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SYNOPSIS A deeper inspection has been made into the physical meaning and mechanism of the soil liquefaction. Emphasis has been laid on the stress condition and stress evolution in saturated cohesionless soils during liquefaction. Typical examples have been described to demonstrate the process of stress evolution in soil liquefaction. Some other related factors such as limit equilibrium condition, strain and pore water pressure evolution in the saturated cohesionless soil mass under cyclic loading or vibration have also been discussed. It has been concluded that the state of stress in saturated cohesionless soils is bounded by limit equilibrium condition and approaches the "hydro-static pressure" in the course of liquefaction, the pore water pressure evolution during soil liquefaction can be correlated with the fabric characteristics and drainage condition of the soil mass, and the strain does not seem to be a proper reference for the definition of soil liquefaction.

INTRODUCTION

There are many interpretations about the term "Liquefaction" or "Soil Liquefaction" according to different opinions or different fields of application. The most eminent ones are those given by Casagrande (1975), Seed (1979) and the Committee on Soil Dynamics of the Geotechnical Engineering Division of the American Society of Civil Engineers (1978) in recent years. The author appreciates all of the statements given by them and wishes to look further into the matter by beginning with the statement given by the Committee which states: "Liquefaction.—The act or process of transforming any substance into a liquid. In cohesionless soil, the transformation is from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress."

STRESS CONDITION AT LIQUEFACTION

The basic distinction between solid state and liquid state of a substance is that the substance in its solid state shows rigidity; i.e., when it is deformed by application of external forces, internal forces are brought into play which oppose the deformation and tend to restore its original shape, whereas the substance in its liquid state does not have this property. So the former can possess both spherical stress tensor ("hydro-static pressure") and deviator stress tensor, but the latter can possess only spherical stress tensor and no deviator stress tensor. Therefore the process of transformation of any substance from a solid state into a liquid state is substantially a process of diminishing in shear resistance of the material so that any deviator stress tensor will die away.

The shear resistance of cohesionless soils is mainly proportional to the intergranular pressure and the coefficient of friction between soil particles, which is usually given by the following relationship:

$$\tau_s = \sigma' \tan\phi' = (\sigma - u)\tan\phi' \tag{1}$$

where τ_s is shear resistance: σ' and σ are effective and total normal pressures respectively; u is pore pressure; ϕ' is angle of internal friction in terms of effective stress. The condition for liquefaction is:

$$\sigma' \rightarrow 0; \text{ or } u \rightarrow \sigma \tag{2}$$

The general stress condition in a saturated soil can be expressed as:

$$\sigma_{ij} = \sigma'_{ij} + \delta_{ij}u \tag{3}$$

where σ_{ij} and σ'_{ij} are total and effective stress tensors respectively; δ_{ij} is Kronecker delta. The stress evolution in the course of liquefaction of a saturated cohesionless soil should be:

$$\sigma'_{ij} \rightarrow 0; \text{ and } \sigma_{ij} \rightarrow \delta_{ij}u \tag{4}$$

The stress condition in the liquefied soil should be:

$$\sigma'_{ij} = 0; \text{ and } \sigma_{ij} = \delta_{ij}u \tag{5}$$

If we define

$$\left.\begin{aligned} q &= \tfrac{1}{2}(\sigma_1 - \sigma_3) = \tfrac{1}{2}(\sigma_1' - \sigma_3') \\ p &= \tfrac{1}{2}(\sigma_1 + \sigma_3) = p' + u \\ p' &= \tfrac{1}{2}(\sigma_1' + \sigma_3') \end{aligned}\right\} \quad (6)$$

where σ_1 and σ_3 are maximum and minimum total principal stresses (pressures) respectively; σ_1' and σ_3' are maximum and minimum effective principal stresses (pressures) respectively; then the condition for liquefaction will be:

$$q \to 0;\ p' \to 0;\ \text{and}\ p \to u \quad (7)$$

and for liquefied soil

$$q = 0;\ p' = 0;\ \text{and}\ p = u \quad (8)$$

THREE TYPICAL MECHANISM OF SOIL LIQUEFACTION

Although the condition for soil liquefaction obeys the same rule as given by Eq.(4) or (7), the mechanism of liquefaction process may be different. Three typical mechanisms of soil liquefaction are to be reviewed briefly.

Liquefaction Caused by Seepage Pressure Only—Sand Boil

If the pore water pressure in a saturated sand deposit reaches and excesses the overburden pressure, the sand deposit will float or "boil" and lose entirely its bearing capacity. This process is nothing to do with the density and volumetric contraction of the sand. Therefore it has been usually considered as a phenomenon of "seepage instability". However, according to the mechanical behavior of the material, it also belongs to the category of soil liquefaction.

Liquefaction Caused by Monotonous Loading or Shearing—Flow Slide

The phenomenon of flow slide has long been recognized. Casagrande (1936 and 1975) has suggested the concept of critical void ratio. The main fact is that the skeleton of loose saturated sands exhibits irreversible contraction in the bulk volume under the action of monotonous loading or shearing, which causes increase of pore water pressure and decrease of effective pressure and finally brings about an "unlimited" flow deformation.

Liquefaction Caused by Cyclic Loading or Shearing—Cyclic Mobility

The phenomenon of cyclic mobility of saturated cohesionless soils has been investigated extensively during the last twenty to thirty years by many investigators with various experimental techniques and testing apparatuses. It has been revealed that cohesionless soils always show volumetric contraction at low shear strain level, but might dilate at higher shear strain level depending on the relative density of the soil. Therefore, under the action of cyclic shearing a saturated cohesionless soil could show liquefaction at time intervals when the shear strain level is low, but might regain shear resistance in time intervals when the shear strain level is higher. A sequence of such sort of intermittent liquefactions would bring about the phenomenon of cyclic mobility with "limited" flow deformation. If the saturated cohesionless soil was loose enough to keep contraction at high shear strain level, then it also could come out to be an "unlimited" flow deformation.

STRESS EVOLUTION DURING SOIL LIQUEFACTION

Following examples are given to show the way of stress evolution during soil liquefaction and the reality of Eqs(4), (5), (7) and (8).

Sand Boil

Suppose a saturated sand deposit with level ground surface is initially without seepage flow through it, then the initial state of stress at depth z is:

$$\left.\begin{aligned} \sigma_{zi} &= \gamma_t z = \gamma' z + \gamma_w z \\ \sigma_{xi} &= K_i' \gamma' z + \gamma_w z \end{aligned}\right\} \quad (9)$$

where σ_{zi} and σ_{xi} are initial vertical and horizontal total normal stresses (pressures) respectively; γ_t and γ' are saturated and submerged unit weights of the sand deposit respectively; γ_w is unit weight of water; K_i' is initial coefficient of lateral earth pressure in terms of effective stress; and the subscript "i" denotes the initial state. The alternative expression is:

$$\left.\begin{aligned} q_i &= \tfrac{1}{2}(1 - K_i')\gamma' z \\ p_i &= p_i' + \gamma_w z \\ p_i' &= \tfrac{1}{2}(1 + K_i')\gamma' z \end{aligned}\right\} \quad (10)$$

Now, if the pore water pressure at depth z is to be raised by an amount of Δu ($\Delta u < \gamma' z$), the state of stress will change to an intermediate state as:

$$\left.\begin{aligned} \sigma_z &= \sigma_{zi} = \sigma_z' + u \\ \sigma_x &= \sigma_x' + u = K'\sigma_z' + u \end{aligned}\right\} \quad (11)$$

where $u = \gamma_w z + \Delta u$; $\sigma_z' = \gamma' z - \Delta u$; $\sigma_x' = K'\sigma_z'$ (in which K' is coefficient of lateral pressure in terms of effective stress in the intermediate state). The alternative expression is:

$$\left.\begin{aligned} q &= \tfrac{1}{2}(1 - K')(\gamma' z - \Delta u) \\ p &= p' + u \\ p' &= \tfrac{1}{2}(1 + K')(\gamma' z - \Delta u) \end{aligned}\right\} \quad (12)$$

Finally, if Δu approaches the value of $\gamma' z$ (or u approaches the value of $\gamma_t z$), the state of stress will approach to its final state as:

$$\sigma'_{zf} \rightarrow 0;\ \sigma'_{xf} \rightarrow 0;\ \text{and}$$

$$\sigma_{xf} \rightarrow u_f \rightarrow \gamma_t z = \sigma_{zf} \quad (13)$$

or

$$q_f \rightarrow 0;\ p'_f \rightarrow 0;\ \text{and}\ p_f \rightarrow u_f = \gamma_t z \quad (14)$$

where the subscript "f" denotes the final state. Equations (13) and (14) are exactly the Eqs(4) and (7). So liquefaction will commence. Now, the upward hydraulic gradient of the seepage flow is:

$$i = \Delta u_f/\gamma_w z \rightarrow \gamma'/\gamma_w = i_{cr} \quad (15)$$

where i_{cr} is well known as the critical hydraulic gradient for sand boil.

<u>Liquefaction in Consolidated Undrained Triaxial Compression Test</u>

The initial state of stress is:

$$\left.\begin{aligned} \sigma_{1i} &= \sigma'_{1i} + u_i \\ \sigma_{3i} &= \sigma'_{3i} + u_i \end{aligned}\right\} \quad (16)$$

or

$$\left.\begin{aligned} q_i &= \tfrac{1}{2}(\sigma_{1i} - \sigma_{3i}) = \tfrac{1}{2}(\sigma'_{1i} - \sigma'_{3i}) \\ p_i &= p'_i + u_i \\ p'_i &= \tfrac{1}{2}(\sigma'_{1i} + \sigma'_{3i}) \end{aligned}\right\} \quad (17)$$

where σ_{1i} and σ_{3i} are initial axial and lateral total principal stresses (pressures) respectively; σ'_{1i} and σ'_{3i} are initial axial and lateral effective principal stresses (pressures) respectively; u_i is back pressure or initial pore water pressure.

In the intermediate stage of test the state of stress is:

$$\left.\begin{aligned} \sigma_1 &= \sigma_{1i} + \Delta\sigma_1 = \sigma'_1 + u \\ \sigma_3 &= \sigma_{3i} = \sigma'_3 + u \end{aligned}\right\} \quad (18)$$

or

$$\left.\begin{aligned} q &= q_i + \Delta\sigma_1/2 \\ p &= p' + u \\ p' &= p'_i - \Delta u + \Delta\sigma_1/2 \end{aligned}\right\} \quad (19)$$

where $u = u_i + \Delta u$; Δu is change of pore water pressure induced by monotonously applied axial stress $\Delta\sigma_1$; $\sigma'_1 = \sigma'_{1i} + \Delta\sigma_1 - \Delta u$; and $\sigma'_3 = \sigma'_{3i} - \Delta u$.

According to the theory of limit equilibrium the value of $\Delta\sigma_1$ should be bounded by the following relationship

$$\Delta\sigma_1 \leqslant B_{cu}\left[(p'_i - \Delta u) - q_i/\sin\phi'\right] \quad (20)$$

where $B_{cu} = 2\sin\phi'/(1 - \sin\phi')$.

From Eq.(20) it is not difficult to prove that the prerequisite for Δu to approach the value of σ'_{3i} is:

$$\sigma_{1i} + \Delta\sigma_1 \rightarrow \sigma_{3i} \quad (21)$$

Then, if Δu increases continuously and approaches the value of σ'_{3i} (or u approaches the value of σ_{3i}), the final state of stress becomes:

$$\sigma'_{1f} \rightarrow 0;\ \sigma'_{3f} \rightarrow 0;\ \text{and}$$

$$\sigma_{1f} \rightarrow u_f \rightarrow \sigma_{3i} = \sigma_{3f} \quad (22)$$

or

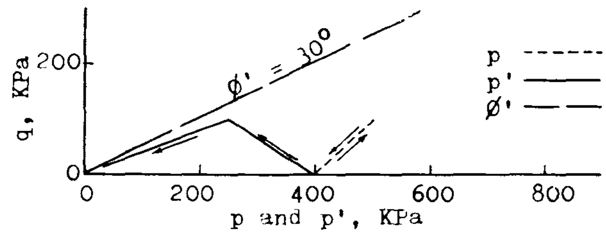
$$q_f \rightarrow 0;\ p'_f \rightarrow 0;\ \text{and}\ p_f \rightarrow u_f \rightarrow \sigma_{3i} \quad (23)$$

Equations (22) and (23) are exactly the Eqs(4) and (7). So liquefaction can commence. Two examples are shown in Fig.1 which are deduced from the data given by Casagrande (1975).

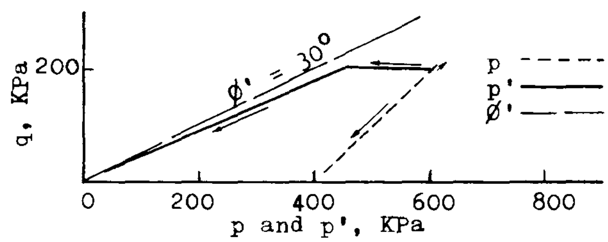
<u>Liquefaction in Consolidated Undrained Cyclic Simple Shear Test</u>

The initial state of stress is:

$$\left.\begin{aligned} \sigma_{zi} &= \sigma'_{zi} + u_i \\ \sigma_{xi} &= \sigma'_{xi} + u_i = K'_i\sigma'_{zi} + u_i \end{aligned}\right\} \quad (24)$$



(a) Isotropically Consolidated Undrained Triaxial Compression Test



(b) Anisotropically Consolidated Undrained Triaxial Compression Test

Fig.1. Examples of Stress Path Diagrams of Consolidated Undrained Triaxial Compression Test on Saturated Loose Sand

or

$$\left.\begin{aligned} q_i &= \tfrac{1}{2}(1 - K_i')\sigma_{zi}' \\ p_i &= p_i' + u_i \\ p_i' &= \tfrac{1}{2}(1 + K_i')\sigma_{zi}' \end{aligned}\right\} \quad (25)$$

where σ_{zi} is initially applied vertical total normal stress (pressure); u_i is back pressure or initial pore water pressure.

After application of cyclic simple shear stress $\Delta\tau_{zx}(t)$ the state of stress will change consecutively as:

$$\left.\begin{aligned} \sigma_z &= \sigma_{zi} = \sigma_z' + u \\ \sigma_x &= K'\sigma_z' + u = \sigma_x' + u \\ \tau_{zx} &= \Delta\tau_{zx}(t) \end{aligned}\right\} \quad (26)$$

or

$$\left.\begin{aligned} q &= \tfrac{1}{2}\sqrt{(1 - K')^2(\sigma_{zi}' - \Delta u)^2 + 4\Delta\tau_{zx}^2(t)} \\ p &= p' + u \\ p' &= \tfrac{1}{2}(1 + K')(\sigma_{zi}' - \Delta u) \end{aligned}\right\} \quad (27)$$

where $u = u_i + \Delta u$: Δu is change of pore water pressure induced by cyclic shearing: $\sigma_z' = \sigma_{zi}' - \Delta u$; and $\sigma_x' = K'\sigma_z'$.

According to the theory of limit equilibrium the value of $\Delta\tau_{zx}(t)$ should be bounded by the following relationship:

$$\left|\Delta\tau_{zx}(t)\right| \leqslant B_{css}(\sigma_{zi}' - \Delta u) \quad (28)$$

where $B_{css} = \frac{1}{2}\sqrt{(1 + K')^2\sin^2\phi' - (1 - K')^2}$.

From Eq.(28) it is not difficult to prove that the prerequisite for Δu to approach the value of σ_{zi}' is:

$$\Delta\tau_{zx}(t) \rightarrow 0 \quad (29)$$

Therefore, if Δu increases consecutively, liquefaction can occur at moments when the condition given by Eq.(29) is met. At these moments:

$$\sigma_z' \rightarrow 0;\ \sigma_x' \rightarrow 0;\ \tau_{zx} \rightarrow 0\text{: and}$$

$$\sigma_x \rightarrow u \rightarrow \sigma_{zi} = \sigma_z \quad (30)$$

or

$$q \rightarrow 0;\ p' \rightarrow 0;\ \text{and } p \rightarrow u \rightarrow \sigma_{zi} \quad (31)$$

Equations (30) and (31) are exactly the Eqs(4) and (7). There are many test results which can substantiate this conclusion such as those given by Peacock and Seed (1968).

<u>Liquefaction in Consolidated Undrained Cyclic Triaxial Test</u>

The initial state of stress is same as that given by Eqs(16) and (17). After application of cyclic axial stress $\Delta\sigma_1(t)$ the state of stress will change consecutively as:

$$\left.\begin{aligned} \sigma_1 &= \sigma_{1i} + \Delta\sigma_1(t) = \sigma_1' + u \\ \sigma_3 &= \sigma_{3i} = \sigma_3' + u \end{aligned}\right\} \quad (32)$$

where $u = u_i + \Delta u$ (in which u_i and Δu are initial and induced pore water pressures respectively); $\sigma_1' = \sigma_{1i}' - \Delta u + \Delta\sigma_1(t)$; and $\sigma_3' = \sigma_{3i}' - \Delta u$.

According to the theory of limit equilibrium the value of $\Delta\sigma_1(t)$ is bounded by the following relationships:

(i) In case $\Delta\sigma_1(t) < \sigma_{3i} - \sigma_{1i}$ (extension)

$$-\Delta\sigma_1(t) \leqslant B_{ct}[(p_i' - \Delta u) + q_i/\sin\phi'] \qquad (33)$$

(ii) In case $\Delta\sigma_1(t) > \sigma_{3i} - \sigma_{1i}$ (compression)

$$\Delta\sigma_1(t) \leqslant B_{cc}[(p_i' - \Delta u) - q_i/\sin\phi'] \qquad (34)$$

where $B_{ct} = 2\sin\phi'/(1 + \sin\phi')$; and $B_{cc} = B_{cu}$.

From Eqs(33) and (34) it is not difficult to prove that the prerequisite for Δu to approach the value of σ_{3i}' is:

$$\sigma_{1i} + \Delta\sigma_1(t) \to \sigma_{3i} \qquad (35)$$

Therefore, if Δu increases consecutively, liquefaction can occur at moments when the condition given by Eq.(35) is met. At these moments the state of stress will be:

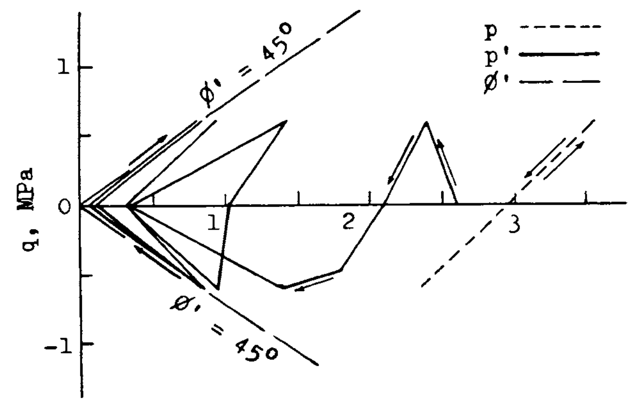
$$\sigma_3' \to 0;\ \sigma_1' \to 0;\ \text{and}$$

$$\sigma_1 \to u \to \sigma_{3i} = \sigma_3 \qquad (36)$$

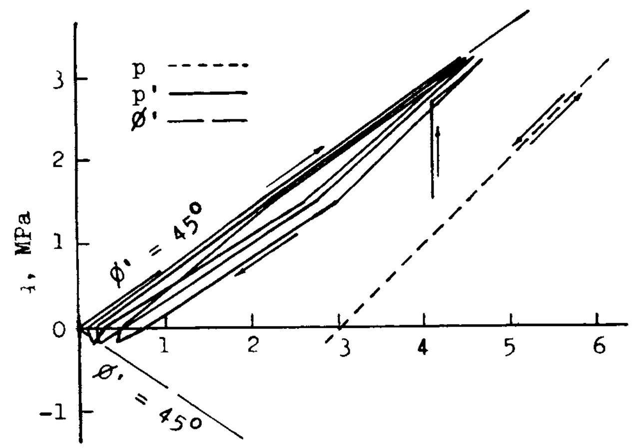
or

$$q \to 0;\ p' \to 0;\ \text{and}\ p \to u \to \sigma_{3i} \qquad (37)$$

Equations (36) and (37) are exactly the Eqs(4) and (7). Two examples are shown in Fig.2 which are deduced from the data given by Banerjee, Seed and Chan (1979).



(a) Isotropically Consolidated Undrained Cyclic Triaxial Test



(b) Anisotropically Consolidated Undrained Cyclic Triaxial Test

Fig.2. Examples of Stress Path Diagrams of Consolidated Undrained Cyclic Triaxial Test on Saturated Dense Coarse Gravel

Liquefaction in Shaking Table Sand Box Test

The initial state of stress in box-contained saturated sand is very much the same as that given by Eqs(9) and (10). After the shaking table starts, the stress evolution in the saturated sand mass is somewhat complicated, but the average state of stress will follow a similar pattern as that given by Eqs(11) and (12); however, the change of pore water pressure Δu will be related to volumetric strain of the sand skeleton induced by the vibratory action and will be affected by the drainage and boundary conditions of the sand body. If the situation is favorable for pore water pressure to build up and to approach and reach the value of the overburden pressure, then the condition given by Eqs(13) and (14) may be obtained and liquefaction can occur.

COMMENTS ON OTHER RELATED FACTORS

Limit Equilibrium Condition

In the course of stress evolution for soil liquefaction the state of stress is bounded by the limit equilibrium condition which is always to be reached before the commencement of liquefaction. The limit equilibrium condition of saturated cohesionless soils is well defined as:

$$q = p'\sin\phi' \qquad (38)$$

The author (1980) has considered the first arrival of the limit equilibrium condition as the incipient failure margin of the soil, at which the shear resistance does not necessarily approach zero and the soil can still maintain its solid state, while at liquefaction stage the shear resistance of the soil should approach zero.

Strain

The author rather agrees to the comments made by the Committee on Soil Dynamics of the Geotechnical Engineering Division of the American Society of Civil Engineers (1978) under the definition of the term "Liquefaction", which states that the definition of "Liquefaction" is independent of deformation or ground failure movement that might follow the liquefaction. However, in some other interpretations the terms such as "Unlimited" and "Limited" shear strains have been mentioned in connection with the phenomenon of soil liquefaction, because they have practical significance in geotechnical engineering. It seems hard to use strain as a reference to define the term "Liquefaction", but it does have influence on the development and consequence of soil liquefaction.

Pore Water Pressure Evolution due to Cyclic Loading or Vibration

The author (1962 and 1980) has presented his point of view on the mechanism of development, dissipation and dispersion of excess pore water pressures in the saturated cohesionless soil mass due to vibration in previous publications. It has been based on the fabric and volumetric strain characteristics of the cohesionless soils and the law of seepage flow. The main formulation is:

In case without residual rebound

$$\frac{1}{\gamma_w}\mathrm{div}(k\ \mathrm{grad}\ u_d) = \alpha\frac{\partial}{\partial t}(u_d - \sigma_m - u_d^*) \qquad (39)$$

In case with residual rebound

$$\frac{1}{\gamma_w}\mathrm{div}(k\ \mathrm{grad}\ u_d) = \beta\frac{\partial}{\partial t}(u_d - \sigma_m) \qquad (40)$$

where k is coefficient of permeability; u_d is excess pore water pressure due to vibratory action for natural drainage condition; u_d^* is excess pore water pressure determined by consolidated undrained vibration or cyclic loading tests with similar stress condition as in the field; α and β are coefficients of volumetric compression and rebound or recompression of the soil respectively; and t is time.

Equations (39) and (40) accompanied with the equation of equilibrium may be used to estimate the pore water pressure evolution in the saturated cohesionless soil mass due to vibration or cyclic loading.

CONCLUSIONS

1. As the "Soil Liquefaction" is considered as the transformation of the soil from a solid state into a liquid state, the stress evolution in the course of liquefaction of saturated cohesionless soils should follow the process as given by Eqs(4) and (7).

2. During the course of soil liquefaction the state of stress is bounded by the limit equilibrium condition as given by Eq.(38). The limit equilibrium condition is always to be reached before the commencement of soil liquefaction.

3. The mechanism of pore water pressure evolution in a saturated cohesionless soil mass due to vibration or cyclic loading can be investigated based on the fabric characteristics of the soil skeleton. The study made on this line has led to the formulation of Eqs(39) and (40).

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