SOILS AND FOUNDATIONS Vol. 46, No. 5, 695–700, Oct. 2006
Japanese Geotechnical Society

EFFECTS OF PORE FLUID COMPRESSIBILITY ON LIQUEFACTION RESISTANCE OF PARTIALLY SATURATED SAND

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ABSTRACT

It has been recognized that the soil resistance to liquefaction increases significantly as the degree of saturation decreases. However, the effect of the degree of saturation reported in the literature varies widely between researchers. In this study, influential factors of the liquefaction resistance of partially saturated sand are derived from theoretical consideration and effects of the factors are examined through a series of triaxial tests. It was confirmed that the degree of saturation has a significant effect on the liquefaction resistance. It also appeared that the liquefaction resistance depends on the initial confining pressure and the initial pore pressure; the higher the confining pressure and the lower the initial pore pressure, the higher the liquefaction resistance of partially saturated sand. A unique relationship between liquefaction resistance ratios and the potential volumetric strain was found, which enable to estimate the liquefaction resistance of partially saturated sand with the effects of the three factors taken into account.

Key words: confining pressure, degree of saturation, liquefaction, sand, triaxial test, volumetric strain (**IGC**: D6)

INTRODUCTION

Natural soil deposits below the ground water table are usually fully or nearly saturated with water (Tsukamoto et al., 2002). Recent investigations revealed, however, that injection of air in a soil could lower the degree of saturation of the soil substantially (Tokimatsu et al., 1990; Okamura et al., 2003) and the unsaturated condition of the desaturated soil lasts for a long time, typically more than tens of years (Okamura et al., 2006). This fact suggests that desaturation of soils could be an effective way to enhance the soil resistance to liquefaction in the field. It is, therefore, necessary to establish a practical method to estimate qualitatively the liquefaction resistance of partially saturated soils.

The effect of degree of saturation on the liquefaction resistance has been studied through laboratory tests. In the early research works, degree of saturation of tested specimens was mostly in the range close to 100%, because the primary objective in those studies was to establish the standard for the laboratory cyclic shear test to avoid undesirable unsaturated condition which resulted in overestimation of the liquefaction resistance (e.g. Martin et al., 1978). Thereafter, partially saturated sands with degree of saturation down to 70% were tested by several researchers. Figure 1 depicts some recent test results in the literature summarized in the form of the relationship between the degree of saturation and the liquefaction resistance of the partially saturated sand normalized with respect to that of the fully saturated sand (Huang et al., 1999; Yoshimi et al., 1989; Yasuda et al., 1999; Ishihara et al., 2001; Goto and Shamoto, 2002). Liquefaction resistances reported by any researchers consistently increased with decreasing the degree of saturation. However, the liquefaction resistance ratios were considerably different for different sands tested at different conditions, indicating that the degree of saturation may not be the only factor dominating the normalized liquefaction resistances of partially saturated sands.

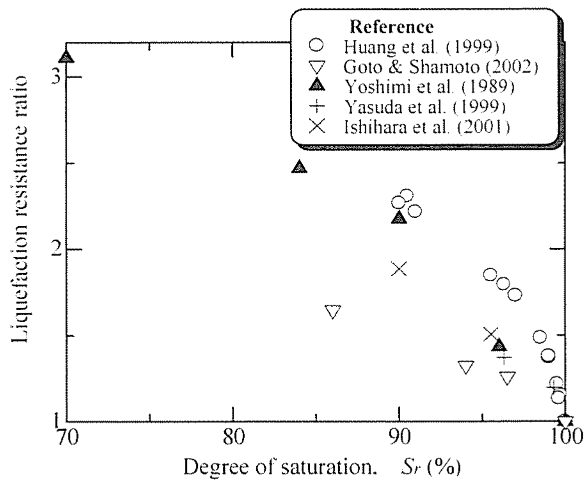

Fig. 1. Results of tests on the effect of degree of saturation on liquefaction resistance (Liquefaction resistance is normalized with that of fully saturated sand)

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The manuscript for this paper was received for review on December 8, 2005; approved on May 29, 2006.
Written discussions on this paper should be submitted before May 1, 2007 to the Japanese Geotechnical Society, 4-38-2, Sengoku, Bunkyo-ku, Tokyo 112-0011, Japan. Upon request the closing date may be extended one month.

As can be seen in Fig. 1, existence of air in a soil significantly enhances the liquefaction resistance. Researchers have paid great attention to saturate specimens completely for laboratory tests and model grounds for shaking table tests to avoid overrating soil resistances to liquefaction. Replacement of air in the void of soils by carbon dioxide followed by introducing deaired water under a vacuum pressure is the typical technique that has been developed. We, however, often observe contradictory phenomenon that a sand deposit in a small container or a bottle, being almost filled with water but contains visible air bubbles in the deposit, can be easily liquefied by shaking the container gently. This also alludes existence of influential factors of the liquefaction resistance of a partially saturated sand other than the degree of saturation.

In this study, influential factors of liquefaction resistance of a partially saturated sand are derived from theoretical consideration and effects of the factors are examined through a series of triaxial tests. Results are summarized in the form which can be easily applied to evaluate liquefaction resistances of partially saturated sands in situ.

FACTORS AFFECTING LIQUEFACTION RESISTANCE OF UNSATURATED SAND

Existence of air in the pore of a soil is considered to enhance the liquefaction resistance of the soil in two ways. The first mechanism is such that air in the pore plays a role of absorbing generated excess pore pressures by reducing its volume. The bulk modulus of the pore fluid changes significantly by the presence of air bubbles. The bulk modulus and change in volume of the pore fluid, that is air water mixture, may be the factors dominating this mechanism. The second is the matric suction of unsaturated soils which increases the effective stress and thus the strength of soil mass (Bishop and Blight, 1963). The matric suction depends not only on the degree of saturation but also on soil particle size. For most liquefiable soils the matric suction is less significant compared to the effective stress of soils at the depth of practical concern, say several meters or deeper. For the fine sand used in tests in this study, for instance, the matric suction is at the highest 4 kPa if degree of saturation goes down to 70%. In this study the first mechanism is focused on. The effect of the matric suction is neglected or the pressures of air and water in the pore are assumed to be the same. Note that for liquefiable soils with higher fines contents, such as non-plastic silt and sand containing considerable amount of fines, the effect of the matric suction could not be negligible. Further investigation is needed on this regard.

Consider a soil mass with the pore filled with air and water. For a small change in the pore pressure, Δp, the volumetric strains of the air and the water can be written by equations;

$$\varepsilon_a = B_a/\Delta p \tag{1}$$

$$\varepsilon_w = B_w/\Delta p \tag{2}$$

and volumetric strain of the fluid (water and air mixture), ε_{vf}, is;

$$\varepsilon_{vf} = \frac{\Delta p}{B_f} = [(1-S_r)\varepsilon_a + S_r\varepsilon_w] = \Delta p\left(\frac{1-S_r}{B_a} + \frac{S_r}{B_w}\right) \tag{3}$$

where S_r is degree of saturation of the soil mass and B_a, B_w and B_f are bulk moduli of the air, the water and the fluid, respectively. Since B_w is much higher than B_a, the second term in Eq. (3), S_r/B_w, is negligible. Introducing Boyle's law and assuming soil grains to be incompressible, we obtain the volumetric strain of the soil mass as;

$$\varepsilon_v = \frac{\Delta p}{B_a}(1-S_r)\frac{e}{1+e} = \frac{\Delta p}{p_0+\Delta p}(1-S_r)\frac{e}{1+e}$$
$$\leq \frac{\sigma_c'}{p_0+\sigma_c'}(1-S_r)\frac{e}{1+e} = \varepsilon_v^* \tag{4}$$

where p_0 and e denote the absolute pressure of the fluid and the void ratio of the soil mass, respectively. The highest value of the volumetric strain for the soil is achieved when the Δp attains its possible maximum value which is equal to the effective confining stress, σ_c'. This highest value of the volumetric strain is hereafter in this paper termed as potential volumetric strain, ε_v^*.

TRIAXIAL TEST

In this study, effects of the factors derived in the preceding section were investigated through a series of triaxial tests. Three testing parameters including the initial effective confining pressure, σ_c', the back pressure, p_0, and the degree of saturation, S_r, were varied between tests while the void ratio of the specimens was kept constant throughout the test series.

Preparation of Specimen

Toyoura sand was used in tests conducted in this study. The specific gravity of the sand is 2.64 and the minimum and the maximum void ratios are $e_{min}=0.609$ and $e_{max}=0.973$, respectively. Triaxial specimens were either saturated or partially saturated sand. Test specimens were prepared as follows. Wet sand with a water content of 5% was tamped to a relative density $D_r=40\%$ in a mold with internal dimensions of 50 mm in diameter and 100 mm in high. The sand was set in the triaxial cell and deaired water was introduced from the pedestal for a while. Then the back pressure of 98 kPa was applied and volume of water pushed into the specimen was measured. The measurement was continued for an hour since the volume was observed to increase gradually probably due to dissolution of air in the pore water. Volume of air in the specimen was estimated from Boyle's law with the measured volume of the water pushed into the specimen. The procedure of introducing deaired water and the air volume estimation was repeated until the specimen contained predetermined volume of air. It should be noted that the adsorption path and the desorption path of the soil-water characteristic curves are generally different.

Table 1. Triaxial test conditions

Rerative density, D_r (%)	Degree of saturation*, S_r (%)	Effective confining pressure, σ_c' (kPa)	Absolute back pressure, p_0 (kPa)	Potential volumetric strain, ε_v^*
39–43	100	49, 98	199	0
	98 (97.0–98.5)	19.6	199	0.00084
		49		0.00184
		98		0.00305
	96 (95.5–96.0)	19.6	199	0.00165
		49		0.00362
		98		0.00601
	90 (89.0–92.0)	98	199	0.0151
			297	0.0113
	80 (78.5–83.0)	98	199	0.0300
			297	0.0225
	70 (70.0–72.0)	98	199	0.0451
			297	0.0338

*: Degree of saturation in the parentheses was estimated from the measured volume of water pushed into the specimen by increasing the back pressure.

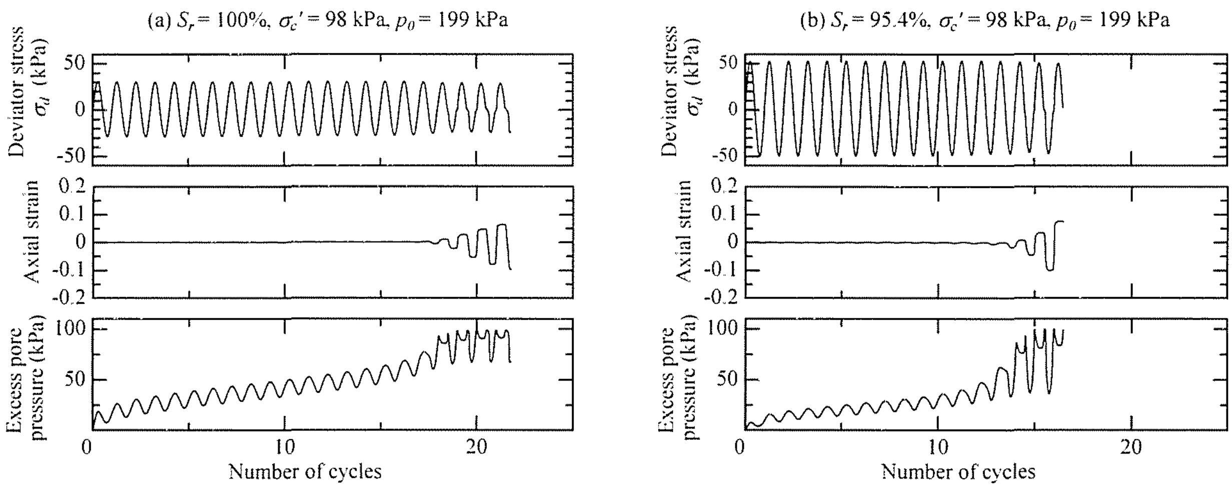

Fig. 2. Typical time history from tests on fully saturated and partially saturated specimens

However, the volume of the water pushed in and expelled from the specimen during increasing and decreasing the back pressure was essentially the same. This is probably due to the matric suction of this particular sand being very low in the range of degree of saturation tested in this study.

For the saturated specimen, deaired water was introduced until the Skempton's B value became 0.95 or higher. The effective confining pressure was kept constant to 10 kPa throughout the course of the preparation. On completion of preparation, the initial effective stress and the back pressure were applied and the specimens were subjected to the cyclic shear stress with a frequency of 0.01 Hz under undrained condition.

Testing Parameters

Three testing parameters derived in the previous section were varied between tests as shown in Table 1. It should be noted that the back pressure, p_0, used throughout this paper is the absolute pressure instead of the ordinary used gauge pressure. The range of the parameters tested was wide enough so that the range of ε_v^* covers that of possible field situation which might be encountered in practice; the initial effective confining pressure was varied between 19.8 kPa and 196 kPa and the range of S_r was similar to that of the in-situ soils desaturated by the sand compaction pile installation (Okamura et al., 2003, 2006). The values of the potential volumetric strain at the beginning of cyclic shearing, ε_v^*, are also given in Table 1.

It should be mentioned that the frequency of shear cycles in the tests being much lower than those of earthquake motions may allow air in the specimen to dissolve in pore water in accordance to generated excess pore pressure during cyclic shearing. According to the Henry's law, a maximum of 3.8 cm^3 in volume of air can dissolve for the test condition shown in Table 1 when the specimen generates a 100% excess pore pressure ratio. However, it was observed in preliminary tests that amount of air dissolved in the pore water in an hour after applying a back pressure of 98 kPa was very limited, approximately 0.3 cm^3 corresponding to an increase in S_r of 0.3%.

RESULTS AND DISCUSSIONS

Stress-strain Relationship

Figure 2 shows typical time histories obtained from tests on saturated and partially saturated ($S_r=95.4\%$) specimens at the same confining pressure ($\sigma_c'=98$ kPa) and the back pressure ($p_0=199$ kPa). For the saturated specimen, the excess pore pressure increased with the number of cycles. The axial strain started to increase swiftly as soon as the excess pore pressure approached to the initial effective confining pressure. This response is typical of a fully saturated loose sand. The response of the unsaturated sand indicated in Fig. 2(b) is quite similar to those of the saturated sand except for the applied cyclic stress amplitude being considerably higher.

Effect of the Factors

This section discusses effects of the three factors, that is the degree of saturation, the confining pressure and the back pressure on the liquefaction resistance. Figure 3 depicts the relationship between cyclic stress ratios and the number of cycles, N, to cause double amplitude axial strain, DA, of 5% for cases with $\sigma_c'=98$ kPa and $p_0=199$ kPa. As the degree of saturation decreases, the cyclic stress ratio increased irrespective of the number of cycles. The cyclic stress ratio almost doubled as the degree of saturation decreased from 100% to 90%, while in the range of the degree of saturation lower than 90% it increased at a lower rate with decreasing the degree of saturation. Hereafter, in this paper, the cyclic stress ratio to cause DA = 5% in 20 cycles is termed as the liquefaction resistance.

Illustrated in Fig. 4 are variations of the cyclic stress ratio with number of cycles for tests in which the confining pressure was varied between tests while the

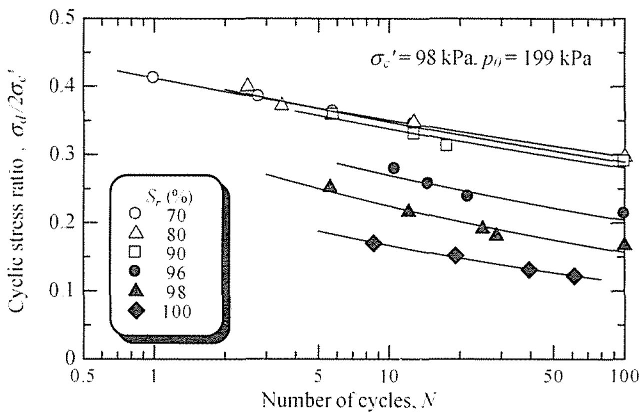

Fig. 3. Effect of degree of saturation on the relationship between cyclic stress ratio and number of cycles

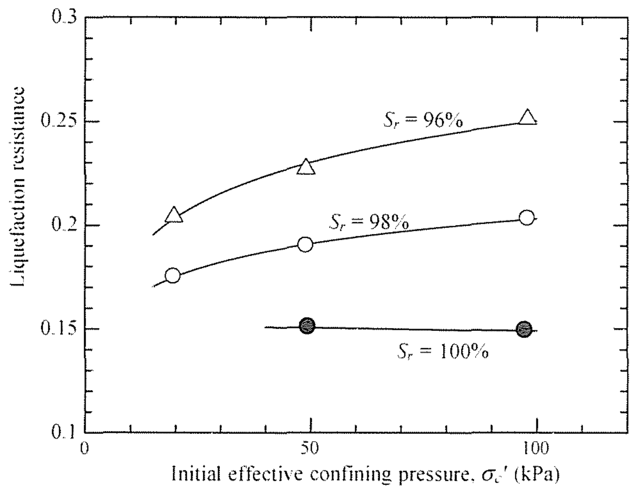

Fig. 5. Revolution of liquefaction resistance with an increase in initial effective confining pressure

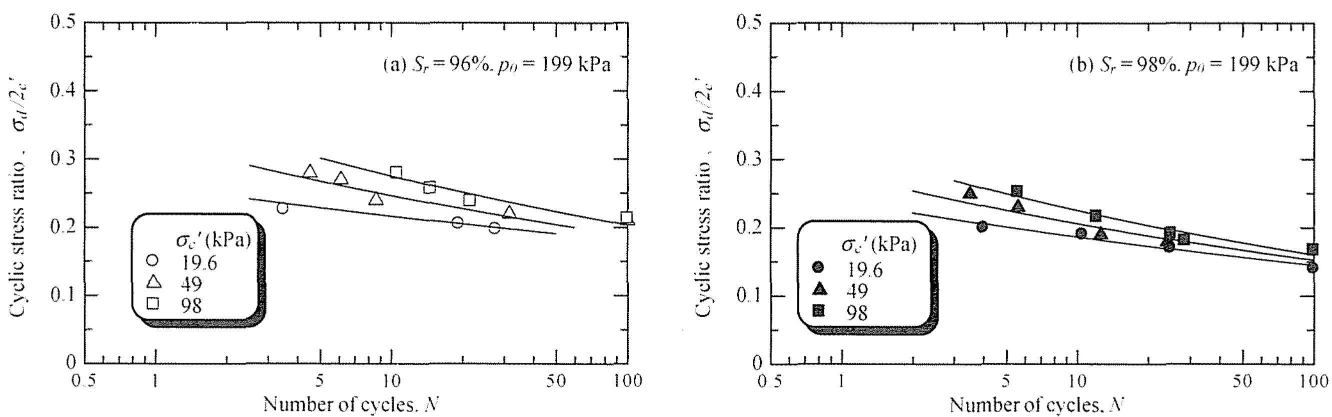

Fig. 4. Effect of initial effective confining pressure on the relationship between cyclic stress ratio and number of cycles

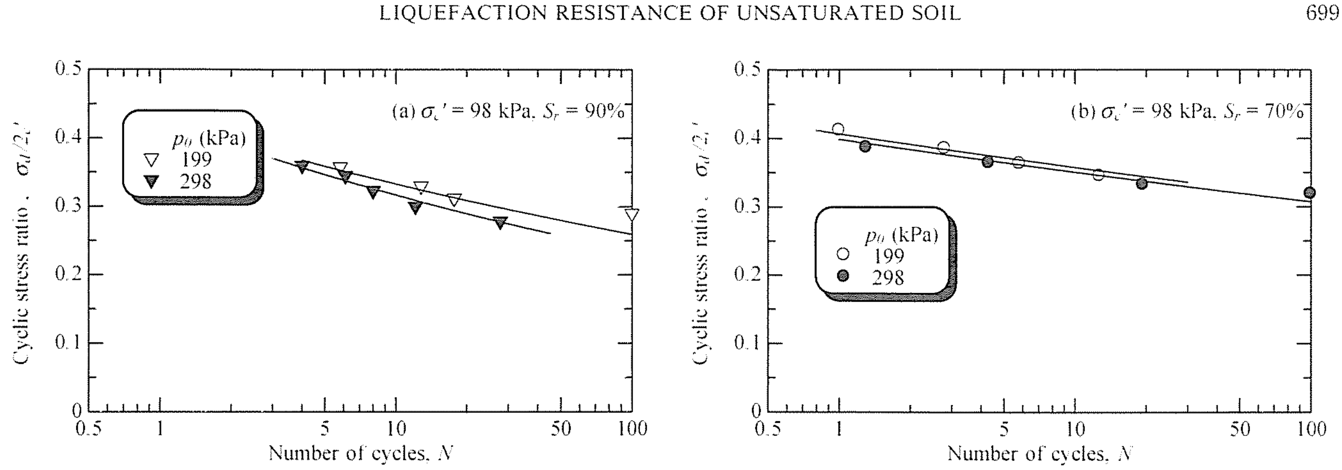

Fig. 6. Effect of initial pore pressure on the relationship between cyclic stress ratio and number of cycles

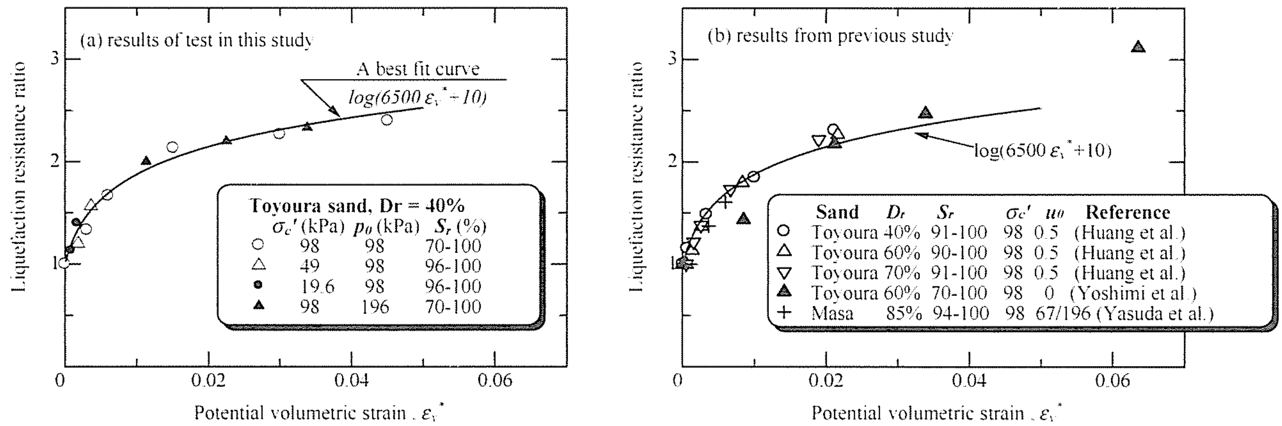

Fig. 7. Relationship between hypothetical volumetric strain and liquefaction resistance of partially saturated sand normalized with that of fully saturated sand

degree of saturation and the back pressure were kept constant. The cyclic stress ratio of the partially saturated sand is apparently dependent on the initial confining pressure. Liquefaction resistances are plotted against initial confining pressures in Fig. 5. The liquefaction resistances of the partially saturated sand increase with the initial confining pressures, with the liquefaction resistances being higher for lower S_r. The liquefaction resistance of the partially saturated sand seems to approach to that of fully saturated sand as the confining pressure decreases to zero. In other words, degree of saturation has a smaller effect on the liquefaction resistance of sand under a low confining pressure. Figure 6 indicates effects of the back pressure on the cyclic stress ratio. The liquefaction resistance of the partially saturated sand apparently depends on the back pressure, which is not the case for saturated sand. In opposition to the effect of the confining pressure, the liquefaction resistance decreases as the back pressure increases.

Liquefaction Resistance of Desaturated Sand

Finally, the effect of the potential volumetric strain, ε_v^*, given by Eq. (4) on the liquefaction resistance is discussed in this section. All the effects of three influential factors on the liquefaction resistance discussed above qualitatively support the idea of the first mechanism that, air in the pore plays a role of absorbing generated excess pore pressures by reducing its volume. Thus, the liquefaction resistance ratio, which is the liquefaction resistance of a partially saturated sand normalized with respect to that of the fully saturated sand, is plotted against the potential volumetric strain in Fig. 7(a). All the data lies along a unique curve, confirming that the potential volumetric strain is the determining factor of the effect of degree of saturation on this specific sand at relative density of 40%. Data retrieved from the literature is also shown in Fig. 7(b) in the same manner. The data plotted in this figure was obtained from the tests on specimens prepared using different sand at different relative density and subjected cyclic loading at different confining pressures as summarized in the figure. Despite these different conditions, all the data lies along the same curve as that in Fig. 7(a). This confirms that the effect of the degree of saturation on liquefaction resistance, which is arisen from the first mechanism, can be estimated using this curve.

Figure 7 and Eq. (4) indicate that the liquefaction resistance of a soil under a very low confining pressure is

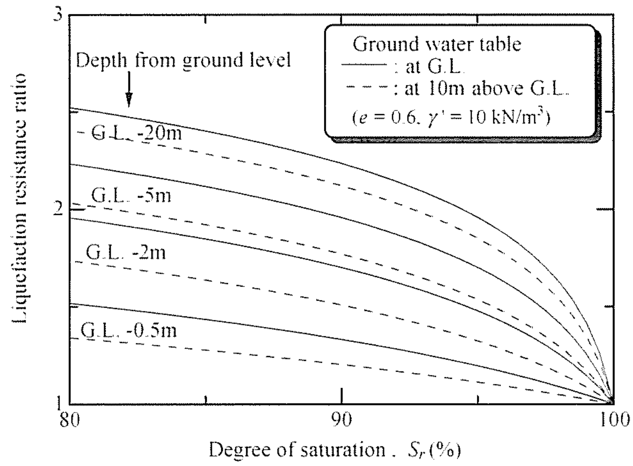

Fig. 8. Variations of liquefaction resistance ratio with S_r at several depths

essentially the same irrespective of the degree of saturation and the back pressure. This must be a reason why the soil in a small scale model for a shaking table test at 1 g environment can easily liquefy even if the model ground contains considerable amount of air bubbles (Okamura and Teraoka, 2005).

The relationship indicated in Fig. 7 and Eq. (4) make it possible to evaluate the liquefaction resistance ratio for field conditions. Figure 8 depicts variations of the liquefaction resistance ratio with the above discussed three factors for a fully submerged uniform sand deposit with buoyant unit weight $\gamma' = 10\ \mathrm{kN/m^3}$, void ratio $e = 0.6$ and water table being coincided with the ground level (G.L.) or at 10 m above G.L. It can be seen that the effects of S_r and the effective confining pressure are more significant than that of the water table or the initial pore pressure.

CONCLUDING REMARKS

Resistance to liquefaction of partially saturated sand was investigated through a series of triaxial tests in this study. Three parameters obtained from theoretical consideration, that is the degree of saturation, the initial confining pressure and the initial fluid pressure, were selected as testing parameters in the series of undrained cyclic triaxial tests.

It is confirmed that the degree of saturation has significant effect on the liquefaction resistance of sand. The liquefaction resistance also depends on the initial confining pressure and the initial pore pressure. The effect of the existence of air on the liquefaction resistance is more significant for soil under the higher confining pressure and the lower initial pore pressure. This fact implies that the effect of the degree of saturation of soil is not that significant for small scale models at 1 g but for larger models and centrifuge models. In field conditions, provided that liquefiable foundation soil is desaturated in someway with an intention to enhance liquefaction resistance, a significant effect can be expected except for soils at a shallower depth.

It was found that there is a unique relationship between the normalized liquefaction resistance and the potential volumetric strain. The liquefaction resistance of partially saturated sand can be reasonably estimated from that of fully saturated sand in conjunction with the potential volumetric strain using the relationship obtained in this study. This relationship may be limited for soils with a low matric suction. Further investigation is needed for the liquefaction resistance of partially saturated such as non-plastic silt and sand containing considerable amount of fines.

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