LIQUEFACTION POTENTIAL OF SATURATED SAND - THE STIFFNESS METHOD

by

R. Dobry[I], D.J. Powell[II], F.Y. Yokel[III] and R.S. Ladd[IV]

SUMMARY

The paper proposes a new stiffness method for evaluating the liquefaction potential of horizontal saturated sand layers (level sites) during earthquakes. The method is based on field measurements of the shear modulus of the sand at small strains, G_{max}, using geophysical techniques. Once G_{max} is known for the layer, the threshold ground surface acceleration, $(a_p)_t$ which just generates an excess pore pressure in the layer, can be determined. This calculation is based on experimental evidence showing that sands possess a threshold strain, $\gamma_t \simeq 10^{-2}\%$. Simplified design charts for layer depths between 0 and 30 feet are developed for practical use, and are shown to be generally consistent with reported liquefaction behavior during past earthquakes.

INTRODUCTION

A major cause of failure of civil engineering facilities during earthquakes is the liquefaction of saturated sand deposits. For sites which are level or almost level, reported effects have included occurrence of sand boils, flotation of buried concrete tanks, cracking of pavements, settlement and tilting of structures and lateral spreading failures. Kuribayashi and Tatsuoka (1975) and Dobry et al. (1980) have listed about 60 earthquakes between 1872 and 1980 which induced liquefaction. The best known is the 1964 Niigata, Japan earthquake, where tilting and failure of multistory buildings due to liquefaction of the foundation sand was widespread. In the last few years, liquefaction has been reported in a number of earthquakes including the following: 1974 Haicheng and 1976 Tangshan earthquakes, both in China; 1977 San Juan, Argentina earthquake; 1978 Miyagi-Ken-Oki, Japan earthquake; and 1979 El Centro, USA earthquake.

Present state-of-the-art procedures are available to predict liquefaction potential of level sties. They are mainly based on the cyclic stress approach recently summarized by Seed (1979). These procedures compare the cyclic stress ratio induced by the earthquake in the field, $(\tau/\sigma'_v)_e$, with the cyclic stress ratio needed to cause liquefaction of the soil in a comparable number of cycles, $(\tau/\sigma'_v)_{cs}$. The value of $(\tau/\sigma'_v)_{cs}$ (also called cyclic strength ratio) is obtained either from empirical correlations using Standard Penetration Test (SPT) results, or from laboratory cyclic tests performed on representative sand specimens. In practice, the determination of $(\tau/\sigma'_v)_{cs}$ is plagued with problems and uncertainties, as discussed in detail by Seed (1979),

I,II Assoc. Prof. and Grad. Student, respectively, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, New York, U.S.A.

III Leader of Geotechnical Engineering Group, National Bureau of Standards, Washington, D.C., U.S.A.

IV Associate and Laboratory Director, Woodward-Clyde Consultants, Clifton, New Jersey, U.S.A.

Peck (1979) and Dobry et al. (1980). This is especially true if laboratory tests are used. It is very difficult or impossible to test truly representative sand specimens. In addition, the results of these tests are very sensitive to such factors as relative density, fabric, overconsolidation, geologic age and seismic history of the soil deposit. In many cases, laboratory determinations underestimate the field value of $(\tau/\sigma'_v)_{cs}$ and provide answers that are too conservative (Peck, 1979). The use of empirical correlations would seem to be more reliable, but it has also problems (Seed, 1979). Due to these problems and to the strictly empirical nature of the SPT (what property of the sand does it measure?), it is very difficult to refine or extend these correlations to other situations which are not included in the original data base.

The authors have proposed elsewhere an alternative cyclic strain approach to the prediction of pore water pressure buildup and liquefaction (Dobry and Swiger, 1979; Dobry and Ladd, 1980, Yokel et al., 1980; Dobry et al., 1980). This new approach is based on laboratory evidence showing that shear strains rather than shear stresses control settlement and pore water pressure buildup of sands during cyclic loading. The strain approach predicts that, other things being equal, stiffer sand deposits are less likely to liquefy. This increased stiffness of the sand layer can arise from a combination of different factors such as increased relative density, increased overconsolidation, stronger fabric, cementation due to geologic aging under pressure, or prestraining due to prior seismic history. Note that these factors are the same which have been shown to increase the cyclic strength of sands in the laboratory. However, detailed knowledge of these factors for a particular site is not absolutely necessary, as the stiffness (shear modulus) of the soil at small strains, G_{max}, can be measured directly in the field using geophysical techniques.

This paper proposes a stiffness method for evaluating liquefaction potential of saturated level sand layers based on field measurements of G_{max}. The method uses the threshold strain concept as described below. Additional information about the method is presented by Powell (1979) and Dobry et al. (1980).

MAXIMUM SHEAR MODULUS AND THRESHOLD STRAIN

Nondestructive geophysical methods for field measurement of the soil's shear wave velocity, V_s, have developed rapidly in the last few years. A number of these techniques are now available (Woods, 1978). Of these methods, the most widely used for geotechnical earthquake engineering purposes is the cross-hole technique, which can provide a detailed profile of V_s versus depth at a given site. Once V_s is known for a soil layer, the maximum shear modulus of the layer, G_{max}, is obtained from Eq. 1:

$$G_{max} = \rho V_s^2 \qquad (1)$$

where ρ = mass density of soil layer = total unit weight/acceleration of gravity.

Several authors have discussed the existence of a threshold cyclic shear strain, $\gamma_t \simeq 10^{-2}\%$, for sands. If cyclic strains below this value are induced in the soil, there is neither densification of dry sands nor pore

water pressure buildup of saturated sands (Drenevich and Richart, 1970; Pyke, 1973; Dobry and Grivas, 1978; Dobry and Swiger, 1979). Further evidence about γ_t is presented in Fig. 1, which shows the residual pore pressure after 10 loading cycles in strain-controlled cyclic triaxial tests recently performed by the authors on saturated Monterey No. 0 Sand. Figure 1 clearly demonstrates the existence of the threshold, $\gamma_t \simeq 1.1 \times 10^{-2}\%$, which appears to be independent of Relative Density for the range 45% to 80%.

STIFFNESS METHOD FOR LIQUEFACTION POTENTIAL

A simplified soil model of a level sand site is used to develop the proposed stiffness method (see Fig. 2). This model is essentially the same used by Seed and Idriss (1971) and Seed (1979) as part of the stress approach to liquefaction discussed above. The static state of stresses on a soil element at depth z is defined by the effective vertical pressure, σ'_v, by the effective horizontal pressure, $\sigma'_h = K_o \sigma'_v$, and by the total vertical pressure, $\sigma_v = \sigma'_v + \gamma_w(z-z_w)$, where $\gamma_w = 62.4$ lb/ft^3 is the unit weight of water. The peak horizontal ground surface acceleration, a_p, is associated with a peak shear stress, τ_p, acting on the horizontal and vertical planes of the soil element. The value of τ_p is

$$\tau_p = \frac{a_p}{g} \sigma_v r_d \qquad (2)$$

where g = acceleration of gravity = 32.2 ft/sec^2, and $r_d \leq 1$ is the soil flexibility coefficient defined by Seed and Idriss (1971). At the ground surface, $r_d = 1$ and at a depth z = 30 feet, $r_d = 0.92$.

Following Seed (1979), a representative cyclic shear stress caused by the earthquake, τ_e, can be estimated to be about 65% of the peak value, τ_p. Therefore:

$$\tau_e = 0.65 \frac{a_p}{g} \sigma_v r_d \qquad (3)$$

and the associated cyclic shear strain, $\gamma_e = \tau_e/G$ is:

$$\gamma_e = 0.65 \frac{a_p}{g} \frac{\sigma_v r_d}{G_{max} (G/G_{max})_{\gamma_e}} \qquad (4)$$

where G = equivalent shear modulus of the soil and $(G/G_{max})_{\gamma_e}$ = modulus reduction coefficient, both determined at a strain equal to γ_e(Seed and Idriss, 1970).

For a cyclic strain equal to the threshold, $\gamma_e = \gamma_t = 10^{-4}$ $(10^{-2}\%)$, $(G/G_{max})_{\gamma_e} = 0.65$ to 0.90 for sands (Seed and Idriss, 1970; Iwasaki et al., 1978). For $\gamma_e = 10^{-4}$, a representative value $(G/G_{max})_{\gamma_e} = 0.75$ was selected. Therefore, Eq. 4 becomes:

$$(a_p)_t = 1.154 \times 10^{-4} \frac{G_{max}}{\sigma_v r_d} \quad \text{(g's)} \qquad (5)$$

Equation 5 is the basis of the stiffness method proposed here. If the value of G_{max} corresponding to the sand layer at depth z is measured in the

field using the cross-hole technique, Eq. 5 can be used to determine the threshold peak ground surface acceleration, $(a_p)_t$ needed to start the development of excess pore water pressure at that depth. If the design earthquake acceleration for the site, a_p is less than $(a_p)_t$ determined from Eq. 5, there will be no pore pressure buildup and there is no danger of liquefaction. On the other hand, if $a_p > (a_p)_t$, the design earthquake will induce a pore pressure buildup at depth z and further liquefaction studies are necessary. For this latter case, the actual occurrence of liquefaction and its effects on civil engineering facilities will depend on factors such as the duration of the earthquake, and the relative density and drainage boundaries of the sand layer.

A study of several dozen liquefaction case histories available from the literature was performed by the authors using Eq. 5 and estimating G_{max} from available SPT values. The results have been published elsewhere (Dobry et al., 1980). This study showed that in all sites that had liquefied, the earthquake peak ground surface acceleration was larger or about equal to the threshold, $a_p \gtrsim (a_p)_t$. The only two sites for which $a_p \simeq (a_p)_t$ corresponded to a large magnitude (M = 8.3) earthquake which caused a very long duration of shaking.

SIMPLIFIED DESIGN CHARTS

Equation 5 was used in conjunction with Fig. 2 to develop the simplified design charts presented in Figs. 3 and 4. A total unit weight = 115 lb/ft^3 was assumed for the soil at all depths, and a stiffness coefficient, A, was defined as follows:

$$A = \frac{G_{max}}{816.5\ \sqrt{\sigma'_v}} \qquad (lb^{1/2}/ft) \tag{6}$$

where G_{max} and σ'_v are in lb/ft^2. This definition of A is related to the equation for G_{max} suggested by Seed and Idriss (1970):

$$G_{max} = 1{,}000\ K_{2max} \sqrt{\frac{1}{3}\ (1 + 2K_o)\ \sigma'_v} \tag{7}$$

In this work, A is used rather than the coefficient K_{2max}, to avoid the need for assuming the value of K_o, which is difficult to determine. Measured values of A in sands range between about 35 (for very loose, normally consolidated recent deposits) and more than 150 (Dobry et al., 1980). If it is assumed that $K_o = 0.5$, then $A = K_{2max}$ and Eqs. 6 and 7 become one and the same.

By replacing Eq. 6 into Eq. 5 and by making $\sigma_v = 115\ z$ (lb/ft^2) and $\sigma'_v = 115\ z_w + (115 - 62.4)\ (z - z_w)$ (lb/ft^2), Eq. 8 can be obtained:

$$(a_p)_t/A = 8.2 \times 10^{-4}\ \frac{\sqrt{62.4\ z_w + 52.6\ z}}{z\ r_d} \tag{8}$$

The simplified Eq. 8 is valid below the water table, $z \geq z_w$. Eq. 8 has been plotted in Fig. 3 as a function of z_w and for depths z = 10, 20 and 30 feet (this covers the range of depths where liquefaction most frequently occurs). The same equation is plotted as $(a_p)_t$ versus z_w in Fig. 4 for the depth, z = 20 feet and for A = 35, 100 and 150. Fig. 4 clearly shows the influence of

the groundwater table depth, z_w, and of the stiffness parameter, A, on liquefaction potential. For a very loose sand layer (A = 35), at 20 feet and if z_w = 0, the threshold is $(a_p)_t \simeq 0.05g$. This corresponds approximately to a Modified Mercalli Intensity, MMI ≃ VI, and suggests that no liquefaction is possible for peak ground accelerations lower than about 0.05g and MMI < VI, even for the worst soil conditions and for large earthquake magnitudes causing the longest durations of shaking. This conclusion is generally consistent with reported cases of liquefaction during earthquakes (Dobry et al., 1980).

Especially impressive in Fig. 4 is the influence of stiffness A on $(a_p)_t$. Let us assume that we have two adjacent soil profiles, both with the groundwater level at z_w = 10 feet and potentially liquefiable sand layers at z = 20 feet. The design surface acceleration is also the same, a_p = 0.15g and corresponds to a Magnitude = 8 (long duration earthquake). In Site 1, the sand layer has a measured A = 35. Therefore, $(a_p)_t \simeq 0.06g$ and this layer will most probably liquefy. In Site 2, the sand layer has a measured A = 150. Therefore, $(a_p)_t \simeq 0.26g$. This layer will not even start developing an excess pore pressure, let alone liquefy during the design earthquake.

DISCUSSION AND CONCLUSIONS

The proposed stiffness method for sand liquefaction potential has several advantages when compared with other available techniques:

i) It is based on field measurements of a basic soil parameter (G_{max}), for which reliable techniques are available. Therefore, it avoids both the problems associated with laboratory cyclic testing and the empiricism of the Standard Penetration Test.

ii) It replaces a number of soil parameters believed to influence liquefaction resistance by the stiffness of the soil, thus allowing a considerable simplification of the problem.

iii) It is based on laboratory cyclic tests showing that there is an approximately constant threshold shear strain for sands, $\gamma_t \simeq 10^{-2}\%$.

In addition, the method provides results which are consistent with liquefaction experience during past earthquakes. The method is by essence a conservative procedure (it predicts the threshold of pore pressure buildup rather than liquefaction itself). Despite this conservantism, the charts in Figs. 3 and 4 show that very stiff sand layers can sustain accelerations of 0.15g or larger without developing significant pore pressures.

CONVERSION TO STANDARD UNITS

The table below is presented to facilitate conversion to SI units.

To convert	to	Multiply by
ft	m	3.05×10^{-1}
lb/ft^2	Pa	37.9
lb/ft^3	N/m^3	157

ACKNOWLEDGEMENTS

This work was sponsored by Program Build of Rensselaer Polytechnic Institute (RPI) and by the Center for Building Technology, National Engineering Laboratory, National Bureau of Standards (NBS), U.S. Department of Commerce. The tests on Monterey No. 0 Sand included in Fig. 1 were conducted at Woodward-Clyde Consultants' Soil Dynamics Laboratory under the sponsorship of NBS. Prof. T. Nogami and Mr. Greg Thomas of RPI reviewed the paper and made useful suggestions.

REFERENCES

Dobry, R. and Grivas, D.-A (1978) "Stress-Strain Relation for Soils Under Earthquake Loading - Technical Background", Research Report 78-2, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, NY, July.

Dobry, R. and Swiger, W.F. (1979) "Threshold Strain and Cyclic Behavior of Cohesionless Soils", Proc. Third ASCE/EMDE Specialty Conference, Austin, Texas, 17-19 September, pp. 521-525.

Dobry, R. and Ladd, R. (1980) Discussion to "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground during Earthquakes", by H.B. Seed and "Liquefaction Potential: Science versus Practice", by R.B. Peck, Journal of the Geotechnical Division, ASCE, June.

Dobry, R., Yokel, F.Y., Ladd, R.S. and Powell, D.J. (1980) "Prediction of Pore Pressure Buildup and Liquefaction of Sands During Earthquakes By the Cyclic Strain Method" (in preparation), Research Report, National Bureau of Standards, U.S. Dept. of Commerce, Washington, D.C.

Drenevich, V.P. and Richart, F.E., Jr. (1970) "Dynamic Prestraining of Dry Sand", Journal of the SMFD, ASCE, 96, SM2, March, pp. 453-469.

Iwasaki, T., Tatsuoka, F. and Takagi, Y. (1978) "Shear Moduli of Sands under Cyclic Torsional Shear Loading", Soils and Foundations, Vol. 18, No. 1, March, pp. 39-56.

Kuribayashi, E. and Tatsuoka, F. (1975) "Brief Review of Liquefaction During Earthquakes in Japan", Soils and Foundations, Vol. 15, No. 4, December.

Peck, R.B. (1979) "Liquefaction Potential: Science Versus Practice", Journal of the Geotechnical Engineering Division, Vol. 105, No. GT3, March, pp. 393-398.

Powell, D. (1979) "A Cyclic Strain Approach to Liquefaction of Level Sand Sites", MS Thesis, Rensselaer Polytechnic Institute.

Pyke, R. (1973) "Settlement and Liquefaction of Sands under Multi-Directional Loading", Ph.D. Dissertation, University of California, Berkeley.

Seed, H.B. (1979) "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes", Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT2, February, pp. 201-255.

Seed, H.B. and Idriss, I.M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analyses", Earthquake Engineering Research Center, Report No. EERC 70-10, University of California, Berkeley.

Seed, H.B. and Idriss, I.M. (1971) "A Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9, September, pp. 249-274.

Woods, R.D. (1978) "Measurement of Dynamic Soil Properties", State-of-the-Art Report, Proceedings Specialty Conference on Earthquake Engineering and Soil Dynamics, ASCE, Vol. I, pp. 91-178, Pasadena, California, July.

Yokel, F.Y., Dobry, R., Ladd, R.S. and Powell, D.J. (1980) "Liquefaction of Sands During Earthquakes - The Cyclic Strain Approach", Proceedings International Symposium on Soil Under Cyclic and Transient Loading, Jan. 7-11, Swansea, U.K.

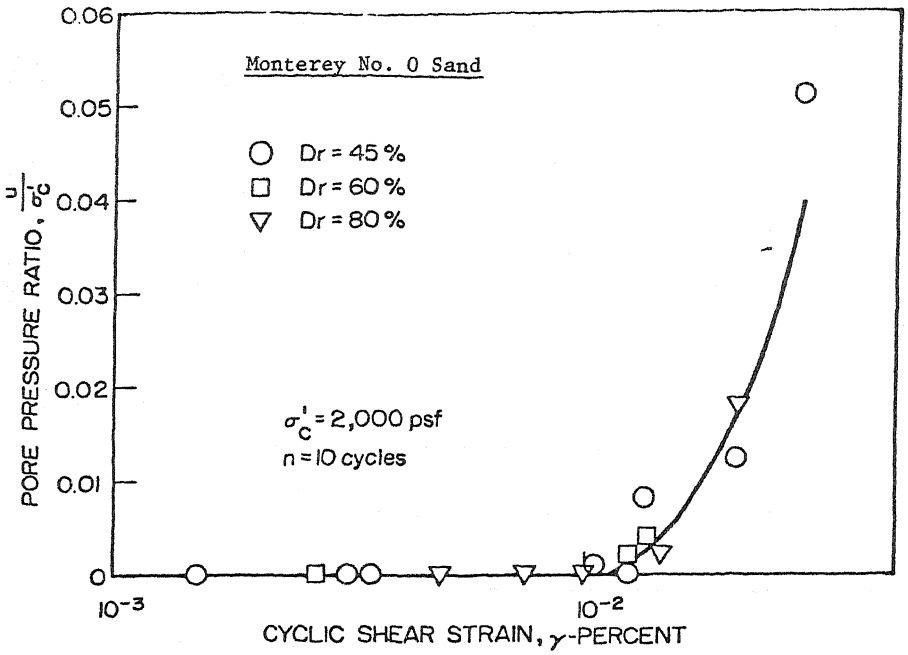

Figure 1 - Pore Pressure Buildup After Ten Loading Cycles, Cyclic Triaxial Strain-Controlled Tests

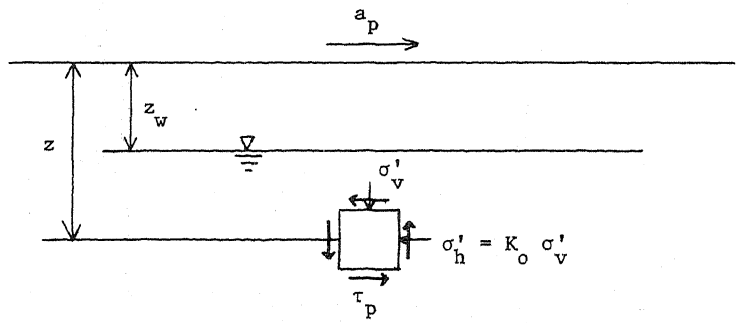

Figure 2 - Simplified Soil Profile

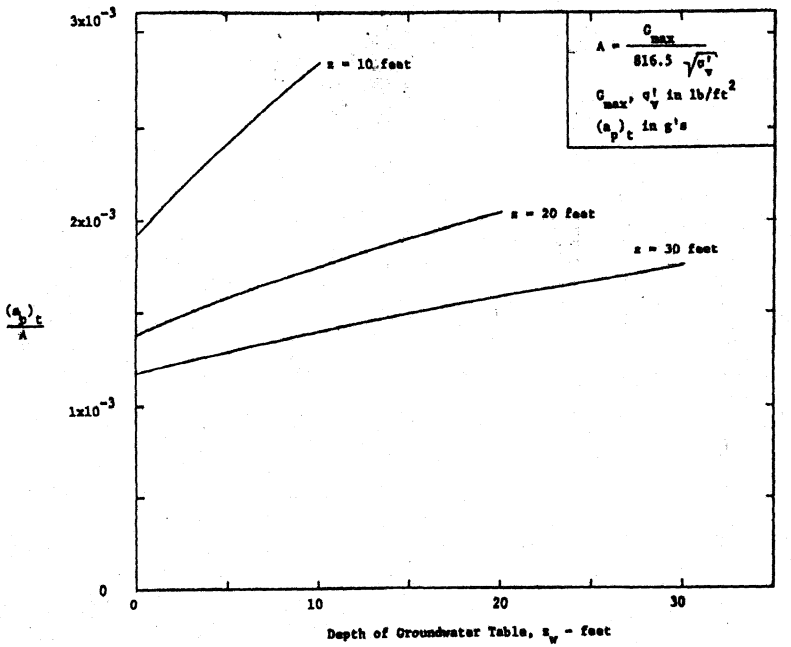

Figure 3 - Liquefaction Chart for Threshold Peak Ground Surface Acceleration, $(a_p)_t$.

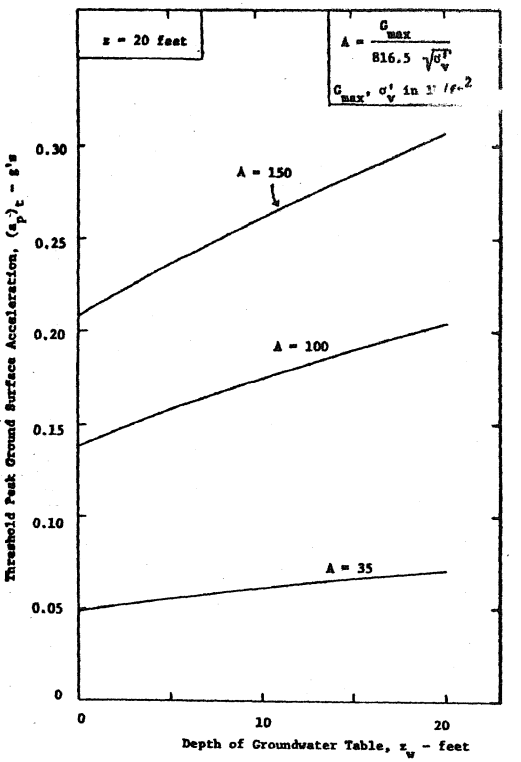

Figure 4 - Liquefaction Chart for Threshold Peak Ground Surface Acceleration, $(a_p)_t$, at z = 20 feet.