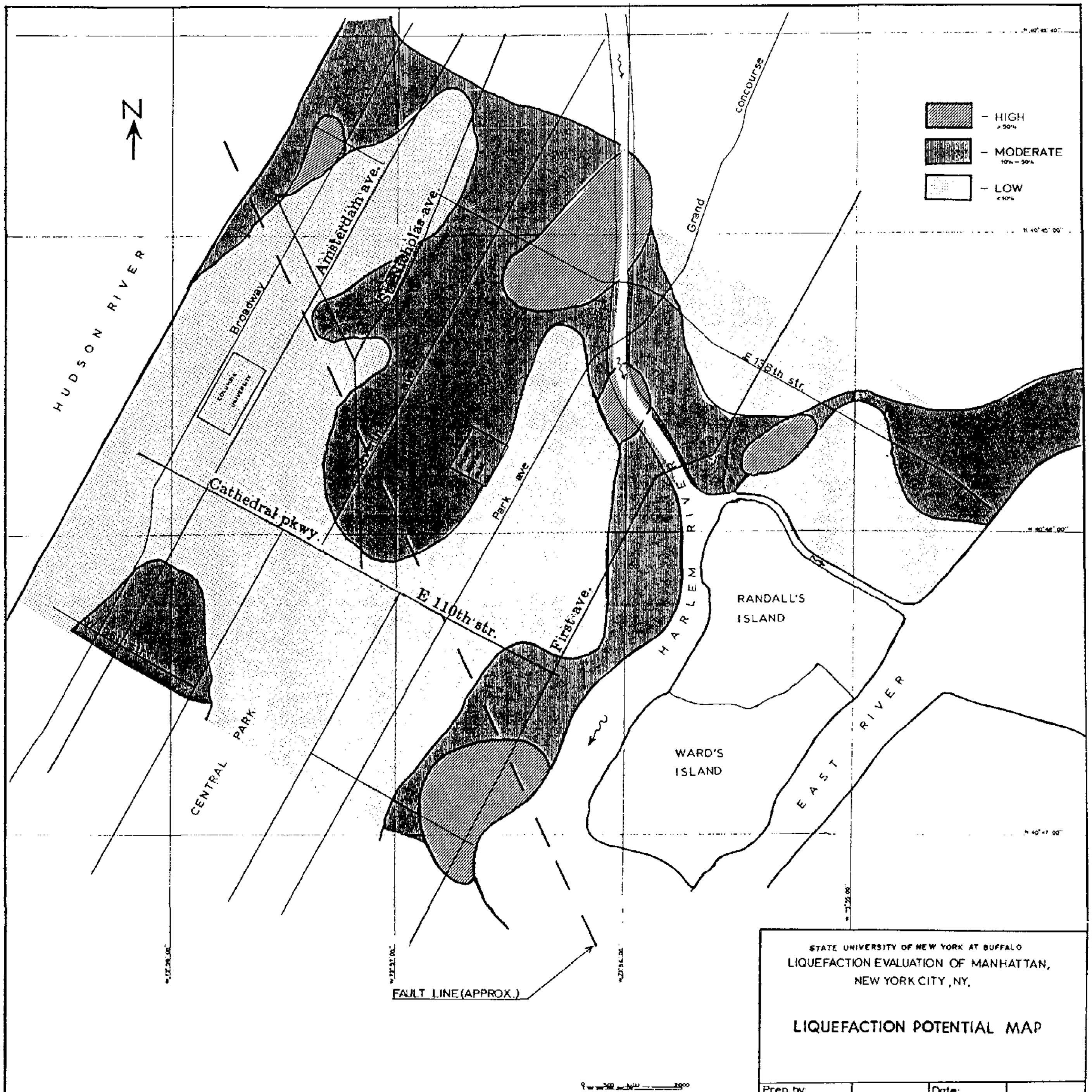


Figure 5. Liquefaction potential map for the upper Manhattan study area.

Approximately one half of the study area has a high to moderate probability of liquefaction. The high risk areas, as would be expected from the discussion in Section 2.1, are adjacent to the shore of the Harlem River. The area generally to the north-west of Second Avenue has a low probability of liquefaction and does not appear to have any significant risk of ground failure.

Even though Ward's and Randall's Islands were included in the study area, they have not been evaluated because of a lack of SPT values.

5.2.2 Buffalo

The analysis of the Buffalo region has not progressed to the same degree as the study of Manhattan. The data for the former became available at a much later date than the Manhattan data. Our examination of the Buffalo data is more qualitative and the evaluations have been based more on the geological nature of the soils identified in the borehole logs. The quantitative evaluation of the Buffalo data is continuing.

Fig. 6 shows a preliminary liquefaction potential map of the waterfront area of Buffalo. Two areas on Kelly Island, an industrialized section of the waterfront, show high probabilities for liquefaction. These areas contain deposits of very loose silty-sands.

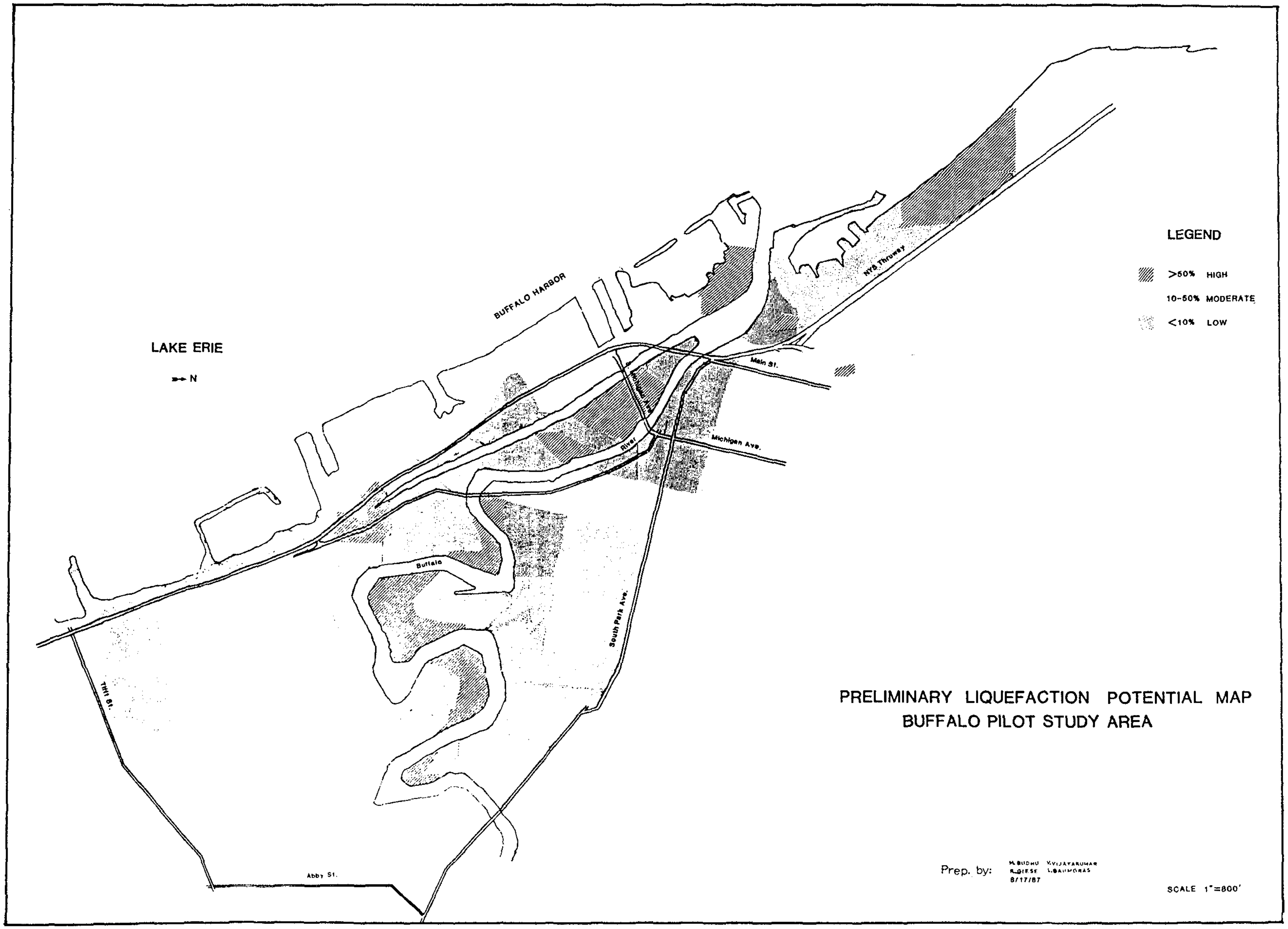


Figure 6. Liquefaction potential map for the Buffalo study area.

SECTION 6

DATABASE DESIGN

As part of the study of the liquefaction potential of New York State, a comprehensive data base system was designed and largely implemented. The purpose of the data base was to store the large number of bore hole data which were accumulated during the investigation of that area of Manhattan bounded by 145th and 96th Streets (East-West), and the Hudson and East Rivers (North-South), as well as the central part of Buffalo. The total number of soil and SPT values accumulated from bore hole investigations are on the order of 20,000 for Manhattan alone.

A data base system serves several purposes. Two are of prime importance for this project: 1) storage of the collected data for future use, and 2) the ability to retrieve selected values for the evaluation of the liquefaction potential of the two areas in question. Since it is not possible to foresee future uses for these data, the data base has been designed in a very general manner allowing easy addition of future applications.

6.1 Specifications

A data base can assume many forms and perform many functions. In view of the complexity of the liquefaction study described in earlier sections, a relatively small number of functions were decided upon so as to simplify the design of the data base and facilitate its implementation. The functions of this minimal data base are:

	Function	Operation	Current Status
1	STORAGE	store geotechnical data from different sites and bore holes on disk files	implemented
2	EDITING	the ability to recall data and alter them	implemented
3	DISPLAY DATA	display stored data as required	implemented
4	DISPLAY LOCATIONS	show the positions of bore holes on a map of the study area	partially implemented (Manhattan only)
5	DISPLAY CROSS SECTIONS	show the soil types, geotechnical data, and correlations for selected bore holes	partially implemented
6	OUTPUT DATA	create an output file containing selected data for subsequent analysis	not implemented

6.2 Implementation

The development of the data base (termed **NCEER** at present) has proceeded in two phases. The initial implementation was done in a commercial data base system (dBase III) designed especially for use on IBM PCs or similar machines. This allowed the authors to rapidly try various configurations thus gaining experience in the use of the data base. The code was written in the dBase III language and debugged on an IBM XT equipped with a hard disk and having 640 kbytes memory. Functions 1 through 3 were perfected using this system. It was clear from the start that dBase III would not completely satisfy the requirements for the data base because the language does not allow graphics output to the computer screen nor graphics output to a plotter. There are ways around these limitations (principally by using third party software), but it was decided that this would create a non-standard package that might be difficult to maintain and modify.

The second phase was to transfer the code developed in dBase III to another language, one which was capable of handling text data, numerical calculations, and graphics. We chose Pascal because of the large body of experience gained in this language, and the existence of a very high quality implementation for the IBM PC (Turbo Pascal). In practice, the transfer of the dBase III code to Pascal was relatively simple. On the other hand, the graphics part of the code (functions 4 and 5) have been more time consuming than was initially thought. The program presently consists of approximately 2000 lines of code.

6.3 Current Status

At present, the major functions of the code are completely or partially working. The data base is being used to store the data collected during this first year of work. As far as the authors are aware, this is the only data base in existence for the storage and manipulation of geotechnical data related to earthquake hazard assessment. The data base is of general use for the storage of normal geotechnical data even though it is now being used primarily for the evaluation of liquefaction potentials.

6.4 Future Activities

Most of our effort during this first year has gone into designing and implementing the data base code. There remain several major tasks-

1. We have created a minimal data base which is used primarily for testing the program; it contains too few bore hole records to serve any practical purpose. Continued work on the data base will focus largely on adding information to the data base so that we will be able to manipulate any of the thousands of pieces of information which we have collected to date.

2. We have spent little time working on the code necessary for the subsequent analysis of the bore hole data (function 6). A major effort will be made to create the software necessary to completely analyze the bore hole data of the two regions for liquefaction potential.

3. We will modify the program so that it will be capable of a more flexible graphics output, both to the monitor screen and to a printer/plotter.

4. Design ways in which the data base could be made accessible to a wider group of users. A dial-up access is one such mechanism.

SECTION 7
CONCLUSIONS

Liquefaction potential maps of two heavily populated urban areas in New York State - upper Manhattan and central Buffalo - have been assembled from subsurface borehole data. The liquefaction potential has been determined by a method which is partly empirical and partly statistical and assumes an earthquake of magnitude 7.5 with a peak ground acceleration of 0.15 *g*. Given an event of this magnitude and surface acceleration, soils adjacent to the Harlem River in Manhattan and along the Buffalo waterfront would be highly susceptible to liquefaction

Many parts of the two study areas, especially those bordering water, are reclaimed land formed by infilling with assorted debris. It was not possible to evaluate the liquefaction potential of these areas since we have no historical record of how this type of artificial deposit will behave when subjected to earthquake stresses. One would suspect that this type of artificial fill would be likely to amplify earthquake induced ground motions and to shift the frequency spectrum of the motions to longer periods.

The liquefaction potential maps are not intended to suggest that presently existing structures in areas which are identified as being underlain by highly liquefiable soils are at risk since such an evaluation is beyond the scope of the present study. The maps provide useful information for: 1) preliminary structural design considerations, 2) planning of emergency procedures, and 3) the assessment of land use.

The geotechnical data base which is being assembled in connection with this project should be expanded to take in larger sections of heavily populated areas in New York State

SECTION 8

LIMITATIONS

The liquefaction potential maps presented here were zoned into categories of high, medium, and low liquefaction potential. The zoning criteria however were relative and adopted as per the opinion of the authors. These maps have some critical limitations with regard to their interpretation and actual use. The important limitations are :

1. The liquefaction evaluation procedure involved correlations made using data for other areas in the nation as well as abroad and not from the region of study.

2. The established empiricisms specify a percentage of silt and clay content, whereas the assessment of these quantities at each site was based on the qualitative classifications of the soils. Clean sand or silt layers were not observed near the surface within the study area. Rather, a mixture of gravels, sands, silts, and clays are common - the effects of which are not taken into account in the evaluation procedure. It should be noted that such effects have the tendency of reducing the liquefaction potential, making the results of the present study conservative.

3. The mapping was not attempted for Ward's and Randall's Island, due to sparse and inadequate data. This can be done once adequate data become available. This is also true for the Buffalo map.

4. The maps indicate zones having different liquefaction potentials; this does not necessarily represent the relative tendency for ground failure. For example, thin subsurface layers having a single low value for the blow count were considered to be liquefiable. Thin layers at greater depths would probably not cause ground failure even if they liquefy. These layers might, however, cause sand "boiling" which could cause some surface damage.

5. The boundaries of the liquefaction susceptibility zones are approximate. However, it is not necessary to delineate these zones precisely because the methodology itself is imprecise.

6. The liquefaction potentials shown on the preliminary maps presented here were based on the existence of cohesionless subsurface materials susceptible to liquefaction. There can be sensitive clay layers within the study area which could cause damage of different kinds during earthquakes. For example, a sensitive clay can amplify the ground motion leading to increased surface damage. These materials were not considered in this report.

7. Earthquakes are capable of causing several other kinds of ground failure apart from liquefaction. The preliminary liquefaction potential map only cover liquefaction-based ground failures. The evaluation considered soils lying within 50' of the surface. Greater depths could be examined, if necessary, because many of the borelogs go to much greater depths.

SECTION 9

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SECTION 10

ACKNOWLEDGMENTS

The authors are particularly indebted to a number of people who helped us assemble the data needed for this study. These are-

Mr. Aroon Shaw
Chief Structural Engineer
and Mr. Andrew Berson
New York City Housing Authority

Mr. J. Blas
Manager
Empire Soils Investigations, Inc.

Mr. Michael Greenman
Geologist
Dept. of General Services
Municipal Building
New York City

Mr. John Lafredo
City Engineer
City of Buffalo

Mr. Steve Pulley
Geotechnical/Materials Engineer
Empire Soils Investigations, Inc.

Mr. James Segun
Computer Analyst
City of Buffalo

SECTION 11

APPENDICES

Appendix 1A and 1B are sections from a Lotus spread sheet which was used to do the calculations of the liquefaction potential - 1A lies directly to the left of 1B in the original configuration. Not all of the borehole data are shown. The headings for the columns are self explanatory except for the following.

Ref. Number	this is the site number on the original borehole log
Sampler Hammer	the sampler hammer ratio, R_s, from equation (1), Section 2.2.2.
r_d	the stress reduction coefficient in equation (3), Section 2.2.3.
Correction for SPT	the correction factor from equation (2), Section 2.2.2.
C_n	the correction factor to normalize the overburden pressure ($\sigma_o^{-1/2}$) with σ_o in tsf.
N_{60}	corrected normalized SPT. The *x* ordinate of Fig. 1 and Fig. 2
τ_{av}/σ_o	average cyclic shear stress in equation (3)
Prob. liq	the probability of liquefaction for the susceptible layer from Fig. 2

Appendix 1A. Sample borehole data used for the computation of liquefaction potentials. See text for details.

Ref. Number	Elev. Ref. Mean SL (ft)	LAT. (N) 40 deg + _'	LONG (W) 73 deg + _'	Hammer Weight (lbs)	Hammer Fall (in)	Spoon OD (in)	Sampler Hammer	Log Number	Shift Lat.	LONG.	Log LAT.	Position LONG.	Surface elev.	G. W. elev.	Suscept. from	Layer to
45	2.750	47 20/60	54 47/60	300	18.00	2.50	1.16E-05	5.00	108	-113	47.351	54.759	2.20	2.20	-5.80	-10.80
72	2.608	48 13/60	54 21/60	265	18.00	2.50	1.32E-05	SES 6	-1450	1840	47.975	54.750	6.7	1.7	-1.3	-6.3
		48 13/60	54 21/60	265	18.00	2.50	1.32E-05	SES 8	-750	1050	48.092	54.579	6.40	0.80	-1.60	-5.60
		48 13/60	54 21/60	300	18.00	2.50	1.16E-05	SES12	-75	450	48.204	54.448	6.10	1.70	-7.90	-17.90
				300	18.00	2.50	1.16E-05				0.000	0.000	6.10	1.70	-17.90	-21.90
		48 13/60	54 21/60	265	18.00	2.50	1.32E-05	SES16	640	-490	48.323	54.243	5.40	1.80	-7.60	-13.60
				265	18.00	2.50	1.32E-05				0.000	0.000	5.40	1.80	-13.60	-25.60
106	2.750	48 13/60	55 47/60	300	18.00	2.50	1.16E-05	1.00	-158	30	48.190	55.790	0.00	0.00	-33.00	-69.00
		48 13/60	55 47/60	300	18.00	2.50	1.16E-05	14.00	170	-58	48.245	55.771	6.10	0.00	-12.90	-26.90
108	2.75	48 51/60	56 6/60	300	18.00	2.50	1.16E-05	2.00	50	-112	48.858	56.076	5.20	-1.40	-15.80	-28.80
				300	18.00	2.50	1.16E-05				0.000	0.000	5.20	-1.40	-28.80	-85.80
		48 51/60	56 6/60	300	18.00	2.50	1.16E-05	10.00	-13	238	48.848	56.152	7.50	-1.30	-14.50	-37.50
				300	18.00	2.50	1.16E-05				0.000	0.000	7.50	-1.30	-37.50	-77.50
		48 51/60	56 6/60	300	18.00	2.50	1.16E-05	15.00	-363	-250	48.790	56.046	7.10	-1.20	-15.90	-29.90
				300	18.00	2.50	1.16E-05				0.000	0.000	7.10	-1.20	-29.90	-42.90
110	?	49 10/60	56 2/60	300	18.00	2.50	1.16E-05	2.00	-42	-60	49.160	56.020	6.00	-1.30	-2.00	-16.00
				300	18.00	2.50	1.16E-05				0.000	0.000	6.00	-1.30	-16.00	-44.00
111	2.750	48 25/60	56 3/60	300	18.00	2.50	1.16E-05	6.00	-20	-70	48.413	56.035	7.50	-0.50	-16.50	-24.50
				300	18.00	2.50	1.16E-05				0.000	0.000	7.50	-0.50	-24.50	-32.50
134	2.750	48 54/60	57 23/60	300	18.00	2.50	1.16E-05	3A	0	0	48.900	57.383	30.00	20.00	20.00	2.00
145		47 6/60	56 44/60	300	18.00	2.50	1.16E-05	37.00	-40	0	47.093	56.733	8.20	-1.60	-17.80	-36.80
		47 6/60	56 44/60	300	18.00	2.50	1.16E-05	49.00	-280	-120	47.053	56.707	6.60	-0.90	-6.40	-17.40
				300	18.00	2.50	1.16E-05				0.000	0.000	6.60	-0.90	-17.40	-33.40
				300	18.00	2.50	1.16E-05				0.000	0.000	6.60	-0.90	-33.40	-58.40
		47 6/60	56 44/60	300	18.00	2.50	1.16E-05	69.00	-30	-760	47.095	56.568	7.80	-0.60	-20.20	-26.20
				300	18.00	2.50	1.16E-05				0.000	0.000	7.80	-0.60	-26.20	-43.20
		47 6/60	56 44/60	300	18.00	2.50	1.16E-05	107.00	48	130	47.108	56.762	9.50	-0.80	-7.50	-17.50
				300	18.00	2.50	1.16E-05				0.000	0.000	9.50	-0.80	-17.50	-25.50
				300	18.00	2.50	1.16E-05				0.000	0.000	9.50	-0.80	-25.50	-50.50

Appendix 1B. Sample borehole data used for the computation of liquefaction potentials. See text for details.

Fines %	Clay %	Field Penet.	Unified Soil Classif.	r_d	Layer Thick-ness	Layer Depth (ft)	Total Stress (psf)	Effect Stress (psf)	Correc-tion for SPT	Std. SPT N	C_n	N_{60}	t_{av}/sig-ma_o CSR	Prob. liq Liao et. al (1987)
50.00		2	SM	0.97	5.0	10.5	1155.0	499.8	1.21	2.42	2.00	5	0.218	81
		17	GM	0.97	5.0	10.5	1155.0	811.8	1.32	22.45	1.57	35	0.134	0
20.00		5	SM	0.97	4.0	10.0	1100.0	825.4	1.32	6.60	1.56	10	0.126	30
20.00		7	SM	0.96	10.0	19.0	2090.0	1179.0	1.21	8.45	1.30	11	0.166	40
		11	SP	0.94	4.0	26.0	2860.0	1512.2	1.21	13.28	1.15	15	0.173	30
100.00	50.00	5	ML	0.96	6.0	16.0	1760.0	986.2	1.32	6.60	1.42	9	0.167	50
		40	SW	0.94	12.0	25.0	2750.0	1414.6	1.32	52.83	1.19	63	0.178	0
100.00	50.00	10	ML	0.80	36.0	51.0	5610.0	2427.6	1.21	12.08	0.91	11	0.180	48
		10	SP	0.94	14.0	26.0	2860.0	1618.2	1.21	12.08	1.11	13	0.162	40
50.00		6	SM	0.94	13.0	27.5	3025.0	1720.8	1.21	7.25	1.08	8	0.161	52
20.00		12	SM	0.70	57.0	62.5	6875.0	3386.8	1.21	14.49	0.77	11	0.139	30
		8	SP	0.91	23.0	33.5	3685.0	2143.7	1.21	9.66	0.97	9	0.153	65
20.00		8	ML	0.70	40.0	65.0	7150.0	3643.1	1.21	9.66	0.74	7	0.134	45
		11	SP	0.93	14.0	30.0	3300.0	1945.9	1.21	13.28	1.01	13	0.154	30
20.00		11	SM	0.84	13.0	43.5	4785.0	2588.5	1.21	13.28	0.88	12	0.151	32
		4	SP	0.96	14.0	15.0	1650.0	1169.5	1.21	4.83	1.31	6	0.131	70
20.00	20.00	10	ML	0.89	28.0	36.0	3960.0	2169.1	1.21	12.08	0.96	12	0.159	32
		7	SP	0.92	8.0	28.0	3080.0	1832.0	1.21	8.45	1.04	9	0.150	65
		13	SP	0.89	8.0	36.0	3960.0	2212.8	1.21	15.70	0.95	15	0.156	18
	5.00	39	SP	0.96	18.0	19.0	2090.0	1528.4	1.21	47.09	1.14	54	0.128	0
		10	SW	0.89	19.0	35.5	3905.0	2301.3	1.21	12.08	0.93	11	0.148	45
100.00	20.00	1	ML	0.96	11.0	18.5	2035.0	1348.6	1.21	1.21	1.22	1	0.141	70
		15	SW	0.91	16.0	32.0	3520.0	1991.2	1.21	18.11	1.00	18	0.157	8
20.00		11	SM	0.80	25.0	52.5	5775.0	2967.0	1.21	13.28	0.82	11	0.152	36
100.00	50.00	2	CL	0.91	6.0	31.0	3410.0	1999.8	1.21	2.42	1.00	2	0.151	72
20.00		7	SM	0.84	17.0	42.5	4675.0	2547.2	1.21	8.45	0.89	7	0.150	55
95.00		2	SM	0.95	10.0	22.0	2420.0	1689.9	1.21	2.42	1.09	3	0.133	62
		12	SP	0.91	8.0	31.0	3410.0	2118.3	1.21	14.49	0.97	14	0.142	18
5.00		11	SP	0.82	25.0	47.5	5225.0	2903.7	1.21	13.28	0.83	11	0.144	40