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FOURTH ANNUAL ISET LECTURE—RECENT DEVELOPMENTS IN THE EVALUATION OF SOIL LIQUEFACTION

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ABSTRACT

Earthquake induced liquefaction of saturated cohesionless deposits is a dramatic cause of damage to structures due to sinking, excessive tilting and uneven settlements. Light structures may float upwards. Foundation may displace laterally causing structural failure. While liquefaction has been reported in numerous earthquakes, it was most dramatic during the Niigata and Alaska earthquakes of 1964. Much information has been gained recently related to earthquake induced liquefaction.

It would seem that the design engineer may evaluate the liquefaction potential of a cohesionless soil deposit based upon the data obtained from the laboratory test results or by the guidance derived from the field performance of sand deposits during the recent earthquakes. Insitu characteristics like standard penetration test value are of considerable utility. The understanding of the liquefaction has now advanced to a point where comparable results can be obtained whichever procedure is used as the basis for desion decisigns.

INTRODUCTION

One of the most dramatic causes of damage to engineering structures during earthquakes has been the development of liquefaction in saturated sand deposits, manifested either by the formation of boils and mud-spouts at the ground surface (see Fig. 1), seepage of water through ground cracks and in some causes by the development of quicksand-like conditions over substantial areas. Where the latter phenomenon occurs, buildings may sink substantially into the ground or tilt excessively, light-weight structures may float upwards to the ground surface and foundations may displace laterally causing structural failures.

While liquefaction has been reported in numerous earthquakes (Seed, 1967), nowhere has the phenomenon been more dramatically illustrated in recent years than in the Niigata, Japan earthquake of 1964 and the Alaska earthquake of the same year. However much information has been gained for recent studies related to other earthquakes and it is the purpose of this paper to summarize these recent developments.

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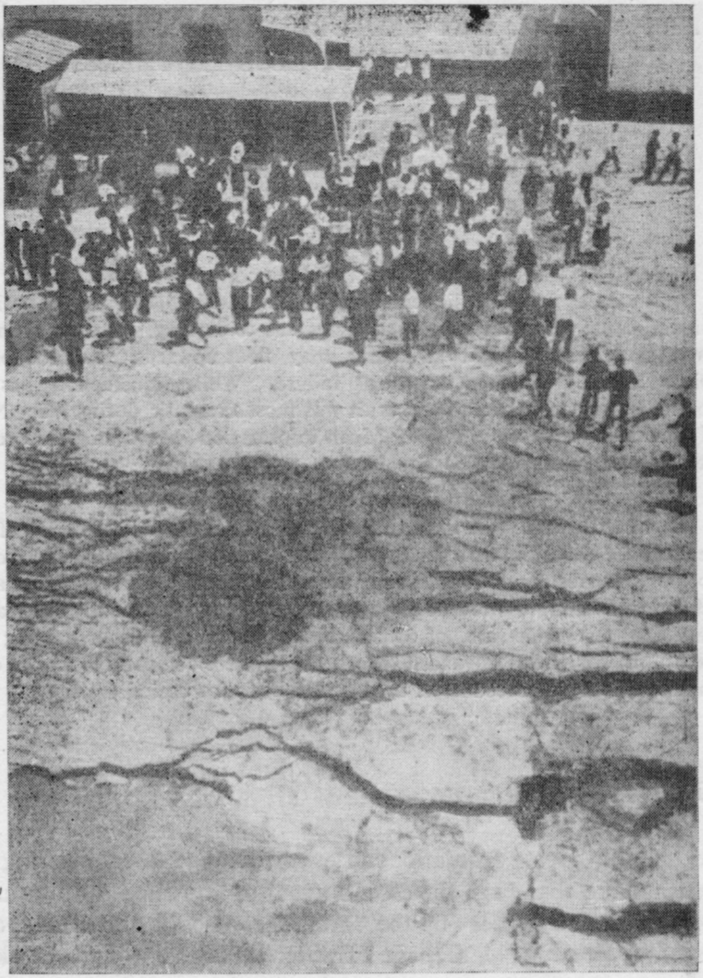
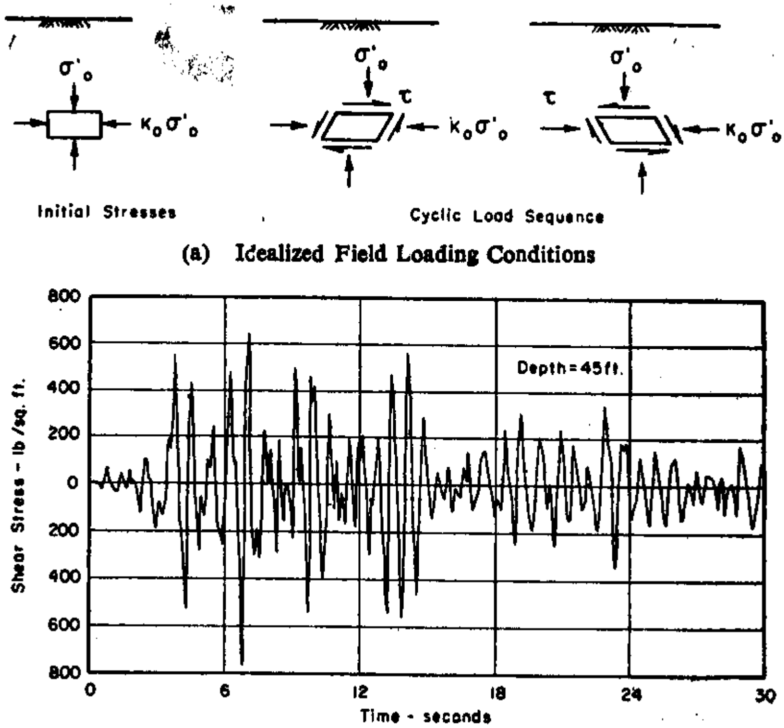


Fig. 1. Initial Stages of the Water Flow from Ground, Niigata (1964)

CAUSES OF SOIL LIQUEFACTION

The cause of liquefaction of sands has been understood, in a qualitative way, for many years. If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume; if drainage is unable to occur, the tendency to decrease in volume results in an increase in pore-water pressure, and if the pore-water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and it develops a liquefied state.

In more quantitative terms, it is now generally believed that the basic cause of liquefaction in saturated cohesionless soils during earthquakes is the build-up of excess hydrostatic pressure due to the application of cyclic shear stresses induced by the ground motions. These stresses are generally considered to be due primarily to upward propagation of shear waves in a soil deposit, although other forms of wave motions are also expected to occur. Thus, soil elements can be considered to undergo a series of cyclic stress conditions as illustrated in Fig. 2, the stress series being somewhat random in pattern but nevertheless cyclic in nature as shown in Fig. 2.



(a) Idealized Field Loading Conditions

(b) Shear Stress Variation determined by Response Analysis

Fig. 2. Cyclic Shear Stress on a Soil Element during Ground Shaking

As a consequence of the applied cyclic stresses, the structure of the cohesionless soil tends to become more compact with a resulting transfer of stress to the pore water and a reduction in stress on the soil grains. As a result, the soil grain structure rebounds to the extent required to keep the volume constant, and this interplay of volume reduction and soil structure rebound determines the magnitude of the increase in pore water pressure in the soil (Martin et al., 1975). The basic phenomenon is illustrated schematically in Fig. 3. The mechanism can be quantified so that the pore pressure increases due to any given sequence of stress applications can be computed from a knowledge of the stress-strain characteristics, the volume change characteristics of the sand under cyclic strain conditions, and the rebound characteristics of the sand due to stress reduction.

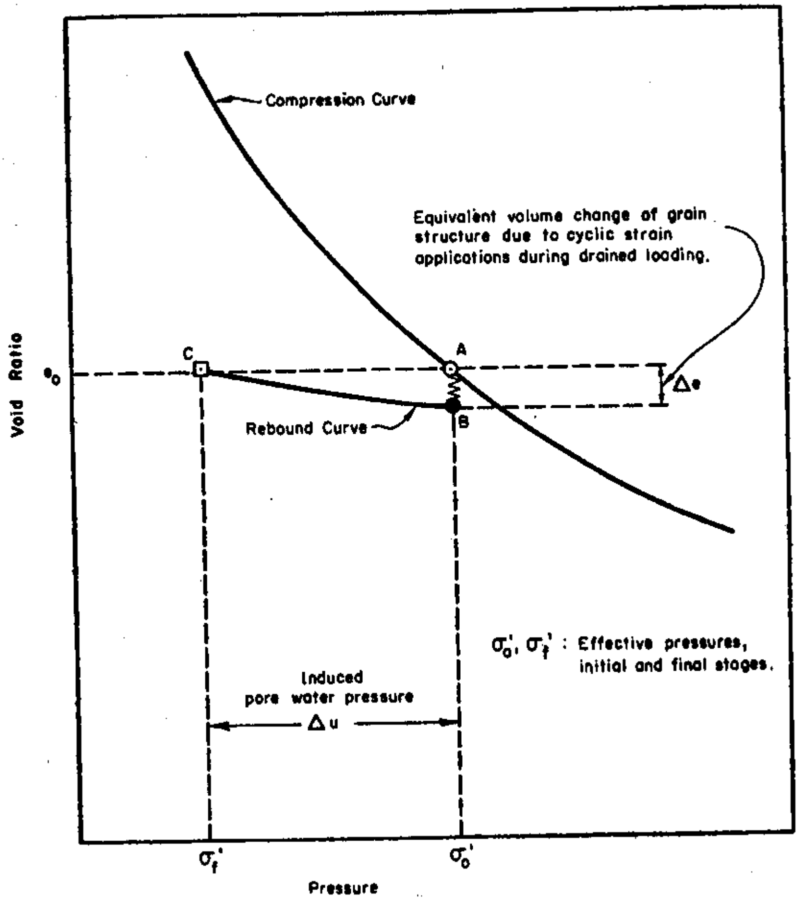


Fig. 3. Schematic Illustration of Mechanism of Pore Pressure Generation during Cyclic Loading

As the pore water pressure approaches a value equal to the applied confining pressure, the sand begins to undergo deformations. If the sand is loose, the pore pressure will increase suddenly to a value equal to the applied confining pressure, and the sand will rapidly begin to undergo large deformations with shear strains which may exceed ± 20 percent or more. If the sand will undergo virtually unlimited deformations without mobilizating significant resistance to deformation, it can be said to be liquefied. If, on the other hand, the sand is dense, it may develop a residual pore water pressure, on completion of a full stress cycle, which is equal to the confining pressure (a peak cyclic pore pressure ratio of 100%), but when the cyclic stress is reapplied on the next stress cycle, or if the sand is subjected to monotonic loading, the soil will tend to dilate, the pore pressure will drop if the sand is undrained, and the soil ultimately develop enough resistance to withstand the applied stress. However, it will have to undergo some degree of deformation to develop the resistance, and as the cyclic loading continues, the amount of deformation required to produce a stable condition may increase. Ultimately, however, for any cyclic loading condition, there appears to be a cyclic strain level at which the soil will be able to withstand any number of cyclies of a given stress without further increase in maximum deformation (DeAlba et al., 1976). This type of behaviour is termed "cyclic mobility" and it is considerably less serious than liquefaction, its significance depending on the magnitude of the limiting strain. It should be noted, however, that once the cyclic stress applications stop, if they return to a zero stress condition, there will be a residual pore water pressure in the soil equal to the overburden pressure, and this will inevitably lead to an upward flow of water in the soil which could have deleterious consequences for overlying layers.

Liquefaction of a sand in this way may develop in any zone of a deposit where the necessary combination of in-situ conditions and vibratory deformations may occur. Such a zone may be at the surface or at some depth below the ground surface, depending only on the state of the sand and the induced motions.

However, liquefaction, of the upper layers of a deposit may also occur, not as a direct result of the ground motions to which they are subjected, but because of the development of liquefaction in an underlying zone of the deposit. Once liquefaction develops at some depth in a mass of sand, the excess hydrostatic pressures in the liquefied zone will dissipate by flow of water in an upward direction. If the hydraulic gradient becomes sufficiently large, the upward flow of water will induce a "quick" or liquefied condition in the surface layers of the deposit. Liquefaction of this type will depend on the extent to which the necessary hydraulic gradient can be developed and maintained; this, in turn will be determined by the compression characteristics of the sand, the nature of ground deformations, the permeability of the sand, the boundary drainage conditions, the geometry of the particular situation, and the duration of the induced vibrations.

It is now possible to analyze the generation and dissipation of pore water pressure in soil deposits during and following earthquakes and the results of such studies can provide valuable insights into possible site behavior in some cases. However, the level of analytical capability used in these studies has probably outstretched our engineering ability to provide details of soil profile stratification and soil property determinations with sufficient accuracy to make the analytical results reliable. Furthermore in dealing with sands, silty sands and silts, for which most liquefaction problems occur, dissipation effects during an earthquake are not significant. Accordingly it is customary to base evaluations of soil liquefaction or cyclic mobility potential on the assumption that all sand layers are undrained during the period of earthquake shaking. If under undrained conditions, it can be shown that every layer in a soil profile has an adequate margin of safety against the development of liquefaction or cyclic mobility, then no significant pore pressures will be generated and consideration of pore pressure dissipation is unnecessary. This approach is followed in the procedures outlined in the following pages for evaluating the liquefaction potential of soil deposits.

GENERAL PROCEDURES FOR EVALUATION OF LIQUEFACTION POTENTIAL

There are basically two methods available for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

1. Using methods based on field observations of the performance of sand deposits in previous earthquakes and involving the use of some in-situ characteristic of the deposits to determine probable similarities or dissimilarities between these sites and a proposed new site with regard to their potential behaviour.

2. Using methods based on an evaluation of the cyclic stress or strain conditions likely to be developed in the field by a proposed design earthquake and a comparison of these stresses or strains with those observed to cause liquefaction of representative samples of the deposit in some appropriate laboratory test which provides an adequate simulation of field conditions, or which can provide results permitting an assessment of the soil behaviour under field conditions.

These are usually considered to be quite different approaches, since the first method is based on empirical correlations of some in-situ characteristic and observed performance, while the second method is based entirely on an analysis of stress or strain conditions and the use of laboratory testing procedures.

In fact, however, because of the manner in which field performance data are often expressed, the two methods involve the same basic approach and differ only in the manner in which the field liquefaction characteristics of a deposit are determined.

Thus, for example, it has been found that a convenient parameter for expressing the cyclic liquefaction characteristics of a sand under level ground conditions is the cyclic stress ratio; that is, the ratio of the average cyclic shear stress $(\tau_h)_{av}$ developed on horizontal surfaces of the sand as a result of the cyclic or earthquake loading to the initial vertical effective stress σ_0' acting on the same layer before the cyclic stresses were applied. This parameter has the advantage of taking into account the depth of the soil layer involved, the depth of the water table, and the intensity of earthquake shaking or other cyclic loading phenomena.

The cyclic stress ratio developed in the field due to earthquake shaking can readily be computed from an equation of the form (Seed and Idriss, 1971):

$$\frac{(\tau_h)_{av}}{\sigma_0'} \simeq 0.65\,\frac{a_{max}}{g}\cdot\frac{\sigma_0}{\sigma_0'}\cdot r_d. \tag{1}$$

where a_{max} = maximum acceleration at the ground surface

σ_0 = total overburden pressure on sand layer under consideration

σ_0' = initial effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of 1 at the ground surface to a value of 0.9 at a depth of about 30 ft.

and values of this parameter have been correlated, for sites which have and have not liquefied during actual earthquakes, with parameters indicative of soil characteristics such as relative density based on penetration test data (Seed and Peacock, 1971), some form of corrected penetration resistance (Castro, 1975; Seed et al., 1975) the electrical characteristics of soil deposits (Arulmoli et al., 1981) or the flat dilatometer test (Marchetti, 1982). Thus in evaluating the liquefaction resistance of a new site for a given level of shaking, the stress ratio induced by the earthquake can be determined by Eq. (1), or a procedure similar to that on which this equation is based, and compared with the stress ratio required to cause liquefaction of the soil determined either

1. by use of the field correlations discussed above or

2. by means of laboratory tests on representative samples of the soil deposit involved.

The evaluation procedure may be conducted in terms of stress ratio, stress, or strain. However, no matter which of these parameters is used, the in-situ properties can only be evaluated reliably if appropriate tests are performed on in-situ deposits or on undisturbed samples. Obtaining truly undisturbed samples which accurately reflect the in-situ liquefaction characteristics of sands presents great difficulties and for denser sands, sampling disturbance can lead to very misleading results as evidenced by the test data shown in Figs. 4 and 5. Figure 4 shows the measured cyclic loading resistance of two sets of samples taken from the same sand deposit, one set by hand trimming block samples and the other set by good quality "undisturbed sampling" in thin wall tubes. The results are different by 100% and neither set is likely to reflect the true in-situ properties of the sand (Marcuson and Franklin, 1979).

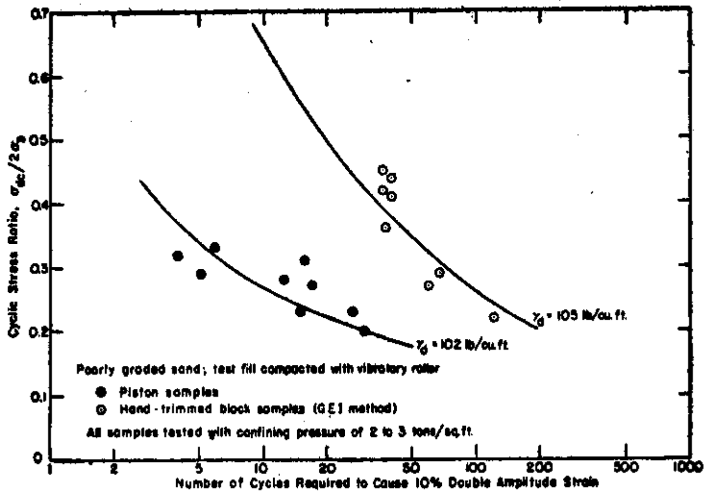


Fig. 4. Influence of Method of Sampling on Cyclic Loading Resistance of Dense Sand

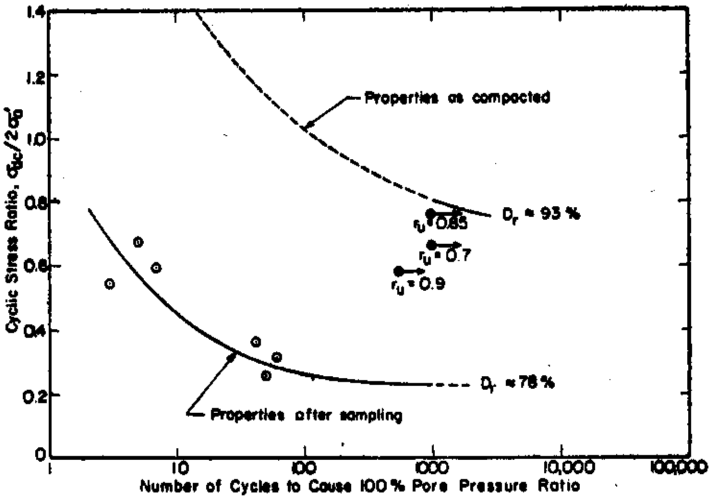


Fig. 5. Effect of Sample Disturbance on Cyclic Loading Resistance of Dense Sand

Figure 5 shows a comparison of the known cyclic load resistance of a large block of dense sand and the measured resistance of high quality undisturbed samples taken in thin wall tubes from the same block. In this case the measured cyclic loading resistance of the "undisturbed samples" was only about 30% of that of the sand block from which they were extracted (Seed et al., 1981). The effects of sampling disturbance on the cyclic load characteristics of medium dense sands is likely to be much smaller than the values indicated above, and may in many cases be of minor significance, but because of the great difficulties in obtaining and testing truly undisturbed samples of sand deposits, many engineers have prefered to adopt the field performance correlation approach since it circumvents this aspect of the problem.

While in principle, soil liquefaction characteristics determined by field performance can be correlated with a variety of soil index parameters such as standard penetration resistance, cone penetration resistance, electrical properties, DMT data, shear wave velocity and perhaps others, there is very little field data available to establish good correlations of field performance with any soil characteristics other than the standard penetration resistance. This situation will no doubt change with time as other index parameters are determined for soils whose liquefaction resistance has been established by actual earthquakes and possibly improved correlations will be developed. Furthermore other parameters can potentially be measured more accurately, over a wider depth range, and in more difficult environmental conditions than can the standard penetration resistance (SPT).

However because the SPT has been so widely used in the past, the great bulk of available field performance data are currently only correlated with this index of soil characteristics and it is the purpose of this report to summarize the available information concerning these correlations.

THE STANDARD PENETRATION TEST

Various studies in recent years have shown the potential variability in the conditions utilized in this supposedly standardized test procedure which was intended to measure the number of blows (of a 140 lb hammer falling freely through a height of 30 inches) required to drive a standard sampling tube (2" O.D. and 1-1/2" I.D.) 12 inches into the ground. For example, Kovacs, et al., (1977, 1978), made careful investigations of the energy delivered by the hammer at its impact with the top of the sampling rod-anvil system, when using the conventional practice of lifting the hammer by means of a rope wrapped around a rotating drum, as compared with an ideal triggering device giving a truly free fall to the 140 lb drive weight. It was found that typically the energy delivered by the hammer when using the rope and drum procedure with two turns of the rope was only about 55% of that theoretically delivered by a free-falling weight; other minor variations were introduced by using old or new rope and changing the speed of the pulley. The authors concluded that an energy standard should be adopted as a criterion for the SPT test and in the meantime, all pertinent test conditions should be made a standard part of the boring log to aid in interpreting the results.

From recent comprehensive theoretical and field studies of the standard penetration test at the University of Florida, Schmertmann (1977) concluded that the results may also be significantly influenced by such factors as: (1) The use of drilling mud versus casing for supporting the walls of the drill hole; (2) the use of a hollow stem auger versus casing and water; (3) the size of the drill hole; (4) the number of turns of the rope around the drum; (5) the use of a small or large anvil; (6) the length of the drive rods; (7) the use of nonstandard sampling tubes; and (8) the depth range (0 to 12 in. or 6 in. to 18 in.) over which the penetration resistance is measured.

Both Schmertmann and Kovacs, et al. conclude that a necessary prerequisite to the satisfactory use of the standard penetration test as a measure of any soil characteristic is an increased degree of standardization. Schmertmann (1977) suggests that this is particularly true with respect to: (1) The amount of energy delivered into the drilling rods; and (2) the use of rotary drilling methods and a drill hole continuously filled with drilling mud.

If this approach is adopted, much of the variability can be eliminated by adopting standard test conditions and applying corrections for others. Thus in the present report, the loss of driving energy which results from using a short length of rods is corrected by multiplying the measured N values in the depth range 0 to 10 ft by a factor of 0.75 and other aspects of the test are standardized by using data from tests performed under the following conditions:

1. the use of a rope and drum system, with two turns of the rope around the drum, to lift the falling weight

2. drilling mud to support the sides of the hole

3. a relatively small diameter hole, approximately 4 inches in diameter and

4. penetration resistance measured over the range 6 inches of 18 inches penetration into the ground.

While it is recognized that these conditions do not represent the standard prescribed in the ideal test procedure, they represent conditions widely used for many years both in North America and in other countries throughout the world, and they have been used in establishing much of the field data available for liquefaction correlations. Thus their adoption for the purposes of this report is justified for this reason alone. Where test conditions deviate from those listed above, such as, for example, the use of a free-fall hammer, appropriate corrections to the measured results should be made before using the correlation charts presented herein.

CORRELATIONS OF SPT WITH THE PERFORMANCE OF SAND DEPOSITS IN PREVIOUS EARTHQUAKES

It was not until the Alaska and Niigata earthquakes of 1964 that geotechnical engineers took serious interest in the general phenomenon of earthquake-induced liquefaction or cyclic mobility, or the conditions responsible for causing them to occur in the field. Following the Niigata earthquake, a number of Japanese engineers (Kishida, 1966; Koizumi, 1966; Ohsaki, 1966) studied the areas in Niigata where liquefaction had and had not occurred and developed criteria, based primarily on the Standard Penetration Resistance of the sand deposits, for differentiating between liquefiable and nonliquefiable conditions in that city.

From this beginning, similar studies have been made at various locations where some evidence of liquefaction or no liquefaction is known to have taken place during earthquakes and used as a basis to determine the relationship between field values of cyclic stress ratio τ_h/σ_0' (in which τ_h = the average horizontal shear stress induced by an earthquake, and σ_0' = the initial effective overburden pressure on the soil layer involved) and the Standard Penetration Resistance of sands determined as described previously. The results have been compiled over a 14 year period (1969—present) and the most recent compilation of this field data collection is shown in Fig. 6 (after Seed, Idriss and Arango, 1983). Values of cyclic stress ratio known to be associated with some evidence of liquefaction or no liquefaction in the field are plotted as a function of the normalized penetration resistance N_1 of the sand deposit involved. In this form of presentation N_1 is the measured penetration resistance corrected to an effective overburden pressure of 1 ton/

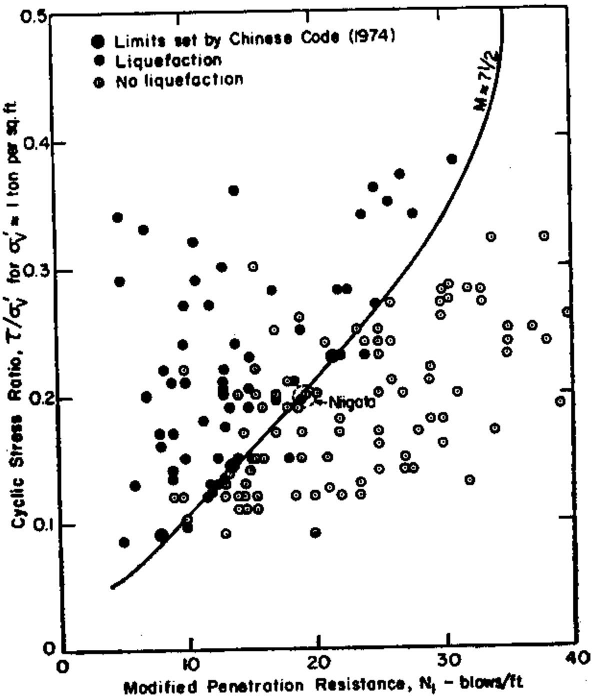


Fig. 6. Correlation between Field Liquefaction Behaviour of Sands under Level Ground Conditions and Standard Penetration Resistance

sq ft or 1 ksc, and can be determined from the relationship:

$$N_1 = C_N . N \tag{2}$$

where C_N is a function of the effective overburden pressure at the depth where the penetration test was conducted. Values of C_N may be determined from the chart shown in Fig. 7 which is based on recent studies conducted at the Waterways Experiment Station (Bieganousky and Marcuson, 1976; Marcuson and Bieganousky, 1976).

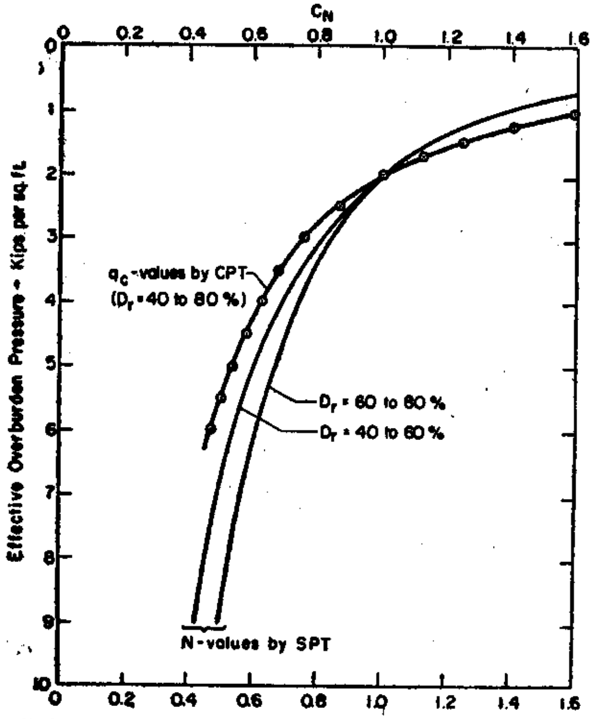


Fig. 7. Relationships between C_N and Effective Overburden Pressure

Thus for any given site and a given value of maximum ground surface acceleration, the possibility of cyclic mobility or liquefaction can readily be evaluated on an empirical basis with the aid of this chart by determining the appropriate values of N_1 for the sand layers involved, reading off a lower bound value of τ_{av}/σ_0' for sites where some evidence of liquefaction is known to have occurred (such as the line shown in Fig. 6) and comparing this value with that induced by the design earthquake for the site under investigation (computed from Eq. 1). The data points shown in Fig. 6 are from site studies in the United States, Japan, China, Guatemala, and Argentina and thus represent a wide range of geo .aphical locations and conditions. The extent and consistency of the data used to define liquefiable conditions in Magnitude 7–1/2 earthquakes, shown in Fig. 6, provides a reasonably reliable basis for evaluating the liquefaction characteristics of sands at other sites from SPT data.

CORRELATIONS FOR DIFFERENT MAGNITUDE EARTHQUAKES

The results presented in Fig. 6 provide a realistic basis for developing correlations between standard penetration tests and the liquefaction characteristics of sands for Magnitude 7-1/2 earthquakes. Unfortunately similar collections of data are not available for other magnitudes of earthquakes. The results shown in Fig. 6 can be extended to other magnitude events, however, by noting that from a liquefaction point of view, the main difference between different magnitude events is in the number of cycles of stress which they induce. Statistical studies show that the number of cycles representative of different magnitude earthquakes is typically as shown in Table 1. Furthermore a representative shape for the relationship between cyclic stress ratio and number of cycles required to cause liquefaction shows that the relative values of stress ratio required to cause liquefaction in different numbers of cycles are typically close to those shown in the table (Seed et al., 1983).

TABLE 1

Earthquake Magnitude, M	No. of Representative Cycles at $0.65\,\tau_{max}$	$\dfrac{(\tau_{av}/\sigma_0')_{l-M=M}}{(\tau_{av}/\sigma_0')_{l-M=7.5}}$
8–1/2	26	0.89
7–1/2	15	1.0
6–3/4	10	1.13
6	5	1.32
5–1/4	2–3	1.5

Thus by multiplying the boundary curve in Fig. 3 by the scaling factors shown in column (3) of Table 1, boundary curves separating sites where liquefaction is likely to occur or unlikely to occur may be determined for earthquakes with different magnitudes. Such a family of curves for sands is shown in Fig. 8, providing a basis for evaluating the liquefiability of sands in earthquakes of any magnitude.

FIELD DATA FOR SILTY SANDS

A study of sites at which liquefaction did and did not occur in the Miyagiken-Oki earthquake in Japan (Mag. $\simeq 7.5$) by Tokimatsu and Yoshimi (1981) has provided an extensive set of field data points for silty sands ($D_{50} < 0.15$ mm). Japanese engineers (e.g. Tatsuoka, Iwasaki et al., 1980) have considered for the past several years, on the basis of laboratory test

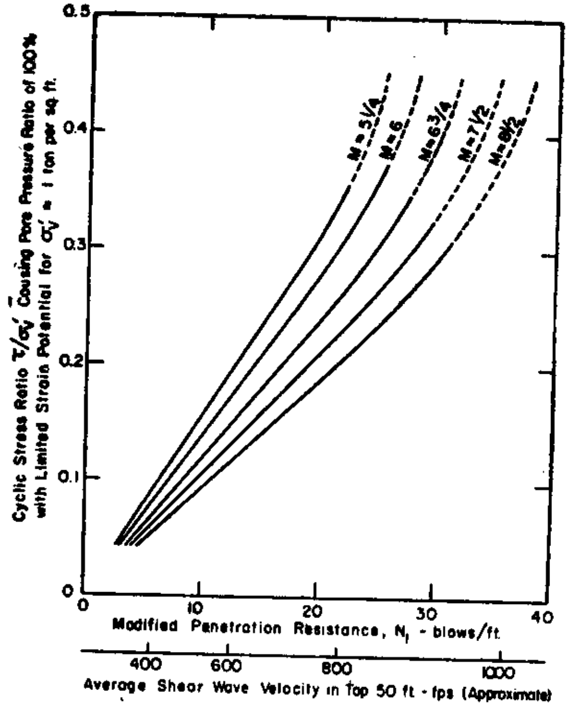


Fig. 8. Chart of Evaluation of Liquefaction Potential for Different Magnitude Earthquakes

data, that silty sands are considerably less vulnerable to liquefaction than sands with similar penetration resistance values and the field studies conducted by Tokimatsu and Yoshimi provide good field corroboration that this is in fact the case. The data for silty sands, for sites which liquefied and sites with no apparent liquefaction, are presented in the same form as the data in Fig. 6 and Fig. 9. Also shown in Fig. 9 are a reasonable boundary separating sites where liquefaction occurred and sites where no liquefaction occurred for these silty sand deposits, and the boundary line for clean sands taken from Fig. 6. It may be seen that the boundary line for silty sands is significantly higher than the boundary line for sandy soils, although the two lines are essentially parallel. In fact for any value of stress ratio, the normalized standard penetration resistance, N_1, for sands with $D_{50} > 0.25$ mm is essentially equal to that for silty sands ($D_{50} \nless 0.15$ mm) plus about 7.5. It may be concluded therefore that the boundary lines previously established for sands can be used for silty sands, provided the N_1 value for the silty sand site is increased by about 7.5 before entering the chart.

It is interesting to note that Zhou (1981) reached a similar conclusion on the basis of field studies in China following the Tangshan earthquake. From a comparison of the behaviour of different types of soil, Zhou proposed

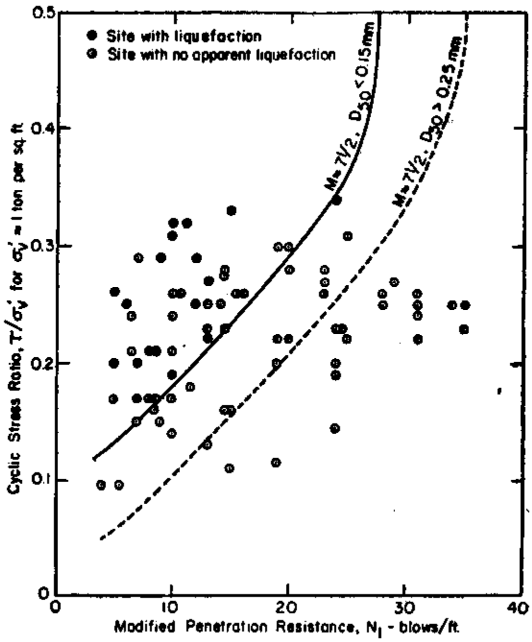


Fig. 9. Correlation between Field Liquefaction Behaviour of Silty Sands under Level Ground Conditions and Standard Penetration Resistance

that the difference in liquefaction characteristics could be taken into account by an appropriate increase in penetration resistance (in this case the static penetration resistance), the magnitude depending on the fines content. Interestingly, for soils with about 30% fines which would correspond approximately to soils with $D_{50} < 0.15$ mm, the desirable increase in static cone resistance was found to be about 27 kg/cm^2 which corresponds, for the site conditions involved, to an increase in N_1 value of about 6. This is in remarkably good agreement with the value of 7.5 indicated by the results presented previously.

CORRELATION OF LIQUEFACTION CHARACTERISTICS WITH CPT DATA

While the Standard Penetration Test (SPT) has been widely used for many years, in many cases it may be more expedient to explore the variability of conditions within an extensive sand deposit using the static cone penetration test (CPT). The main advantages of this procedure are that it provides data much more rapidly than does the SPT test, that it provides a continuous record of penetration resistance in any bore hole, and it is somewhat less vulnerable to operator error than the SPT test.

The main disadvantages of the test, from the point of view of predicting the liquefaction resistance of a site, is that it has a very limited data base to provide a correlation between soil liquefaction characteristics and CPT values. This data base may remain meager for some time pending the generation of new data from new earthquakes. In the meantime, however, the test can be used in conjunction with the extensive data base for the Standard Penetration Test by either:

(1) Conducting preliminary studies at each new site to establish a correlation between CPT data and N values for the sand at the site.

(2) Using available correlations between SPT test data and CPT test data based on test programs previously conducted. Thus the average relationship between CPT data in ksc units and N values in SPT tests are approximately (Schmertmann, 1978):

(a) for clean sands $q_c \simeq 4$ to $5\ N$, and

(b) for silty sands $q_c \simeq 3.5$ to $4.5\ N$.

Using such relationships the data obtained from CPT test programs can readily be converted to equivalent N values for the sand and then used in conjuction with the charts in Figs. 6, 8 and 9 to evaluate liquefaction resistance. By this means full advantage can be taken of the advantages of the CPT test procedure and the extensive data base of the SPT correlation with field liquefaction characteristics.

Alternatively, the critical boundaries separating liquefiable from non-liquefiable conditions shown in Figs. 6, 8 and 9 can be expressed in terms of a static Cone Penetration Resistance corresponding to an overburden pressure of 1 ton per sq ft, q_{c1}, by using the relationships

$$q_{cl} \simeq 4 \text{ to } 5N_1 \text{ for clean sands}$$

and $$q_{cl} \simeq 3.5 \text{ to } 4.5\ N_1 \text{ for silty sands.}$$

This would lead to plots relating values of cyclic stress ratio causing liquefaction with q_{cl} values, as shown in Fig. 10.

It is interesting to note that for any sand the value of q_{c1} can be deter- from the value of q_c measured at any depth using the relationship

$$q_{cl}=q_c \cdot C_N \qquad (3)$$

where value of C_N are read off from the curve shown in Fig. 4, which is based on the relationship between $q_{c'}$ effective overburden pressure and relative density proposed by Schmertmann (1978).

In view of the need to introduce a second correlation (between SPT and CPT) this procedure would seem to be less desirable than use of the SPT

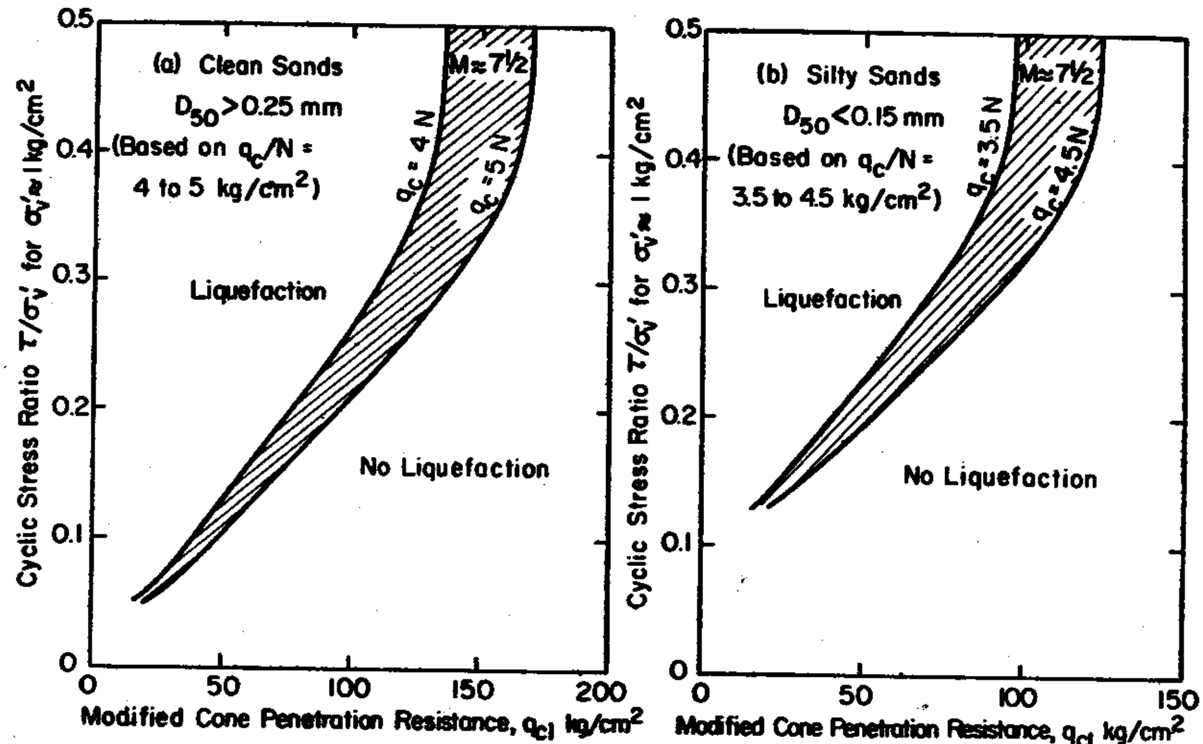


Fig. 10. Proposed Correlation between Liquefaction Resistance of Sands for Level Ground Conditions and Cone Penetration Resistance

directly as an index of liquefaction. However in view of the other advantages of the CPT test (continuous records of soil characteristics and more rapid testing) and the fact that site-specific correlations can be developed where appropriate, this procedure may well prove advantageous in many cases.

CHINESE BUILDING CODE (1974) CORRELATION OF LIQUEFACTION RESISTANCE WITH SPT DATA

It is interesting to note that liquefaction studies in China conducted along similar lines to those used in the United States over the period 1970-83, led to the use of a correlation between earthquake shaking conditions causing cyclic mobility or liquefaction and the standard penetration resistance of sands in China. In this correlation, the critical value of the standard penetration resistance, N_{crit}, separating liquefiable from nonliquefiable conditions to a depth of approximately 50 ft was determined by

$$N_{crit}=\bar{N}[1+0.125(d_s-3)-0.05(d_w-2)] \tag{4}$$

in which d_s = depth to sand layer under consideration in meters; d_w = depth of water below ground surface in meters; and $\bar{N}$ = a function of the shaking intensity as follows:

Modified Mercalli Intensity	*$\bar{N}$, in blows per foot*
$\simeq$VII	6
$\simeq$VIII	10
$\simeq$IX	16

This correlation, for a water table depth 2 m, reduced to the same parameters as those used in Fig. 6, with the aid of the correlation between earthquake shaking intensity and maximum ground acceleration developed by Trifunac and Brady and that used in China is plotted in Fig. 11 where it is also compared with the lower bound line for sites showing evidence of some degree of cyclic mobility or liquefaction shown in Fig. 6. It may be seen that there is a high degree of agreement between the critical boundary determined in this way and that shown in Fig, 6. It is significant and remarkable that such a great similarity both in procedures and criteria should have evolved in countries with so little technnical communication at the time the individual plots were developed.

CORRELATION OF LIQUEFACTION RESISTANCE WITH SHEAR WAVE VELOCITY

As in the case of CPT data, it is not difficult to extend the correlation between field liquefaction characteristics and SPT results to include shear wave velocity data. Many studies have been conducted to relate N values

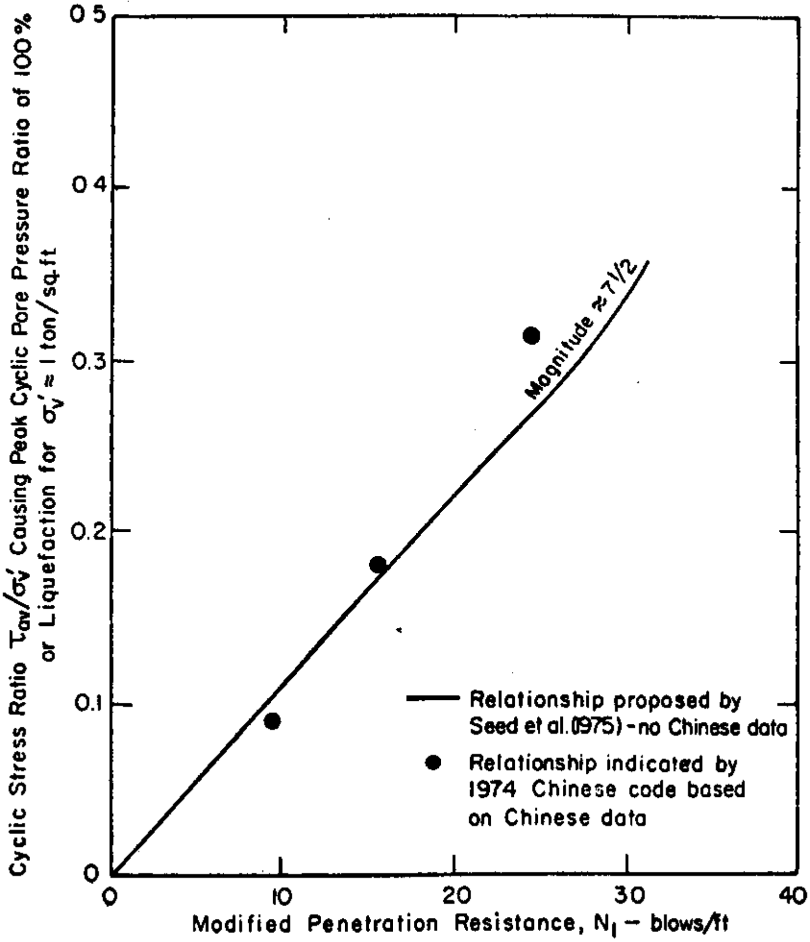


Fig. 11. Comparison of Empirical Chart for Predicting Liquefaction with Recommendations of 1974 Chinese Code

with the dynamic shear modulus of sands, a typical result being that proposed by Ohsaki and Iwasaki (1973):

$$G_{max} \simeq 120\ N^{0.8}\ \text{ksc} \tag{5}$$

which is closely approximated, for practical purposes by the simpler expression

$$G_{max} \simeq 65N\ \text{ksc} \tag{6}$$

Noting that $$v_s = \sqrt{\frac{G \cdot g}{\gamma}}$$

It follows that $$v_s \simeq \sqrt{\frac{65N \times 10^3 \times 981}{2}}\ \text{cm/sec}$$

$$\simeq 55\sqrt{N}\ \text{m/sec.} \tag{7}$$

Noting that $N = N_1/C_N'$ leads to:

$$v_s \simeq \frac{55\sqrt{N_1}}{\sqrt{C_N}} \text{ m/s} \tag{8}$$

In the upper 15 m of a sand deposit, the effective overburden pressure, σ_0', will be less than 4000 psf. and for values of σ_0' below this value, C_N is typically in the range 0.7 to 1.6 (see Fig. 4). The corresponding values of $\sqrt{C_N}$ will be in the range of 0.85 to 1.25 so that a reasonable average value is about 1.05. Thus from the above equation:

$$v_s \simeq \frac{55\sqrt{N_1}}{1.05} = 52\sqrt{N_1} \text{ m/sec} \tag{9}$$

for depths up to about 50 ft. This approximate relationship can be plotted along the abcissa of Fig. 8, to provide an approximate correlation between values of stress ratio causing liquefaction in the field and the average shear wave velocity of the upper 50 ft of soil.

It may be noted that Fig. 8 indicates that liquefaction will never occur in any earthquake if the shear wave velocity in the upper 50 ft of soil exceeds about 1000 fps. This is in good agreement with the finding of Youd and Hoose (1980) that Holocene sand deposits, typically having $v_s < 700$ fps have been more disturbed by liquefaction that Pleistocene deposits for which $v_s \geqslant$ 1100 fps.

EVALUATION OF PROPERTIES OF UNDISTURBED SAMPLES

One of the primary reasons for developing a relationship between soil liquefaction characteristics and in-situ properties of sands such as the N_1-value discussed above, was the great difficulty in obtaining undisturbed samples of natural deposits which could be tested in the laboratory. Several examples of this difficulty were discussed earlier. However the potential advantages of determining laboratory data by tests on truly undisturbed samples are readily recognized and as a result, increasing emphasis has been given to the problems of sample disturbance in recent years.

Detailed studies (Seed et al., 1982, Yoshimi et al., 1978, have shown:

1. For medium dense sands, the effects of sample disturbance on the liquefaction characteristics of samples tested in the laboratory may be relatively small due to compensating errors introduced by sample disturbance effects.

2. For dense sands, the effects of samples disturbance are not compensating and thus large errors may be involved if tests are performed on conventional types of undisturbed samples (see Figs. 4 and 5).

3. It is possible to obtain truly undisturbed samples of sand by freezing the soil in place prior to sampling, and field procedures for accomplishing this have now been developed in Japan (Yoshimi, et al., 1978).

The increased understanding of the problem and the development of field procedures for obtaining truly undisturbed samples can eliminate the objections to the use of laboratory testing as a means of determining the liquefaction characteristics of sands making the use of stress analysis techniques combined with laboratory testing procedures a viable approach to liquefaction evaluation for all deposits.

CONCLUSION

It would seem that the design engineer confronted with the need to evaluate the cyclic mobility or liquefaction potential of a deposit has two basic choices if he considers it appropriate to neglect the possible effects of drainage occurring during the period of cyclic stress applications:

1. To calculate the stresses induced in the ground by the design earthquake, using either ground response analysis or simplified procedures, whichever seems most appropriate, and to compare these stresses with those required to cause cyclic mobility or liquefaction of representative samples in the laboratory. In doing this, the main problem will lie in correctly assessing the characteristics of the in-situ deposit from laboratory tests performed on even good quality undisturbed samples. To accomplish this satisfactorily it will be necessary either to take the best possible undisturbed samples and then try to evaluate their true field characteristics by allowing for the effects of sample disturbance on the basis of reasonable judgement or by taking undisturbed samples using an appropriate freezing procedure and using these as a basis for property evaluation.

2. To be guided by the known field performance of sand deposits correlated with some measures of in-situ characteristics, such as the standard penetration test, on the grounds that most factors that tend to improve cyclic mobility or liquefaction resistances also tend to increase the standard penetration resistance or the results of any other in-situ test that may be adopted as a possible indicator of field liquefaction behavior.

It would be imprudent to ever neglect the guidance to be derived from records of past experience and this should always be considered in any site evaluation. Whichever method is used, however, it is likely to be necessary to use judgement either to interpret the results of the laboratory test data or the results of in-situ test procedures. Never-the-less, I believe it is fair to say that our understanding of the liquefaction phenomenon has now advanced to the point where comparable results can be obtained whichever procedure is used as the basis for design decisions.

SYMBOLS

a_{max}—maximum acceleration at ground level

C_n—constant of proportionality relating N and N_1

d_s—depth of sand layer

d_w—depth of water table below ground level

D_r—relative density

e—void ratio

G_{max}—maximum value of the shear modulus

K_o—coefficient of at rest earth pressure relating the vertical and lateral pressures

N—standard penetration test value (SPT)

N_1—normalized SPT value

$\bar{N}$—function of shaking intensity

N_{crit}—critical value of SPT

q_c—cone penetration resistance

q_{c1}—modified cone penetration resistance

r_d—stress reduction factor

u—pore water pressure

v_s—velocity of shear waves

Δe—change in void ratio

Δu—change in pore water pressure

σ_o—initial over burden pressure (total)

σ'_o—initial effective over burden pressure

σ'_f—final effective vertical pressure

τ—shear stress

$(\tau_h)_{av}$—average shear stress on horizontal plane

$(\tau)_{max}$—maximum shear stress

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