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Liquefaction potential assessment for unsaturated soil based on liquefaction mechanism of unsaturated sand

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ABSTRACT :

In this study, author suggests the liquefaction mechanism of unsaturated sand and introduces a method of hazard evaluation based on it. Even under the unsaturated condition, sand with a highly compressible soil particle structure may lose effective stress due to cyclic shear and may reach the liquefaction. Based on the effective stress of unsaturated soil, it can be understood that the liquefaction is achieved when the pore air pressure equals pore water pressure and also equals the total stress. When the pore air can be assumed to be an ideal gas, a consideration of the volume change of pore air between the initial and final full liquefaction states should be established with the relation of the Boyle-Charles law. Therefore, the amount of volumetric compressions required for the unsaturated soil to liquefy will be determined theoretically. The equation of the unsaturated soil liquefaction was proposed in this study. So, it can be understood that the void ratio change required to cause liquefaction is a function of the initial degree of saturation, the confining stress, and initial pore pressure. A new scheme to evaluate the liquefaction potential is proposed. The proposed method for laboratory tests showed good performance when applied to a laboratory test.

KEYWORDS: Unsaturated sand, liquefaction potential, volumetric compression

1. Introduction

1.1.Background and objective of the study

In recent earthquakes, mud-flow type slope failures occurred in gentle slope fills. The typical case is the failure of a gentle fill slope occurred at Dateshita in Tsukidate town during the 2003 Sanriku-Minami earthquake in Japan. Most of the failures were surface failure, and the sediment of failure was probably in unsaturated sand. The saturation of the failed ground soil in the case of Tsukidate town was about 70% to 90%. Authors have been researching Seismic behavior of surface soil, examining unsaturated cyclic triaxial tests. As a result, some important points were acquired. The first point is that Effective stress of unsaturated sandy soil decreased during cyclic loading. The second is that effective stress of some specimen reached to zero as so-called liquefaction, even if the degree of saturation is relatively low. The third is that in order for unsaturated soil to liquefy, since a pore air pressure increases, a volumetric compression is required. The final and the most interesting point is that the amount of volumetric compressions required to liquefaction of unsaturated sand is obtained. In this paper, author suggests the liquefaction mechanism of unsaturated sand and introduces a method of hazard evaluation based on it.

1.2.The triaxial cyclic test results of the unsaturated sand

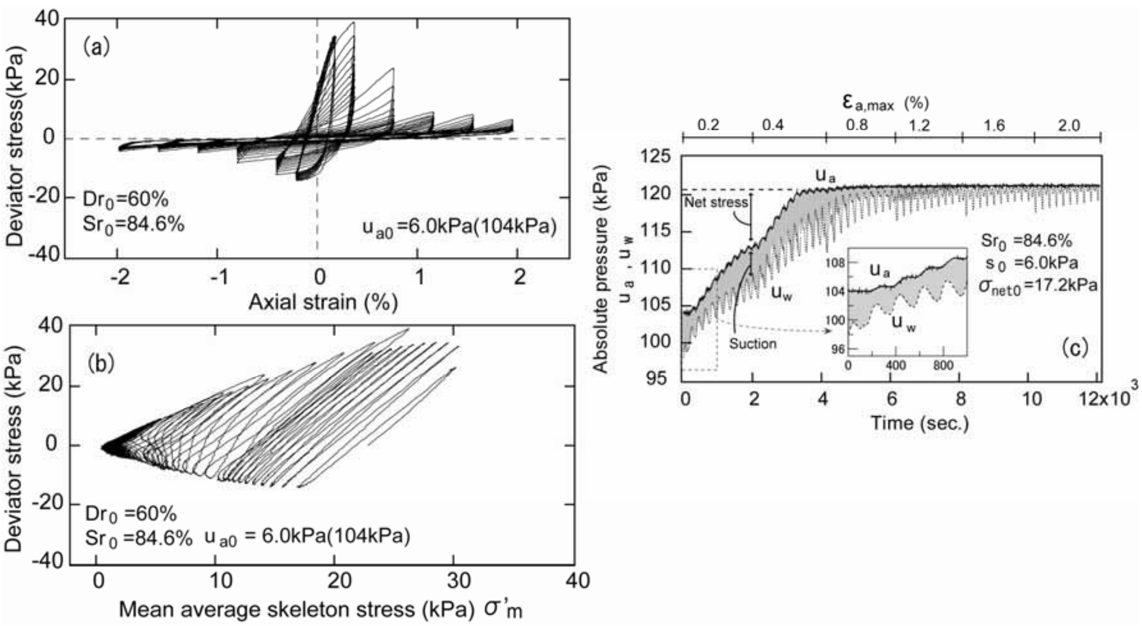
Kazama et al. 2006 and Unno et al. 2008 reported the cyclic shear behavior of unsaturated sand considering suction, pore water pressure, pore air pressure and volumetric compression at previous study. Here, it is well known that there are many definitions of an effective stress for unsaturated soils. For simplicity, in these studies, authors have used the average skeleton stress to evaluate the effective stress, as shown in Equation (1.1).

$$\sigma_m' = (\sigma_m - u_a) + \chi(u_a - u_w) \tag{1.1}$$

$$s = u_a - u_w \tag{1.2}$$

In the equation, u_a, u_w and c represent the pore air pressure, the pore water pressure, and the material parameter, respectively. The suction s is defined by Equation (1.2). Several definitions of parameter c have been proposed by many researchers (e.g. Bishop et al., 1963; Vanapalli et al., 1996; Gallipoli et al., 2002). In these studies, parameter c adopts the degree of saturation $S_r/100$. Tests were conducted for specimens with different initial suctions or initial saturation and the same dry density under undrained conditions for both air and water.

In the tests results, unsaturated sand specimens lost their effective stress under cyclic shear loading even if the degree of saturation is about 80%. For example, Figure 1 shows the test result for unsaturated sand with a relatively low degree of saturation. The specimen is Toyoura sand (initial relative density D_{r0}=60%, initial suction s_0=6.0 kPa, initial degree of saturation S_{r0}=84.6 %). In this figure, (a)-(c) show the stress-strain relationships, the effective stress paths, and the time histories of the pore air and water pressure during cyclic shear, respectively. In Figure 1(c) the difference between u_a and u_w, which is indicated by shading, represents suction. As well as a gradual decrease in the peak of the deviator stress with increasing strain amplitude, the stiffness decreased with each loading cycle. Consequently, the mean effective principal stress reached zero. It is noteworthy that, even in cases with a considerably small degree of saturation, the soil particle skeleton is degraded by the cyclic shear and reaches a zero effective stress state as so-called liquefaction, thereby causing a failure in the microstructure and engendering the reduction of the soil shear strength. Regarding on the cyclic shear behavior of unsaturated sand, another literature written by the authors (Unno et al., 2008) is referred to.



(a) Stress-strain relationship (b) Stress path (c) Time history of suction

Figure 1 cyclic shear behavior of unsaturated sand (D_{r0}=60%, S_{r0}=84.6%) (Unno et al., 2008)

2. LIQUEFACTION STATE OF UNSATURATED SAND

Even under the unsaturated condition, specimens with a highly compressible soil particle structure may lose average skeleton stress (i.e. effective stress) due to cyclic shear and may reach the liquefaction state. Based on the test results and Equation (1.1), it can be understood that the complete liquefaction state for unsaturated soils is achieved when both the pore air and water pressure are the same as the initial total confining pressure. If the pore air can be assumed to be an ideal gas, a consideration of the volume change of pore air between the initial and final full liquefaction states should establish the following relation from the Boyle-Charles law.

$$u_{a0}V_{a0} = u_{a,liq}(V_{a0} - \Delta V_a) = const. \tag{2.1}$$

where V_{a0} is the initial volume of pore air. It should be noted that pore air pressure is an absolute value including atmospheric pressure. Equation (2.1) should be established not only in the initial and final state, but also in the optional time during cyclic shear. When there is zero effective stress state in Equation (1.1), the pore air and water pressure should be equal to the initial total pressure. Thus, $u_{a,liq}$ can be replaced by σ_{m0} in Equation (2.1); the following relationship is established in the zero effective stress state.

$$u_{a0}V_{a0} = \sigma_{m0}(V_{a0} - \Delta V_a) \tag{2.2}$$

Under the undrained condition, the volume change of pore air ΔV_a is equal to the volume change of the soil particle structure, which represents the volume change required to cause complete liquefaction. Here, since the volume compressibility of air is significantly larger than that of soil particles and water, soil particles and water are assumed uncompressible. From Equation (2.2), because the volume change ΔV_a corresponds to the void ratio change (e_0 - e_{liq}), the relationship expressed in Equation (2.2) can be rewritten as follows:

$$e_0 - e_{liq} = (1 - \frac{S_{r0}}{100})(1 - \frac{u_{a0}}{\sigma_{m0}})e_0 \tag{2.3}$$

$$e_{liq} = (\frac{u_{a0}}{\sigma_{m0}} + \frac{S_{r0}}{100} - \frac{u_{a0}}{\sigma_{m0}}\frac{S_{r0}}{100})e_0 \tag{2.4}$$

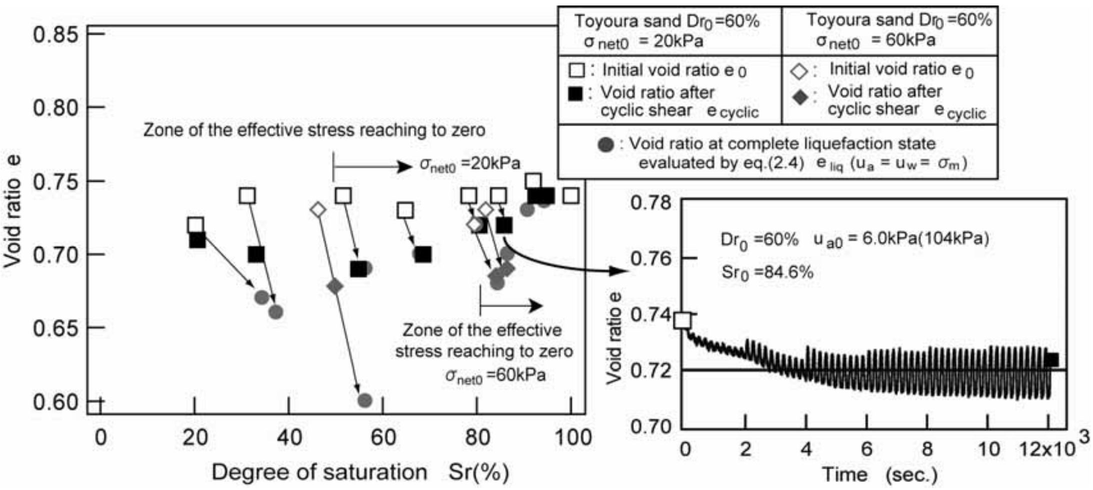


Figure 2 Relationship between the void ratio and the degree of saturation before and after cyclic loading

Here, e_{liq} is the void ratio at the zero effective stress state, in which both pore air and water pressure are identical to the initial mean total confining pressure. It can be understood that the void ratio change required to cause liquefaction is a function of the initial degree of saturation, the confining stress, and initial pore pressure. From Equation (2.3), as was mentioned earlier, liquefaction is more likely to occur as the initial degree of saturation is high and as the initial effective stress is low, and only a small volume change is required to cause liquefaction.

Figure 2 shows the relationship between the void ratio and the degree of saturation before and after cyclic shear. In the figure, the open square mark the circle mark e_{liq} depicts the void ratio change required to obtain a state of complete liquefaction as calculated by Equation (2.4). Since the structure of the soil is loose, the soils liquefy easily because of their high compressibility. It can be seen that the test specimens reached the liquefaction state when the void ratio after cyclic shear e_{cyclic} was equivalent to e_{liq}. On the other hand, the sample did not reach liquefaction state in cases where the e_{cyclic} did not reach e_{liq}.

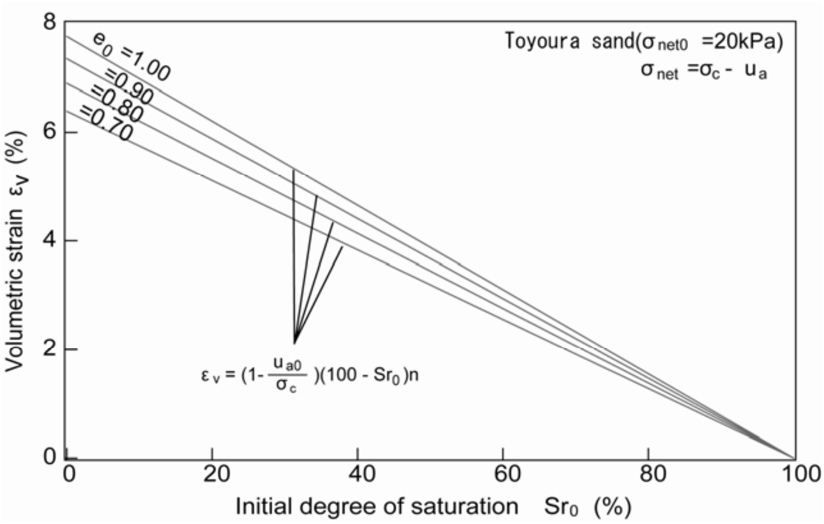


Figure 3 volumetric strain required for liquefaction

The volumetric compression of the specimen is an important key of liquefaction of an unsaturated soil. Figure 3 shows the volume strain required for liquefaction. Volume compression required for liquefaction becomes small, so that an initial void ratio is small.

3.0 LIQUEFACTION POTENTIAL ASSESSMENT FOR UNSATURATED SOIL GROUND

3.1. Volume compressibility of soil subjected to cyclic shear history

Volume compressibility under cyclic shear is the key issue in the liquefaction of unsaturated soil. In order to evaluate the volume compressibility of soil, the authors studied the volume change of sand subjected to cyclic shear (Unno et al. 2006b). In the study, several series of strain-controlled cyclic shear triaxial tests were carried out under both undrained and drained conditions, and the volume change of the soils under the drained cyclic shear was compared with those reconsolidated after the undrained cyclic shear with the same shear strain history. Figure 4 shows the comparison of volume change between two kinds of drainage conditions. Figure 4(a) shows the test procedures and Figure 3(b) shows the test result. Here, it is known that the accumulated shear strain path is an effective index to represent the degree of shear strain history (Sento et al., 2003). The accumulated shear strain is defined by the following equation.

$$\gamma_{acm} = \int |\dot{\gamma}(t)| dt \tag{3.1}$$

$\dot{\gamma}(t)$ is the velocity of the shear strain at time t. In the following discussion, it is assumed that such a relationship also exists under the unsaturated condition. Under the same cyclic shear strain histories, the laboratory test results illustrated that the volumetric strain in the drained cyclic shear test of dry sand was the same in the reconsolidation test after the undrained cyclic shear of saturated sand.

The volumetric change under cyclic shear depended only on the strain history and not on the stress path. Regarding on the void ratio due to cyclic shear, another literature written by Unno et al, 2006c is referred to. In conclusion, the void ratio change Δe_{cyclic} during arbitrary cyclic shear history can be obtained by the drained cyclic shear test for dry sand rather than the reconsolidation test after undrained cyclic shear in the case of fully saturated sand.

3.2. Proposed method

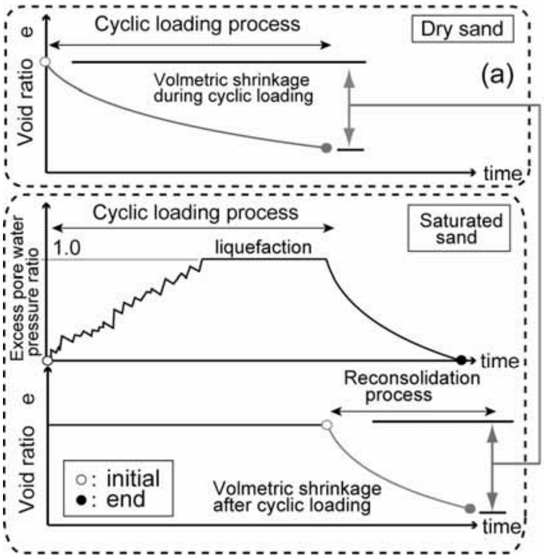
Figure 5 shows a new assessment scheme of the liquefaction potential for unsaturated soil ground as a function of the degree of saturation. The concept and procedure are explained below:

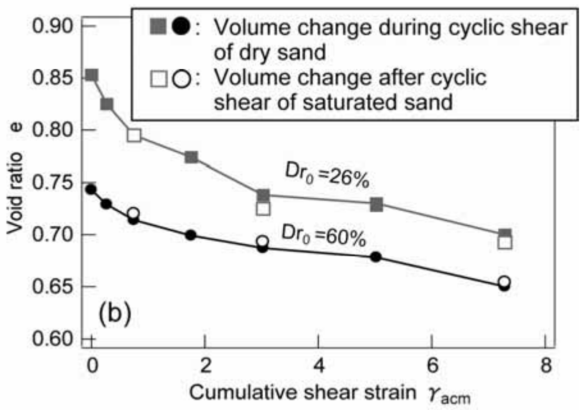
(1) The ground conditions are given as assessment parameters, and include the dry density, the total stress σ_m and the shear wave velocity as well as others. Here, if the soil-water characteristic curve (SWCC) is given by laboratory test for the soils, the distribution of the degree of saturation in the depth direction above the water table is estimated as shown in Figure 5(1).

(2) The void ratio change of the soils subjected to an arbitrary shear history is obtained from the cyclic shear test under the dry condition described in the previous section. If the accumulated shear strain γ_{acm} in the ground generated by the design earthquake can be obtained from a certain earthquake response analysis, by using the relationship between γ_{acm} and the void ratio, γ_{acm} can be used to evaluate the void ratio change Δe_{cyclic} of soils expected for the designed seismic loads. This is shown in Figure 5(2).

(3) In the case where the initial pore water pressure is assumed to be the atmospheric pressure $u_{a0} = 98$ kPa, the amount of volume contraction required to reach the zero effective stress state ($e_0 - e_{liq}$) can be calculated by Equation (2.3) in Figure 5(3).

(4) Finally, a comparison of the Δe_{cyclic} and (e_0 - e_{liq}) can be used to evaluate the possibility of liquefaction occurrence at the target depth. Naturally, in the case of $\Delta e_{cyclic} < (e_0 - e_{liq})$, liquefaction will not occur. On the contrary, in case of $\Delta e_{cyclic} > (e_0 - e_{liq})$, the possibility of liquefaction occurrence will be higher with larger differences. Furthermore, the assessment of the depth direction will provide a zone with high liquefaction potential.





(a) Test procedures (b) Test results

Figure 4 volume compression behavior of sand under the same cyclic history

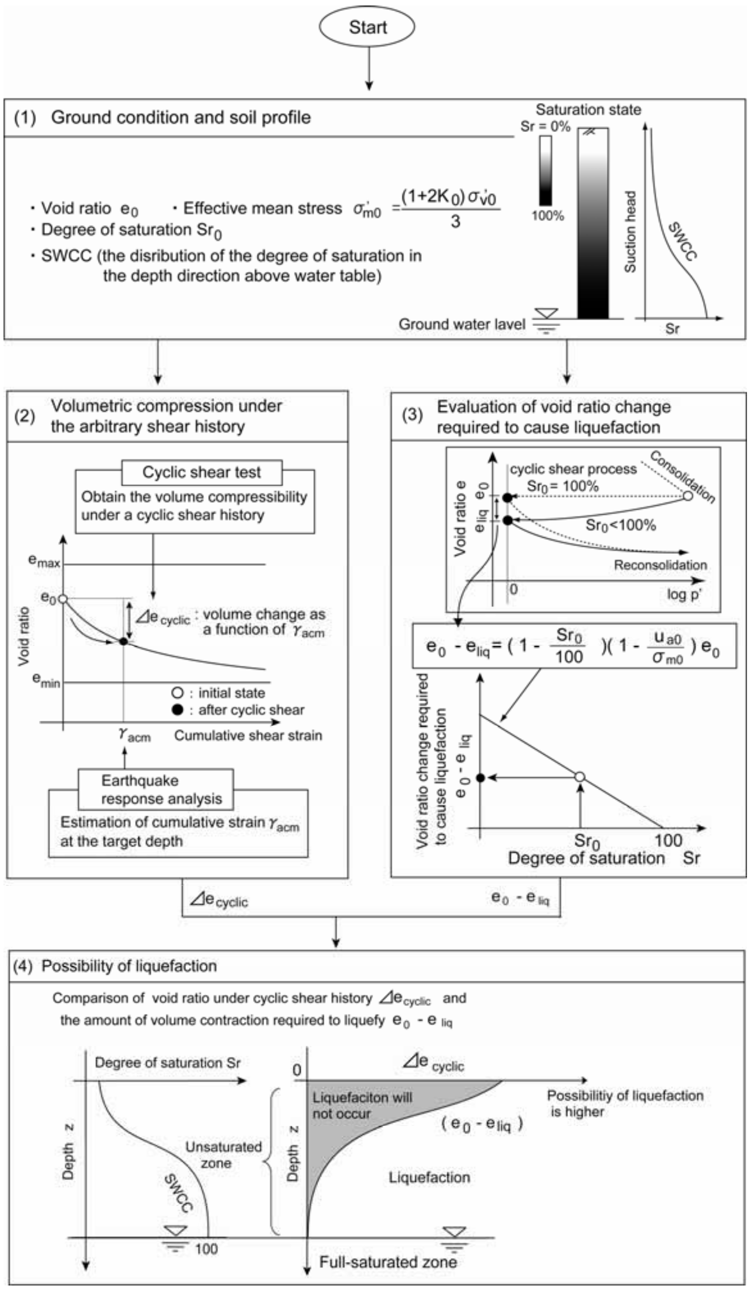


Figure 5 The assessment scheme of the liquefaction potential for unsaturated soil ground

3.3. The example of application to a test result

To confirm the applicability of the assessment method proposed, we have compiled the laboratory test results of test series-a without the seismic load consideration. That is, in the actual condition, the cyclic shear strain should be given for a designed seismic load, but in our investigation this process was ignored and the same shear strain history was applied as the seismic load. Figure 6 shows the volume change of the test specimens versus the degree of saturation both before and after cyclic shear. In the figure, the volume changes of some of the specimens are plotted after undrained cyclic shear. It is noteworthy that volume compression is not only produced during the cyclic shear process, but also during the reconsolidation process after cyclic shear. For example, in the full saturation case, the specimen shows no volume change during cyclic shear and the volume change occurred only during reconsolidation process. From the results, it can be seen that the boundary between non-liquefaction and liquefaction are in good agreement with the theoretical boundary evaluated. In conclusion, the proposed method performs well.

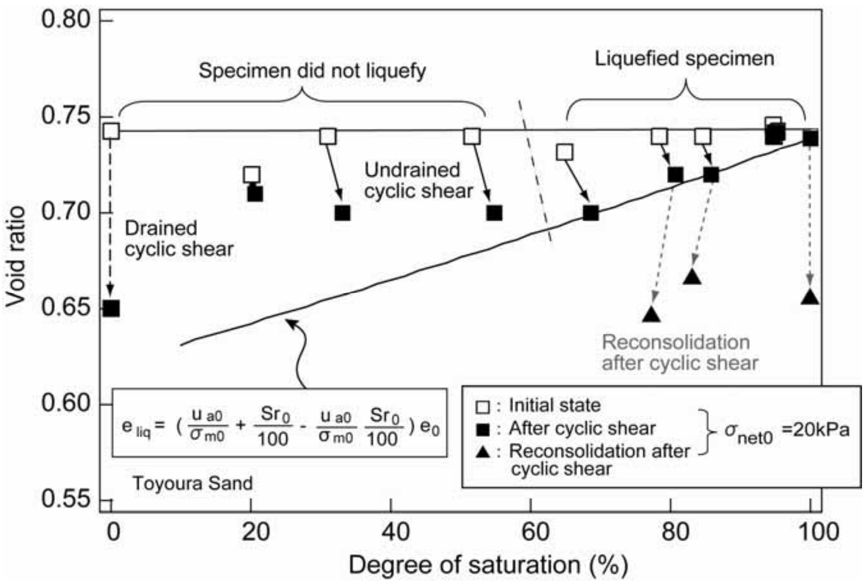


Figure 6 Volume change of the test specimens versus the degree of saturation
(Initial relative density D_{r0}=60%)

3.4. Suction and pore air pressure

According to the Japanese Geotechnical Society Standard (JGS-standard 0527-1998) and previous researches of unsaturated soil, an initial suction state can be produced to introduce certain pore air pressure, which is usually larger than an atmospheric pressure, to an unsaturated specimen in triaxial shear test. In such a condition, the pore air pressure may be different from the atmospheric pressure. On the other hand, when the suction is at or near the ground surface, capillary tension is demonstrated, the pore air pressure is thought to be identical to the atmospheric pressure and the pore water pressure is smaller than the atmospheric pressure. This point must fully be paid attention. It is because the point may influence the decision of liquefaction. When initial pore air pressure is difference, the amount of volumetric compression required for the specimen to liquefy changes. Sufficient consideration is required for the setup of suction.

CONCLUSIONS

In this paper, the author shows new method was proposed to evaluate the liquefaction potential based on our findings of the liquefaction mechanism of unsaturated soils. The conclusions can be summarized as follows:

1) By using the definition of the effective stress (i.e. average skeleton stress) for unsaturated sand, in which the contributions to the effective stress consists of a net stress and a suction, the liquefaction state for unsaturated soils can be defined as the zero effective stress state, as is the case for saturated soils.
2) Our test results reveal that, even in the case where the degree of saturation is quite small, the soil particle skeleton is degraded by cyclic shear and reaches a zero effective stress state, thereby causing a failure of the microstructure and engendering the reduction of the soil

shear strength.

3) To increase the pore air pressure, a certain volume change of the soil particle skeleton is necessary. The amount of volume change required to reach the zero effective stress state depends on the volume compressibility of the soil particle skeleton, the degree of saturation and the initial confining pressure. In addition, the required volume change can be calculated by the Boyle-Charles law.

4) By comparing the volume compressibility of the soil particle skeleton and the volume change required to cause liquefaction, the liquefaction potential of unsaturated soil can be evaluated. The proposed method to laboratory tests showed good performance when applied to a laboratory test.

Hazard evaluation in this study allows us to consider the water states of ground, which has not been considered in past seismic disaster studies. The result of this study probably gives effective knowledge on issues of ground disasters, composed of earthquakes and heavy rain caused by global warming.

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