LIQUEFACTION POTENTIAL OF SATURATED SAND - THE STIFFNESS METHOD

by

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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, $\gamma_t \approx 10^{-2}\%$. 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),

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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:

$$G_{max} = \rho \, V_s^2 \tag{1}$$

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

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:

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

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):

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

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:

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

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

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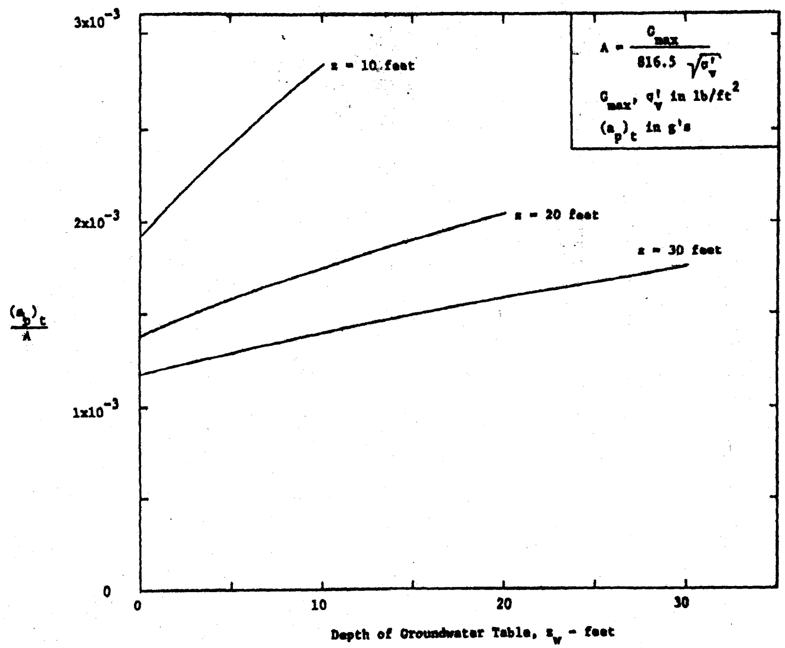

Figure 3 - Liquefaction Chart for Threshold Peak Ground Surface Acceleration, $(a_p)_t$.

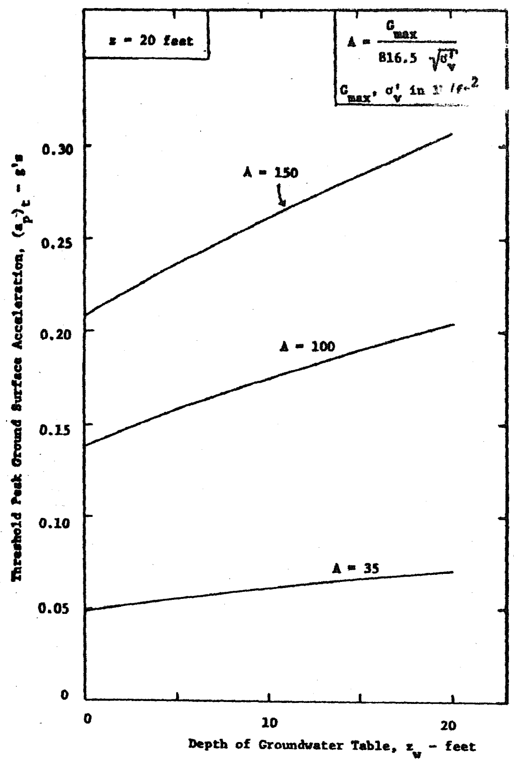

Figure 4 - Liquefaction Chart for Threshold Peak Ground Surface Acceleration, $(a_p)_t$, at z = 20 feet.