Liquefaction and Cyclic Mobility of Saturated Sands[a]

Discussion by David A. Howells[2]

The writer disagrees with the statement by the author that "it is possible to distinguish at least two clearly different phenomena that will be referred to herein as 'liquefaction' and 'cyclic mobility' respectively." and suggests that the term liquefaction as commonly used is the extreme case of cyclic mobility as defined by the author, not a different phenomenon.

Although a laboratory study may not represent the effect of an earthquake very realistically, nevertheless the observed increase of pore pressure in undrained conditions appears to be the same phenomenon as that which occurs in some soil deposits during an earthquake. This buildup of pore pressure has the obvious effect of reducing the effective normal stress while leaving the effective shearing stress unchanged so that the effective stress state of the soil is moved closer to the failure envelope. In the extreme case, soils that are particularly liable to this increase of pore pressure may fail under the shearing stresses, static plus earthquake where these are additive, which exist in the natural state. This is the usual meaning of the term liquefaction. It is doubtful if the critical state is a necessary condition for this to happen. Failure may occur in deposits which are denser than critical. They could be expected to dilate on failure so that post-failure behavior may be that of the critical state.

Soils that show cyclic mobility are unlikely to show loss of shear strength as defined by the friction angle. Nevertheless, foundations built on them need to be designed to lower induced shearing stress levels in order to have the same factor of safety in earthquake conditions as those built on "firmer" soils.

Discussion by Mishac K. Yegian,[3] A. M. ASCE and Issa S. Oweis,[4] M. ASCE

The author presented a valuable plot (Fig. 8) for evaluating liquefaction potential based on reported field case histories.

The writers attempted to employ this plot in their liquefaction studies for a site along the east coast of the United States. The postulated earthquake for this site is of magnitude 5.0, with a peak ground surface acceleration of 0.22 gravity at the site. A liquefaction analysis was made of a sand layer extending from 32 ft–50 ft below ground surface. The water table is at a depth of 11 ft below ground surface. The average standard penetration resistance, N, in the sand layer is 22 blows/ft. Measured relative densities from undisturbed

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[a] June, 1975, by Gonzalo Castro (Proc. Paper 11388).

[2] Consulting Engr., London, England.

[3] Sr. Staff Engr., Woodward-Clyde Consultants, Clifton, N.J.; Grad. Student, Massachusetts Inst. of Tech., Cambridge, Mass.

[4] Project Engr., Woodward-Clyde Consultants, Clifton, N.J.

sample had an average value of 75%. Results from dynamic response analyses and liquefaction testing on "undisturbed" samples indicated ample safety factors against liquefaction. Using the author's plot, the likelihood for liquefaction would be marginal at a depth of 32 ft and strong at a depth of 50 ft. The discrepancy

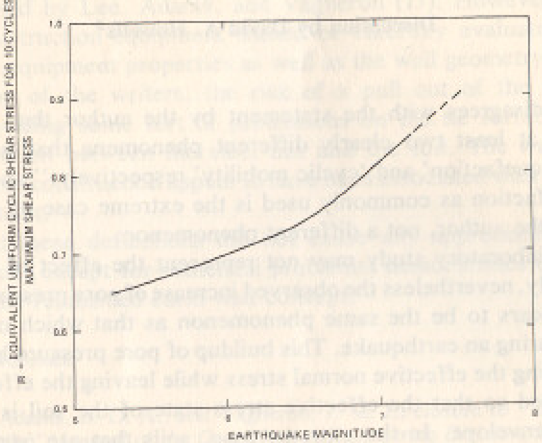

FIG. 12.—Shear Stress Conversion Factor R for 10 Significant Cycles as Function of Earthquake Magnitude (11)

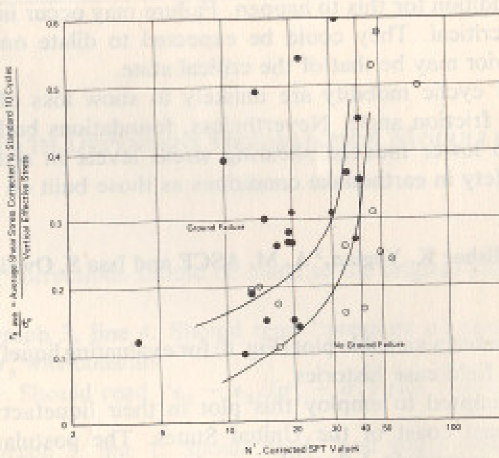

FIG. 13.—Shear Stress Normalized to 10 Significant Cycles Versus Corrected SPT Values

between the two conclusions prompted the writers to examine the assumptions made by the author with regard to the use of a constant conversion factor of 0.7 for all magnitudes and the calculation of shear stresses based on a rigid body assumption.

Whitman (20), who used a format very similar to Fig. 8 for organizing field

observations, introduced the effect of earthquake durations by correcting the dynamic shear stresses to correspond to a standard duration. To correct the maximum dynamic shear stresses to a duration that would produce 10 uniform significant cycles, the author acknowledged that an earthquake magnitude-dependent conversion factor, R, should be employed. However, because of the scatter present in Lee and Chan's (11) data, the author decided to adopt a constant factor of 0.7. In doing so, in effect, the author did not take into account the influence of earthquake duration in his interpretation of the data. Fig. 12 presents the relationship between R and earthquake magnitude M as suggested by the average relationships from Lee and Chan (11). This figure indicates that the shear stress conversion factor, R, on the average, varies between 0.65 and 0.85 for magnitudes ranging between 5.2 and 7.2. This range of values of R may even be larger if greater magnitudes are also considered.

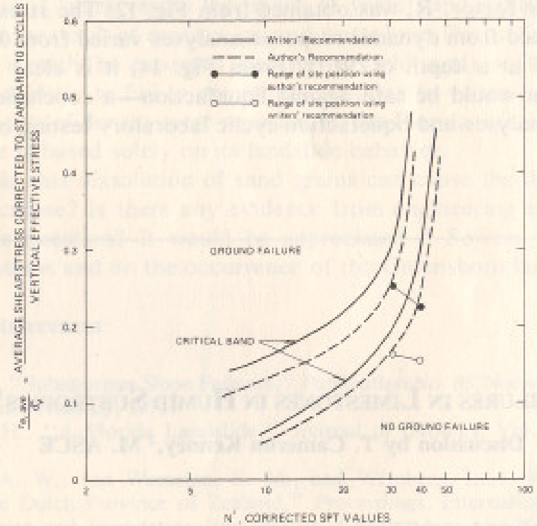

FIG. 14.—Application of the Author's and Writers' Recommendations to East Coast Site

Inasmuch as 80% of the case histories cited by the author are for magnitudes greater than 7.2 and about 95% of the data points plotted within or close to the critical band suggested by the author also correspond to magnitudes greater than 7.2, it is the writers' opinion that the author's choice of a constant value of R equal to 0.7 is overly conservative.

The writers reexamined the data used by the author and decided to employ Fig. 12 to incorporate magnitude-dependent shear stress converson factor R. For the cases involving magnitudes greater than 7.2 which, as previously stated, constituted 95% of the critical data points, a constant value of R equal to 0.85 was adopted. This value of R corresponds to a magnitude of 7.2 as suggested by Fig. 12. For the rest of the available data, the value of R was chosen on the basis of Fig. 12. Fig. 13 presents the writers' data and suggested critical band.

The position of the critical band is dictated primarily by the data points corresponding to magnitudes greater than 7.2. The critical band thus obtained by the writers could have also been arrived at by shifting the critical band suggested by the author vertically up by the ratio of 0.85 to 0.7 as shown in Fig. 14. To use the plot suggested by the writers, it is essential that the user employs a conversion factor of $R = 0.85$ for cases involving magnitudes greater than 7.2 and Fig. 12 for magnitudes less than 7.2.

The author assumed that the stress attenuation with depth for the case histories studied was negligible. This assumption is certainly reasonable for near-surface (up to 25 ft) conditions, which constitute the great majority of the case histories cited by the author. However, the writers believe that for depths greater than 25 ft, a stress reduction factor should be used in conjunction with Fig. 13.

Finally, the liquefaction potential of the sand layer described was reevaluated based on the writers' recommendation previously described. The value of the stress conversion factor, R, was obtained from Fig. 12. The stress attentuation factors as obtained from dynamic response analyses varied from 0.78 at a depth of 32 ft to 0.75 at a depth of 50 ft. From Fig. 14, it is clear that the sand layer in question would be safe against liquefaction—a conclusion compatible with response analyses and liquefaction cyclic laboratory testing on undisturbed samples.

Failures in Limestones in Humid Subtropics[a]

Discussion by T. Cameron Kenney,[2] M. ASCE

The writer enjoyed reading Sowers' lucidly-written paper on the complex and obviously important subject of foundation difficulties arising from dissolution of limestone. The writer has no experience with such problems and therefore cannot contribute in this way to the paper, but the paper did bring back to memory the case of a landslide in Miocene clayey sand in Florida which, from the description of the landslide, apparently was in a loose condition. The question of why this very old clayey sand might exist in a loose condition has not been answered, and Sowers' opinion is sought whether or not it could be related to solution of carbonates.

Jordan (2) described the landslide which occurred at Greensboro near Tallahassee, Fla., in "reddish, partially indurated clayey sands of the Hawthorne formation" which the author stated as being probably of Miocene age. His description of the material would lead a reader to believe the material to be primarily sand and "commonly admixed with varying amounts of clay." The description of the landslide scar perfectly fits that of many landslides in very sensitive clay; it occurred in a flat-lying region into which a stream valley had been eroded, the slide crater was circular with a diameter of about 900 ft and a depth of about 50 ft, the soil had flowed out through a narrow bottleneck into the stream valley and the slide had developed by retrogressive slumping from the sides of the crater towards the center. It occurred after a year of unusually heavy rainfall, including 16 in. (410 mm) during 30 days preceding the slide.

[a] August, 1975, by George F. Sowers (Proc. Paper 11521).

[2] Prof., Dept. of Civ. Engrg., Univ. of Toronto, Toronto, Canada.

Landslides having somewhat similar characteristics have occurred in silty and clayey sands along the coast of The Netherlands, along the banks of large rivers and in fjords (2,3,4), and the similarities between the behaviors of these soils and the very sensitive clays in which the same landslide mechanisms occur are believed to be due to the looseness and resulting metastable nature of the structures of the soils. The important difference between the Florida soil and those along the Netherland coast, Mississippi River, and in Norwegian fjords is age (millions of years versus thousands of years), and under normal circumstances and as the result of erosion, ground-water level fluctuations, and tectonic disturbances, it might be expected that the very old sandy soils would be much denser and not likely to fail in comparable fashion to young loose sandy soils. No measurements of density were reported for the Florida soil and the suggestion that it is loose is based solely on its landslide behavior.

Is it possible that dissolution of sand grains can cause the density of a sand deposit to decrease? Is there any evidence from engineering experience which relates to this question? It would be appreciated if Sowers would comment on these questions and on the occurrence of the Greensboro landslide.

Appendix.—References

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