Definition of liquefaction

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ABSTRACT

Different definitions are used for the onset of liquefaction in engineering practice. For example, NCEER, JRA, and AIJ use γ=3 %, DA = 5 %, and γ= 5 %, respectively. The state where the excess porewater pressure becomes equal to the initial confining stress is used to identify the liquefaction in the field during earthquake and laboratory tests such as a shake table test. In addition, the number of cycles causing liquefaction, N_c is 15 in NCEER and AIJ whereas it is 20 in JRA. In total, 44 liquefaction strength tests of soils sampled from the natural deposits are used to evaluate the liquefaction strength under different definitions, and the relationships between various definitions are evaluated and discussed. The liquefaction strength at N_c = 15 is about 4 to 5 % larger than that at N_c = 20. The soil with large fines frequently does not become a zero confining stress state. The liquefaction strength of NCEER and JRA is nearly the same, and that of AIJ is about 7 % larger than them.

Keywords: liquefaction, design specification, excess porewater pressure, shear strain

1 INTRODUCTION

Since soil liquefaction is an important phenomenon because it has caused serious damage to structures, it is important to define it clearly in engineering practice. The definition of liquefaction has two meanings. One is the mechanism (e.g., Yoshida, 2023) and the other is the quantitative definition of the onset of liquefaction. The latter is very important in the engineering practice, because different definitions are used depending on the design specifications and in practice as shown in the next chapter.

This paper reviews the history of the definition of liquefaction and makes the differences clear based on the liquefaction strength tests.

2 BRIEF HISTORY OF STATE OF ARTS

Although the liquefaction strength curve was shown in the research by the research group represented by Seed, the liquefaction is not clearly defined. It is possibly because liquefaction is considered to be a state where the effective confining stress becomes zero under which the soil shows liquid-like behavior. The early design specifications in Japan, such as Earthquake resistant design of highway bridges and commentary (JRA, 1972) and Recommendations for design of building foundations (AIJ, 1974) used the critical *N*-value method. Thus, the definition of liquefaction was not necessary.

One of the bases of the North American method is Seed et al. (1982, 1983). However, the definition of liquefaction is not clearly shown. For example, they showed a liquefaction strength curve by referring to Ishihara and Koga (1981), in which many liquefaction strength curves with different definitions, i.e., initial liquefaction, double amplitude axial strain DA = 5 %, and DA = 10 % are shown, but the reason why they choose only one liquefaction strength curve is not shown.

The first quantitative definition in the design specification is probably Tokimatsu & Yoshimi (1983), which is the basis of Recommendations for the design of building foundations (hereinafter AIJ) at present. They summarized the test results using the frozen sample, a highly undisturbed sample, and showed an approximate equation for the liquefaction strength $\sigma_d/(2\sigma_o')$ as

$$\frac{\sigma_d}{2\sigma_o'} = a\left\{\frac{D_r}{100} + \left(\frac{D_r}{C}\right)^n\right\} \tag{1}$$

where σ_0' denotes initial effective confining stress, D_r denotes the relative density, and a = 0.45 and n = 14 are constants. The value of C is defined as

$$C_a = 97 - 19\log DA \quad \text{(Triaxial test. DA: \%)} \tag{2}$$

$$C_s = 94 - 19\log\gamma \quad \text{(Simple shear test. } \gamma\text{: \%)} \tag{3}$$

AIJ uses Eq.(3) with γ= 5 % from the 1988 edition (AIJ, 1988).

Idriss & Boulanger (2008) refer to Alba & Seed (1976) (Excess pore water pressure ratio, r_u =100 %), Yoshimi, et al. (1984) (DA = 5%), and Vaid & Sivathayalan (1996) (γ= 3%), but they did not show one definition. The numeral definition is not shown in

NCEER Workshop (NCEER, 1997) (hereinafter NCEER), but the relationship between the liquefaction strength and the equivalent N-value $(N_1)_{60}$, is shown. The design curve is differentiated by fines contents F_c. On the other hand, Seed et al. (1985) showed a similar design curve for clean sand under different shear strains. These two curves are overlayed in Fig. 1. The curve for $F_c \leq 5$ % and $\gamma = 3\%$ agrees nearly perfectly. Thus, we can consider that NECCR uses $\gamma = 3\%$ as the definition of liquefaction.

The definition DA = 5 % is shown at first in the Specifications for Highway Bridges in 2015 (JRA, 2015) (hereinafter JRA). This definition was written in Matsuo (2004), the explanation of the background of the 1996 edition, but it is not written in the 1996 edition.

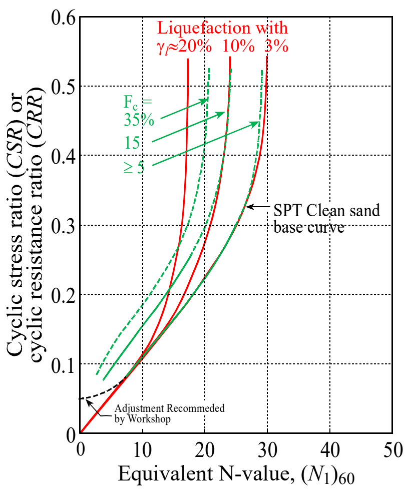


Fig. 1 Overlaying Seed et al. (1985) and NCEER Workshop (NCEER, 1997).

As shown in the preceding, the quantitative definition was not made in the early stage of research, possibly because the research is mainly focused on loose sand in which case the effective stress becomes zero under the cyclic loading. However, many types of soils began to be investigated as the progress of the liquefaction research and there appeared cases where the effective stress does not become zero although the shear strain becomes large. It is also considered liquefaction based on the definition of soil liquefaction by NCEER (1997). Other design specifications also have similar ideas. This is the reason why the definition began to be made by shear strain.

The definitions of liquefaction are different depending on the design specifications. NCEER (NCEER, 1997) uses $\gamma = 3\%$, JRA (2015) uses $DA = 5\%$, and AIJ (1988) uses $\gamma = 5$ %. Many liquefaction strength tests in Japan are conducted following the standard method by the Japanese Geotechnical Society (JGS Committee on laboratory test standard , 2020) (hereinafter JGS standard), in which 5 criteria are suggested as shown in Fig. 2, i.e., $DA = 1$, 2, 5, and 10 %, and $r_u = 0.95$. The definition of $DA = 5$ % is usually used in Japanese practice.

A different definition is also used in the shake table test and centrifugal test because measurement of the strain is nearly impossible. The onset of liquefaction in the field during the earthquake is also the same situation. The zero effective stress or sand boil is used to identify the liquefaction in the former case and only the sand boil is used in the latter case. This may be the same definition of $r_u = 0.95$ in the JGS standard.

In summary, four definitions are used in practice, $\gamma =$ 3 and 5 %, DA = 5%, and r_u = 0.95 at present. It is also noted on the number of cycles causing liquefaction N_c. NCEER and AIJ use $N_c = 15$, whereas JRA and JGS use $N_c = 20$. If the different definitions are used, the engineer cannot talk with the same background. Thus, we need to know the relationships between them.

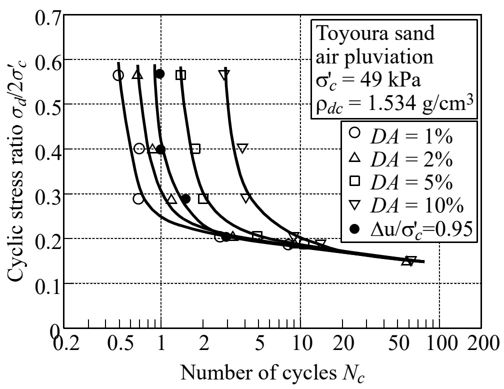


Fig. 2 Example of liquefaction strength test (JGS, 2020).

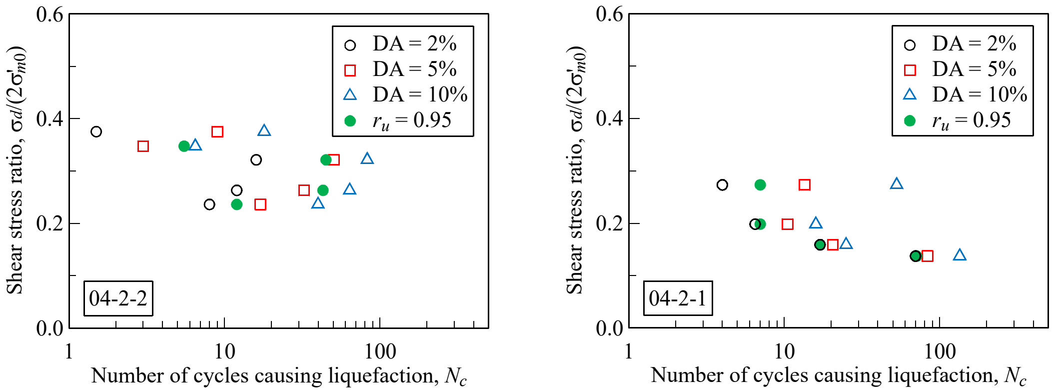
3 LIQUEFACTION STRENGTH TEST

Totally 52 liquefaction strength tests are used. Soils were sampled in the Kanto area in Japan, Tokyo, Saitama, Chiba, and Ibaraki Prefectures, by tube sampling method (Geology and Geotechnical Research Group , 2016). A triaxial test apparatus is used in the liquefaction strength test in which the initial confining stress is set 2/3 times of σ'_v in order to make the initial effective stress the same as in-situ effective stress (K_0 = 0.5 is assumed). The loading is continued up to 500 cycles if liquefaction does not occur.

Since at least 4 specimens are used in one liquefaction strength test, a total of 228 specimens are tested. It is noted that soil samples are not identical even in the same tube. Thus, the liquefaction strength curve is usually drawn by hand looking at test data. However, this process may cause additional human error. Since the number of cycles N_c = 15 and 20 are interested here, data points that sandwich these N_c are important. We omit particular test data or data points that seem extraordinary.

For example, data points scatter significantly in Fig. 3(a). Thus, we do not use this test result. On the other hand, data points also scatter in Fig. 3(b), but data points look natural if we omit data points for $\sigma_d/(2\sigma_0) = 0.198$. Thus, we use this test result. Finally, we selected 44 test data that are used in the following.

We need the liquefaction strength curves



(a) Unused test data because of large scatter. (b) Used data although data scatter.

Fig. 3 Unused test data because of large scatter.

corresponding to $\gamma = 3$ %, $\gamma = 5$ %, DA = 5%, and r_u = 0.95. Among them, the liquefaction strength curves for DA = 5% and r_u = 0.95 are in the original data.

The relationship between the shear strain γ and the axial strain amplitude ε_d is calculated under the undrained test (no volume change is assumed) by

$$\gamma = \varepsilon_a - \varepsilon_r \approx 1.5\varepsilon_a \tag{4}$$

where ε_a and ε_r are axial and lateral strains in the triaxial test. Considering that DA is a double amplitude, $\gamma = 3$ and 5 % become DA = 4 and 6.667 %, respectively. The liquefaction strength curves for DA = 4 and 6.667 % are linearly interpolated. Then, the shear stress ratios at N_c = 15 and 20 are calculated by linear interpolation. The semi-log axis such as Fig. 3 is used in the interpolation.

4. COMPARISON OF VARIOUIS LIQUEFACTION STRENTHS

Fig. 4 shows the liquefaction strengths for all criteria under N_c = 15 and 20. Soils are classified into three categories depending on fines content F_c based on JRA. The fines content is not shown in the two tests which are shown at the right of the figure.

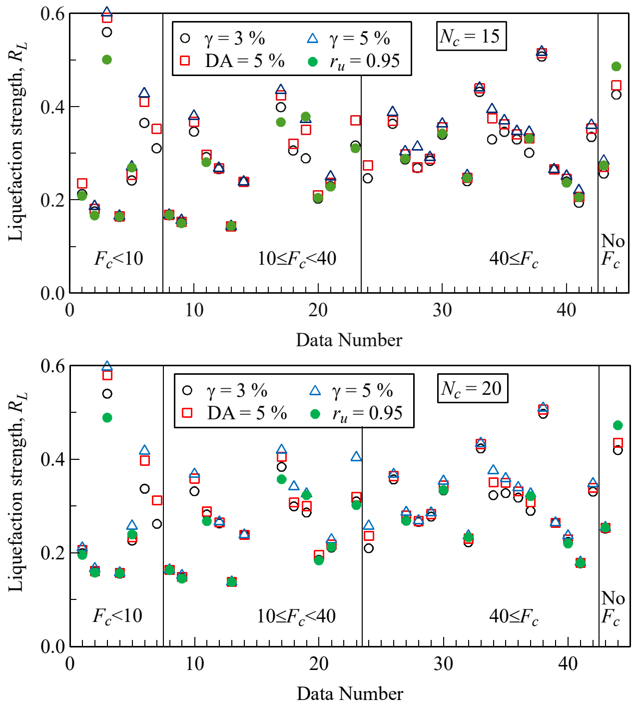
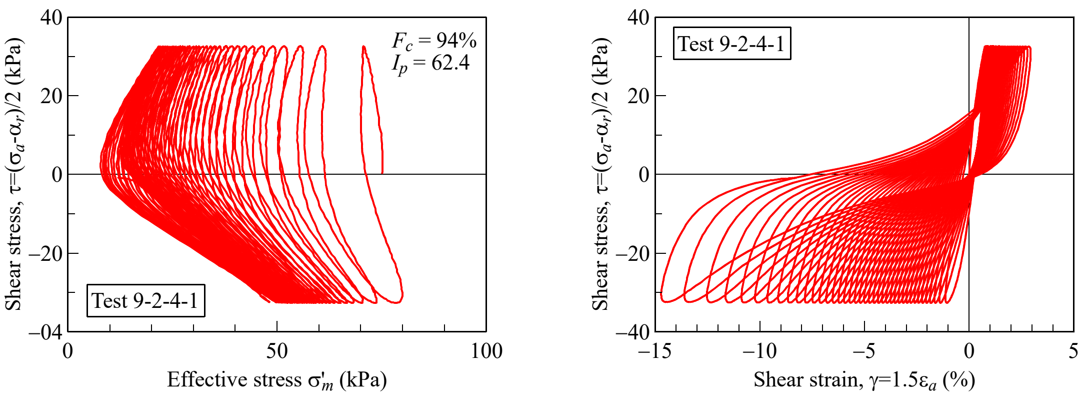


Fig. 4 Liquefaction strengths for N_c = 15 and 20.



(a) Stress path (b) stress-strain curve

Fig. 5 A case that effective stress does not come with zero.

The green solid circle ($r_u = 0.95$) sometimes does not exist especially when the fines content is large. This means that liquefaction did not occur. An example is shown in Fig. 5, where I_p denotes the plasticity index. The double amplitude shear strain reaches more than 15 %. In this case liquefaction does not occur if zero effective stress is used as the definition of liquefaction.

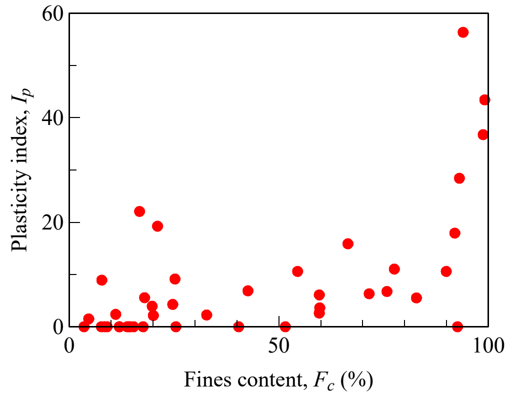


Fig. 6 Relationships between I_p and F_c.

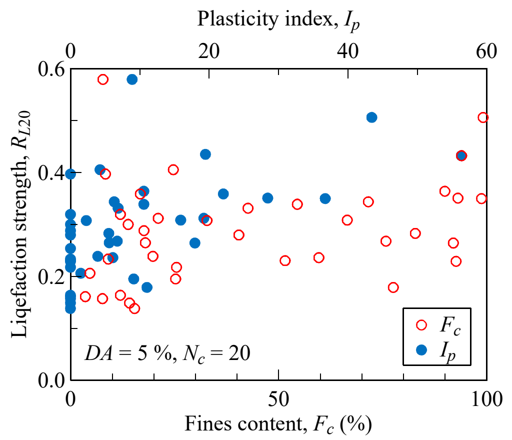


Fig. 7 Liquefaction strength depending on F_c and I_p

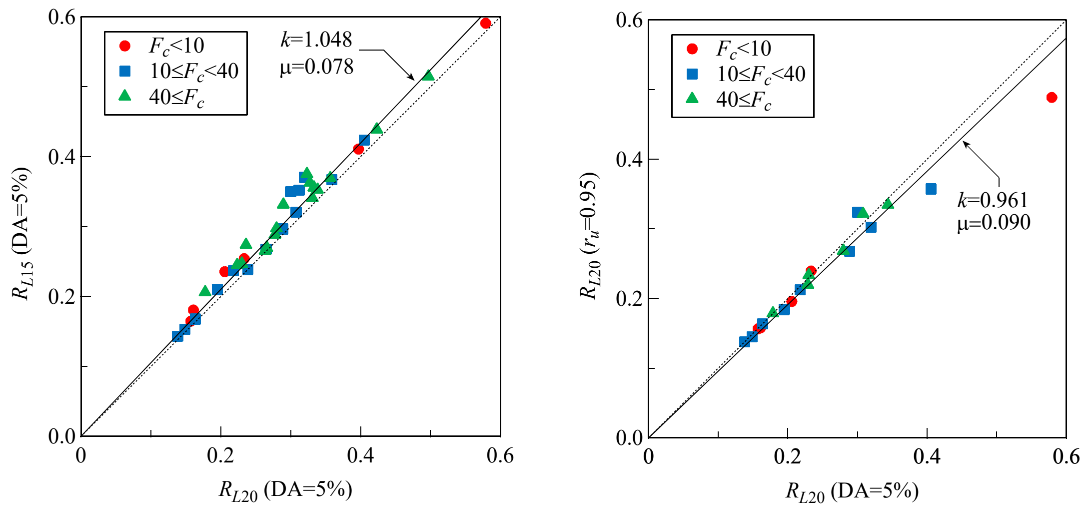
This is one of the reasons why the shear strain is used to define the liquefaction. This behavior is sometimes discussed as a clayey component. Then, Fig. 6 compares I_p and F_c. The I_p increases as F_c in general, but data scatter.

The design liquefaction strengths increase as F_c increases in many design specifications. Fig. 7 shows the relationships between shear strengths R_{L20} (liquefaction strength under 20 cycles of loading) and F_c. Since the SPT N-value is not considered in the figure, it is difficult to discuss the quantitative nature. However, it is noted that even soil with fines content near 100 % can liquefy. The disadvantage of using F_c in evaluating the increase of the liquefaction strength is silt, but it is known that there are two types of silt, plastic and nonplastic silt. It cannot be identified from F_c. The plasticity index may be a better index. Thus, the relationships between R_L and I_p is also shown in Fig. 7. Many data point lie I_p smaller than 20, which indicates that I_p is a better index than F_c to consider the plastic nature of sand.

Fig. 8 shows examples of differences between definitions, where k denotes average and μ denotes standard deviation.

Fig. 8(a) compares the number of cycles N_c = 15 and 20. The gradient k = 1.048 means that the liquefaction strength at N_c = 15 is 4.8 % larger on average than that at N_c = 20. The fines content does not seem to affect the relationships.

Fig. 8(b) compares different definition, r_u = 0.95 and DA = 5%. As explained in the preceding, the number of data points is smaller than that of, for example, Fig. 8(a), because some specimens do not liquefy under 500 cycles of loading for r_u = 0.95. In the case that F_c < 10%, both liquefaction strengths are almost the same. Fig. 9 shows an example of a case with small fines content. When $F_c \geq 10\%$, the data scatter a little but data points lie close to the 1:1 line (dotted line). The average gradient k = 0.961 seems to be affected by two data at large R_L values. Since



(a) N_c = 15 vs 20 (b) r_u = 0.95 vs DA = 5 %

Fig. 8 Comparison between different definitions

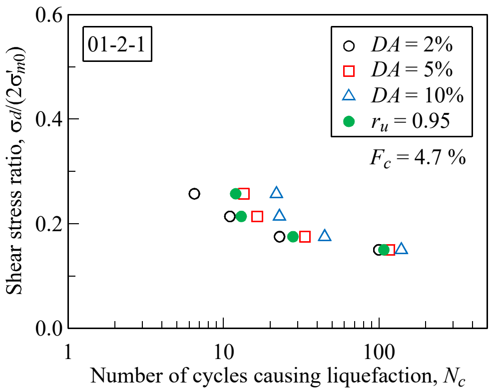


Fig. 9 Liquefaction strength of test 01-2-1 (F_c = 4.7 %)

the data point is too short, we cannot discuss the effect with large liquefaction strength.

Table 1 summarizes all the cases under the same number of cycles causing liquefaction, N_c = 15 and 20. It is noted that the data without liquefaction, which case frequently occurs under the r_u = 0.95 criterion, is not considered here. In other words, $R_{L,ru=0.95}/R_{L,\ DA=5}$ = 0.953, but it may be larger because there are nonliquefied data with $R_{L,ru=0.95}$.

Table 2 shows the effect of N_c. The liquefaction strength under N_c = 15 is larger than that under N_c = 20 for several percent.

Fig. 10 compares design specifications with average gradient k and standard deviation μ. The average gradient k is 0.993 between NCEER (N_c = 15, γ = 3%) and JRA (N_c = 20, DA = 5%), which means that both design specifications give almost the same liquefaction strength. The R_L increases as N_c decreases. On the other hand, R_L decreases as the strain at liquefaction decreases. Both effects cancel out resulting in the same R_L.

On the other hand, R_L by AIJ is 7.1 % larger than that by JRA. Both the decrease of N_c and increase of strain at liquefaction work to increase R_L.

Table 1 Summary of statistical analysis

	N_c = 15		N_c = 20	
	k	μ	k	μ
$R_{L,\gamma=5}/R_{L,\gamma=3}$	1.078	0.0987	1.083	0.114
$R_{L,ru=0.95}/R_{L,\gamma=3}$	1.013	0.122	1.001	0.080
$R_{L,\gamma=3}/R_{L,DA=5}$	0.946	0.082	0.955	0.069
$R_{L,\gamma=5}/R_{L,DA=5}$	1.030	0.046	1.039	0.086
$R_{L,ru=0.95}/R_{L,\ DA=5}$	0.953	0.104	0.961	0.090
$R_{L,ru=0.95}/R_{L,\ \gamma=5}$	0.918	0.081	1.426	0.484

Table 2 Effect of N_c

R_{L15}/R_{L20}	k	μ
γ=3 %	1.038	0.060
DA=5 %	1.048	0.078
γ=5 %	1.042	0.060
r_u=0.95	1.055	0.049

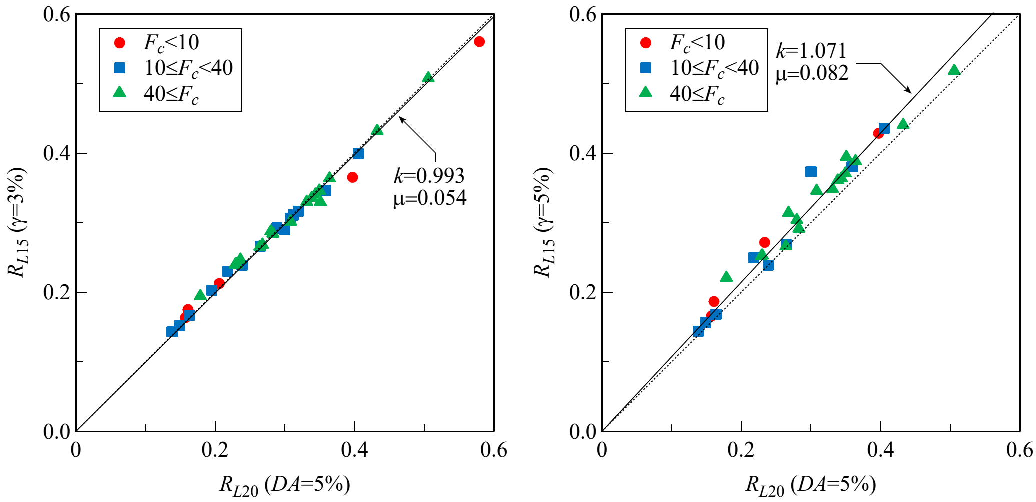
CONCLUDING REMARKS

The criteria to define the liquefaction strength are different depending on the design specifications. After the state of arts on the definition of liquefaction is briefly reviewed, the relationships between them are investigated from about 50 liquefaction strength tests of naturally deposited soils in the Kanto area of Japan.

The liquefaction strength at N_c = 15 is about 4 to 5 % larger than that at N_c = 20. The soil with large fines frequently does not become a zero confining stress state. The plasticity index I_p may be a better index than F_c to consider the increase of the liquefaction strength due to plastic characteristics because I_p can consider the plastic behavior of soil.

The liquefaction strength of NCEER is nearly the same as that of JRA, but that of AIJ is about 7 % larger than JRA on average.

It is noted that the liquefaction strengths in this research use soils with tube sampling. However, recent design specifications are based on the liquefaction strength test by using highly undisturbed samples such



(a) NCEER vs JRA (b) AIJ vs JRA

Fig. 10 Comparison between design specifications as frozen samples in Japan. Therefore, conclusions here may be modified considering the effect of disturbance.

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