UNDRAINED STRENGTH OF GRAVELLY SOILS WITH DIFFERENT PARTICLE GRADATIONS

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SUMMARY

A series of undrained cyclic triaxial tests were performed in this study on sand-gravel materials with different particle gradation and relative density. Despite large differences in particle gradation, a good agreement in the liquefaction strength was obtained for specimens having the same relative density. The post-liquefaction undrained monotonic shear strength was much lager for well-graded soils than that for poorly-graded soil having the same relative density. This indicates that fatal structural failure with large post-liquefaction soil strain is difficult to develop in well-graded granular soils compared to poor-graded sands with the same relative density, although they are almost equally liquefiable.

INTRODUCTION

Liquefaction of gravelly soils has increasingly been witnessed during recent earthquake. During the 1995 Hyogoken Nambu earthquake in Japan, reclaimed ground in Kobe filled with decomposed granite soil, called Masado containing large quantity of gravel or fines fraction, liquefied extensively despite a widely accepted perception that gravelly soil was harder to liquefy than sand because of larger uniformity coefficient and larger dry density. Besides these cases, liquefaction of gravelly soils was also reported during several earthquakes, such as the 1983 Borah Peak earthquake in Idaho (Andrus [1]), the Hokkaido Nansei-oki earthquake (Kokusho [2]), etc. In the above mentioned Hokkaido Nansei-oki earthquake, rock debris avalanche gravel with 70 to 80% gravel content liquefied.

Gravelly soils in natural deposits are normally well-graded compared to poorly graded sands. Dry densities and the uniformity coefficient (Uc) are much higher than typical liquefiable loose sands. However, those liquefied gravelly soils sometimes exhibit quite low N-values and S-wave velocity (Kokusho [3]).

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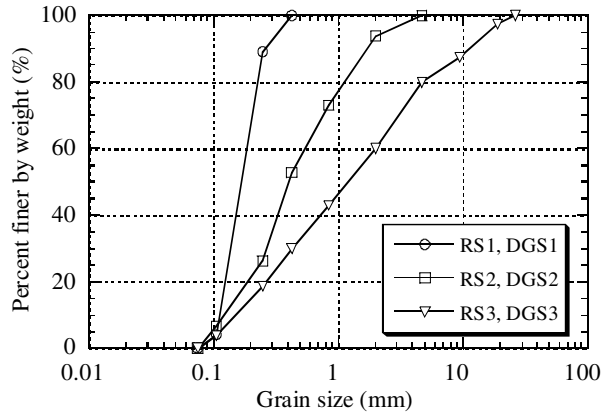[3] Nihon Hodou Incorporated., JAPAN.

Fig.1 Grain size distribution curves

Table.1 Physical properties of RS and DGS soils

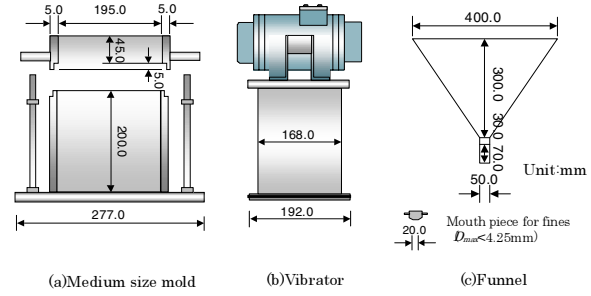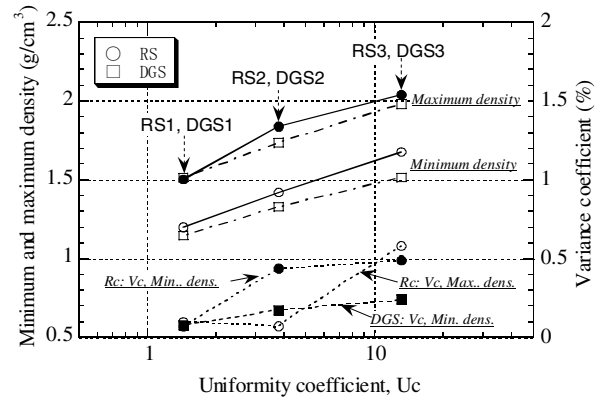| Soil name	D_{50} (mm)	Uc	ρ_s (g/cm³)	ρ_{dmax} (g/cm³)	ρ_{dmin} (g/cm³)	e_{max}	e_{min}
RS1	0.14	1.44	2.696	1.502	1.198	1.250	0.794
RS2	0.40	3.79	2.697	1.839	1.421	0.898	0.467
RS3	1.15	13.1	2.655	2.038	1.675	0.585	0.303
DGS1	0.14	1.44	2.649	1.514	1.146	1.312	0.750
DGS2	0.40	3.79	2.647	1.737	1.328	0.993	0.524
DGS3	1.15	13.1	2.640	1.979	1.515	0.743	0.334

Fig.2 Minimum and Maximum density test apparatus

Fig.3 Minimum and Maximum density tests result

Undrained cyclic strength of gravelly soils has not been investigated so much as sands with regard to their density, particle gradings, etc., though they have significance in liquefaction potential evaluation for seismic design. In earlier time, Wong [4] made large scale undrained cyclic triaxial tests of poor-graded gravelly soils of the same uniformity coefficient Uc = 1.3 and found that the cyclic stress to cause initial liquefaction in gravelly soils was somewhat larger than sand possibly due in part to the artificial effect of membrane compliance in the test specimens. Tanaka [5] made undrained cyclic triaxial tests for granular soils with different Uc and found that the stress ratio of gravelly soils corresponding to 5% double amplitude (DA) axial strain, which was not modified by the membrane compliance effect, was larger than sand. More recently, Evans [6] carried out undrained cyclic triaxial tests to quantify the effect of gravel content on the liquefaction resistance of sandy gravel composites. They made soil specimens by mixing poor-graded sand and poor-graded gravel in different ratios to make gap-graded specimens with different gravel contents. In order to reduce the membrane compliance effect in the triaxial tests, test specimens were sluiced with sand. They found that gravelly soils showed evidently larger liquefaction resistance than sand with the same relative density.

In this experimental research, granular river soils consisting of hard particle are reconstituted to have smooth grain size curves analogous to natural sandy or gravelly soil with uniformity coefficient Uc, varying from 1.44 to 13.1. Systematic undrained triaxial tests are performed by loading either cyclically or monotonically on the granular specimens with different relative density Dr and different Uc. In addition, the effect of particle crushability on the undrained strength is also investigated by comparing test results on decomposed granite soils of weathered soil particles possessing the same grain size with those on the river soils of hard particles.

SOIL MATERIALS AND MINIMUM & MAXIMUM DENSITY TEST

Soil materials tested are two types; river soils and decomposed granite soils abbreviated as RS and DGS, respectively, hereafter. The former is reconstituted from river sands or gravels originated from a river. The particles are roundish and hard to cursh. The latter is reconstituted from decomposed weathered granite originated from reclaimed ground in Higashinada in Kobe city, where extensive liquefaction took place during the 1995 Hyogoken Nambu earthquake. This soil was originated from the Rokko granite mountains at the back of the Kobe city and had been used for filling reclaimed land in the coastal areas. As the particles are angular and weathered, the DGS soils may result in higher crushability than the river soils. Soil grain-size distribution curves are plotted in Fig.1 and soil physical properties are listed in Table.1.

As will be discussed later, relative density, Dr, is a pertinent parameter to evaluate undrained cyclic strength of granular soils of different particle gradations and defined by the minimum and maximum dry density, ρ_{dmin}, and ρ_{dmax}. They were determined by a test mold utilizing a medium-sized metal soil mold (195mm inner diameter and 200mm depth), a vibrator disc and a large-sized metal funnel as indicated in Fig.2. For the minimum density ρ_{dmin}, soil was fed into the mold through a large-sized metal funnel elevated slowly about 0.15mm/sec with zero drop height. For the maximum density ρ_{dmax}, soil was compacted for four minutes in the mold by the vibrating disc in five layers. The number of repetition in the density tests was 18 and 9 for the minimum and maximum density for all materials. Details of the test method are described in Kokusho [7]. As indicated in Fig.3, the average minimum and maximum densities for RS and DGS materials obviously increase with increasing uniformity coefficient. Their coefficients of variances also tend to increase with the uniformity coefficient although the values are lower than 0.5%.

TEST METHOD

In a triaxial apparatus used in this research, the specimen size is 100mm diameter and 200mm height. The soil specimen can be cyclically loaded from above as a stress-control test and monotonically loaded from below as a strain-control test as indicated in Fig.4. Two small accelerometers were stuck near the top and bottom specimen sides. The S-wave data are measured with the accelerometers during consolidation prior to shear tests by striking an axial road lightly.

The soil specimens were prepared by wet tamping because other preparation methods such as air-pluviation or water-plubiation tend to intensify soil particle segregation for well-graded granular soils. The relative density of the RS specimen was adjusted by tamping to approximate six target values, $Dr=$ 20%, 30%, 50%, 70%, 90%. The specimens of the DGS materials were created for only about 50% relative density. The specimen was fully saturated by using CO_2-gas and de-aired water and isotropically consolidated by the effective stress of 98 kPa with the back-pressure of 294 kPa. The Skempton's B-value lager than 0.90 was measured in all tests, indicating that almost prefect saturation was attained Kokusho [8].

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%. In the undrained monotonic loading tests, the axial strain was increased with the strain rate of 0.09%/min. The effect of membrane penetration during cyclic loading was taken into consideration by employing a modification method proposed originally by Tokimatsu [9] and modified for using pore pressure response to low stress amplitude (Tanaka [10]). The difference in undrained cyclic strength due to this effect was found generally small, because the specimen surfaces were actually smooth because of
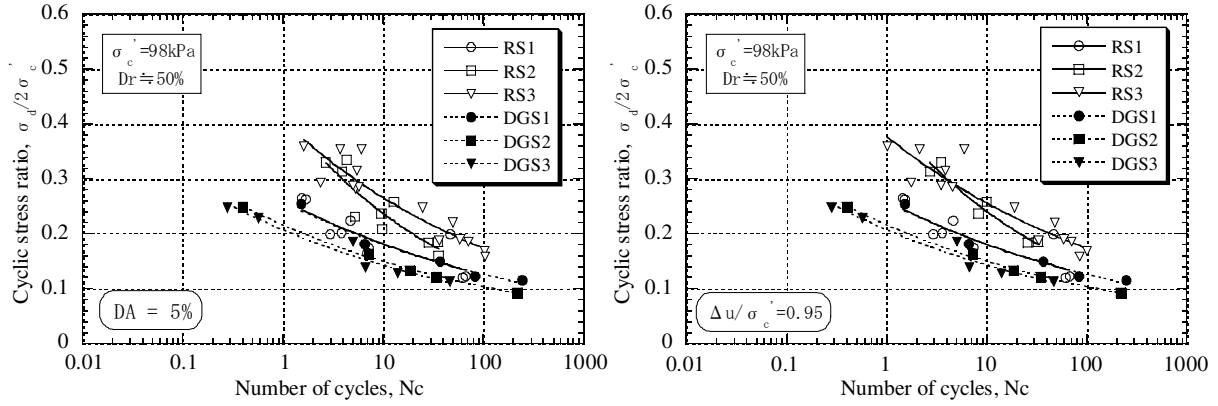
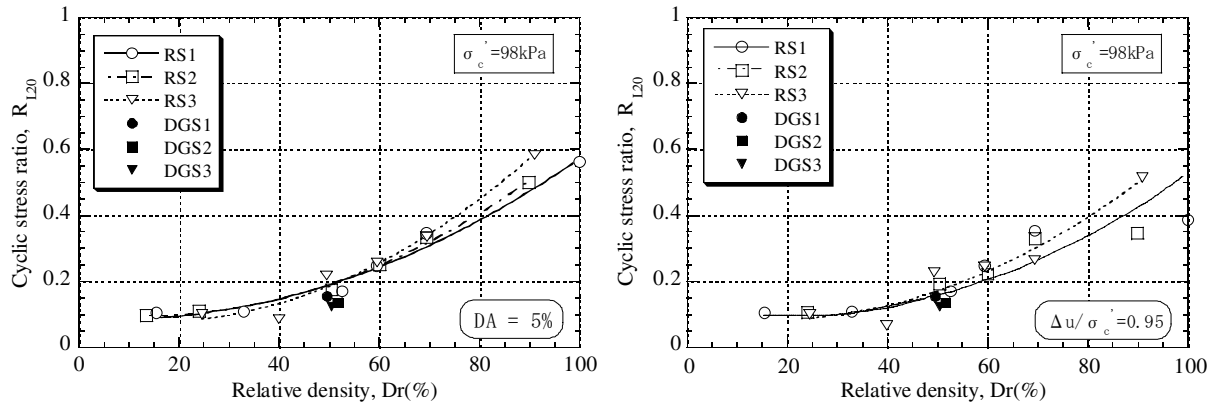Fig.4   Liquefaction strength curve

Fig.5   Relationship between the cyclic stress ratio versus relative density

rich content of sand. Hence, the membrane penetration effect does not seem to have significant influence on the test results in general.

UNDRAINED CYCLIC SHEAR BEHAVIOR OF RIVER SOILS AND DECOMPOSED GRANITE SOILS

Open symbols in Fig.4 exemplifies typical relationship between the cyclic stress ratio, R_L ($\sigma_d/2\sigma_c'$; σ_d = single axial stress amplitude and σ_c' = effective confining stress), for attaining 5% double amplitude or 0.95 excess pour-water pressure ratio, $\Delta u/\sigma_c'$ (Δu = pore-water pressure increment) and the number of loading cycles N_C for the river soils and decomposed granite soils for Dr about 50%. This stress ratio defined by the 5% DA strain is almost identical with that defined by nearly 100% pore-water pressure buildup (Hara [11]). Obvilusly, the strength of DGS is smaller than RS of the same particle gradation particularly for coarser soils with larger Uc, while that of DGS1 is almost equivalent to RS1. Based on this and other results, stress ratios corresponding to N_C =20, R_{L20}, versus relative density Dr are plotted in Fig.5. The data points for the river soils seem to be almost coincidental with each other. The strength of DGS is smaller than RS particularly for coarser soils with large Uc. Hence, it may be said that the undrained cyclic strength is not so much dependent on Uc or particle gradation in contrast to its large
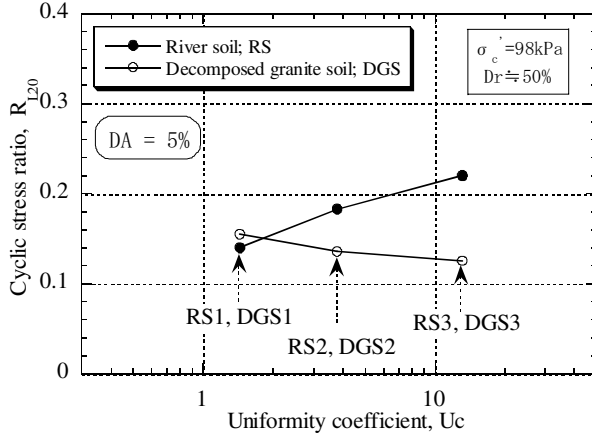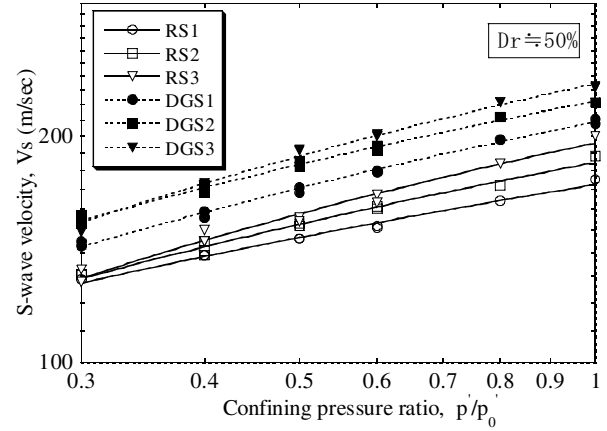
Fig.6 Cyclic stress ratio versus uniformity coefficient

Fig.7 S-wave velocity versus confining pressure ratio

dependency on *Dr*. In other words, the undrained cyclic strength defined by the stress ratio for attaining DA=5% or $\Delta u/\sigma_c' = 0.95$ may be considered largely dependent on the relative density despite large difference in absolute density due to the different in particle gradations. In order to clearly see the effect of grain size distributions on the strength, Fig.6 indicates a relationship between R_{L20} and *Uc* for *Dr* about 50%. Small differences in *Dr* for individual plots on Fig.5 are adjusted based on the slopes of the regression curves to evaluate R_{L20}. The stress ratio R_L small increases with increasing *Uc* for the river soils, while it decreases a little for the decomposed soils.
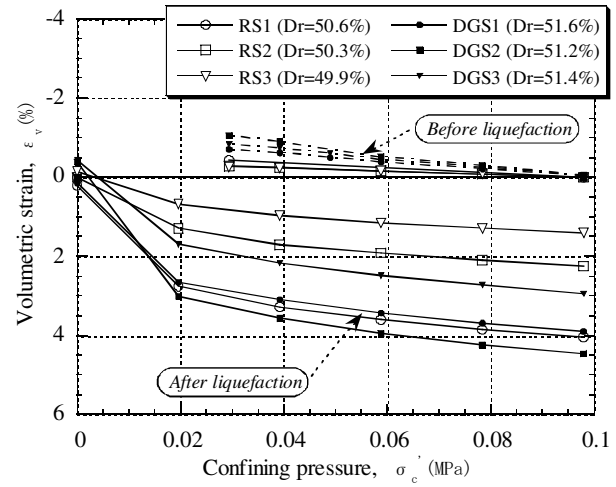
Fig.7 shows the S-wave velocity (*Vs*) versus confining pressure ratio p/p_0 (p_0 = the confining

Fig.8 Consolidation tests result

pressure 98 kPa) relationship obtained by the consolidation tests carried out before the undrained cyclic loading tests for the river soil and the decomposed granite soils for *Dr* about 50%. The S-wave velocity for all the soils is almost linearly dependent on confining pressure σ_c' on the full logarithm scale. The values are much higher for the river soils than the decomposed granite soils.

Fig.8 shows volumetric strain (ε_v) versus isotropic stress (*p'*) relationship obtained by consolidation tests carried out before and after undrained cyclic loading tests for the river soils and decomposed granite soils. The volumetric strain of the decomposed granite soils is larger than the river soils after liquefaction in particular presumably because of higher particle crushability. For the river soils, post-liquefaction volumetric strain is larger for soils with smaller *Uc*, because sands with smaller *Uc* has larger void ratio than coarser soils with higher *Uc* for the same relative density. DGS1 shows almost identical consolidation strain to the river sand, RS1. However for soils with higher *Uc*, the volumetric strain is evidently larger for the decomposed granite soils than for the river soils. The majority of the post-liquefaction settlement occurs at the initial stage of consolidations.
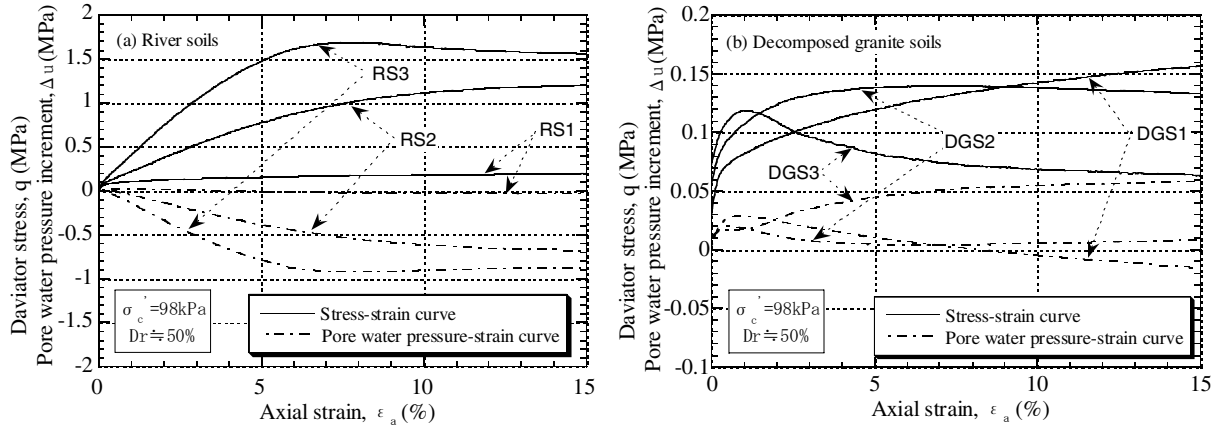
Fig.9   Stress, pore-water pressure versus axial strain obtained in undrained monotonic loading tests
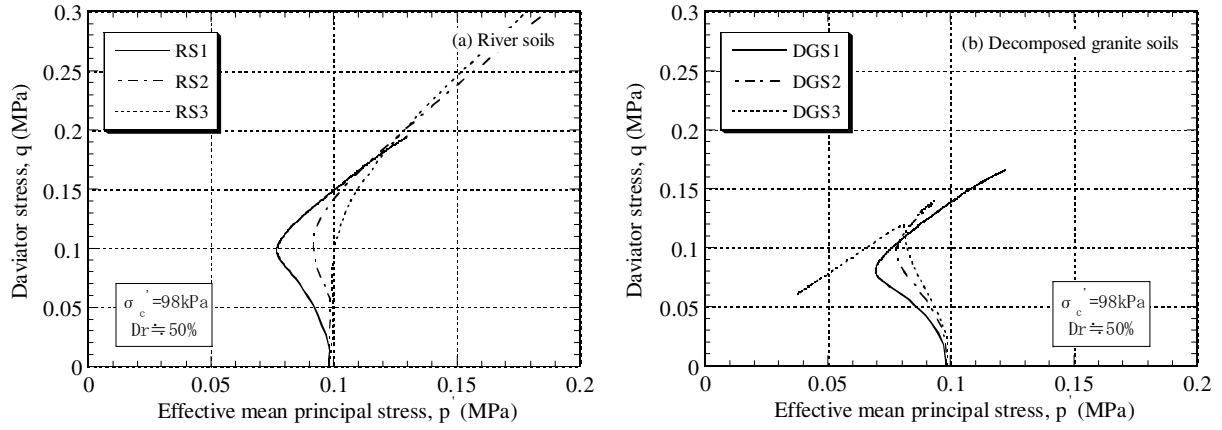

Fig.10   Effective stress path obtained in undrained monotonic loading tests

UNDRAINED MONOTONIC SHEAR BEHAVIORS OF RIVER SOILS AND DECOMPOSED GRANITE SOILS

Fig.9 shows undrained monotonic shear behavior for the river soils and the decomposed granite soils with the relative density Dr about 50% in terms of the deviator stress q ($= \sigma_1 - \sigma_3$, where σ_1 and σ_3 are the axial and lateral stresses) versus the axial strain ε_a and the pore-water pressure increment Δu versus the axial strain ε_a relationship. Although the soils have almost the same relative density, their behavior is surprisingly different. The peak stress q_{max} is much higher for RS3 than RS2 or RS1 and the difference is too large to be comparable with those of the undrained cyclic strength of the same soils already discussed. The pore-water pressure tends to decrease largely for the RS3 with higher Uc compared to the RS1 and RS2 with lower Uc. In contrast, the strength obtained in the stress-strain relationship from the decomposed granite soils is not so different despite the difference in Uc. The obvious difference in the shear strength between soils of different grain size distributions in the river soils is hardly recognizable in the DG soils. The pore-water pressure tends to decrease due to shearing for all DGS soils. This
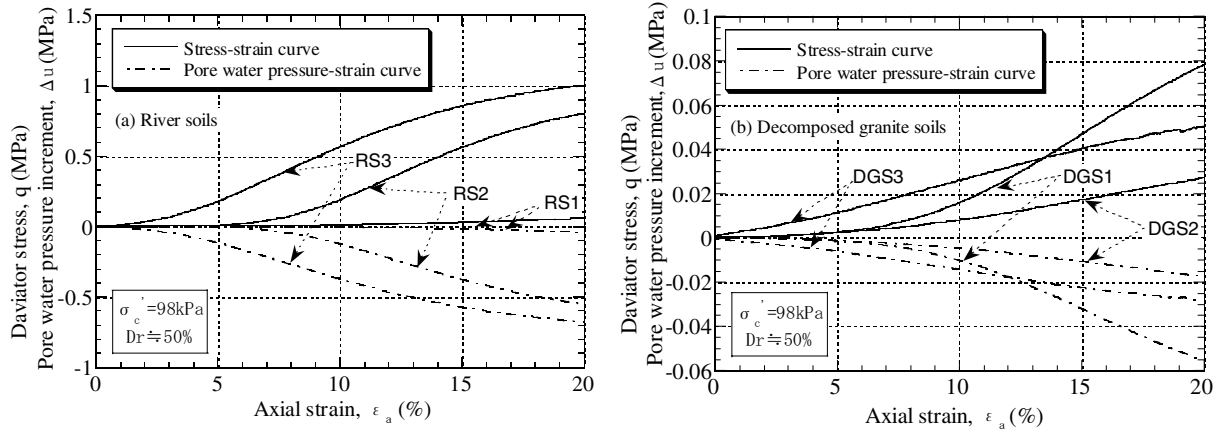
Fig.11 Relationship between stress, pore-water pressure and axial strain after liquefaction

considerable difference in shear behavior may be mainly attributable to the difference in crushability of soil particles of the decomposed granite soils.

Fig.10 shows the corresponding effective stress paths drawn on the mean effective stress p' (= ($\sigma_1 + 2\sigma_3$,)/3) versus the deviator stress q plane for the river soils and the decomposed granite soils. The effective stress paths for three soils eventually go up along straight failure lines. A comparison with the stress paths of the river soils and the decomposed granite soils in Fig.10 reveals that in the soils of DGS2 or DGS3, the pore-water pressure cannot develop so much in negative direction as the river soils RS2 or RS3, and suddenly turns to the positive direction

Fig.12 Relationship between deviator stress and uniformity coefficient

resulting in smaller undrained shear strength. The internal friction angle ϕ' for effective stress can be calculated in Fig10(a) as 36.9° for RS1, 39.7° for RS2 and 41.5° for RS3, respectively, indicating a upward tendency with increasing Uc. While it can be calculated in Fig.10(b) as 36.3° for DGS1, 36.8° for DGS2 and 31.8° for DGS3, respectively, smaller than the river soils and does not indicate a clear upward tendency with the uniformity coefficient Uc. The minimum internal friction angle ϕ' is much smaller for DGS3 than DGS1 or DGS2. This test result may also reflect the crushability of the decomposed granite soils.
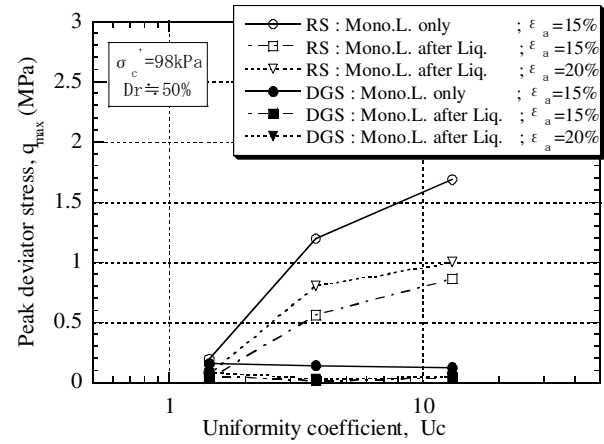
POST-LIQUEFACTION UNDRAINED MONOTONIC SHEAR BEHABIOR OF RIVER SOILS AND DECOMPOSED GRANITE SOILS

Fig.11(a) shows relationships between the deviator stress q versus the axial strain ε_a and the pore-water pressure increment Δu versus the axial strain ε_a obtained in undrained monotonic loading tests carried-out just after the cyclic loading tests for the river soils with the relative density of about 50%. In the cyclic loading tests, all specimens attained almost 100% pore pressure buildup and about 10% DA axial strain.
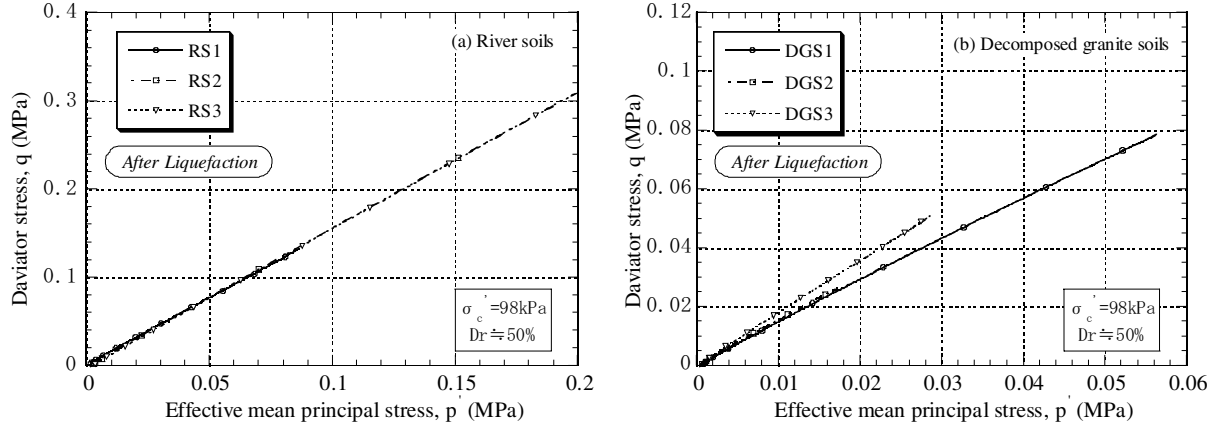
Fig.13 Effective stress path obtained in post liquefaqction undrained monotonic loading tests

The initial response to monotonically increasing strain in Fig 11(a), in which principal stress difference and pore-water pressure change very gradually below some threshold strain, is quite different from that of specimens without preceding cyclic loading shown in Fig.9(a). Fig.11(b) shows relationship between the deviator stress q versus the axial strain ε_a and the pore-water pressure increment Δu versus the axial strain ε_a obtained in undrained monotonic loading tests carried out just after the cyclic loading tests for the decomposed granite soils with the relative density of about 50%. Comparing this with similar stress-strain curves of the river soils in Fig.11(a), considerable decrease in post-liquefaction undrained strength can be noticed for coarser soils, DGS2 and DGS3. Fig.12

Fig.14 Internal friction angle versus uniformity coefficient

indicates relationship between the peak deviator stresses q_{max} which are read off either at ε_a = 15% or 20% of the stress-strain curve versus the uniformity coefficient Uc for the river soils and the decomposed granite soils. Drastic increase in shear resistance of the river soils for large strain with increasing Uc is evidently seen whether or not the soil is subjected to cyclic loading. However, the shear resistance of the decomposed granite soils stays almost constant with increasing Uc both before and after liquefaction.
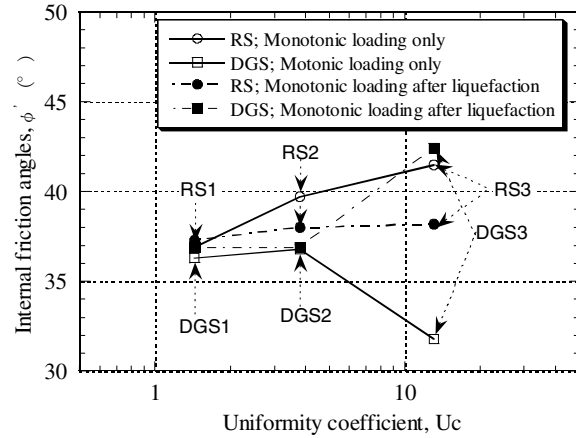
Fig.13 shows the corresponding effective stress paths drawn on the mean effective stress p' versus deviator stress q plane for the river soils and the decomposed granite soils. All paths start near the origin after the full pore pressure buildup and go up along straight failure lines. The stress-path of the decomposed granite soils does not develop so much as the river soils. Fig.14 indicate the internal friction angle ϕ' for effective stress read off from Fig.10 and Fig.13 versus the uniformity coefficient Uc for the river soils and the decomposed granite soils. The internal friction angle ϕ' for effective stress of the river soils stay almost constant with increasing Uc after liquefaction. There value is decreasing about 3%-10% in comparison with the internal friction angle for effective stress ϕ' before liquefaction except for DGS3.

CONCLUSIONS

Liquefaction and undrained monotonic shear behavior of the river soils and the decomposed granite soils have been studied by cyclic and monotonic loading triaxial tests to investigate the effect of the particle gradation on the undrained shear characteristics. Soil materials tested are two types; river soils and decomposed granite soils. Major conclusions obtain in this study are;

1) Liquefaction strength of granular soils may be roughly evaluated by the index of relative density, Dr, despite large difference in particle gradations.
2) Liquefaction strength and S-wave velocity of the decomposed granite soils are small compared with the river soils which have the same particle distribution and relative density about 50%.
3) Undrained monotonic loading strength of the river soils corresponding to larger axial strain of 15% shows drastic increase with increasing uniformity coefficient, Uc, for the same relative density.
4) Post-liquefaction undrained strength of the river soils for larger strain of 20-25%, too, is not uniquely determined by relative density but largely dependent on particle gradations.
5) Strength of the decomposed granite soils measured by undrained monotonic loading tests does not increase so much with increasing Uc unlike the river soils consisting of non-crushable, which have the same particle distribution and relative density about 50%. This may be attributable to soil crushability which impedes development of soil dilatancy and negative pore pressure because crushable larger size particles cannot bear high pressure.

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