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CONFERENCE SPECIALE/SPECIAL LECTURE

EVALUATION OF THE DYNAMIC CHARACTERISTICS OF SANDS BY IN-SITU TESTING TECHNIQUES

EVALUATION DES CARACTERISTIQUES DYNAMIQUES DE SABLES PAR ESSAIS EN PLACE

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Introduction

Interest in the dynamic properties of soils has increased considerably in recent years, primarily as a result of the increased concern of both engineers and the public in the problems of earthquake safety. As a result many engineers are becoming increasingly involved in methods of evaluating the seismic stability of soil deposits, primarily saturated sands, and the dynamic response of these deposits during earthquake shaking.

The dynamic soil characteristics of primary interest in studying these problems are 1) the resistance to liquefaction of cohesionless soils such as sands and silty sands and 2) the shear moduli of the soils comprising the deposits. The use of *in-situ* testing techniques for evaluating these characteristics of sands forms the subject of this paper.

General Procedures for Evolution 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 dis-similarities between these sites and a proposed new site with regard to their potential behavior.

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 behavior 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 a 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 τ_h 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 sand 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):

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

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 dilatomer 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 undistrubed samples. Obtaining truly undisturbed samples which accurately reflect the

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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. 1 and 2. Fig. 1 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. 2 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

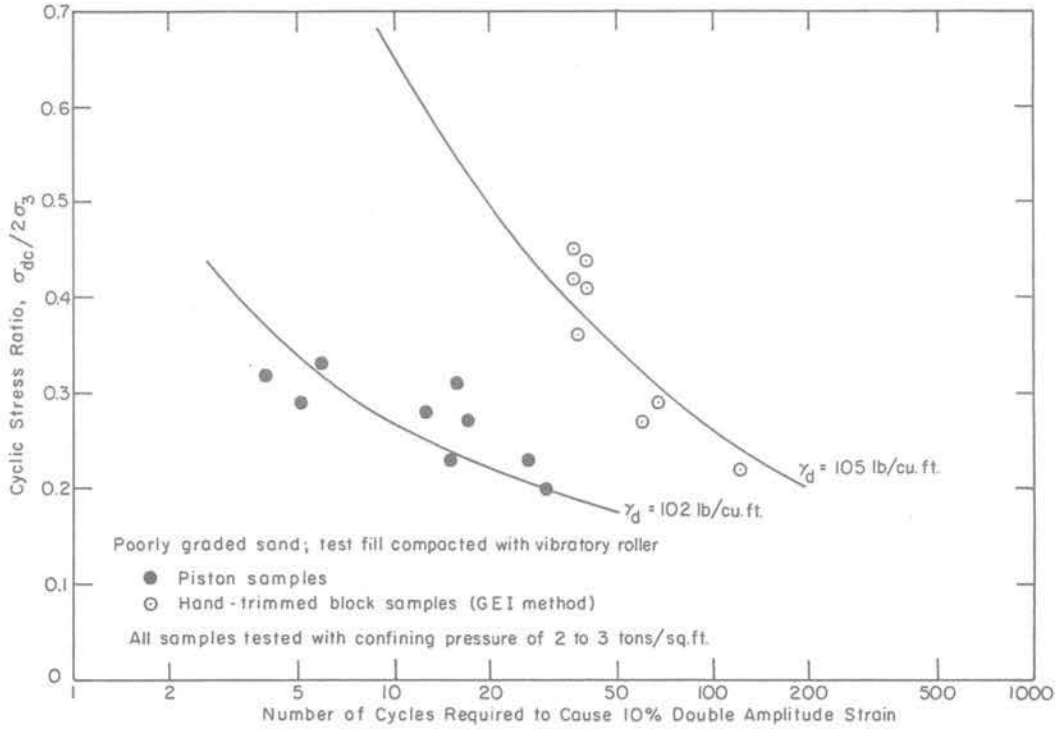

Fig. 1: Influence of method of sampling on cyclic loading resistance of dense sand

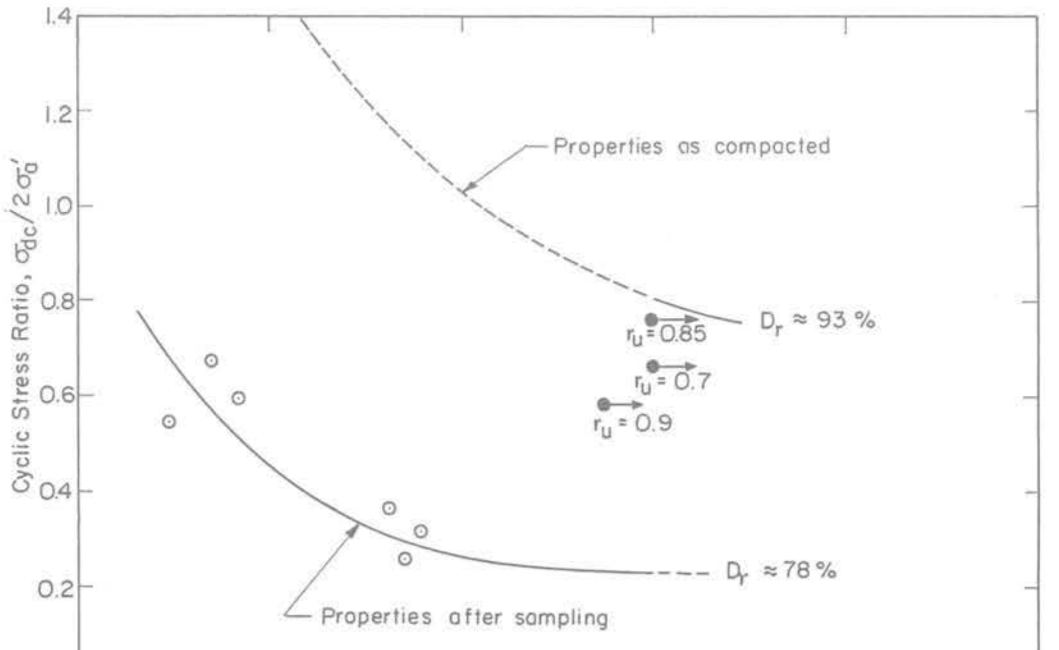

Fig. 2: Effect of sample disturbance on cyclic loading resistance of dense sand

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 preferred 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 in 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 in the hammer at impact when using the rope and drum procedure with two turns of the rope was only about 55 to 60 % of the 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) 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
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.

Correlation 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 earthquakeinduced liquefaction or cyclic mobility of 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 in the U.S. over a 14 year period (1969-present) and the most recent compilation of this field data collection is shown in fig. 3 (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

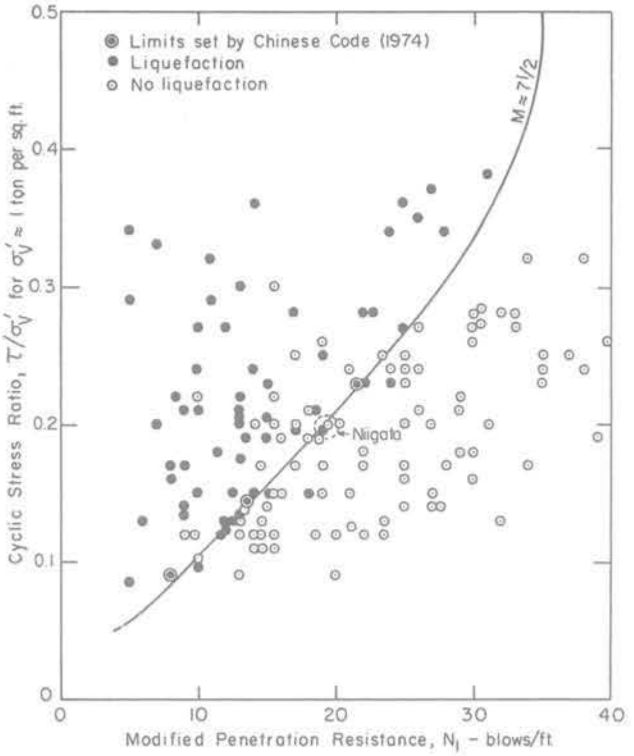

Fig. 3 : Correlation between field liquefaction behavior of sands under level ground conditions and standard penetration resistance.

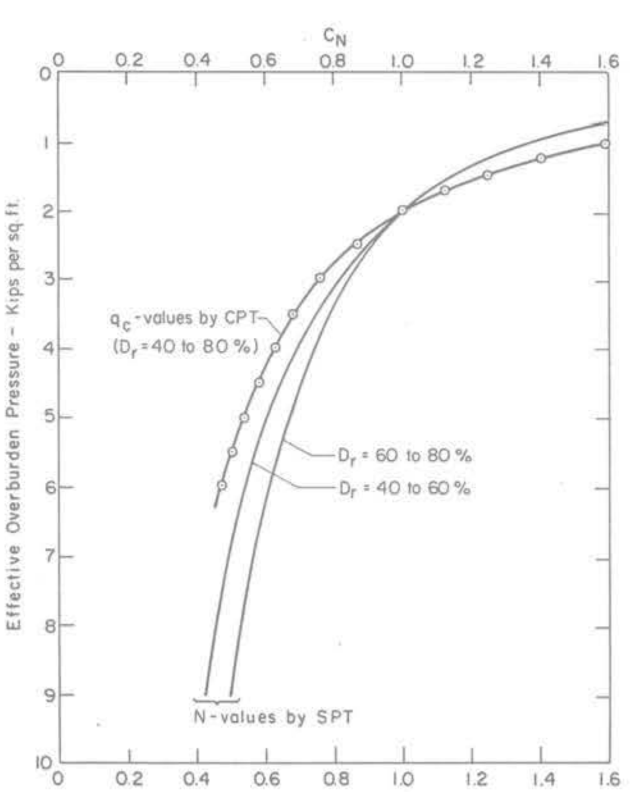

Fig. 4 : Relationships between C_N and effective overburden pressure

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

$$N_1 = C_N \, . \, N \qquad (2)$$

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. 4 which is based on recent studies conducted at the Waterways Experiment Station (Bieganousky and Marcuson, 1976; Marcuson and Bieganousky, 1976).

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 inf fig. 3) 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. 3 are from site studies in the United States, Japan, China, Guatemala, and Argentina and thus represent a wide range of geographical locations and conditions. The extent and consistency of the data used to define liquefiable conditions in Magnitude 7-1/2 earthquakes, shown in fig. 3, 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. 3 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. 3 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 Table1. 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).

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

Table I

Earthquake Magnitude, M	No. of Representative Cycles at 0.65 τ_{max}	$\dfrac{\left(\frac{\tau_{av}}{\sigma'_0}\right)_{\ell - M = M}}{\left(\frac{\tau_{av}}{\sigma'_0}\right)_{\ell - M = 7.5}}$
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

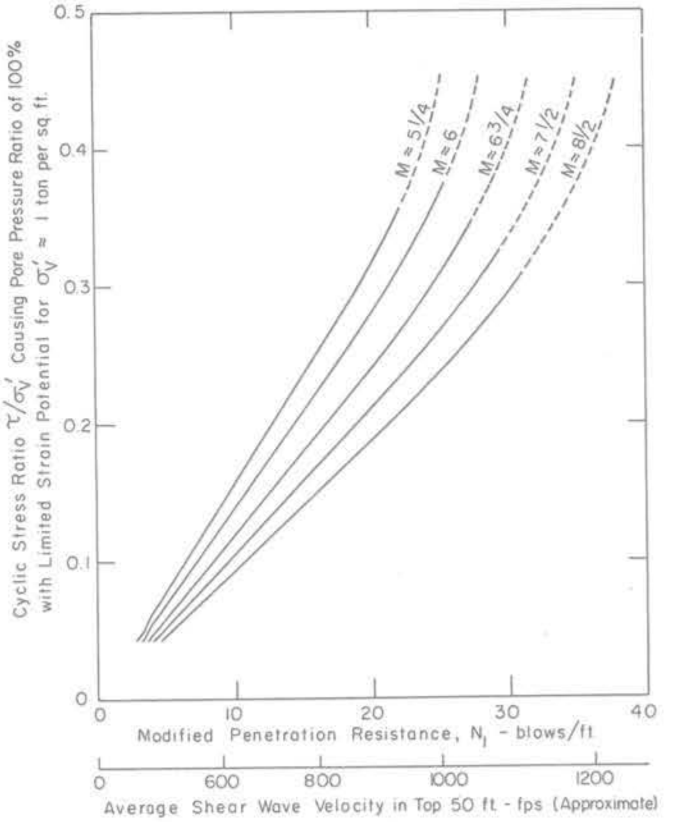

Fig. 5 : Chart for evaluation of liquefaction potential for different magnitude earthquakes.

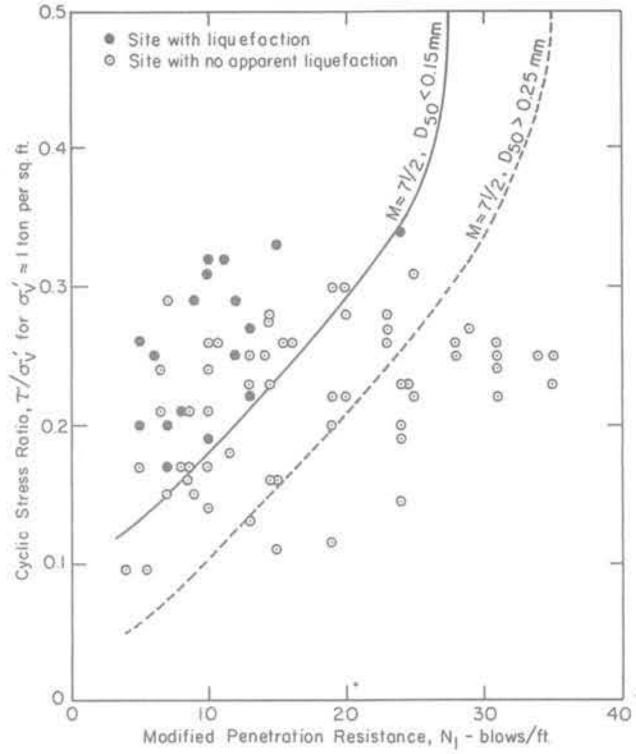

Fig. 6 : Correlation between field liquefaction behavior of silty sands under level ground conditions and standard penetration resistance.

with different magnitudes. Such a family of curves for sands is shown in fig. 5, 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 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. 3 in fig. 6. Also shown in fig. 6 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. 3. 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 ($D_{50} < 0.15$ 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 behavior of different types of soil, Zhou proposed 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/cm² 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 meagre 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 conjunction with the charts in figs. 3, 5 and 6 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 nonliquefiable conditions shown in figs. 3, 5 and 6 can be expressed in terms of a static Cone Penetration Resistance corresponding to an overburden pressure of 1 ton per sq ft, q_{cl}, by using the relationships

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

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. 7.

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

$$q_{cl} = q_c \cdot C_N \tag{3}$$

where values 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 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 non-liquefiable 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
$\simeq$ XI	16

This correlation, for a water table depth of 2 m, reduced to the same parameters as those used in fig. 3, 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. 8 where it is also compared with the lower bound line for sites

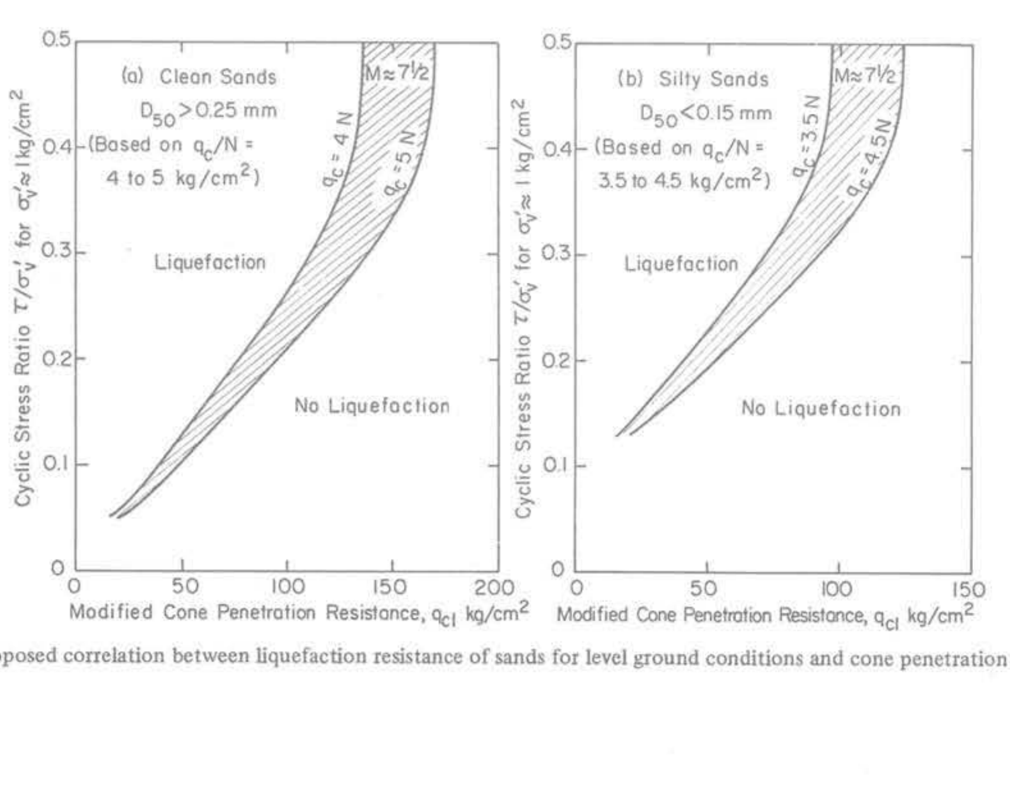

Fig. 7: Proposed correlation between liquefaction resistance of sands for level ground conditions and cone penetration resistance.

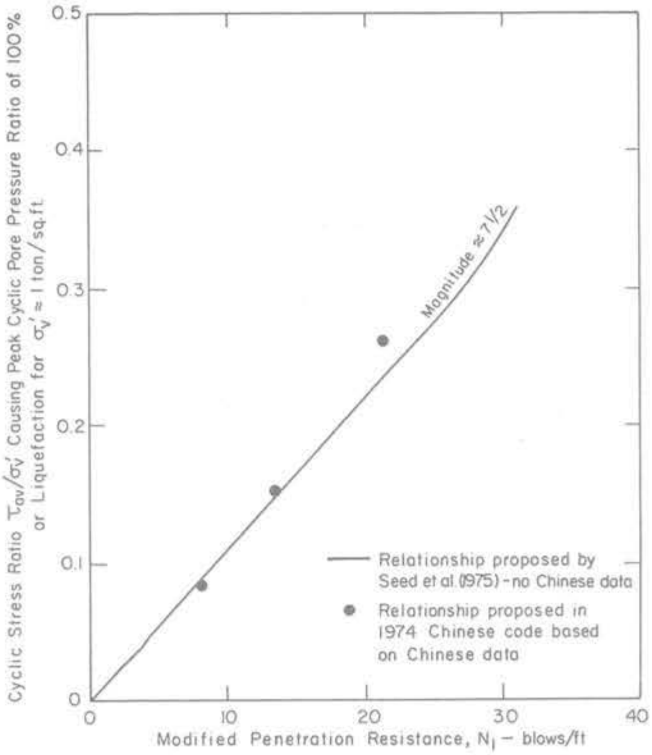

Fig. 8: Comparison of empirical chart for predicting liquefaction with recommendations of 1974 chinese code.

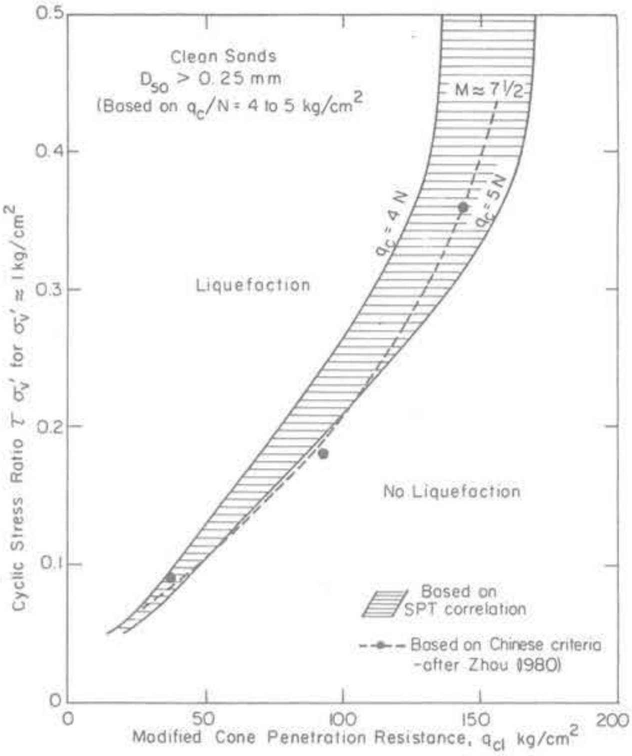

Fig. 9: Comparison of proposed correlation between liquefaction resistance and cone penetration resistance with criteria developed in China.

showing evidence of some degree of cyclic mobility or liquefaction shown in fig. 3. 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. 3. It is significant and remarkable that such a great similarity both in procedures and criteria should have evolved in countries with so little technical communication at the time the individual plots were developed.

Chinese Correlation of Liquefaction Resistance with CPT Data

Field studies in China at sites affected by the Tangshan earthquake ($M \simeq 7.5$) have also led to a method of evaluating the liquefaction resistance of clean sands based on cone penetration test data (Zhou, 1980). In this procedure, a critical value of cone penetration resistance, q_{crit}, separating liquefiable from non-liquefiable deposits of clean sands at depths up to 15 m below the ground surface is determined from the equation

$$q_{crit} = q_{co}\,[1 - 0.065\,(H_w - 2)]\,[1 - 0.05\,(H_0 - 2)] \tag{5}$$

where

H_0 = depth of sand layer under consideration (meters)

H_w = depth of water level below ground surface (meters)

q_{co} = function of shaking intensity as given by the following table

Modifiel Mercalli Intensity	VII	VIII	IX
Max. surface accn. (Chinese Code)	0.1 g	0.2 g	0.4 g
Value of q_{co} (kg/cm²)	47	117	180

This correlation, for a water table depth of say 2 m, can also readily be reduced to the same form as that shown in fig. 7 and it leads to the results shown in fig. 9. Again it may be seen that there is remarkably good agreement between the results developed in different countries and using different data sources.

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

$$G_{max} \simeq 120\,N^{0.8}\ \text{ksc} \tag{6}$$

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

$$G_{max} \simeq 65\,N\ \text{ksc} \tag{7}$$

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

it follows that

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

$$\simeq 55\sqrt{N}\ \text{m/sec.} \tag{8}$$

Noting that $N = N_1 C_N$, leads to:

$$v_s \simeq \frac{55\sqrt{N_1}}{\sqrt{C_N}} \tag{9}$$

In the upper 15 m of a sand deposit, the effective overburden pressure, σ_0', will be less than 4 000 psf and for values of σ_0' below this value, C_N is typically in the range 0.7 to 1.3 (see fig. 4). The corresponding values of $\sqrt{C_N}$ will be in the range of 0.85 to 1.15 so that a conservative average value might be about 0.9. Thus from the above equation:

$$v_s \simeq \frac{55\sqrt{N_1}}{0.9} \simeq 60\sqrt{N_1} \text{ m/sec} \tag{10}$$

for depths up to about 50 ft. This approximate relationship can be plotted along the abscissa of fig. 5, 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. 5 indicates that liquefaction will never occur in any earthquake if the shear wave velocity in the upper 50 ft of soil exceeds about 1 200 fps. This is in good agreement with the finding of Youd and Hosse (1980) that Holocene sand deposits, typically having $v_s \leqslant 700$ fps have been more disturbed by liquefaction than Pleistocene deposits for which $v_s \geqslant 1\,100$ fps.

Conclusion

In the preceding pages relationships have been proposed for evaluating the dynamic properties of sands by means of several *in-situ* tests. Specifically these include:

1. Liquefaction resistance from SPT data
2. Liquefaction resistance from CPT data
3. Liquefaction resistance from shear wave velocity data
4. Wave velocity and shear modulus from SPT data

Probably the best defined relationship is that between liquefaction characteristics and SPT because it is founded on such a large body of field performance data, and for this reason it is probably the most useful empirical approach for evaluating the liquefaction characteristics of sand at the present time. However it should be noted the Standard Penetration Test cannot be performed conveniently at all depths (say deeper than 100 ft or through large depths of water) or in all soils (such as those containing a significant proportion of gravel particles). Thus, it is desirable that it be supplemented by other *in-situ* test methods which can also be correlated with soil liquefaction potential. In many cases the Static Cone test, which can be performed more rapidly and more continuously, may provide a good means for evaluating liquefaction potential especially if it is correlated on a site-dependent basis with SPT results. However this procedure also is limited to sands and silty sands. In dealing with soils containing large particles or in difficult environments, other *in-situ* characteristics such as the shear wave velocity, dilatometer modulus (DMT), or the electrical characteristics of the soil may provide a more suitable means for assessment of liquefaction potential. And in due course any or all of these *in-situ* test methods may have their own detailed correlation with field performance to validate their usefulness as meaningful indicators of liquefaction characteristics. It seems likely however that for onshore sites and with deposits of sand up to 100 ft deep or so, the correlation of liquefaction characteristics with Standard Penetration Test data will provide the most direct emprical means of evaluating field liquefaction potential for some years to come. Other methods however have a significant role to play and should be developed to the fullest extent possible to provide information for different soil types and environments.

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