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EVALUATION OF LIQUEFACTION POTENTIAL

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1. INTRODUCTION

Following the Niigata and Alaskan earthquakes in 1964, geotechnical engineers were motivated to investigate earthquake induced liquefaction and its cause.Since then, many field documented cases where liquefaction has and hasnot occured have been used to construct correlations with earthquake activity. Parameters of importance were shown to include intensity and duration of the earthquake, distance from earthquake source, ground acceleration, depth of water table, overburden pressure, soil density, and soil type.

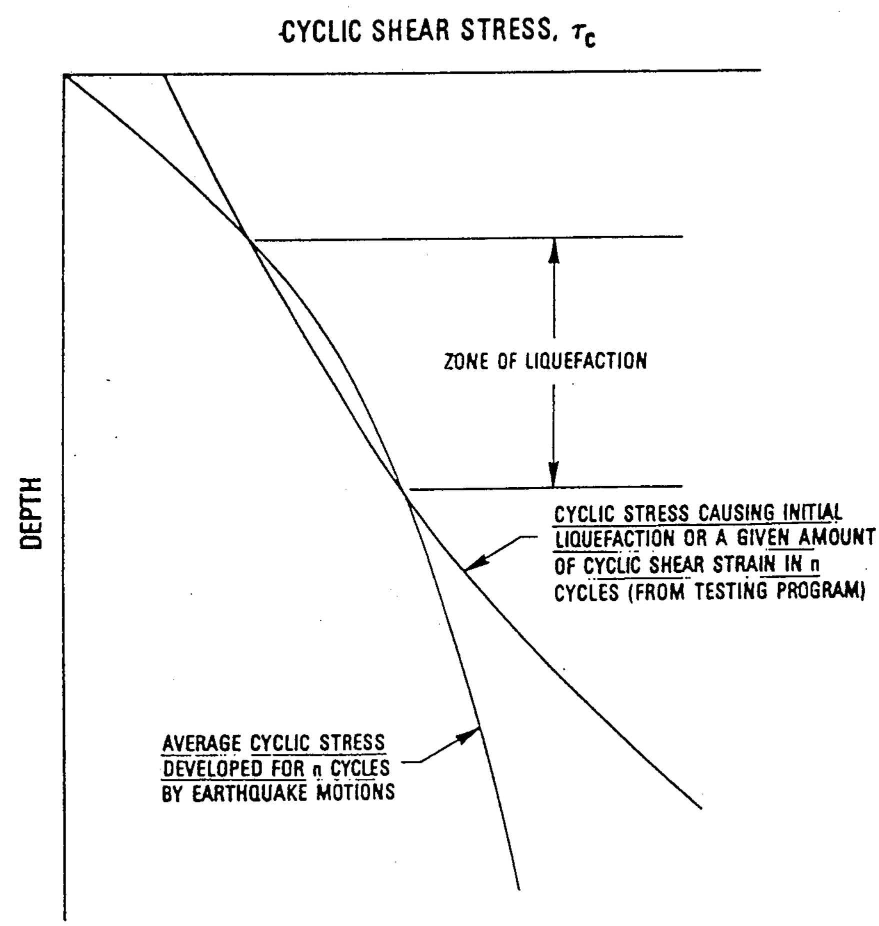
The purpose of this paper is to review some of the procedures commonly used for evaluating the liquefaction potential of soils particularly those containing a significant amount of fines. Because of the different procedures involved in the methods, the corresponding proposed mechanisms will be briefly discussed.

2. LIQUEFACTION MECHANISM AND THE CORRESPONDING EVALUATION METHODS

2.1. Concept of Pore Pressure Generation

The liquefaction of soils is considered to be a result of excess pore pressures generated during earthquake shaking. This occurs because when a loose saturated sand is subjected to vibrations, it tends to decrease in volume. If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore water pressure. If the pore pressure develops to the point at which it is equal to the overburden pressure, the effective stresses become zero, the sand loses its strength completely, at which point it is considered to be liquified (Seed and Idriss, 1982).

depth of the soil layer involved, the depth of the water table, and the intensity of earthquake shaking. The liquefaction resistance of the soil is therefore determined as the ability of the soil to resist cyclic shear stresses. The basic principle used in applying this concept in the early approaches involved comparison of the normalized cyclic shear stress causing initial liquefaction as determined from laboratory tests, τ_l/σ_o' , to the normalized cyclic shear stress expected to be developed in the field by earthquake, τ_d/σ_o'. Any zone where τ_d/σ_o' greater than τ_l/σ_o' was designated as *zone of liquefaction* (Fig. 2).



<u>Fig. 2 : Comparison of cyclic shear stress developed by earthquake and those determined in the laboratoriy or by other method</u>

A classic example of the method 1 was set out by Seed and Idriss (1971). It used laboratory data from tests conducted using uniform cyclic loads to simulate the soil behavior under the irregular cyclic loading of an earthquake. The test data were presumed to define the resistance of the soil to liquefaction by virtue of its response to a given number of load cycles. The driving shear stress to cause liquefaction involved the computation of the equivalent uniform cyclic shear stress, :f.tau sub av :ef., induced at any point in a soil deposit using the relationship :

$$\tau_{av} = 0.65 \bullet \gamma h \bullet \frac{a_{max}}{g} \bullet r_d \qquad \text{[eq. 1]}$$

in which a_{max} = maximum horizontal ground surface acceleration, γ . = total unit weight; h = depth below ground surface; r_d . = depth reduction factor. The 0.65 factor assumes that the equivalent uniform shear stresses, τ_{av}, is 65% of the absolute maximum earthquake induced shear stress. The depth reduction factor, r_d , recognizes that the soil is deformable and does not behave as a rigid body. A range of typical values of r_d . is from 1.0 at the ground surface to 0.5 at about 100 ft below ground surface.

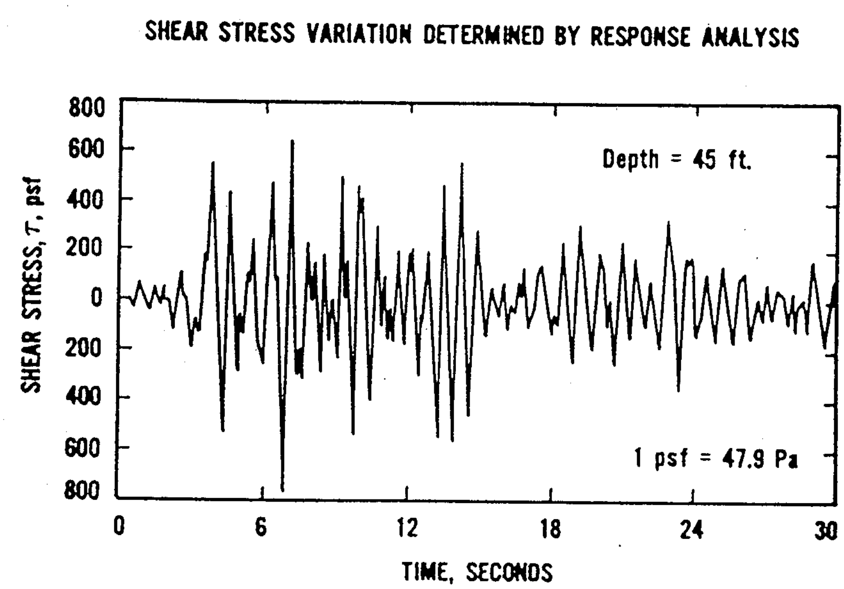


Fig. 3 : Typical Variation of shear stress from ground response analysis

and therefore are available to drive the mass should the soil lose sufficient strength. Driving shear stresses correspond to those that one would calculate in a stability analysis. Driving shear stress exists, for example, in the soil supporting a heavy building, within an earth embankment and its foundation and beneath natural slopes. The concept based on steady state shear strength suggests that the soil can be liquefied under conditions of either static or dynamic loading. Liquefaction failure occurs at the time when the undrained shear strength of the soil is exceeded by the driving stress present in the soil mass during undrained loading. The development of excess pore pressure and cyclic shear strain in a soil due to earthquake shaking tends to push the soil into the steady state condition.

When loose saturated cohesionless materials are subjected to shear loading, the pore water pressure increases, the effective stresses in the soil become very small, and the strength of the soil is reduced. If the shear stresses in the mass are larger than the reduced shear strength of the soil, the mass will flow like a viscous liquid until equilibrium is reached and the soil mass is stable. Casagrande (1936) termed this behavior *spontaneous liquefaction.*

It is generally understood that the steady-state concept is a derivation of the Casagrande's critical void ratio concept. Casagrande introduced the idea of a critical void ratio at which a cohesionless soil can undergo any amount of deformation without volume change. During shear deformation in a drained condition, a loose sand would tend to compress until it reached the *critical void ratio* . In an undrained condition, the tendency for volume reduction would result in increased pore water pressure and reduced effective stress.

In 1975, Casagrande proposed that during undrained loading, the effective stress in loose sand is reduced until it reaches the steady state line as illustrated in Fig. 4. The soil is initially at an effective minor principal stress, σ'_{3c} , prior to liquefaction. At liquefaction, with no volume change, the increased pore pressure results in a reduced effective stress, σ'_{3f} ,but no change in void ratio. The critical void ratio and the effective stress at flow with constant volume and velocity are uniquely defined by the steady state line. Thus, it was proposed that liquefaction and flow are related to large reductions in effective stress, with the final value of effective stress depending on the in situ void ratio.

The Steady State Line (SSL) is defined as the locus of all points in void ratio - stress space at which a soil mass deforms under conditioof constant effective stress, void ratio and velocity. When load is applied to soil, after sufficient straining, the soil will reach a point on the steady state line. This line is supposedly unique for a given soil and independent of the initial

According to the steady state concepts, the undrained steady state strength of a soil, S_{us} , is only a function of its void ratio, and thus the same value of S_{us} applies whether the soil is loaded monotonically or cyclically. Test data supporting this have been presented by Castro (1969, 1975) and Castro et al. (1982). Castro (1987) compared the stress strain behavior of soil under cyclic loading to the behavior under monotonic loading (Fig. 5.) for two cases. In the first case, the undrained steady state strength, S_{us} , is lower and in the other, S_{us} . is higher than the driving shear stress τ_d. The stress strain behavior under cyclic loading for case (1) is very similar to that under monotonic loading, i.e., the peak strength is overcome by sufficient straining which is either caused by a single or by repeated loading. The strain at which the resistance of the soil starts decreasing towards S_{us} is about the same in both cases and they end up at the same value of S_{us} since the void ratio is the same. In case (2) cyclic loading causes an accumulation of strain often accompanied by an increase in pore pressure. The shape of the stress strain curve following cyclic loading may be different from the monotonic loading case, however they both reach the same value of S_{us}.

There are substantial differences in the field behavior that correspond to cases (1) and (2) and also in the method of analysis to predict the field behavior. In case (1), the consequence of cyclic loading is a massive failure such as a flow slide. In case (2) cyclic loading induces a limited deformation without changing the stable configuration of the soil mass.

Evaluation of liquefaction potential using the steady state concept involves the following steps:

1.) Determination of the steady state line from laboratory consolidated undrained triaxial tests conducted on compacted samples.
2.) Careful field fixed piston sampling for determination of in situ void ratio.
3.) Determination of the undrained steady state shear strength of the undisturbed sample using consolidated undrained triaxial tests.
4.) Construction of a field steady state line based on the undrained steady state shear strength of the undisturbed sample and the assumption that the field line is parallel to the line determined for the compacted samples.
5.) Determination of the in situ steady state shear strength corresponding to the in situ void ratio.
6.) Calculation of a factor of safety against liquefaction based on the in situ steady state shear strength and the in situ driving stresses.

combination of void ratio and mean effective stress, sands and silty sands behave similarly if test conditions assure an equal proximity to the steady state.

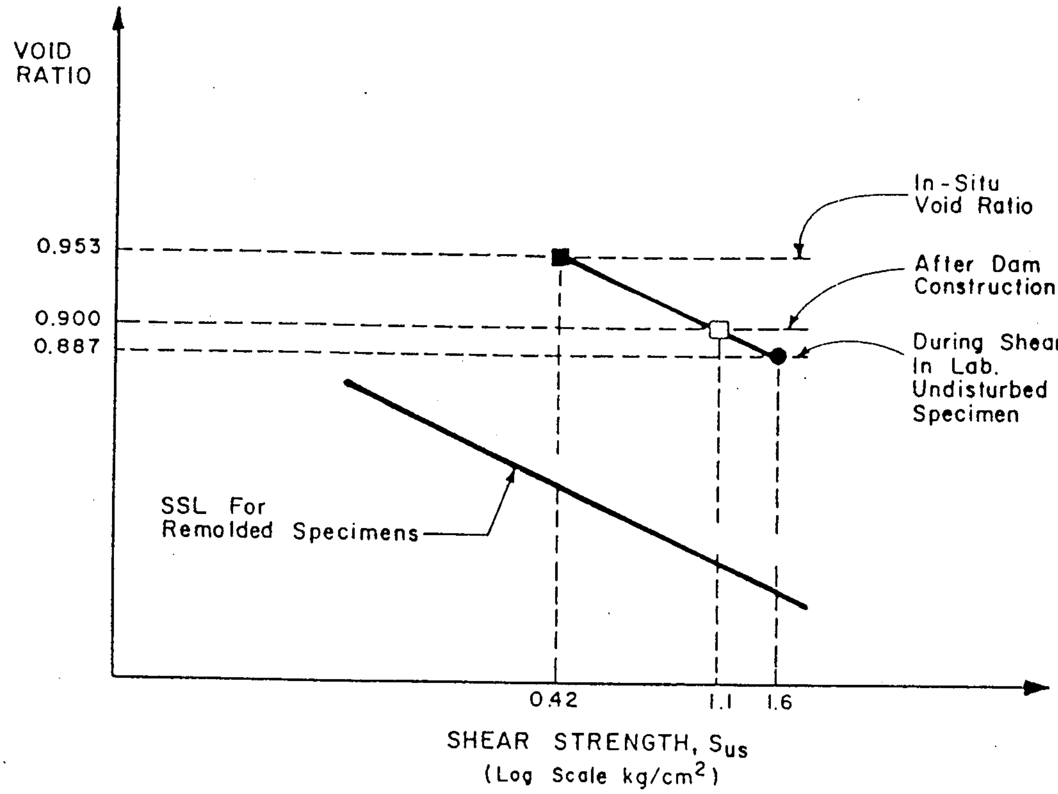


Fig.6 : Steady state Diagram Showing the Use of SSL for estimating the steady state undrained behavior

The *proximity* to the steady state is identified as the *state parameter* which is defined as the void ratio difference between the initial sand state and the steady state conditions at the same mean effective stress, σ_m' (Fig. 7). The choice of is based on the assumption that the deviatoric component of stress will be reflected directly in the sand fabric parameter. Been and Jefferies (1985) used the symbol ψ to represent the state parameter.

Been and Jefferies postulated that state parameter can be used to describe much of the behavior of granular materials over a wide range of stresses and densities. It was successfully applied to normalize large strain behaviors where the influence of initial fabric is small. The selection of steady state as reference line to the state parameter is based on the fact that at steady state conditions, sands and silty sands have :hp1.a unique structure :ehp1.which is not influenced by the initial conditions. Some authors (Rowe, 1962; Schofield & Wroth, 1968) stated that sand has no

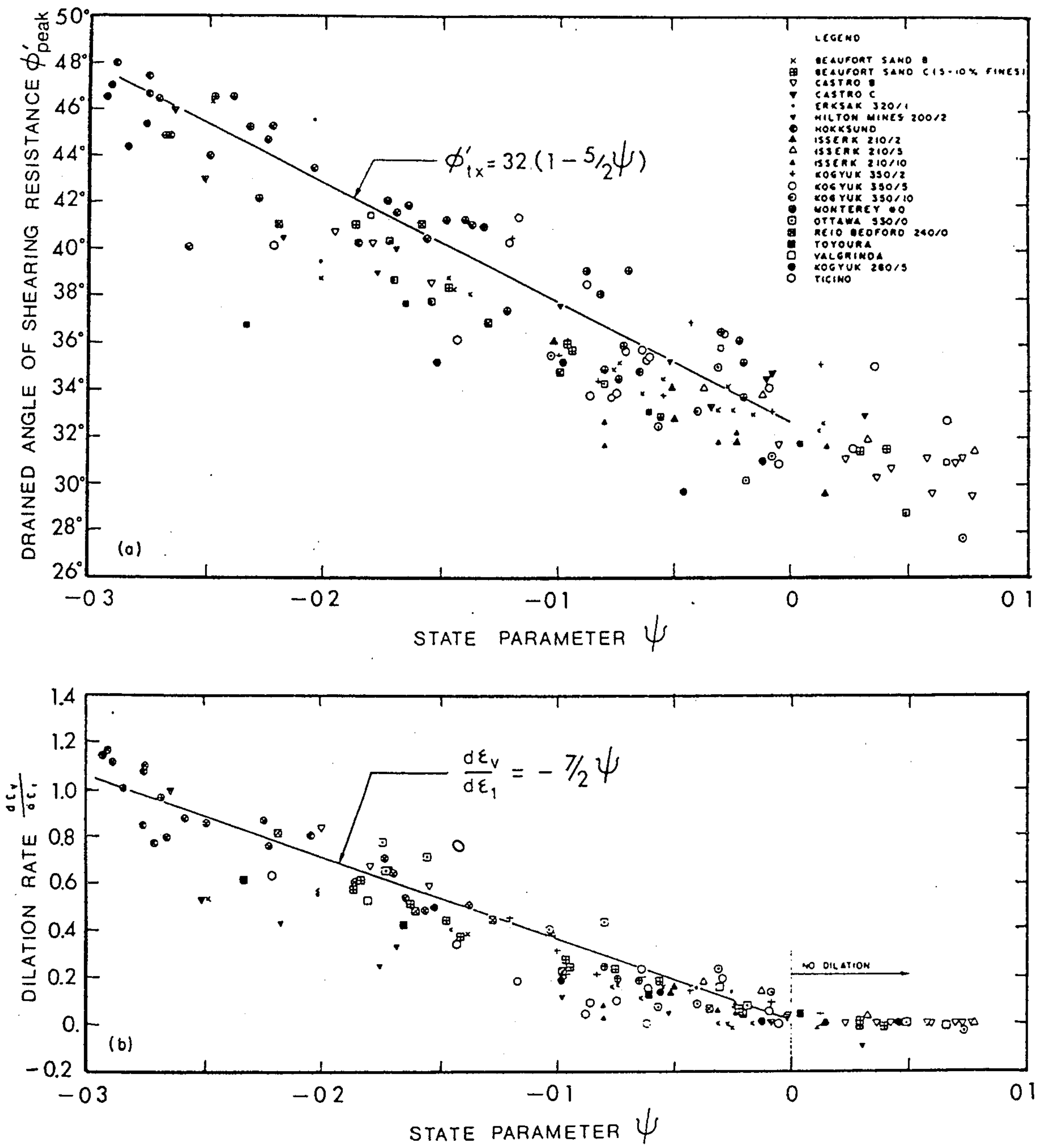


Fig. 8 Relationships of drained angle of shear resistance and dilation rate with state parameter

Observations of data from various earthquakes by Tokimatsu and Yoshimi (1983) indicated that more than half of all the liquefied materials fell within a range of fines content less than five percent and that none of the soils containing more than 20% clays has suffered serious strength loss due to liquefaction. A study by Seed and Idriss (1981) agreed with this conclusion. Meanwhile, Finn (1982) suggested from a study in mainland China that plasticity index could be a promising parameter to estimate the liquefaction resistance of silty soil. From his study, it was recommended that a plasticity index of 10 seems to be the threshold for the possibility of liquefaction to be eliminated.

In regard to in situ tests, Robertson et al. (1985) proposed chart to identify the type of soil that could liquefy based on data obtained from the CPT. Soil that is susceptible to liquefaction falls within area on the chart designated within the area of zone A (Fig. 10) Loose, clean sand with a $D_{50} > 0.25 - mm$ tend to fall within the upper area of zone A with q_c = 30 - 150 kg / cm^2 and friction ratio, FR < 1.0 %. Soil that falls within the lower area of zone A are the loose silty sands and silts since a decrease in mean grain size tends to cause a decrease in penetration resistance.

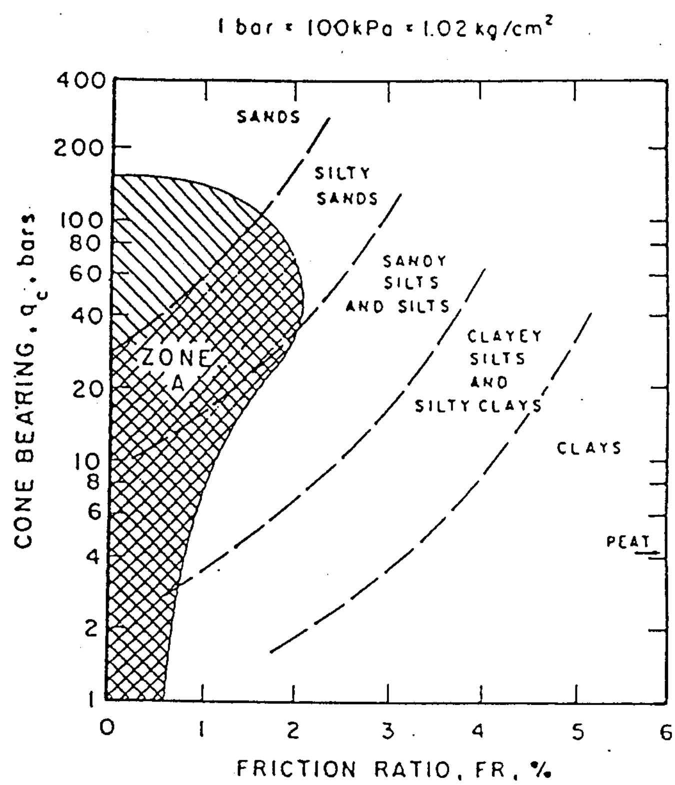


Fig. 10 : Soil Classificaition system (by electric cone) showing proposed zone of liquefiable soils (Robertson & Campanella, 1985)

In the last decade, researchers and practising engineers have focused their attention on the possibility of correlating the cyclic resistance of sand deposits with the cone penetration test (CPT) results (Zhou, 1981; Robertson dan Campenella) and Campanella, 1985; Jamiolkowski et al., 1985; Seed et al, 1986; and Shibata et al., 1987, 1988).

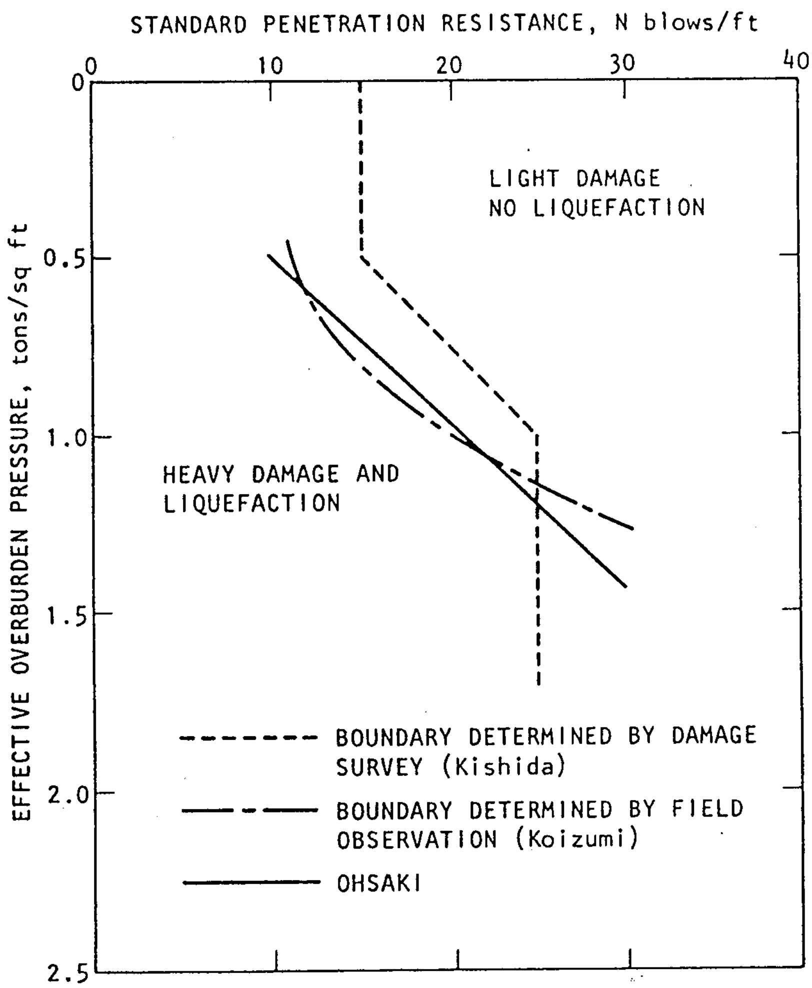


Fig. 11 : Analysis of Field Performance of Liquefaction Potential at Niigata Earthquake (Seed and Iddriss, 1982)

To adjust for the energy variability when using different types of hammers, an SPT blowcount corresponding to an energy ratio of 60%, $N_{1(60)}$ was recommended as a standard. The 60% energy ratio is typical of that obtained with a safety hammer, the most commonly used hammer type.

Based on data for silty sands $D_{(50)} <$ 0.15 mm from Tokimatsu and Yoshimi (1981) in the Miyagiken-Oki earthquake, Seed (1982) found that for a given blowcount, silty sands are less likely to liquefy than clean sands. He proposed a new boundary on his blowcount correlation chart to account for this. In other terms, it was noted that the normalized standard penetration resistance, N_1 for sand with $D_{(50)} >$ 0.25 mm was essentially equal to that of silty sands ($D_{(50)}$ < 0.15 mm) plus 7.5 for purposes of liquefaction potential evaluation. It was concluded that the boundary previously established for sands could be used for silty sands, provided the N_1 value for silty sand is increased by 7.5 before entering the chart. This correction can have a very significant effect on liquefaction evaluations for silty sand deposits. Later Seed et al. (1984) refined the curves to take into account sands containing fines more than 5% (Fig. 12.).

4.1.2. Method proposed by Tokimatsu and Yoshimi, 1983.

Using data from earthquakes in Japan, China and the USA, Tokimatsu and Yoshimi (1983) also examined the use of SPT in the liquefaction evaluation of sands containing fines. In their study, they divided the liquefaction behavior of these soils into four major groups: extensive liquefaction, moderate liquefaction, marginal liquefaction and no liquefaction. Extensive liquefaction is defined as when a sand layer experiences 2% strain during an earthquake or when a heavy structure settles more than 20 cm. For conditions when liquefaction is evident, but the resulting strains are less than the extensive case, the term moderate liquefaction is used. A site where there is no evidence of settlement or sand boils is called no liquefaction, and a marginal condition is used to classify the boundary between moderate liquefaction and no liquefaction. The results of their observation are shown in Fig. 13.

Fig. 14 shows the relationship between both fines contents and mean grain size and SPT N_1 values for liquefied soil It shows a fairly well-defined trend in which the SPT N_1 values for the liquefied soils decrease with either an increase in fines content or decrease in mean grain size. The mean :f. N sub 1 :ef. values for

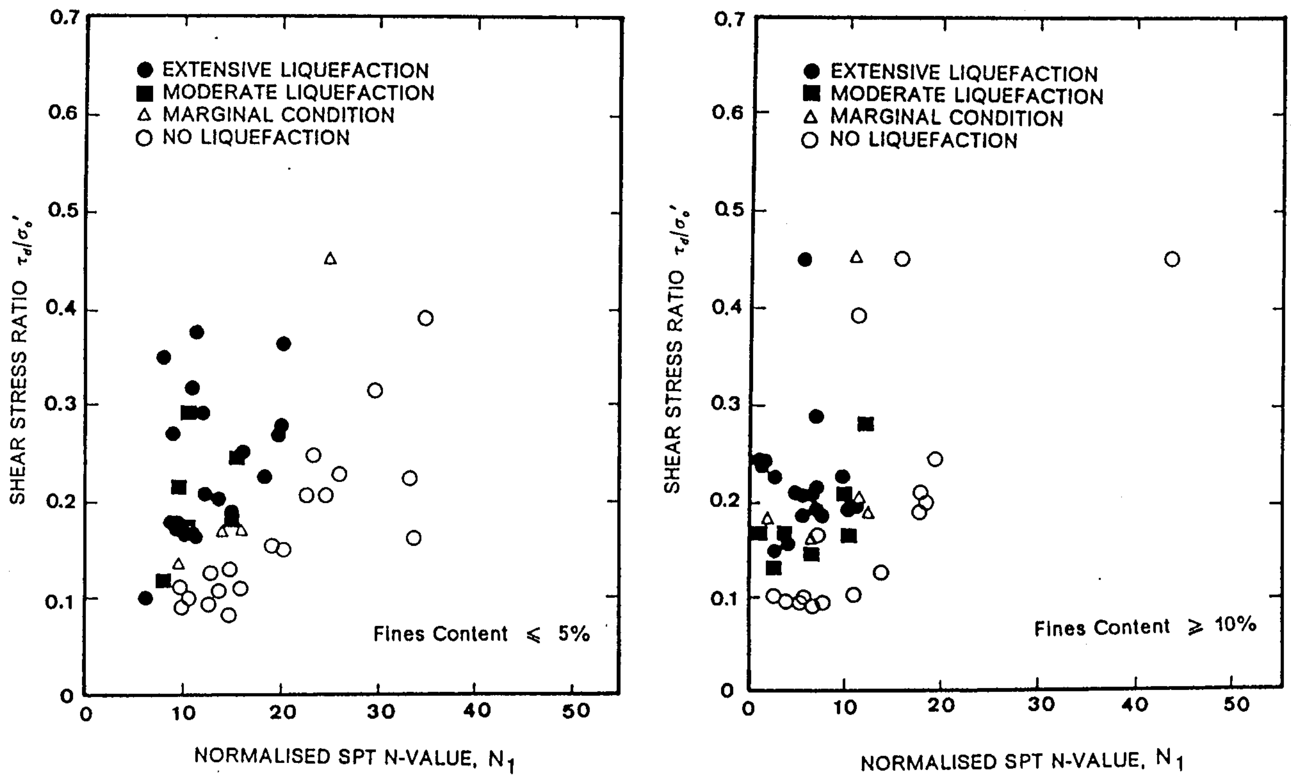


Fig. 13 : Correlation of field Cyclic Stess Ratio and SPT-N1
(Tokimatsu and Yoshimi, 1983)

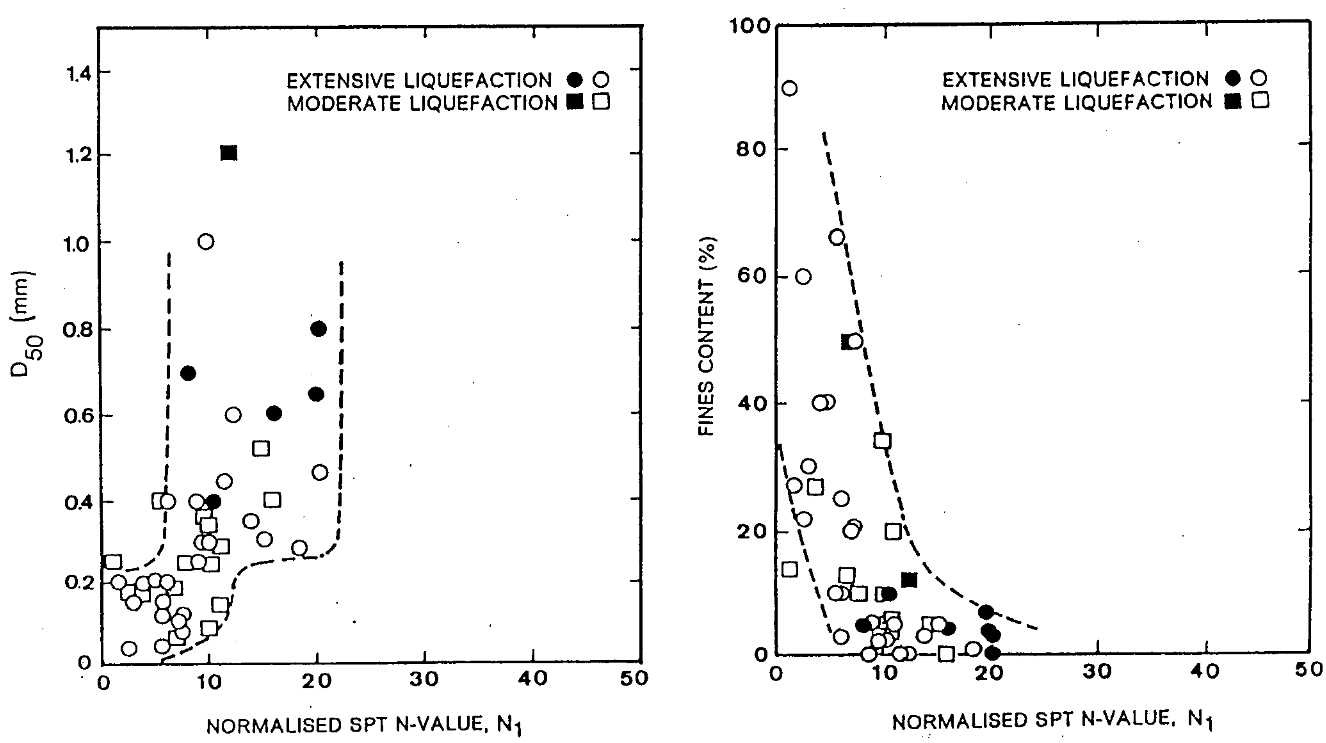


Fig. 14 : Relationship for Normalised SPT N1 with Liquefied Soils
(Tokimatsu and Yoshimi, 1983)

ratio.

Tokimatsu and Yoshimi noted that the method is limited to level ground conditions with no static load on the surface. For conditions where static shear stresses already exist in the ground, the method will likely overestimate the liquefaction potential.

4.1.3. Evaluation of Residual Shear Strength by SPT.

Recognising the importance of field data, Seed (1987) documented cases where major sliding has occured due to liquefaction that allowed back analysis of the strength parameters of the liquefied soil.

Seed developed an empirical relationship between the residual strength of liquefied sands and silty sands to the corrected SPT-N values of the soils (Fig. 15). The residual strength calculated by Seed may be considered to be analogous to the steady state strength described earlier.

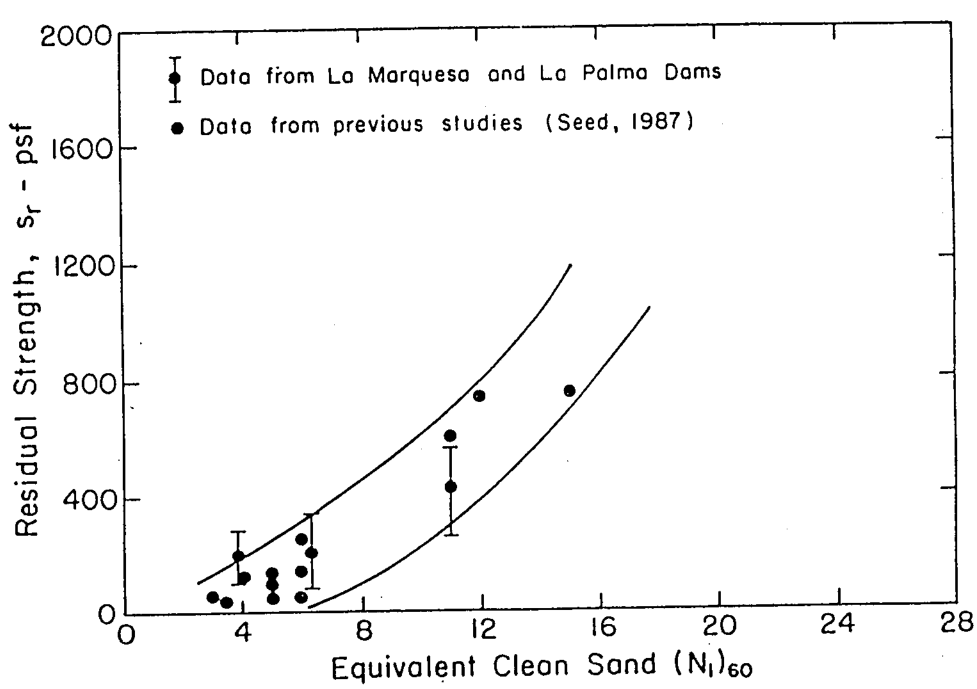


Fig. 15 : Relationship of Undrained Residual Strength Calculated from field data and Equivalent Clean Sand Values of SPT-N (de Alba et. al, 1987)

4.2. Liquefaction Potential Evaluation based on CPT.

4.2.1. Method proposed by Zhou (1981).

Zhou developed a method of evaluating liquefaction potential of sand by CPT based on field performance in the Tangshan (China) earthquake area. The sand layers in Tangshan are mostly clean sands or sands with minor amount of fines. A value of critical tip resistance based on statistical treatment of field data was proposed:

$$q_{cr} = q_{so}[1 - 0.065(H_w - 2)][1 - 0.05(H_o - 2)] \qquad \text{[eq. 7]}$$

where q_{cr} is the critical tip resistance at boundary soil is likely to liquefy during earthquake and :f. q sub so :ef. is the corresponding critical tip resistance for a depth of water table, Hw = 2 m and thickness of overburden pressure, Ho = 2 m. The proposed value of q_{so} depends on the intensity of the earthquake (Zhou, 1980).

This method agreed very well with the behavior in the the Haicheng earthquake. However, considerable deviations were found when the method was used to evaluate liquefaction phenomena over the Luthai area where the soil contained more fines. Zhou suggested that this deviation was due to the difference of the soil conditions between Luthai and the previously studied area, Tangshan. The average size of sand in Luthai is 0.064 - 0.078 mm while in Tangshan 0.076 - 0.61 mm, and the fines content in Luthai is higher (50 - 65 %).

To account for fines content in evaluating liquefaction susceptibility Zhou recommended a correction to q_c , Δq_c ,based on a laboratory study by Masamitsu (1977). Using the conversion factor of 4.5 for the ratio of qc/N, the suggested correction term is :

$$\Delta q_c = 584(\sigma_v' + 0.7)(\Delta R_l)^2 \qquad \text{[eq. 8]}$$

where σ_v' is effective overburden pressure in bars and ΔRl is the correction for cyclic stress ratio for liquefaction due to the increase of fines content. The value of ΔRl is between 0.0022 FC to 0.0075 FC where FC = fines content in percent. In any case ΔRl does not exceed a value of 0.175. Zhou found that eq.8 could be used to explain behavior in the Luthai area. It is interesting to note that correction for the tip resistance, Δqc, corresponds to that of SPT correction for fines content suggested by Seed et al. (1982).

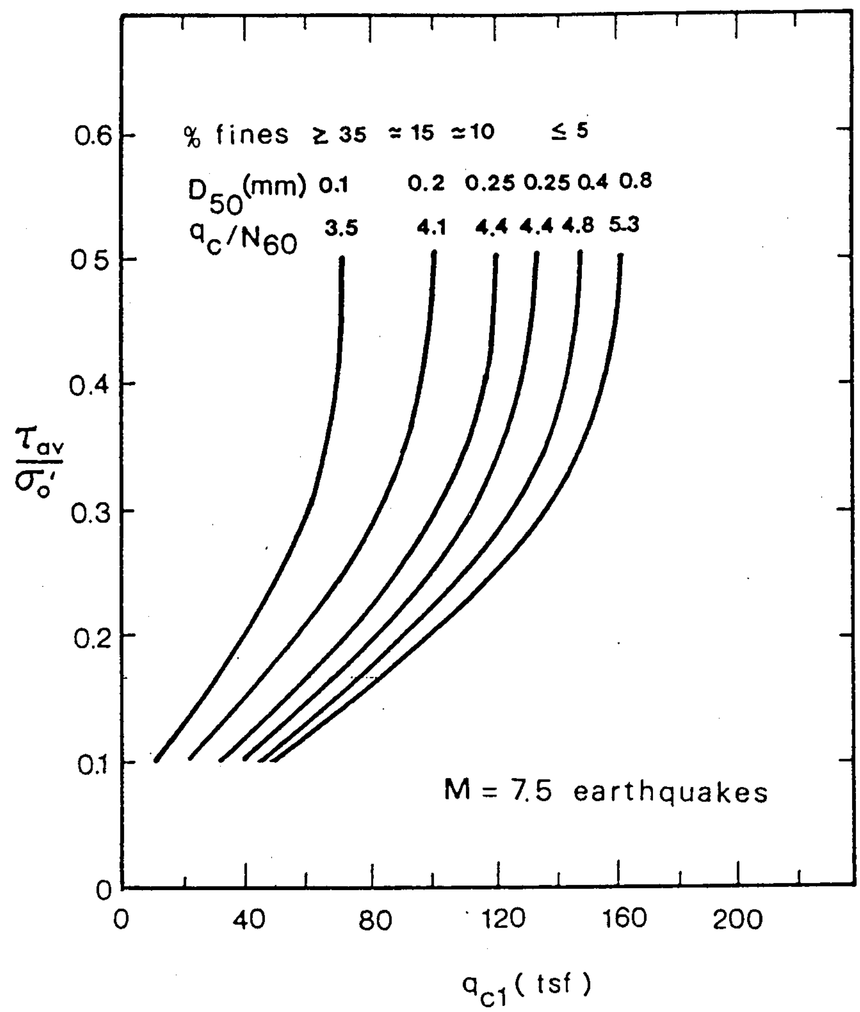
Like the SPT method proposed by Seed, the cone tip resistance is first normalised to an overburden pressure of 1 kg/cm2 using a correction factor Cq which was established using data from Baldi et al. (1981) similar to those proposed by Seed (1983) for SPT. Soils that fall within the upper shaded area of zone A in Fig.10 can be considered sands with $D_{50} > 0.25$ mm and soils that fall within the lower hatched area of zone A can be considered as silty sands or silts with $D_{50} < 0.15$ mm. Fig. 16 is to be used in the same manner as Seed's curves. The site chosen include Vancouver, Niigata and Tokyo Bay, and the USA. The data include field CPT and SPT profiles and laboratory cyclic triaxial test. The data from Canada and from Niigata for sands with $D_{50} < 0.15$ mm plot well above the proposed CPT correlation. It interesting to note that the data for silty sands with $D_{50} < 0.15$ mm plot well above the proposed CPT correlation.

4.2.3. Method proposed by Seed and de Alba (1986).

Seed and de Alba (1986) published a modification of Seed's earlier theory of predicting liquefaction susceptibility using the SPT which incorporates the use of cone penetration test results. Their method is based on the relationship between CPT tip resistance and SPT-N values, and on CPT results from liquefied deposits. As shown in Fig. 2.21., the correlations are presented in the same fashion as in SPT method, and the design procedure employed is much the same. However, unless the grain size characteristics of the soils involved are known with a good degree of reliability, use of this chart may lead to uncertainty owing to the sensitivity to D_{50}.

4.2.4. Method proposed by Shibata and Terapaksa (1987, 1988)

Shibata and Terapaksa (1987,1988) proposed the latest liquefaction assessment method by CPT results for clean sand and silty sand based predominantly on the field performance that has occured during the past several earthquakes. Fig. 18 shows the proposed correlation in the form of the normalised cone resistance, qc1 and cyclic stress ratio, that develop in the field during an earthquake. This applies for both clean sands with $D_{50} > 0.25$ mm and silty sands with $D_{50} < 0.25$ mm. Their findings confirm that there is fairly well defined trend in which the upper bound of qc values for liquefied soils decreases with an increase in mean grain size for $D_{50} < 0.25$ mm. This suggests that given the same value of qc, liquefaction resistance is greater for smaller grain sizes. On the



<u>Fig. 17 : SPT Based Convertion to CPT for Liquefaction Assessment</u>
(Seed and de Alna, 1986)

5. SUMMARY.

The preceeding sections shows that many different theories have been provided for the assessment of the liquefaction potential of soils. Each of these theories differs in the assumptions involved in the analysis, their use of laboratory and in situ data, and in the procedure of evaluation.

Some effects of liquefaction in the field, such as the occurence of sand boils and the differential settlement of structures due to uneven post earthquake densification of the foundation soil, can be explained by the presence of excess pore pressures and associated hydraulic gradients. Other manifestations of seismically-induced liquefaction, which are associated with large or unlimited shear straining of the soil, can be explained by the decrease in shear strength associated with excess pore water pressure. Both pore water pressure generation and the decrease in shear strength are studied in this research and correlated to the CPT.

While Seed and his collegues explained the phenomena from the point of view of the generation of high pore pressure and large accumulated cyclic deformations due to shear stress developed during earthquake, Casagrande and his followers have expressed the importance of the critical void ratio on the behavior of the soil during monotonic and cyclic loading. The approach by Seed et al. is based on the cyclic strength ratio that can be obtained by cyclic triaxial test in laboratory or by empirical correlation based on field evidence. The approach by Casagrande and his colleges requires the existence of driving shear stresses for liquefaction failure to occur, i.e. if the driving shear stress is lower than the undrained steady state strength, then a liquefaction failure will not occur. In such a case, cumulated strain will produce lateral spreading. Also, definition of liquefaction by Castro involves large shear deformations which occur at relatively rapid rate so that it appears to be flowing. The steady state approach is based on laboratory stress-strain behavior and the undrained residual strength.

In an effort to clarify the mechanism of earthquake induced phenomena, Castro (1987) classified the phenomena into three different category, i.e., (1) sand blows and settlement of level ground, (2) flow slide liquefaction and (3) limited deformation or lateral spreading. The main characteristics of the three induced phenomena are presented in Table 3. where the controlling soil properties of mechanism (1) were shear modulus, permeability and compressibility, while the controlling soil parameter for mechanism (2) and (3) was the steady state shear strength. However, Seed and his collegues prefer *relative density* as the key index characterizing the cylic strength of the soil.

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