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LIQUEFACTION POTENTIAL OF SANDS FROM SHEAR WAVE VELOCITY

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SUMMARY

A parametric study of the liquefaction potential of sandy soils was conducted using computer program SHAKE and the cyclic strain approach. The parameters varied were: soil stiffness in terms of shear wave velocity (V_s), the depth and thickness of the liquefiable sand layer, and the characteristics of ground shaking in terms of peak acceleration (a_{max}) and number of equivalent strong-motion cycles (n_c). Liquefaction potential charts were developed which relate a_{max} and V_s for the cases of n_c equal to 10, 20, and 30. The validity of this approach is supported by comparison with liquefaction case histories from Imperial Valley, California.

INTRODUCTION

The 1979 Imperial Valley (M_s = 6.5) and 1981 Westmorland (M_s = 5.6) earthquakes in Southern California resulted in the development of an extensive set of liquefaction case histories. These case histories are unusual because they include: much recorded strong-motion data, detailed field observations of surface manifestations of liquefaction, and extensive field and laboratory testing. Therefore, soil and site conditions and ground motion characteristics which are typical of this part of California were selected as the framework for a parametric study of liquefaction potential. The aim of this study was to investigate the relationship between the shear wave velocity of a liquefiable sand layer and peak ground surface acceleration causing initial liquefaction. Shear wave velocity, V_s, of the sand layer was selected as the parameter to study (instead of penetration resistances) because V_s relates directly to the deformational (stiffness) characteristics during cyclic loading and because it can be measured in situ by various seismic methods. The cyclic strain approach and computer program SHAKE were used in the parametric study as the basis upon which initial liquefaction was evaluated.

PARAMETRIC STUDY

The following parameters were varied in this study: 1. the shear wave velocity of the liquefiable sand layer which ranged from 300 to 500 ft/sec (90 to 150 m/s), 2. the depth to the bottom of the sand layer which was taken as 20, 30 and 40 ft (6, 9 or 12 m), 3. the thickness of the sand layer which was assumed to be 10, 15 or 20 ft (3, 4.5 or 6 m) except in the case when the bottom of the sand layer was at 20 ft (6 m) and then only thicknesses of 10 and 15 ft (3 and 4.5 m) were used, 4. the peak ground surface acceleration on a stiff site which ranged from about 0.1 to 0.6g, and 5. the equivalent number of cycles of strong-motion shaking which was 10, 20 or 30 cycles.

Two general soil profiles were studied. Each profile was represented as a 200-ft (60-m) thick deposit of soil overlying bedrock. The first profile contained the liquefiable sand layer located within the profile as described above. The soil located above and below the sand was assumed to be clay. The clay had a shear wave velocity ranging from 600 ft/sec (185 m/s) just below the sand layer to 1000 ft/sec (300 m/s) at depths of 120 ft (37 m) and greater. Above the sand layer, the clay was given a constant velocity of 350 ft/sec (105 m/s).

The second profile was simply a 200-ft (60-m) thick deposit of clay without any liquefiable sand layer. This profile was used as the reference profile with which to evaluate peak ground surface acceleration in the absence of any liquefaction. The stiffness of the clay layer in terms of shear wave velocity was the same as site one at depths below 40 ft (12 m). However, above 40 ft (12 m), site two was stiffer than site one. The value of V_s ranged from 500 ft/sec (150 m/s) at the surface to 600 ft/sec (185 m/s) at a depth of 40 ft (12 m). Site two was selected to be representative of a stiff soil site in Imperial Valley upon which strong-motion accelerographs were placed.

The strong-motion records used to excite the soil sites were scaled from recorded motions during the 1979 and 1981 earthquakes. Most of the analyses were performed with the Salton Sea record which was recorded on a stiff soil site during the 1981 Westmorland earthquake. This record exhibited an a_{max} of 0.20g and an equivalent number of cycles on the order of 10. If it was desired to have $n_c = 10$ and a different value of a_{max}, scaling was performed simply by multiplying the earthquake record by a preselected value to change a_{max} to the desired magnitude. If it was desired to also change n_c from 10 to 20, then the strong-motion portion of the record was doubled in length. This was accomplished by shifting the strong-motion portion of the record appropriately and adding it to the original record. To have n_c change from 20 to 30, this shifting procedure was simply performed a second time. This scaled motion was then used as the input motion which excited bedrock at a depth of 200 ft (60 m).

Computer program SHAKE (Ref. 1) was used to evaluate stresses and strains in the soil profiles and peak accelerations on the ground surface as a result of the bedrock excitation. The calculation procedure employed in SHAKE is based on an equivalent linear analysis. The variation of shear modulus and material damping ratio with shearing strain for each soil layer is required as input to SHAKE. Since the small-strain shear wave velocities (hence small-strain moduli) were assumed in the parametric study, the modulus reduction factor and material damping were used as input. Figure 1 shows the variation of the modulus reduction factor with shearing strain for two sands (Ref. 2) and for several clays (Ref. 3) in the Imperial Valley. These two curves were used for all clay and sand layers considered in this study. The relationship between material damping and shearing strain for the sands and clays is presented in Fig. 2.

An important point evaluated in this study was the relationship between peak ground surface accelerations at the reference and liquefiable sites. Peak acceleration at the liquefiable sites will be somewhat different than those at the reference site for the same input motion because the relative stiffness of each site is an important variable. For instance, consider two sites which are very close to one another, one site consisting of stiff soil throughout while the other site consists of a soil with a lower shear modulus at shallow depths (such as a liquefiable sand deposit). The motion which is generated by an earthquake will be the same for both sites at great depths. However, when the porewater pressure of the saturated sand begins to build at shallow depths at the liquefiable site, surface accelerations will be lowered relative to the stiff site because the liquefying sand cannot transmit wave energies as effectively as stiffer materials. Thus, the site which remains stiff during the earthquake will have a higher peak horizontal surface acceleration than a similar site in which a sand layer liquefies. This discrepancy increases as the input motion becomes stronger and as more of the sand layer liquefies. It is important that this concept be kept in mind when estimating surface accelerations at sites with which these parametric studies are compared because a_{max} at the stiff site (which corresponds to an accelerograph or strong-motion station at the same distance from the earthquake as the liquefiable sand site) is used as the reference herein.

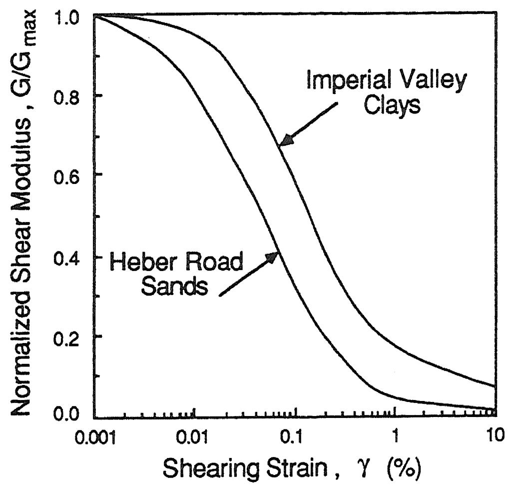

Fig. 1 Variation in Normalized Modulus with Strain for Imperial Valley Soils (Refs. 2 and 3)

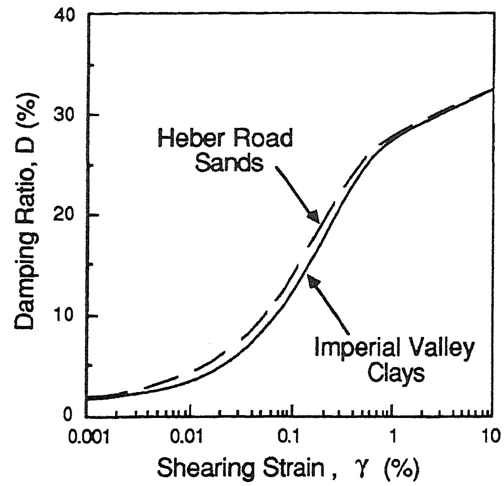

Fig. 2 Variation in Material Damping with Strain for Imperial Valley Soils (Refs. 2 and 3)

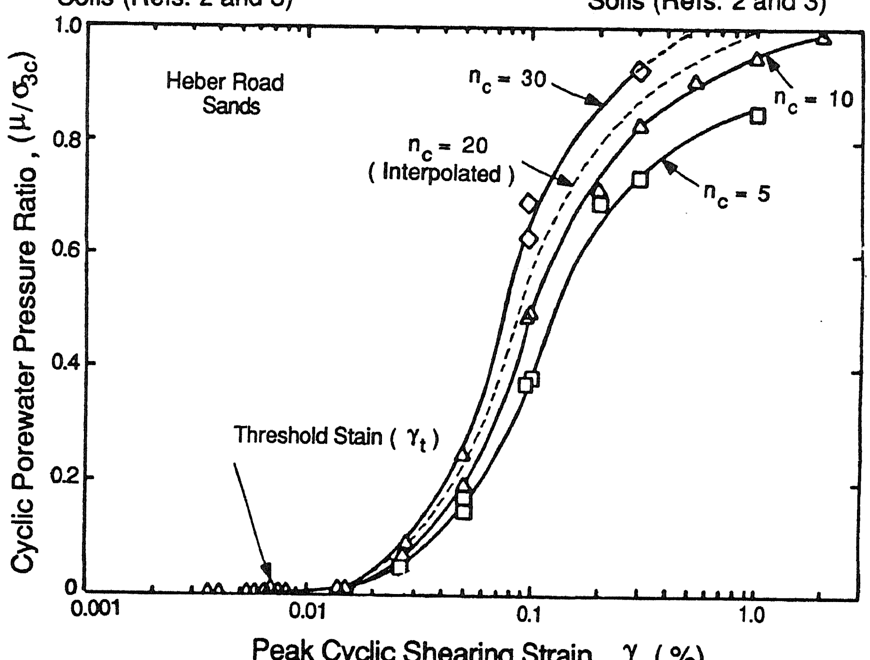

Fig. 3 Variation in Cyclic Porewater Pressure with Number of Cycles of Strain (Ref. 2)

Once the stresses and strains in the liquefiable sand layer were determined for a given input motion, the cyclic strain approach (Ref. 4) was used to determine if initial liquefaction occurred. This was done by comparing cyclic strain levels in the sand with those required to cause an excess porewater pressure ratio of 1.0 to develop. The relationship between excess porewater pressure and cyclic strain for various numbers of cycles of strain is shown in Fig. 3 for two Imperial Valley sands (Ref. 2). This relationship was determined using cyclic triaxial tests and strain-controlled conditions. For equivalent numbers of cycles of 10, 20, and 30, cyclic strains in the liquefiable sand layer equal 2.0, 1.0 and 0.5%, respectively, when initial liquefaction occurs. It is also important to note that no drainage occurred in the tests, and hence it is assumed that no drainage occurs in the field. Also, the behavior shown in Fig. 3 demonstrates an important principle used in the cycle strain approach; that is, for cycling below some threshold strain (on the order of 0.01% for these sands), no excess porewater pressure is generated.

RESULTS

An example of the relationship between the shear wave velocity of the liquefiable sand layer and the peak horizontal ground surface acceleration causing initial liquefaction in ten cycles of strong motion is shown in Fig. 4. In this case, the bottom of the sand layer was held at a constant depth of 40 ft (12 m), and the layer thickness was varied as noted in the figure. The shear wave velocities of the sand layer were varied between 300 and 500 ft/sec (90 and 150 m/s) as shown on the vertical axis in the figure. The horizontal axis gives a_{max} at the surface of each site. It is important to note that the data in Fig. 4 are presented in terms of peak ground surface accelerations of each site itself while the vertical axis represents only the velocity of the sand layer that liquefied. For instance, if the shear wave velocity of the liquefiable sand is 350 ft/sec (105 m/s) and the layer thickness is 20 ft (6 m), a_{max} at the (reference) stiff site is about 0.32g while a_{max} at the site which liquefied is 0.12g. It seems reasonable to assume that, in most analyses, the engineer will more likely have estimates of a_{max} at stiff sites than on the surfaces of liquefied sites. Therefore, it would be prudent and wise to use values of a_{max} at stiff sites in any correlation. With this reasoning, all predictive charts were developed using a_{max} on stiff sites.

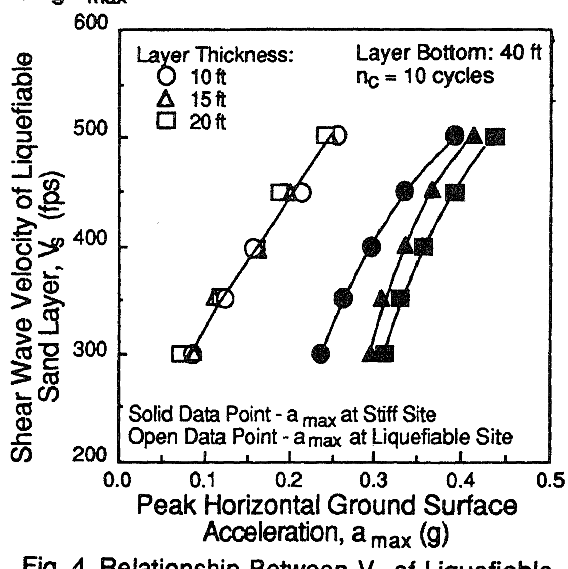

Fig. 4 Relationship Between V_s of Liquefiable Layer, a_{max} at Site when Sand Initially Liquefies, and a_{max} at Reference Site

Based on analyses of figures like Fig. 4, the following general trends were observed. First, the higher the shear wave velocity, the less likely the site is to liquefy for a given a_{max}. Second, the value of a_{max} on the surface of the site experiencing initial liquefaction is essentially independent of the thickness of the sand layer (as demonstrated in Fig. 4 by all open points nearly plotting as one point for a given value of V_s). Third, the greater the thickness of the liquefiable sand layer, the less likely the site is to liquefy for a given V_s (as shown in Fig. 4 by a_{max} at a stiff site increasing as the thickness of the sand layer increases). Finally, the greater the depth to the bottom of the liquefiable sand layer, the slightly more likely the site is to liquefy at a given V_s.

Based on these observations and numerous parametric studies, new liquefaction potential charts which relate the shear wave velocity of a liquefiable sand layer to a_{max} at a stiff site were developed. These charts are shown in Figs. 5, 6 and 7 for earthquake excitation with an equivalent number of cycles of 10, 20 and 30, respectively. Each chart

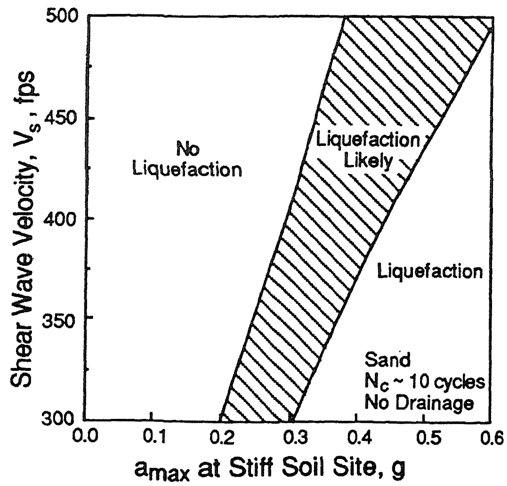

Fig. 5 Liquefaction Potential Chart Based on V_s of Sand Layer and 10 Cycles of Strong Motion

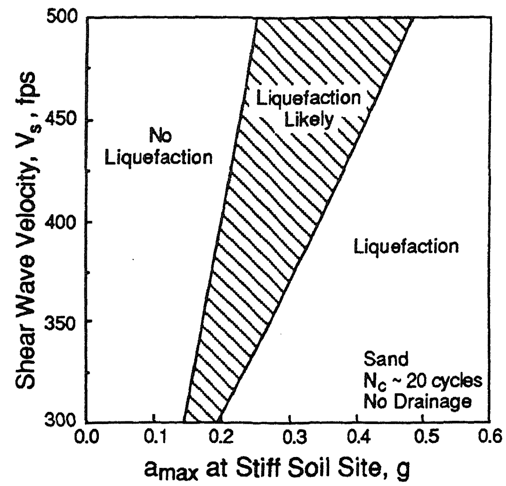

Fig. 6 Liquefaction Potential Chart Based on V_s of Sand Layer and 20 Cycles of Strong Motion

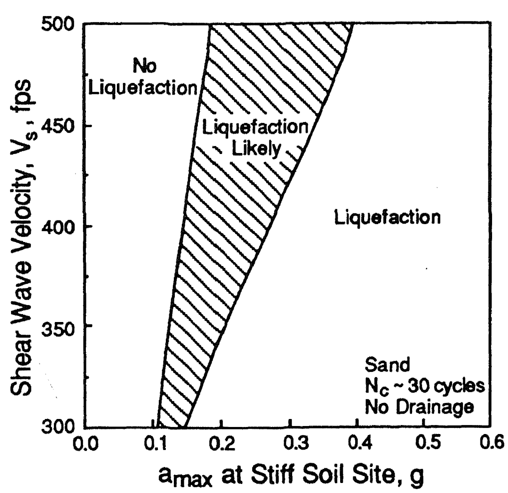

Fig. 7 Liquefaction Potential Chart Based on V_s of Sand Layer and 30 Cycles of Strong Motion

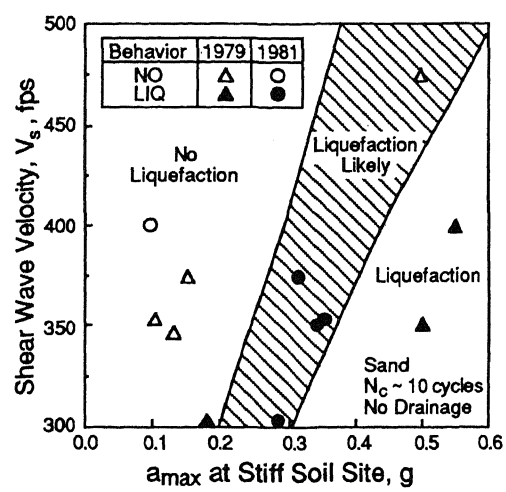

Fig. 8 Comparison of Field Performance and Predicted Behavior in Imperial Valley, California

is divided into the following three zones: 1. no liquefaction (because the sand is too stiff to deform enough to cause initial liquefaction for the given input), 2. liquefaction likely (which means that the performance depends on several variables and a range in behavior should be expected), and 3. liquefaction. With such charts, one can get a feel for the values of shear wave velocity, V_s, which would and would not result in liquefaction of a sand layer when a given a_{max} is predicted at a stiff soil site at the same distance away from the earthquake as the liquefiable site.

To investigate the accuracy of these charts, the performance of seven sites in Imperial Valley which experienced liquefaction in either the 1979 or 1981 earthquakes (Ref. 5) was compared with predictions based on Fig. 5 (because $n_c \cong 10$ for these earthquakes). These comparisons are presented in Fig. 8. The accuracy of the predictions is good which supports the use of V_s as an important sand property in evaluating the liquefaction potential of a site.

CONCLUSIONS

The liquefaction potential of a sand layer can be evaluated from the shear wave velocity of the sand. Important variables which enter this evaluation are layer depth and thickness, peak horizontal ground suface acceleration, and number of cycles of strong-motion shaking. Liquefaction charts relating V_s of the sand and a_{max} at a stiff soil site of comparable distance from the earthquake are presented in Figs. 5, 6 and 7. These charts are based on soil, site and earthquake characteristics that are typical of Imperial Valley, California. Reasonable caution should be exercised in applying the charts to other areas.

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