Liquefaction Strength Of Sand Materials Containing Fines Compared With Cone Resistance In Triaxial Specimens

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ABSTRACT :

Innovative miniature cone penetration tests and subsequent cyclic triaxial tests were carried out on sand specimens containing fines both in the undrained conditions. It has been found that one unique curve relating cone resistance and liquefaction strength can be established, irrespective of relative density and fines content. This indicates that liquefaction strength corresponding to a given cone resistance is constant despite the difference in fines content, which differs from the current liquefaction evaluation practice. The results from the tests on the sustained consolidation specimens plotted a little shifted above the unique curve obtained from the tests on the normal consolidation specimens. This may indicate that, if in situ consolidation effect is considered, liquefaction strength for the same cone resistance tends to increase with increasing fines content.

KEYWORDS: Liquefaction, Sand, Fines, Cone resistance, Triaxial test

1. INTRODUCTION

Liquefaction of gravelly soil and sand containing fines has increasingly been witnessed during recent earthquake. During the 1995 Hyogo-ken Nambu earthquake in Japan, reclaimed ground in Kobe filled with decomposed granite soil, called Masado containing large quantity of gravel or fines fraction. Besides these cases, liquefaction of sands or gravels containing fines was also reported during sevral earthquakes, such as the 1987 Chiba-ken Toho-oki earthquake (Numata and Mori, 2002), the 1993 Hokkaido Nansei-oki earthquake (Kokusho et al., 1995), the 2001 Tottori-ken Seibu earthquake (Yamamoto et al., 2001) etc.

Liquefaction strength is evaluated using penetration resistance of Standard Penetration Tests (SPT) or cone penetration tests (CPT) in engineering practice. If sand contains a measurable amount of fines, liquefaction strength is normally raised in accordance to fines content in most of liquefaction evaluation methods, such as the road bridge design code using SPT N-values. In the case of CPT, Suzuki et al. (1995) carried out in situ penetration tests and soil samplings by in situ freezing technique from the same soil deposits and compared the tip resistance q_t-value and liquefaction strength from laboratory test. In addition, some research that a liquefaction strength and relevance of the cone tip resistance are reported by large container test and a laboratory test result (e.g. Tanigawa et al., 1988; Adachi et al., 1999; Morishige et al., 2007). The comparison showed that the higher fines content tends to increase the liquefaction strength for the same penetration resistance.

In contrast to their finding, however, quite a few laboratory tests show that liquefaction strength clearly decreases with increasing content of low plasticity fines having the same relative density (e.g. Ishihara et al., 1989; Sato et al., 1997; Hara et al., 2005). Thus, a wide gap seems to remain between the current practice for liquefaction potential evaluation and the laboratory experiment on the modification of the liquefaction strength in sands containing fines despite its importance in engineering design.

In order to establish more direct correlations between cone resistance and liquefaction strength considering the effect of fines, a systematic experimental study was undertaken (Kokusho et al., 2003), in which a miniature cone penetration test and subsequent cyclic loading test were carried out on the same triaxial test specimen. An innovative simple mechanism was introduced in a normal cyclic triaxial apparatus enabling a miniature cone to penetrate the sand specimen at a constant speed. Results from the serial tests on the same specimen are compared to develop direct correlations between penetration resistance and liquefaction strength for sands containing various amount of fines.

2. TEST APPARUTUS AND TEST PROCEDURES

In a triaxial apparatus used in this research, the specimen size is 100 mm diameter and 200 mm height. In liquefaction tests, the soil specimen can be loaded cyclically by a pneumatic actuator from above as a stress-controlled test. In order to carry out a cone penetration test in the same specimen prior to undrained cyclic loading, a metal pedestal below the soil specimen was modified as shown in Photo.1 and Fig.1, so that a miniature cone can penetrate into the specimen from below. The pedestal consists of two parts, a fixed circular base to which the cone rod is fixed and a movable metal cap, through the center of which the cone rod penetrates in the upward direction into the overlying specimen. The annulus between the two parts is sealed by O-rings, enabling the cap to slide up and down by water pressure supplied into a water reservoir in between the two parts. During the test, the pedestal cap is initially set up at the highest level by supplying water inside, and specimen is constructed on it. In this stage, the cone already projects into soil specimen by 47 mm. By opening a valve at the start of the cone penetration, the water in the reservoir is drained by the chamber pressure, resulting in settlement of a total body of the specimen together with the disc and piston at the top and realizing the relative upward penetration of the cone by 25 mm. The penetration rate can be set up within the limits of 2 cm/sec from 0.03 cm/sec.

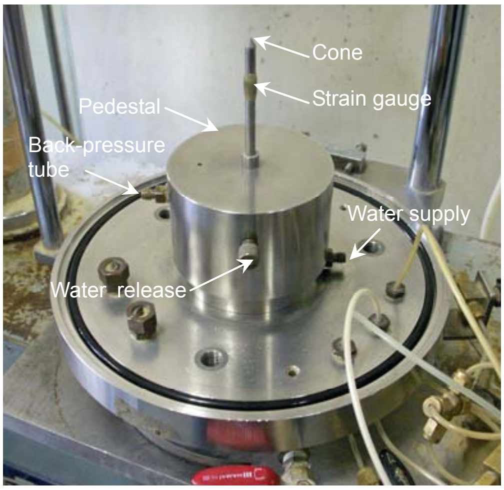

Photo 1 Modified pedestal

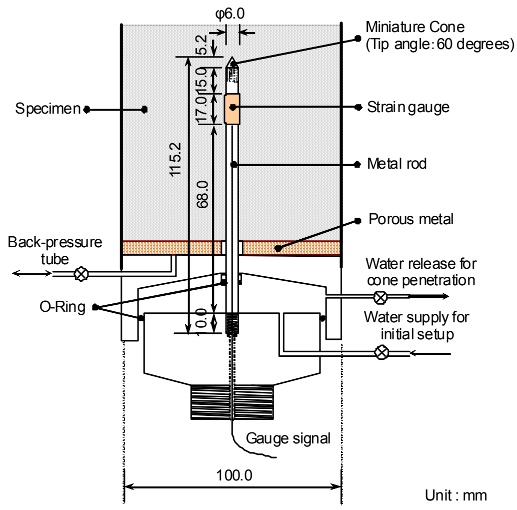

Figure 1 Cress-section of the modified pedestal

The miniature cone is 115.2 mm height, 6 mm diameter and 60 degrees tip angle, about 1/35 smaller in the cross-sectional area than the 10 cm^2 standard cone (JGS, 2006) normally used in engineering practice. The strain gauges are glued at the top part of the rod, 20.2 mm lower than the foot of the cone, meaning that the measured vertical load includes not only the tip resistance but also the skin friction at the top portion of the rod. Though the separation of the tip load is actually needed, the total load is considered as the cone resistance (denoted here by q_c) in the present study because the contribution of the skin friction seems to be considerably smaller than the tip load.

In the test sequence, the penetration test was first carried out after consolidating the specimen. Then, after releasing pore pressure and reconsolidating it under the same confining pressure, the same specimen was cyclically loaded in undrained condition. The cone penetration was basically carried out under the undained condition in the triaxial specimen, though a few drained penetration tests was also conducted to be compared as will be explained later. In undrained conditions, the pore pressure of the specimen was also measured by the electric piezometer in the same way as in normal undrained triaxial tests.

The soil specimens were prepared by wet tamping, that was fully saturated by using CO2-gas and de-aired water and isotropically consolidated by the effective stress of 98 kPa with the back-pressure of 294 kPa. The cell pressure and the pore-water pressure were measured by the electric piezometers and the axial deformation was measured with LVDT outside the pressure chamber. In the undrained cyclic loading tests, the axial stress was cyclically controlled by sinusoidal waves with the frequency of 0.1Hz. The cyclic loading was continued until the double amplitude axial strain (DA) about 10%.

3. SOIL MATERIALS AND MINIMUM & MAXIMUM DENSITY TEST

The soil materials tested are two types; Toyoura sand and River sands abbreviated as TS and RS respectively, hereafter. The former is reconstituted from river sands originated from a Tone river in Japan. A soil grain size distribution curves are plotted in Fig.2 and soil physical properties are listed in Table.1. The same river sand consisting of sub-round particles of hard quality as used in another research by the same authors was used in this test (Hara et al., 2004). Fines mixed with the sand is silty and clayey soils with low plasticity index of Ip ≃6 sieved from decomposed granite in reclaimed ground of the Kobe city, Japan. Soil specimens were prepared by wet tamping method to meet prescribed relative densities as close as possible.

In the test, relative density D_r and fines content F_c of the specimens were parametrically changed to investigate their effects on penetration resistance and undrained cyclic strength. The minimum and maximum densities, ρ_{dmin} and ρ_{dmax}, of the sands containing fines necessary for evaluating relative densities were determined by a method standardized by the Japanese Geotechnical Society (2006). In Fig.3, the variations of ρ_{dmin} and ρ_{dmax}, are plotted versus the fines content. Note that the densities both increase and then decrease with increasing F_c. A threshold fines content may be estimated as around F_c =20%, at which the voids of sands are completely filled with fines and the soil structure changes from a sand skeleton supporting type to a fines matrix supporting type. The density change in Fig.3 is likely to reflect such a change of soil structure with increasing fines content F_c.

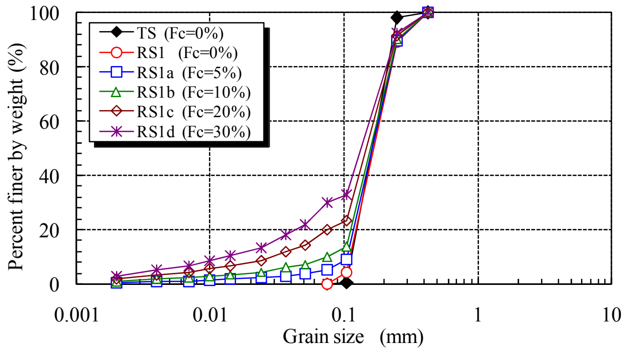

Figure 2 Grain size distribution curves

Table 1 Physical property of TS and RS soils

	F_c (%)	U_c	D_{50} (mm)	ρ_s (g/cm^3)	ρ_{dmin} (g/cm^3)	ρ_{dmax} (g/cm^3)	e_{max}	e_{min}
TS	0	1.64	0.167	2.640	1.335	1.645	0.978	0.605
RS1	0	1.44	0.169	2.696	1.280	1.629	1.106	0.655
RS1a	5	1.68	0.165	2.699	1.310	1.666	1.060	0.620
RS1b	10	2.36	0.158	2.701	1.314	1.746	1.056	0.547
RS1c	20	5.83	0.151	2.706	1.280	1.842	1.114	0.469
RS1d	30	12.2	0.138	2.711	1.173	1.704	1.311	0.591

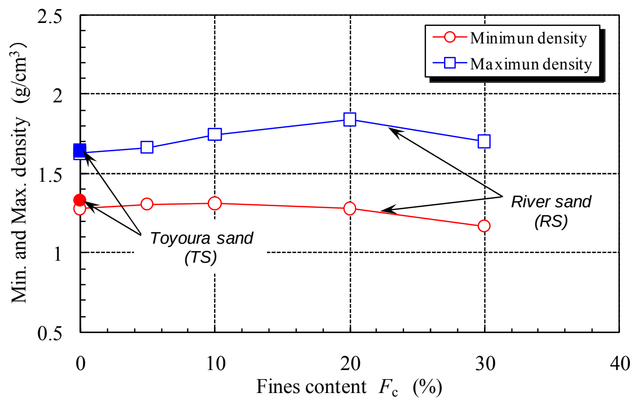

Figure 3 Minimum and Maximum density tests result

4. PENETRATION TESTS

4.1. Toyoura Sand with Different Densities

In Fig.4, relationships between penetration lengths versus elapsed time obtained during preliminary tests by the TS specimens of different densities are shown. The penetration rate is about 1 mm per second, much slower than prototype CPT, and is almost constant despite the difference in relative density of the TS specimens with D_r ≈20 to 70%. Figs.5 (a) and (b) show the cone resistance q_c and the excess pore-water pressure Δu versus the penetration lengths relationship obtained by the TS specimens for D_r about 30%. The penetration rate, v set up to three steps, v = 0.03 cm/sec, 0.06 cm/sec, or 1cm/sec,

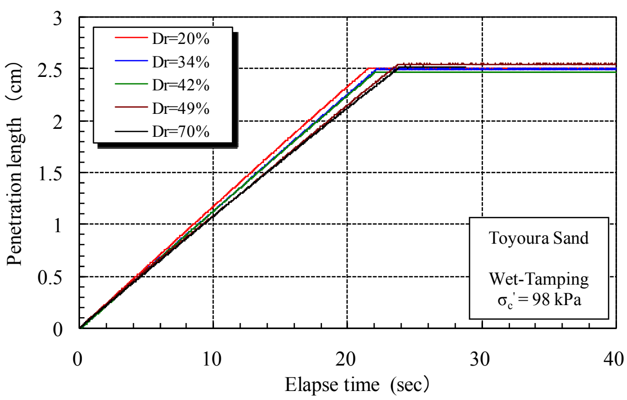

Figure 4 Penetration lengths versus elapse time relationships

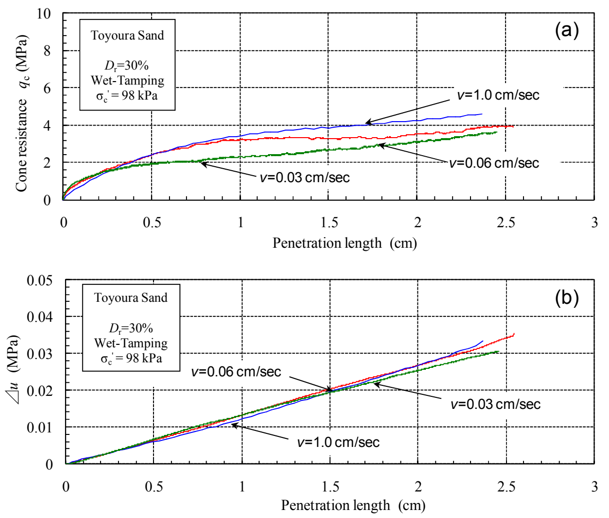

Figure 5 Cone resistance (a) or excess pore-water pressure (b) versus penetration length (Toyoura sand)

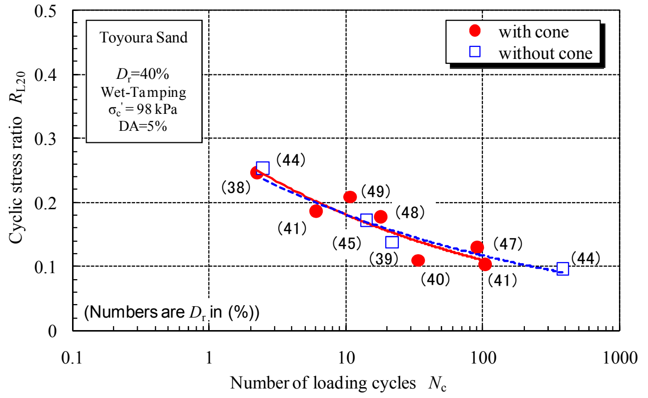

Figure 6 Comparison of liquefaction strength curve between specimens with or without cone rod

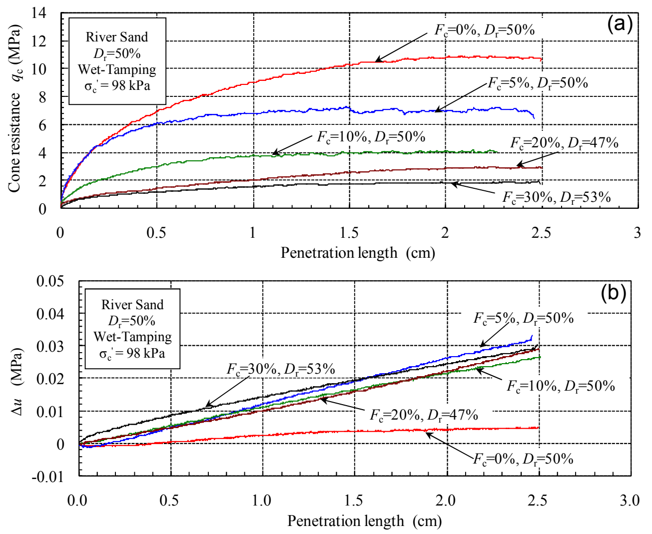

Figure 7 Cone resistance (a) or excess pore-water pressure (b) versus penetration length (River sand)

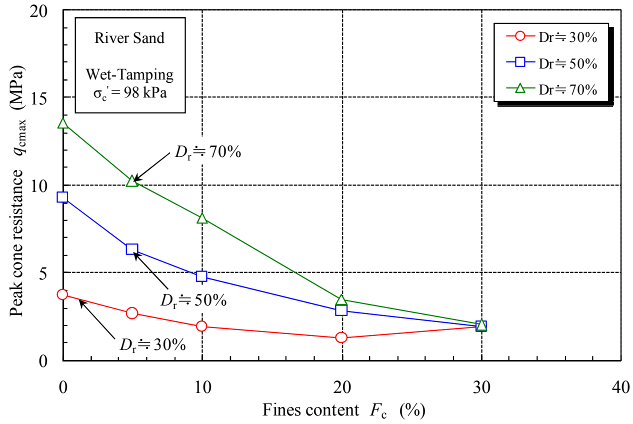

Figure 8 Peak cone resistance versus fines content

and v = 1 cm/sec case supports the prototype CPT tests.

The cone resistance or the excess pore-water pressure is almost same value despite the great different in the penetration rate.

It may well be suspected that, in such a test sequence, the liquefaction strength is possibly influenced by the preceding cone test and subsequent reconsolidation. In order to examine the influence of the test sequences, preliminary comparative tests were conducted in which results by normal liquefaction tests in the same triaxial apparatus without the cone rod were compared with those in this test sequence. Fig.6 shows the comparison of the results with or without the cone, indicating no clear difference between the two corresponding best-fit curves (the solid curve with cone and the dashed curve without cone) correlating stress ratios R_L and number of loading cycles N_c reaching 5% double amplitude strain.

4.2. River Sand with Different Fines Content

In Figs.7 (a) and (b), variations in cone resistance q_c and excess pore-water pressure Δu are shown as a function of the penetration length for the RS specimens of $D_r \approx 50\%$ with different fines content of F_c =0, 5, 10, 20, and 30%, respectively. The penetration rate set up about 0.06 cm/sec. The increase in fines content tends to develop larger pore-water pressure increase and reduce penetration resistance during penetration. The effect of fines content is pronounced even at a small value of F_c =5% particularly on the pore-water pressure build-up and tends to asymptotically converges as it approaches to F_c =30%. The RS materials behavior due to cone penetration probably reflects the change of the soil structure due to

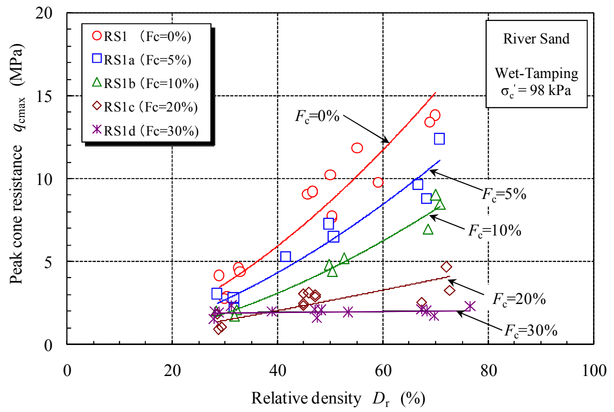

Figure 9 Peak cone resistance versus relative densities

increasing fines content as explained before on the F_c-dependent variations of the minimum and maximum densities. In Fig.8, the peak q_c-values evaluated in this manner are plotted versus corresponding fines content F_c for specimens of $D_r \approx 30$, 50, 70% with different fines content of $F_c = 0$, 5, 10, 20, and 30%, respectively. The peak cone resistance q_c decrease with increasing F_c for the same relative density of $D_r \approx 30$, 50 and 70%, and the change tendency of q_c by the specimens having a higher relative density are larger.

4.3. River Sand with Different Relative Density

The cone penetration resistance q_c to be correlated with relative density or other design parameters such as liquefaction strength should be represented by the value attained after the cone has penetrated a sufficient length at a constant rate. In Fig.9, the peak q_c-values evaluated in this manner are plotted versus corresponding relative densities D_r of the RS specimens. For the clean sand without fines, the plots concentrate along a solid curve on the chart, while the peak q_c-values of fines-containing sand are obviously lower and decrease with increasing fines content for the same relative density of $D_r \approx 30$, 50 and 70%, indicating the dominant effect of fines on cone resistance.

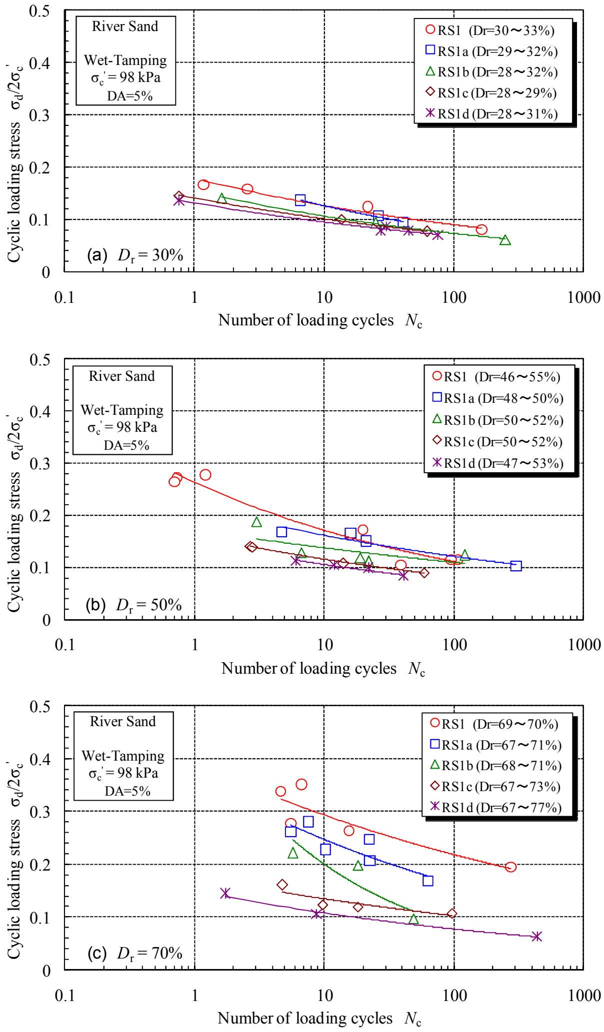

Figure 10 Liquefaction strength curve for River soil

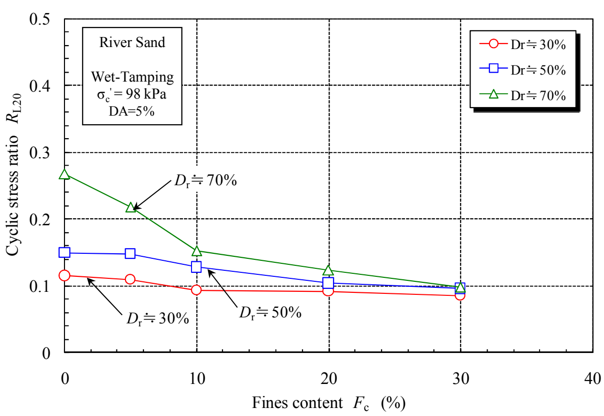

Figure 11 Cyclic stress ratios versus fines content

5. UNDRAINED CYCLIC TRIAXIAL TESTS

Figs.10 (a)～(c) show relationships between the stress ratio R_L and a number of loading cycles N_c for 5% double amplitude axial strain obtained by the series of undrained cyclic loading tests on the RS specimens having $D_r \approx 30\%$, 50%, 70% with fines content, $F_c = 0$, 5, 10, 20, and 30%, respectively. The result from undrained cyclic loading tests conducted on the same test specimens after the cone penetration tests. The stress ratio $\sigma_d/2\sigma_c'$ defined by the stress ratio for 5% double amplitude axial strain, increases with increasing D_r for the RS specimens without fines, while it increase a little for the RS specimens with fines. In order to clearly see the effect of fines on the strength, Fig.11 indicates a relationship between cyclic stress ratios R_{L20}, for 5% double amplitude axial strain in N_c =20, and fines content F_c for the same relative density of D_r ≈30%, 50%, 70% specimens. Small differences in D_r for individual plots on Fig.10 are adjusted based on the slopes of the regression curves to evaluate R_{L20}. The stress ratio R_L small decreases with increasing F_c for D_r ≈30%, 50% specimens, while it large decreases for the D_r ≈70% specimens.

6. PENETRATION RESISTANCE CERSUS LIQUEFACTION STRENGTH

In order to correlate penetration resistances to corresponding liquefaction strengths for the soil specimens with different F_c, the stress ratio R_{L20}, for 5% double amplitude axial strain in N_c =20 are estimated from the line regressed through the filled circles. In Fig.12, the liquefaction strengths R_{L20} thus obtained are plotted on the vertical axis versus cone resistances q_c on the horizontal axis. The data points concentrate in a narrow area which may be represented by a single straight line drawn in the chart (the regression coefficient =0.91). As maybe observed, a unique relationship between R_L and q_c exists despite the large differences in relative density or fines content. This finding seems quite contradictory to the present state of practice in which liquefaction strength corresponding to a given CPT resistance is increased by a certain amount according to increasing fines content. In Fig.13 the same test results are plotted again to compare with field investigation data obtained by Ishihara (1989), Shibata et al. (1988), and Suzuki et al. (1995) combining prototype cone tests in situ and undrained cyclic triaxial tests on intact samples. On the horizontal axis, q_{t1} is taken in place of q_c, to normalize in situ cone tip resistance q_t as, where the vertical overburden stress and q_{t1} = q_c is assumed for the present study. Needless to say, considerable differences exist between the miniature cone tests conducted here and prototype tests in terms of cone size, penetration rate, drainage condition, consolidation effects, etc. Nevertheless, the correlation by the present research is in surprisingly good agreement both qualitatively and quantitatively with in situ data for F_c < 1% in the interval of q_{t1} ≈0~140. It should be noted however that the cone resistance looks quite insensitive to fines content in the present study whereas the in situ data indicate its clear influence. In order to examine the contradiction between the two experimental studies more closely, the effects of long-time consolidation and drainage during cone penetration were further investigated in the present test series.

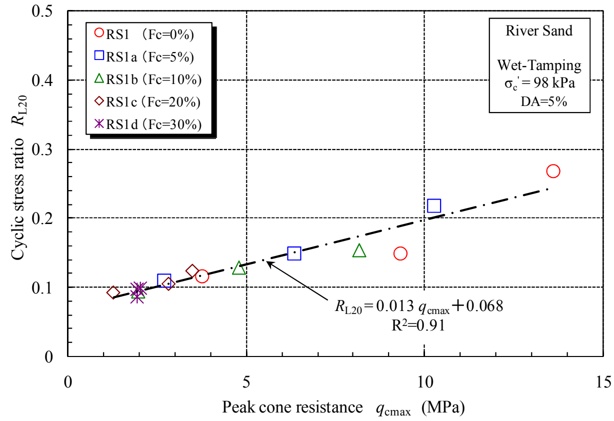

Figure 12 Direct relationship between stress ratio for liquefaction and peak cone resistances for River sand

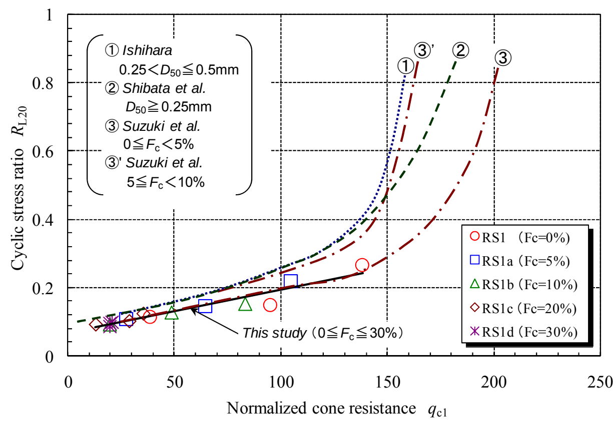

Figure 13 Direct relationship between stress ratio for liquefaction and normalized cone resistances for River sand

7. EFFECT OF SUSTAINED CONSOLIDATION

In the test series, the consolidation sustained about 1 hour before the cone test and also before the undrained cyclic loading test. In order to investigate the effect of longer consolidation time in the both tests, specimens of

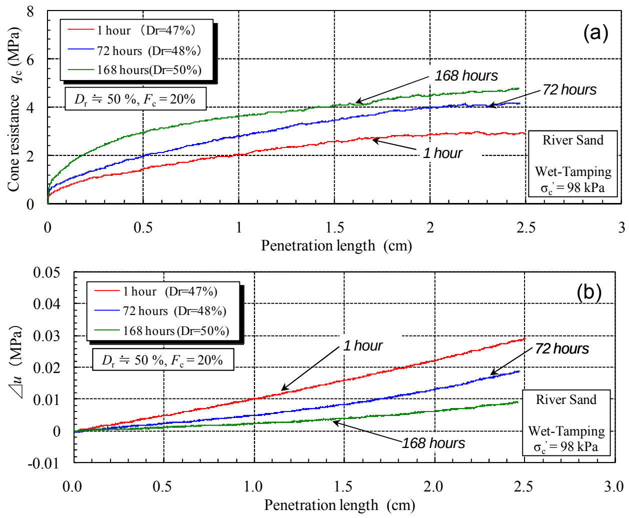

Figure 14 Cone resistance (a) or excess pore-water pressure (b) versus penetration length with different consolidation time

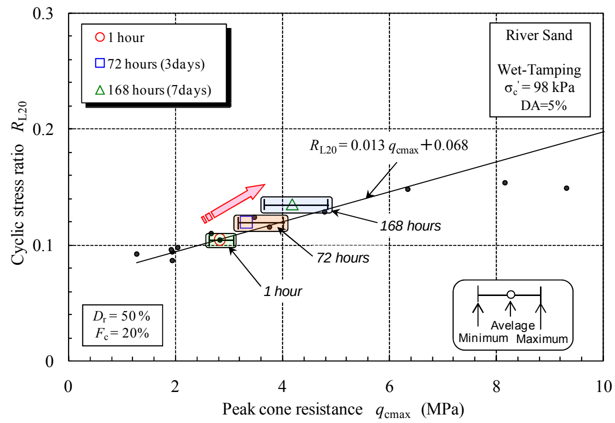

Figure 15 Direct relationship between stress ratio for liquefaction and peak cone resistances with different consolidation time

F_c =20% were chosen, because higher fines content is likely to accelerate the long time consolidation effect in comparison with clean sands. Specimens with the relative density around D_r =50% were consolidated for three different durations; 1 hour, 72 hours (3 days) or 168 hours (7 days).

Figs.14 (a) and (b) compare the typical results of cone resistance q_c and excess pore-water pressure Δu, respectively, plotted versus the penetration length. It is obvious that even several days of sustained consolidation makes measurable difference in the cone resistance. In Fig.15, data points thus obtained for the sustained consolidation are plotted for specimens of F_c =20% with filled symbols encircled as a group to compare with all the other data for 1 hour consolidation. It may well be judged that, despite some data dispersion, the sustained consolidation tends to shift the R_L versus q_c correlation, leading higher liquefaction strength for the same q_c-value. Consequently, even the correlation for very young test specimens looks to be insensitive to fines content and uniquely determined; it is likely that long time consolidation in situ makes the difference. However, much more systematic research is certainly needed to draw more general conclusions considering much longer period and different history of consolidation, different plasticity of fines, etc.

8. CONCLUSIONS

Cone resistance and liquefaction strength of sand materials have been studied by miniature cone penetration test and subsequent cyclic loading test to investigate the effect of fines content. Soil materials tested are two types; Toyoura sand and Rives sand. Major conclusions obtain in this study are;

(1) Cone penetration tests performed prior to cyclic undrained tests have little effect on liquefaction strength of specimens, demonstrating that direct and reliable comparison between penetration resistance and liquefaction strength is possible.

(2) For river sand with or without low plasticity fines, a good correlation between relative density and cone resistance has been found, which is basically consistent with previous research using a prototype cone. With increasing fines content, both cone resistance q_c and liquefaction strength R_L decrease despite the difference in the fines content and relative density.

(3) The results are plotted on the $R_L \sim q_c$ chart, they seem to be located slightly above the unique curve obtained by the normal consolidation tests.

(4) In order to examine long term consolidation effect on the $R_L \sim q_c$ relation, samples with certain fines content were consolidated for variable durations up to a week. The sustained consolidation of such a short time increases both cone resistance and liquefaction strength by measurable amounts. This may indicate that, if in situ consolidation effect is considered, liquefaction strength will increase with increasing fines content.

(5) The present research has been able to provide valuable insights into the liquefaction strength evaluation based on cone penetration. However, more test results on long term consolidation effect and drainage effect for different fines contents are certainly needed before reaching general quantitative conclusions on this important issue.

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