INFLUENCE OF PORE-PRESSURE COEFFICIENT $\overline{B}$ ON SOIL LIQUEFACTION POTENTIAL

by

M.A. Sherif[I], I. Ishibashi[II] and C. Tsuchiya[III]

SYNOPSIS

Liquefaction potential of loose Ottawa sand is investigated for several values of pore-pressure coefficient $\overline{B}$ ranging from 0.25 to near unity. Experiments were conducted in the University of Washington Dynamic Torsional Simple Shear Device (1), which applies nearly uniform cyclic shear strains throughout the tapered donut-like soil specimen.

INTRODUCTION

Soil liquefaction has become an important consideration in the design of earth structures in seismically active regions, especially in places where the water table is high and soil gradation falls within a critical range. Many researchers have in the past attempted to determine the liquefaction potential of soils, but all of these past studies concerned themselves with the liquefaction potential of fully saturated sands.

In this study, the authors first determine the liquefaction potential for partially saturated loose sand as a function of varying pore-pressure parameter $\overline{B}$. This is especially important in practice, in view of the fact that most natural geologic deposits consist of soils that are partially saturated.

EXPERIMENTS

Test Device. - All of the tests in this investigation were conducted in the Torsional Simple Shear Device. This device was developed by the authors (1) and it closely simulates the stress conditions to which soils are subjected in situ and overcomes most of the limitations inherent in the conventional shear box or triaxial type devices.

The shape of the sample used in the Torsional Simple Shear Device is hollow, with 4 inches in outside and 2 inches inside diameters as shown in Fig. 1. The height of the sample is 1 inch at the outside and 1/2 inch at the inside, and the soil can be subjected simultaneously to a vertical stress and outside and inside horizontal stresses. A cyclic shear stress is applied on top of the sample as shown in Fig. 1(c). Since the value of r/h in Fig. 1 is constant, the shear strains automatically remain uniform throughout the sample. A general view of the Torsional Simple Shear Device is shown in Fig. 2.

Test Procedure. - Prior to testing, the required amount of sand for each test was measured and submerged in water overnight. After attaching the inner and outside molds, de-aired water was circulated through the porous stone to remove entrapped air and the sample was carefully placed in the test device.

I Professor of Geotechnical Eng., Dept. of Civil Eng., Univ. of Washington, Seattle, Washington, U.S.A.

II Lecturer and Research Associate, Dept. of Civil Eng., Univ. of Washington, Seattle, Washington, U.S.A.

III Graduate Student, Dept. of Civil Eng., Univ. of Washington, Seattle, Washington, U.S.A., and Research Engineer, Research Laboratory of Japan Highway Public Corporation, Tokyo, Japan.

A qualitative understanding of the degree of saturation was obtained by measuring the pore-pressure parameter $\overline{B}$, which is defined as: $B = \Delta u/\Delta\sigma_c$, where Δu is the pore-pressure increment generated by subjecting the soil specimen to an increment of isotropic stress, $\Delta\sigma_c$, in an undrained condition. Control of the $\overline{B}$ value of the specimen prior to the test was accomplished by controlling the amount of de-aired water circulated through the specimen (percolation method) and by controlling the back water pressure applied to the specimen (back pressure method).

Soil Type and Test Program. - The soil used in this investigation was a clean Ottawa sand (ASTM designation C-109), the physical properties of which are shown in Table 1. Tests were conducted on samples of the above sand with different initial $\overline{B}$ values ranging from 0.25 to almost unity. Throughout this investigation the vertical and horizontal confining stresses were the same and all sinusoidal shear stresses were applied at 2 cycles per second.

TEST RESULTS AND DISCUSSION

It is known that the residual pore pressure in loose saturated sands increases at different rates during the liquefaction process under constant cyclic shear stress application. Fig. 3 shows a typical example of the liquefaction process for a saturated loose sand. During these tests, it was observed that the partially saturated sand showed a pore-pressure rise similar to that of a saturated one and finally liquefied.

In Fig. 3, it is convenient to divide the liquefaction porcess into four stages on the basis of different rates of increase in the residual pore pressure. The characteristics of each of these stages can be summarized as:

- Stage 1 .... During this stage a rapid increase in pore pressure is observed (Initial stage).
- Stage 2 .... During this stage an almost constant rate of increase in pore pressure is noted (Secondary stage).
- Stage 3 .... The rate of pore-pressure buildup gradually accelerates (Initial liquefaction stage).
- Stage 4 .... During this stage the rate of pore-pressure increase becomes zero (Final liquefaction stage).

The initial rapid increase in residual pore pressure is due to the rapid and substantial rearrangement of soil particles to accommodate the instantaneous change in stress conditions due to shear stress application. During the second stage, the volume change (which is responsible for the pore-pressure buildup) continues because of the increase in the stress ratio, τ/σ_c' (shear stress/effective confining pressure). When the stress ratio, τ/σ_c', reaches a certain value, the shear deformation increases rapidly; as a consequence, the residual pore-pressure values accelerate.

Initial liquefaction is defined in two different ways. One involves the critical stress ratio concept; the other relates to the first stress turnover point on the effective stress path (1, 2). The above-mentioned first definition implies that the soil enters the liquefaction stage when the ratio of τ/σ_c' reaches the failure envelope for the first time in Mohr's diagram. In the second concept, initial liquefaction is defined as the first point where the pore pressure decreases while the shear stress remains nearly constant; that is, the effective stress path reverses direction for the first time.

Using the above two different definitions of initial liquefaction, a graph such as the one shown in Fig. 4 is plotted. In this figure, u is the residual pore pressure, and σ_c is the total confining pressure. The data points in Fig. 4 show that the above two criteria defining initial liquefaction yield similar results. It is also interesting to note from Fig. 4 that the ratio of u/σ_c (at the point of initial liquefaction) increases slightly with the number of cycles necessary to induce initial liquefaction.

Fig. 5 shows the measured residual pore-pressure buildup for different $\overline{B}$ value samples at the end of the first cyclic loading. It is seen from this figure that the residual pore-pressure value (at the end of the first cycle) is greater for soils with higher $\overline{B}$ values.

Fig. 6 shows the residual pore-pressure buildup per cycle during Stage 2. It is seen from Fig. 6 that pore-pressure buildup per cycle is lower for soils with low $\overline{B}$ values. A threshold initial stress ratio (τ/σ_c) for zero pore-pressure rise exists and is higher for low $\overline{B}$ value soils.

Fig. 7 shows a relationship between the applied stresses and the number of cycles to liquefaction as a function of the pore-pressure parameter, $\overline{B}$. From this figure it is noted that, although low $\overline{B}$ value soils do liquefy, their potential, however, is low in comparison with that of saturated sands.

CONCLUSIONS

1. It is shown that partially saturated loose sands do liquefy and that the pore-pressure buildup pattern is similar to that generated during tests conducted on fully saturated loose sands.

2. The effective stress ratio criterion and the effective stress path turnover point criterion, defining initial soil liquefaction, correlate very well and essentially yield the same results as shown in Fig. 4.

3. Based on the data shown in Fig. 7, it is concluded that the liquefaction potential for soils decreases with decreasing $\overline{B}$ values.

ACKNOWLEDGMENT

The authors gratefully acknowledge the financial support of the National Science Foundation which made this investigation possible.

BIBLIOGRAPHY

1. Ishibashi, I., and Sherif, M.A., "Soil Liquefaction by Torsional Simple Shear Device," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT8, Proc. Paper 10752, Aug., 1974, pp. 871-888.

2. Shibata, T., Yukitomo, H., and Miyoshi, M., "Liquefaction Process of Sand During Cyclic Loading," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 12, No. 1, March, 1972, pp. 1-16.

TABLE 1. - PROPERTIES OF TEST SPECIMEN

Data	Ottawa sand (ASTM C-109)
Grain Size	D_{100} = 0.60 mm, D_{60} = 0.42 mm, D_{50} = 0.40 mm D_{10} = 0.2 mm, D_0 = 0.15 mm
Uniformity coefficient	2.1
Specific Gravity	2.67
Maximum void ratio, e_{max}	0.76
Minimum void ratio, e_{min}	0.50
Average void ratio through the test	0.67

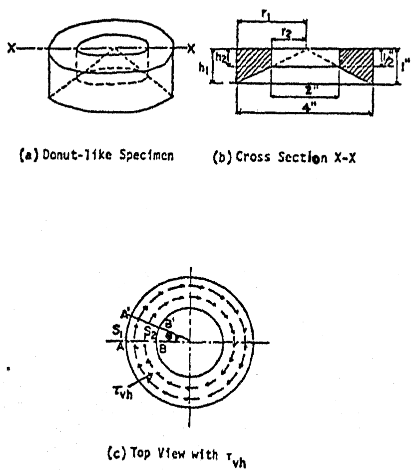

FIG. 1 - SAMPLE SHAPE

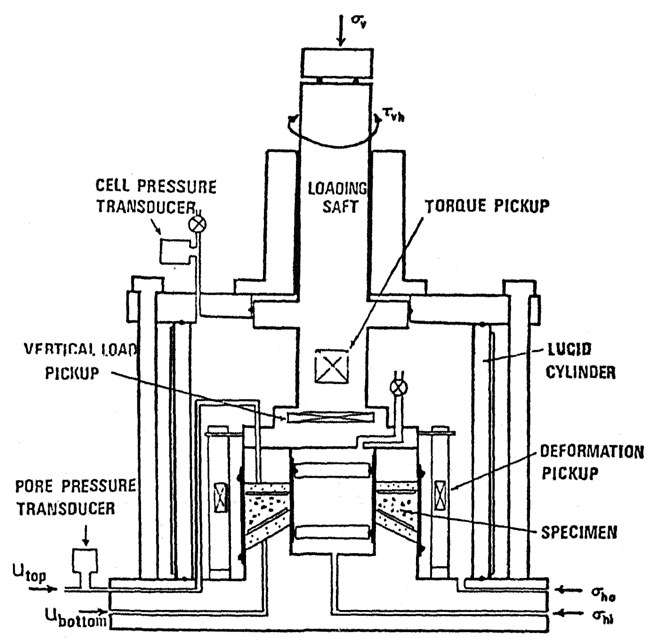

FIG. 2 - TORSIONAL SIMPLE SHEAR DEVICE

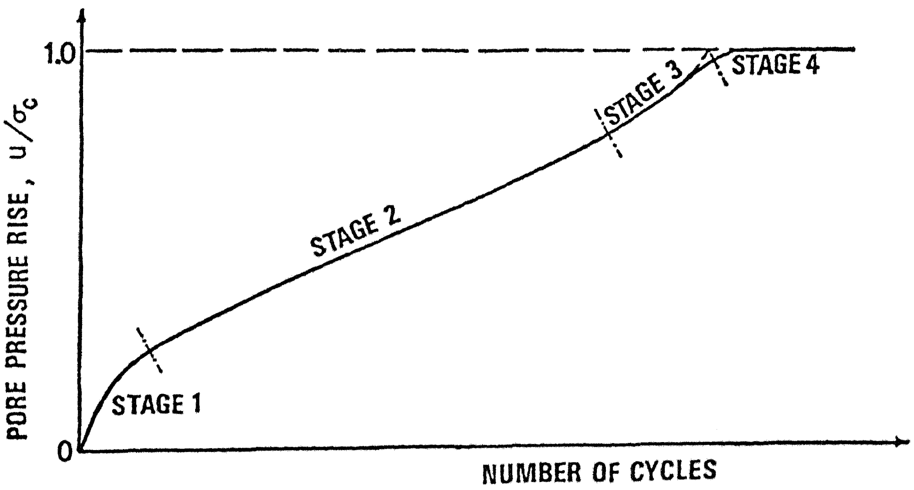

FIG. 3 - PORE PRESSURE RISE UNDER CONSTANT CYCLIC SHEAR STRESS

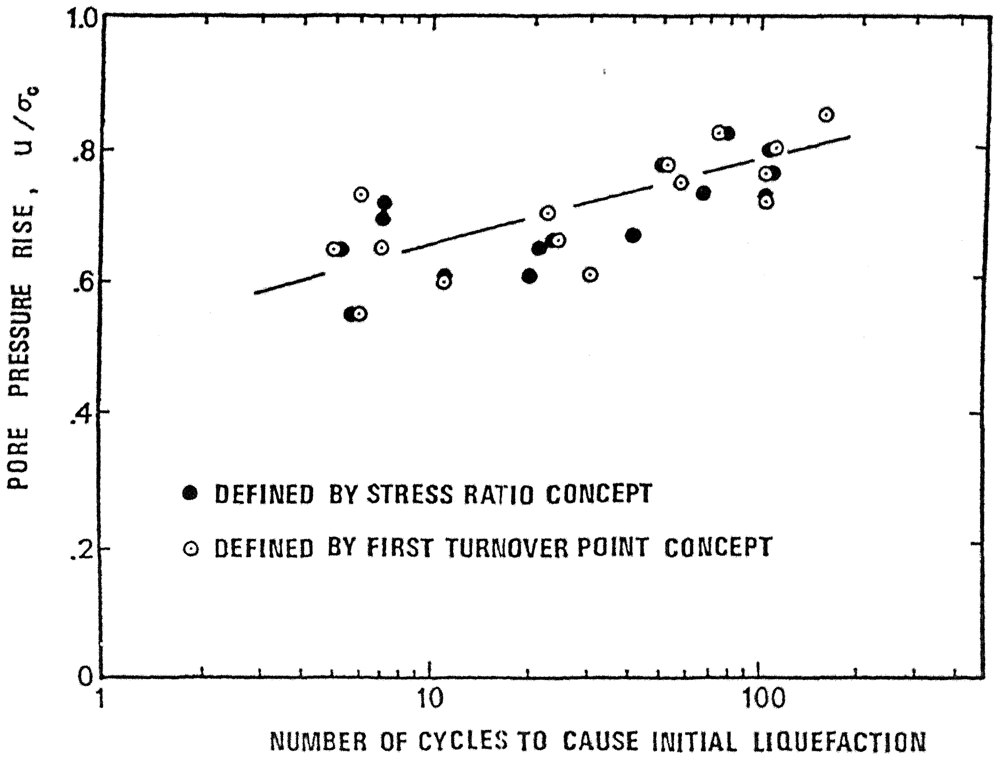

FIG. 4 - STRESS RATIO u/σ_c AT INITIAL LIQUEFACTION BY TWO DEFINITIONS

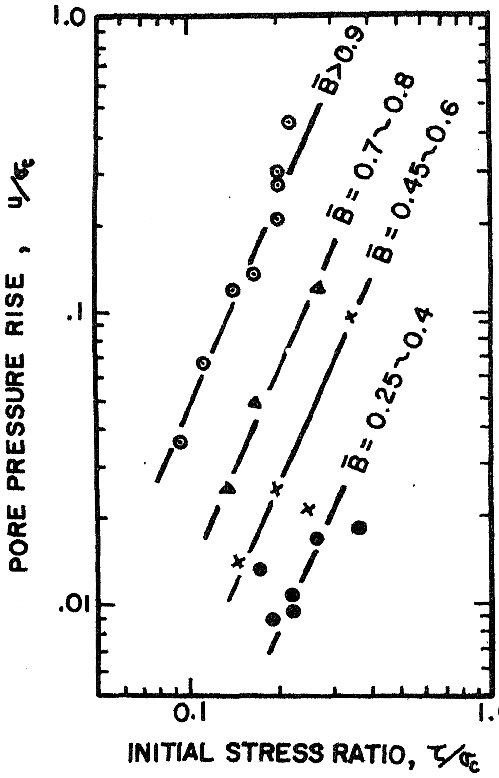

FIG. 5 - PORE PRESSURE RISE AT THE END OF THE FIRST CYCLE

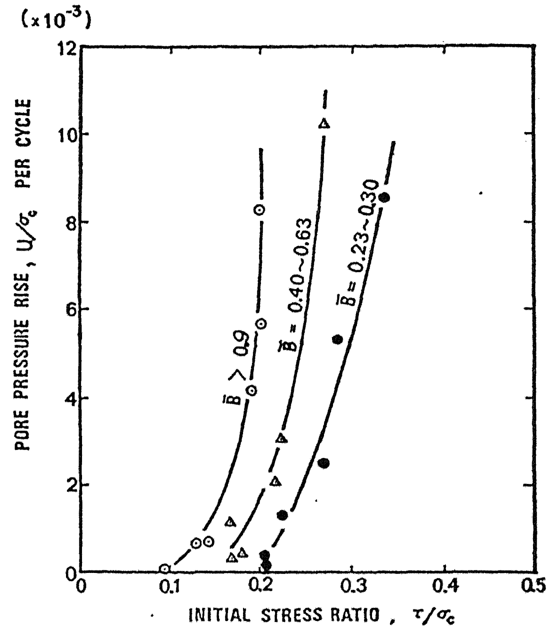

FIG. 6 - PORE PRESSURE RISE PER CYCLE DURING STAGE 2

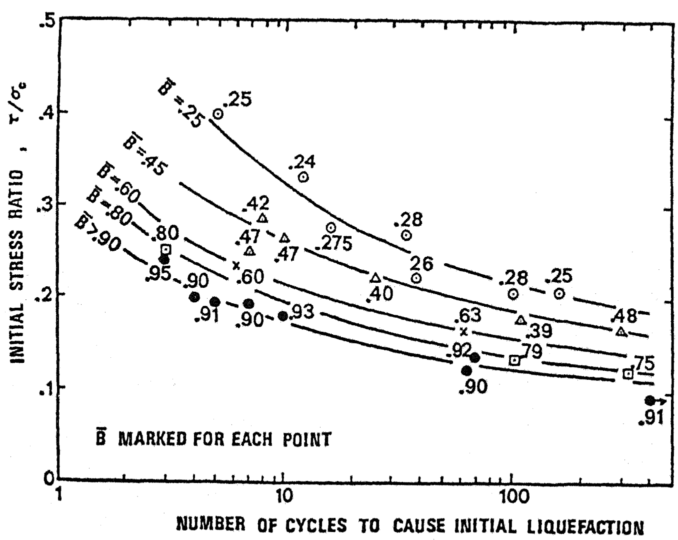

FIG. 7 - EFFECT OF $\overline{B}$ VALUES ON LIQUEFACTION POTENTIAL

DISCUSSION

K. Tanimoto (Japan)

In the definition of initial liquefaction the authors adopt the stage when the stress ratio reaches the failure envelope for the first time in Mohr's diagram.

However, in a cyclic loading test resulting in liquefaction, strength characteristics under various loading conditions may be different from those in a static test (1,2,3), and it is thought that the failure envelope to be used in the definition may not be apriori determined until cyclic tests are carried out.

References:

1) Tanimoto, K. and Suga, Y., (1970), on the dynamic strength of saturated sands, Proc. 3rd Japan Earthq. Engg. Symp - 1970, pp. 191-198.

2) Tanimoto, K. and Umemura, T. (1970), Failure of saturated sands Under cyclic Loading, Trans. JSCE, Vol. 2 part 2, pp. 299-301.

3) Tanimoto, K. and Suga, Y. (1971), Liquefying Process of Saturated Sand Subjected to Dynamic Loading, 4th Asian Reg. Conf. SMFE, pp. 235-239.

Author's closure

In response to Tanimoto's discussion of the Sherif-Ishibashi-Tsuchiya paper, the writers present Fig. 9 (after Fig. 8 in Ref. 1), in which line OB represents the failure envelope based on the definition of initial liquefaction at the first stress turnover point. Line OC represents the static failure envelope of the same soil tested in dry conditions in a direct shear device. Considering the fact that direct shear testing yields slightly higher values for the angle of internal friction, ϕ, coupled with the fact that the ϕ value for dry soils is in general slightly higher than for wet soils, the 2 1/2° discrepancy in the angles of internal friction shown in Fig. 1 does not appear to be excessive. Furthermore, the lower values of the effective failure envelopes obtained by Tanimoto et al. in cyclic triaxial tests may be due to the increase in deviatoric stresses caused by the decrease in the actual effective sample area because of necking effects during the cyclic triaxial tests.

Shibata et al. (2) have also concluded that the mobilized internal friction at initial liquefaction is very close to the true angle of internal friction of the soil.

Based on the above discussion, the writers feel that the failure envelope based on initial liquefaction is reasonably close to the failure envelope determined from static tests.

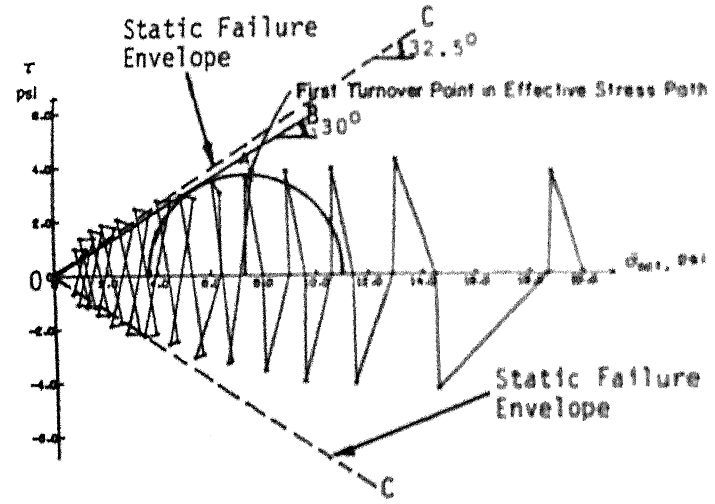

FIG. 9. Effective Stress Path and Definition of Initial Liquefaction

References

1. Ishibashi, I. and Sherif, M.A. (1974), "Soil Liquefaction by Torsional Simple Shear device", Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT8, pp.871-888.

2. Shibata, T., Yukitomo, H. and Miyoshi, M. (1972)"Liqefaction Process of Sand During Cyclic Loading", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 12, No. 1, pp. 1-16.