Experimental Study on Liquefaction Phenomenon of Undisturbed Saturated Sands

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不攪乱砂質土の液状化特性に関する実験的研究

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This investigation deals with the liquefaction characteristics of disturbed and undisturbed sands under cyclic loading conditions using by the dynamic triaxial compression device.

The following conclusions were obtained.

(1) Cyclic stress ratio of undisturbed samples have high value affected by cementation and over consolidation.

(2) Cyclic stress ratio of undisturbed samples is not proportional to relative density.

(3) The liquefaction characteristics of specimens remolded by different compaction procedures is different.

INTRODUCTION

It has been recently recognized that if a saturated sand is subjected to cyclic shear stresses, such as that induced by earthquakes, liquefaction phenomenon occurs and major damages such settlement of sand layer or a slope failure resulting from the loss of its strength will occur. For typical example, the Niigata eartquake of 1964, the Tokachi-oki earthquake of 1968, the Chilean earthquake of 1960 and Alaskan earthquake of 1964 caused extensive damages to buildings and earth embankments (11, 12, 13, 14).

The cause of sand liquefaction by a experimental way has been studied by many investigators for many years. Seed and Lee (11) conducted dynamic experiments, as liquefied by appling cyclic shear stress on the saturated soil specimen, by using cyclic triaxial compression test device and presented the available test data on liquefaction characteristics of saturated sand. Based on the analysis of test data, Seed and Lee have pointed out that the occurance of liquefaction is resulted from the decrement of effective stress due to the increment of residual pore-water pressure resulting from the sequence of cyclic loadings induced by the earthquake under undrained state, and that major factors affecting the development of liquefaction are the void ratio of sand, the confining pressure acting on the sand, the magnitude of the cyclic stress or strain and the number of stress cycles.

Since the first investigation by Seed and Lee, several studies have been undertaken so far in reserch laboratories to make clear the liquefaction characteristics for saturated sand. And then it is considered that the basic studies on liquefaction characteristics have almost been accomplished at present.

These experiments were conducted on the disturbed alluvial sand, such as that diposited at circumference of river or coast. However, it note that undisturbed alluvial sand, diluvium sand and tertiary era sand at in-situ have a latent strength exhibited by soil skeleton between internal grain contact, and these kinds of sand are considered to have high resistance to occurance of liquefaction. Investigation on the characteristics of liquefaction for undisturded sample have not been undertaken so far.

Using the cyclic triaxial compression device, the authors have conducted liquefaction tests on fourteen kinds of undisturbed diluvium or tertiary era sands and sixteen kinds of disturbed sands. The investigations described herein are on the characteristics of liquefaction for undisturbed sands.

SOIL USED IN INVESTIGATION

Soils used in this investigation are sandy soil ranging from sand to sandy loam in the Triangular Classification System. The physical and mechanical properties and grain size distribution curves of the sixteen kinds of sand are shown in Table 1 and Fig. 1. The maximum and minimum void ratios shown in Table 1 were obtained by following methods.

(1) maximum void ratio, e_{max}; pouring dried sample

into the proctor mold of known volume without shock.

(2) minimum void ratio, e_{min}; pouring dried sample into the proctor mold in 2.5cm thick in each layer and hitting the mold ten times horizontally and repeat this step five times.

It is reasonable to consider that the maximum and minimum void ratios would be varied by the effects of difference of grain size distribution, etc. To know these influences, the relationship between the fine contents (percent finer by weight passing No. 200 standard seive) and the maximum and minimum void ratios for sixteen kind of samples are determined and are shown in Fig.2, Fig.3 and Fig.4 compaired with the test results from Watanabe, et al., within the relationships between $e_{max} - e_{min}$ versus mean grain size, D_{50}, and $e_{max} - e_{min}$ versus e_{max}, e_{min} (5).

Table 1. Properties of soil samples

Sample No.	Specific Gravity	Sand C. (>0.074 mm)	Silt C. (0.005∿ 0.074mm)	Clay C. (<0.005mm	Triangular Classification	Uniformity of Coef.	Void Ratio e_{max}	e_{min}	E_{50} 1)	P_y 2)
(A)	2.636	72.0%	16.0%	12.0%	Sandy Loam	40.0	1.635	0.780	44.0	0.80
(B)	2.681	89.8	4.7	5.5	Sand	2.8	1.657	0.836	72.3	3.20(1.9,4.5)
(C)	2.663	97.0	3.0	0	Sand	2.3	1.110	0.610	44.7	1.28(0.93,1.62)
(D)	2.657	98.0	2.0	0	Sand	2.8	1.025	0.625	88.8	3.25(3.0,3.5)
(F)	2.672	95.0	5.0	0	Sand	3.0	1.127	0.619	103.3	≑10
(G)	2.790	71.0	20.0	9.0	Sandy Loam	46.7	1.317	0.600	20.7	3.4
(H)	2.814	87.0	9.5	3.5	Sand	8.8	1.544	0.860	55.0	2.97(2.04,2.37, 4.5)
(I)	2.688	93.0	4.0	3.0	Sand	1.7	1.454	0.802	37.6	3.23(2.1,4.35)
(J)	2.750	90.0	8.0	2.0	Sand	2.1	1.602	0.830	310.0	3.93(2.60,5.25)
(K)	2.640	99.0	1.0	0	Sand	1.2	0.970	0.630	-	-
(L) 3)	2.650	100.0	0	0	Sand	2.0	0.868	0.555	-	-
(M)	2.711	92.6	7.4	0	Sand	2.2	1.490	0.800	28.8	2.09(1.78,2.40)
(N)	2.700	95.8	4.2	0	Sand	2.2	1.430	0.700	39.3	3.20(3.00,3.40)
(O)	2.675	92.2	7.8	0	Sand	4.0	1.500	0.805	44.6	2.25(1.90,2.60)
(P)	2.665	84.0	7.0	9.0	Sand	24.3	1.200	0.580	199.0	≑10
(Q)	2.657	70.0	22.0	8.0	Sandy Loam	39.0	1.279	0.782	204.1	≑10

1) Coefficient of Defomation obtained by Unconfining Compression Tests

2) Consolidated Yield Stress

3) TOYOURA SAND

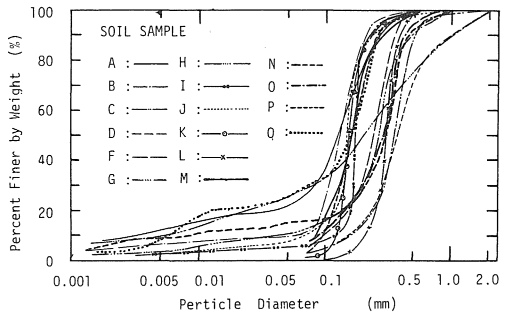


Fig. 1. Grain-size distribution curves

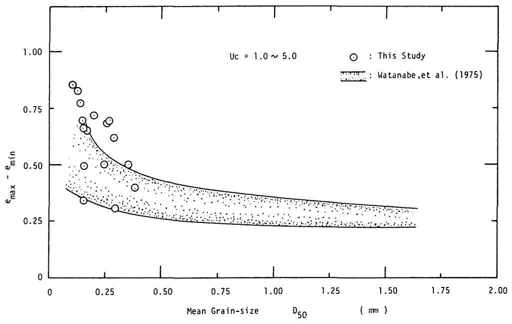


Fig. 3. $e_{max} - e_{min}$ VS D_{50}

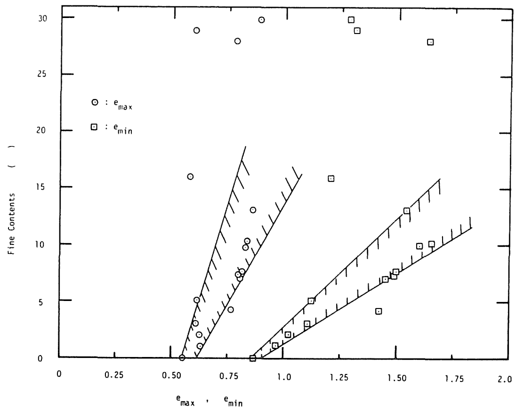


Fig. 2. Fine contents VS e_{max}, e_{mix}

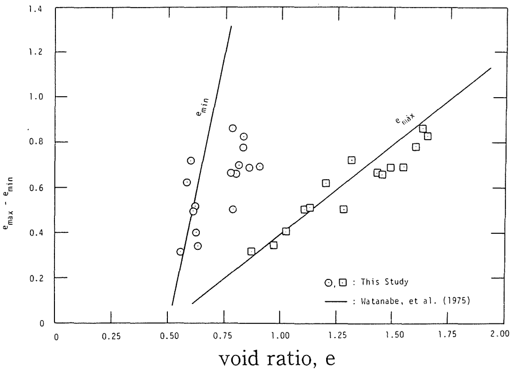


Fig. 4. $e_{max} - e_{min}$ VS e_{max}, e_{min}

EQUIPMENT AND TEST PROCEDURES

CYCLIC TRIAXIAL COMPRESSION TEST EQUIPMENT

During this investigation all tests were conducted in the Cyclic Triaxial Compression Apparatus, as shown in Fig.5. This apparatus essentially consists of a triaxial cell, two loading systems for applying axial and lateral cyclic stresses on the columnar sample, and electronic recording system for measuring the dynamic axial stress, $\Delta\sigma_1$, lateral stress, $\Delta\sigma_3$, axial strain, ε_1, and the pore-water pressure, Δu. For applying constant cyclic stress on the sample, two hydraulic cylinders were controlled by the electrical hydraulic servo system. The sample is 50mm in diameter and 125mm in height.

TEST PROCEDURE

Liquefaction tests on disturbed and undisturbed samples were conducted. The preparation of each sample was that;

1) For disturbed samples; each sample was tested under loose and dense states. For loose state, de-aired saturated sand (voiled from two to three houres) was carefully poured by using a spoon into the water filled specimen mold fixed in triaxial cell. Dense state was obtained by shotting the loose sample using a small hummer.
2) For undisturbed samples, to avoid disturbance of soil-skeleton, which affects the test results, all undisturbed soil were carefully inserted into the cylinder with 70mm in inside and 300mm in height at the field, and these were trimmed by using a trimmer in according to the dimmension and placed into the specimen mold such a standard permeameter test device, which was able to take this mold to two pieces in the triaxial cell at the test and the sample was saturated by for twelve hours with moderate head (ranging from 50 to 100cm), and saturated undisturbed sample was set on the base in triaxial cell. In this way, almost of all samples had obtained value of ranging from 90 to 95 percents in degree of saturation.

For both disturbed and undisturbed samples, the experiments were conducted under the same consolidation condition. ($K_o = 1.0$condition). The cyclic axial and lateral stresses, as which if the axial stress, $\Delta\sigma_3$, increased, the lateral stress, $\Delta\sigma_3$, decreased in equal amount simultaneouslly, were introduced by the aforementioned actuator on a sample at 2.0HZ. For undisturbed sample, especially, to taking a complete saturated condition, the back pressure of 1.0kg per Sq cm applied in the sample. Fig.6 shows the typical record of strain, excess pore-water pressure and lood during cyclic loading test.

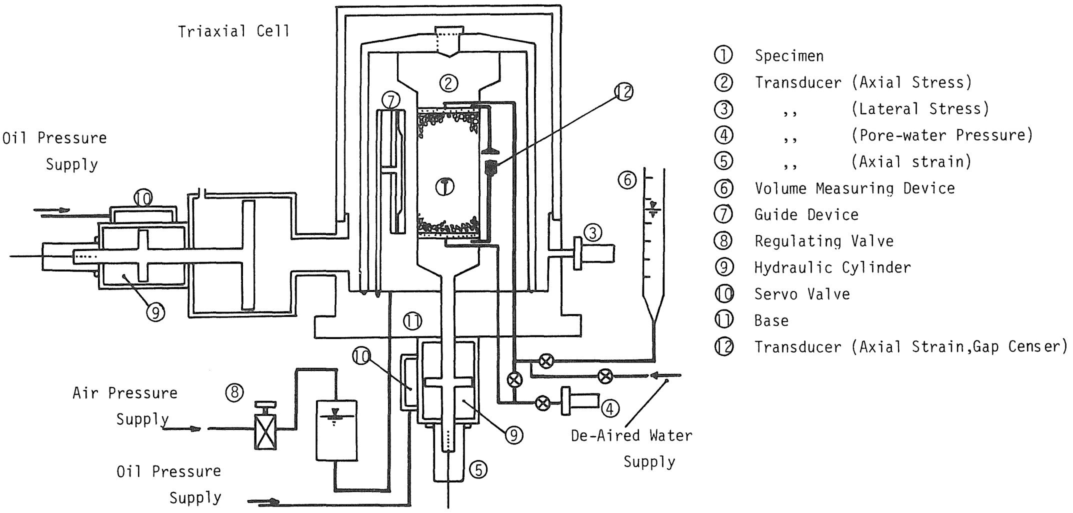


Fig. 5. Schematic drawing of the dynamic triaxial aparatus

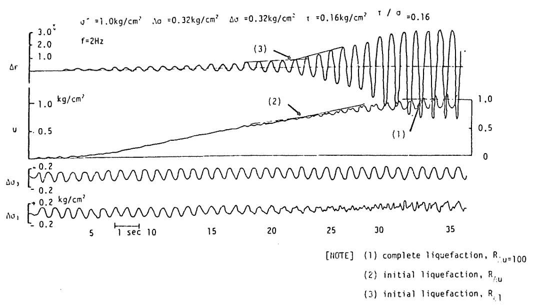


Fig. 6. Typical records of strain, excess pore-water pressure and load during triaxial liquefaction test.

TEST RESULTS

The experiments were conducted under various effective confining pressure and void ratio.

Fig. 7 shows the typical relationship between the cyclic stress ratio, R_{nl}, ($R_{nl} = \tau_d / \sigma'_0$, τ_d : maximum cyclic shear stress, σ'_0 : initial effective confining pressure) and the number of cycles to cause liquefaction, N_l, as a function of void ratio for disturbed saturated sand (TOYOURA-SAND). Similarly, Figs. 8 and 9 show the typical relationship for undisturbed saturated sand. From Fig. 7, it can be seen that the cyclic stress ratio is significantly affected by the value of the void ratio, and in case of representing the liquefaction resistance by stress ratio, there is not a very significant influence on the initial effective confining pressure. This finding is in agreement with the conclusion presented by Seed, Lee and many other investigators. On the other hand, from the test results shown in Figs.8 and 9, it can be seen that there is a high scatter in data points and the manner or the degree of influence of void ratio and initial effective confining pressure on liquefaction occurance is not the same as compaired with the results of disturbed sample shown in Fig.7. The reason of the difference of test result on liquefaction characteristics will refer to the later section.

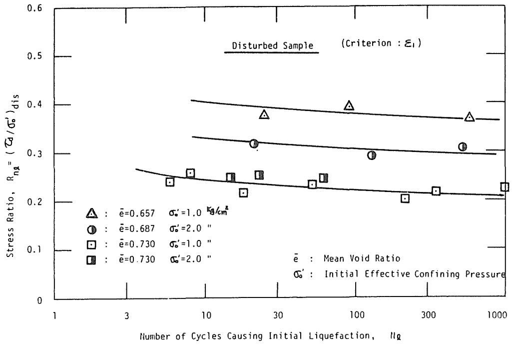


Fig. 7. Typical relationship between stress ratio and number of cycles required to cause initial liquefaction, sample number (K) Toyoura-sand

Almost all experiments for undisturbed samples were conducted under one kind of confining pressure (mainly, σ'_0 equal 1.0kg per Sq cm). Consequently, in this paper only the experimental data points under $\sigma'_0 = 1.0$kg per Sq cm condition were shown.

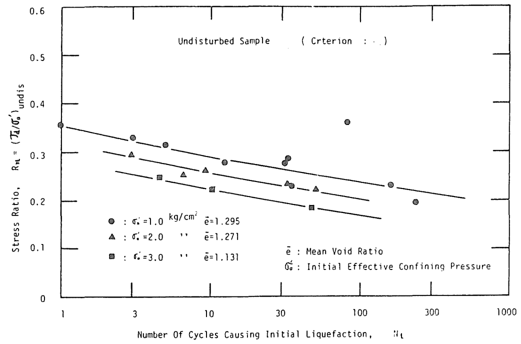


Fig. 8. Typical relationship between stress ratio and number of cycles required to cause initial liquefaction, sample number (M)

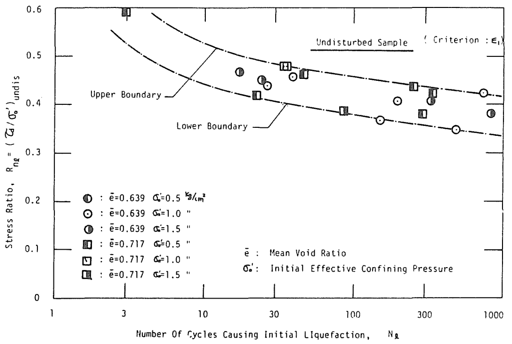


Fig. 9. Typical relationship between stress ratio and number of cycles required to cause initial liquired, sample number (F)

The liquefaction test is very difficulty as compared with a standard soil test, and the value of experimental data will be affected by the difference of the special character of apparatus, test procedure, etc..

In order to check up the special character on the test results obtained from this cyclic triaxial compression device, the test results of disturbed samples are shown in Figs. 10 and 11, together with the other resercher's results.

Fig.10 shows the relationship between the cyclic stress ratio, $R_{nl=10}$, causing liquefaction at ten cycles and the relative density, D_r, for the five kinds of sands contained fine contents of five percents. The full line is the experimental equation ($\tau_l=4.6\times10^{-3}\times D_r\times\sigma_0'$, D_r : %) proposed by Tanimoto. Further, in Ref.5 and 7, Watanabe and Ishihara presented available data on the liquefaction characteristics of wide range of sand type and they proposed from the analysis of these data that the relationship between cyclic stress ratio and density, soil type and soil gradation would be able to be shown by the factor, $e\text{-}e_{min}$, in which $e\text{-}e_{min}$ is a value of the difference between void ratio of sample and minimum void ratio. Based on their proposition, the relationship between the cyclic stress ratio causing liquefaction at twenty cycles, $R_{nl}=20$, and the factor, $e\text{-}e_{min}$, for the all disturbed samples tested during this investigatinn is shown in Fig.11. From the data in Figs.10 and 11, it can be seen that in Fig.10, the mean value of cyclic stress ratio obtained during this investigation is small ranging from 0.03 to 0.07 for $D_r>40$ percents, and in Fig.11, the cyclic stress ratio is also small about 0.09.

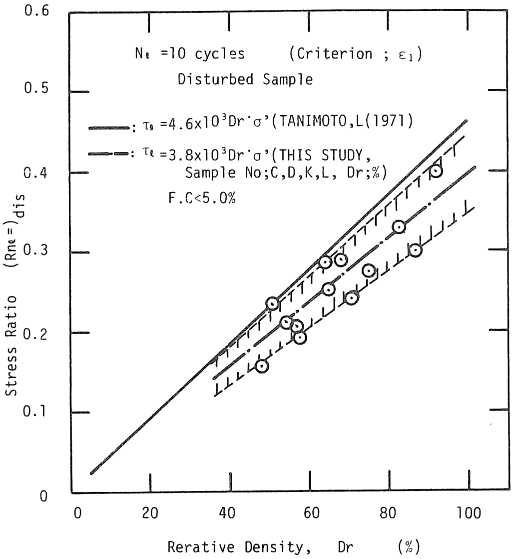


Fig. 10. Comparison of triaxial compression test results (N_l = 10cycles)

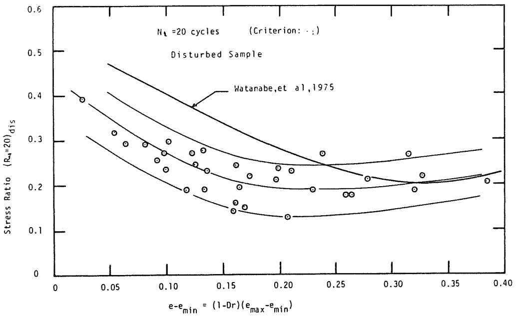


Fig. 11. Stress ratio VS $e-e_{min}$, ($N_l=20$ cycles)

INFLUENCE OF DIFFERENCE ON LIQUEFACTION CRITERION

As pervious mention, liquefaction phenomenon may be considered that the cyclic shear stresses induced by earthquake act on saturated sand under undrained condition, and by this action, the pore-water pressure and the strain are built up to the point of sudden increase which denotes the onset of liquefaction with increasing of number of cycles, and at last, the effective stress become to zero, liquefaction state appears.

For practical purposes, it note that damages of saturated sand layers and earth structurs will occure not only under the complete zero effective stress condition (difined by "complete liquefaction" in this paper) but under the condition of sudden increase of pore-water pressure or strain (difined by "initial liquefaction" in this paper). Then the cyclic stress ratio required to liquefy for all disturbed and undisturbed samples were obtained by three kinds of failure criterion as following;

(1) complete liquefaction when pore-water pressure equals to initial effective confining pressure or becomes constant during cyclic loading.
(Cyclic stress ratio decided with this criterion indicated by $R_{\Delta u=100}$)

(2) initial liquefaction when pore-water pressure suddenly increase during cyclic loading.
(Cyclic stress ratio decided with this criterion indicated by $R_{\Delta u}$)

(3) initial liquefaction when axial strain suddenly increase during cyclic loading.
(Cyclic stress ratio decided with this criterion indicated by $R_{\varepsilon 1}$)

The comparisons of the magnitude of cyclic stress ratio obtained from three criterion for liquefaction occurance are shown in Figs. 12 and 13.

Fig.12 shows the relationship between the ratio of $R_{\varepsilon 1}$ to $R_{\Delta u=100}$ and void ratio, for undisturbed and loose and dense disturbed samples. From this figure, it can be seen that almost all values of ratio, $R_{\varepsilon 1}/R_{\Delta u=100}$, are plotted below 1.0; for disturbed samples, it ranges from about 0.07 to 1.0, and for undisturbed samples, it rangs from about 0.07 to 1.15. Similarly, Fig.13 shows the relationship between the ratio of $R_{\varepsilon 1}$ to $R_{\Delta u}$ and void ratio. It can be seen from this figure that all of the value of ratio, $R_{\varepsilon 1}/R_{\Delta u}$, are near to 1.0. From these findings, it becomes clear that the cyclic stress ratio causing initial liquefaction is small about thirty percents comparied with that of completely liquefaction, for both disturbed and undisturbed samples, and on the criterion of initial liquefaction, the stress ratio is almost unaffected by either criterions, pore-water pressure or axial strain.

INFLUENCE OF SAMPLE PREPARATION PROCEDURE ON THE LIQUEFACTION OCCURANCE

It may be considered that even though the void ratio of sample, the magnitude of cyclic stress, the effective confining pressure acting on the sample and the number of cycles are the same, the resistance to liquefy will differ in consequence of the soil skeleton, the interrocking between soil particles and aration procedure;

Fig.14 shows the relationship between cyclic stress ratio to cause initial liquefaction at twenty cycles and relative density obtained by two sort of sample preparation procedure;

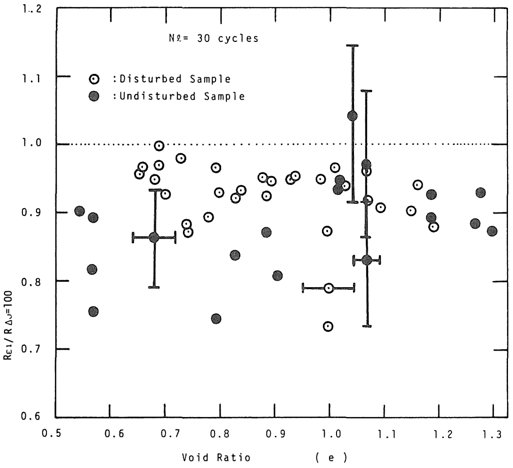
(1) pouring de-aired saturated sand into the water-filled specimen mold using by a spoon.
(2) tampping moist sand using by a tamper.

It can be seen from this figure that the cyclic stress ratio by tampping have a higher resistance to liquefy than that by pouring.

PORE-WATER PRESSURE DEVELOPMENT DURING CYCLIC LOADING

It is a essential particular for liquefaction analysis to estimate the magnitude of pore-water pressure which develops in sands during earthquakes.

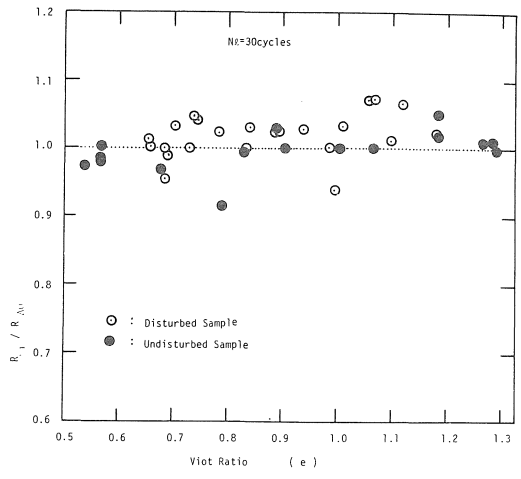
To estimate the magnitude of pore-water pressure development in disturbed sand which is subjected to cyclic shear stress applications, the results of the number of cycles until complete liquefaction state versus residual pore-water pressure are normalized, and relationship between $\Delta u/\sigma_0'$ and N/N_l for the samples having ten percents of fine contents (where, Δu: residual pore-water pressure at N cycles, σ_0': initial effective conffing pressure, N: number of cycles under consideration, and N_l : number of cycles to cause complete liquefaction is plotted in Figs.15.1 to 15.4 at void ratio ranging from 1.04 to 1.15, from 0.89 to 0.94, from 0.78 to 0.84, and from 0.65 to 0.74, respectively, and upper and lower boundery of data points are also shown.



$R_{\varepsilon 1}$: Stress ratio defined as the cycle when the axial strain suddenly increases

$R_{\Delta u=100}$: Stress ratio difined as the cycle when the excess pore water pressure first equals the initial confining pressure or constants

Fig. 12. Comparison of stress ratio obtained by different criterion for liquefaction occurrence



$R_{\Delta u}$: Stress ratio defined as the cycle when the excess pore water pressure suddenly increases

Fig. 13. Comparison of stress ratio obtained by different criterion for liquefaction occurrence

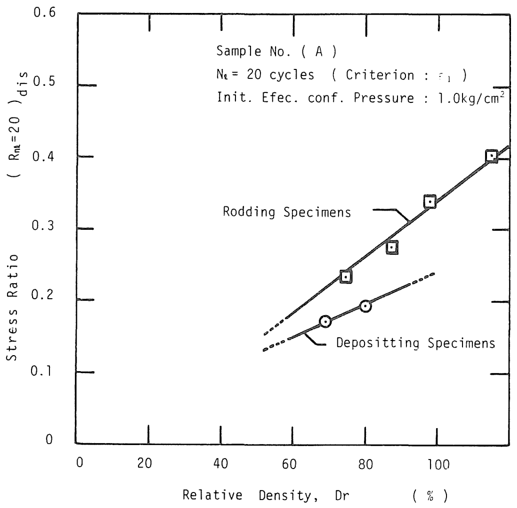


Fig. 14. Comparison of the liquefaction resistance of medium to dense specimens prepared by rodding and depositting

These results relatively have a broad scattering band. Fig.16 shows the mean curve shown in Fig.15 for each mean void ratio.

It may be seen from these figuars that for a given value of N/N_l, value of the ratio $\Delta u/\sigma'_0$ at high void ratio is higher than the value at low void ratio, and that value of the ratio $\Delta u/\sigma'_0$ increase with increasing value of the ratio N/N_l, and that the shape or the magnitude of average pore-water pressure development curves are almost the same for void ratios of 0.92 and 1.10.

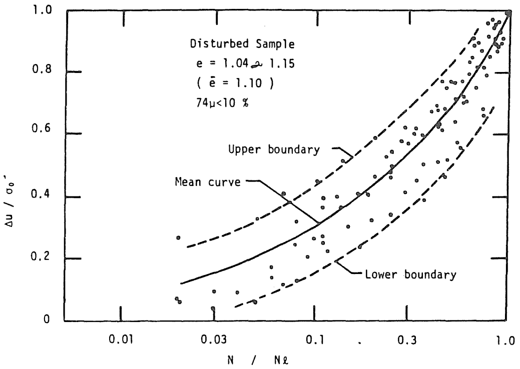


Fig. 15.1 $\Delta u/\sigma'_0$ VS N/N_l (N : spontaneous number of cycle befor liquefaction occurance)

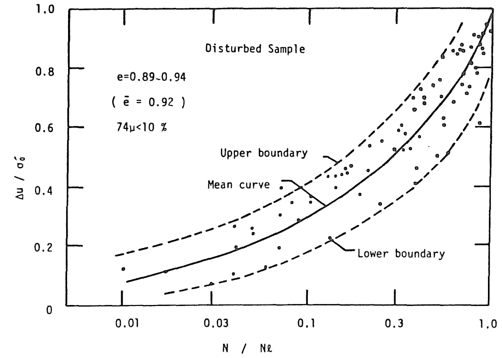


Fig. 15.2 $\Delta u/\sigma'_0$ VS N/N_l

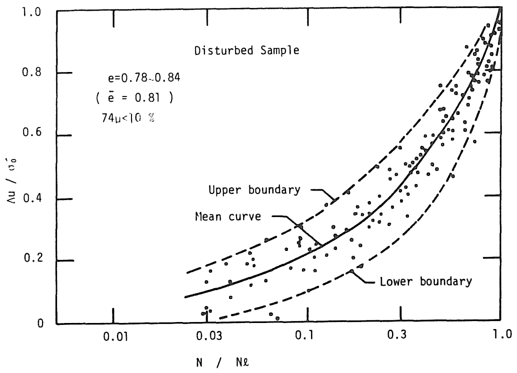


Fig. 15.3 $\Delta u/\sigma'_0$ VS N/N_l

Fig.17 shows representative results for undisturbed samples. From this figuar, it may be seen that the results have a broad band compaired with the results for disturbed sample.

Fig.18 shows the results of disturbed and undisturbed sample for sample number (J). It is evident that in spite of almost the same void ratio, the value of the ratio $\Delta u/\sigma'_0$ for disturbed sample is smaller in all range of N/N_l.

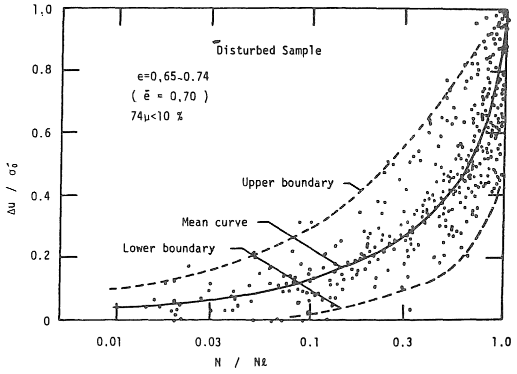


Fig. 15.4 $\Delta u/\sigma'_0$ VS N/N_l

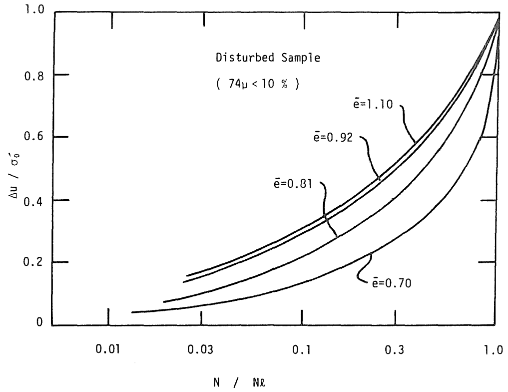


FIG. 16.

Relationship between $\Delta u/\sigma'_0$ and N/N_l for different void ratio, disturbed sample

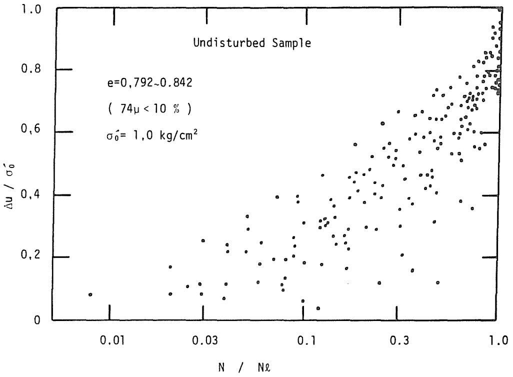


Fig. 17. $\Delta u/\sigma'_0$ VS N/N_l, undisturbed sample

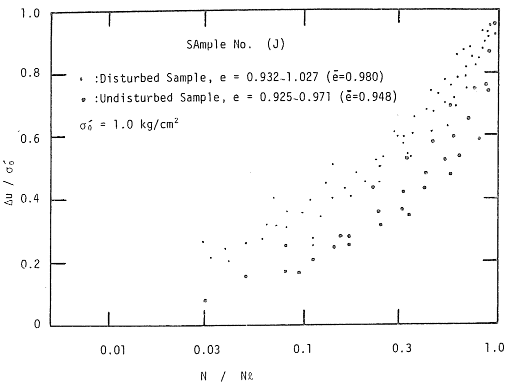


Fig. 18. Conparison of $\varDelta U/\sigma_0'$ and N/N_1 for disturbed and undisturbed samples

CYCLIC STRESS RATIO-AXIAL STRAIN RELATIONSHIP AT LIQUEFACTION OCCURANCE

The plots of cyclic stress ratio, $R_{nl=20}$, versus axial strain, ε_{1l}, at onset of initial liquefaction (defined as axial strain suddenly increase during cyclic loading) are shown in Fig.19. This figure shows the test results of disturbed and undisturbed sample for sample number (C), representatively, and shows in each void ratio, for disturbed sample.

From this figure it may be seen that for disturbed sample, the relationship between $R_{nl=20}$ and ε_1 have a constant form; the value of axial strain enlarge with increasing value of cyclic stress ratio for a given void ratio and with increasing a void ratio for a given cyclic stress ratio. On the other hand, for undisturbed sample completely have a different tendency as compared with the results of disturbed sample; the value of axial strain increase with decreasing the value of cyclic stress ratio untill axial strain of about 0.6 percents.

LIQUEFACTION CHARACTERS OF UNDISTURBED SANDS

In order to evaluate the liquefing strength of undisturbed sands, the relationship between stress ratio and relative density for all undisturbed samples are investigated in the same manner as that of disturbed samples. This results are shown in Fig.20, together with the results of the disturbed samples shown in Fig.10. From this figuar, it can be seen that for the experimental results of disturbed samples, the magnitude of stress ratio increases in accordance with increment of relative density and the stress ratio is in proportion to the relative density. This finding is in agreement with the traditional conclusions. On the other hand, it can be seen from the experimental results of undisturbed samples shown in the same figure that the value of stress ratio is far high as compared with the results of disturbed samples, and there is not a linear relationship between stress ratio and relative density.

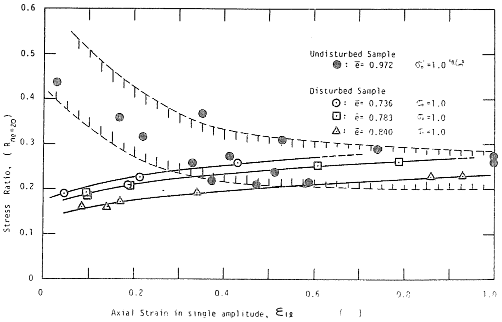


Fig. 19. Relationship between stress ratio and axial strain at onset of initial liquefaction

Thus, liquefaction characters of undisturbed sands are different compared with disturbed sands. The cause of this difference may be assumed that, in the disturbed sample, specimens were prepared by pouring the de-aired saturated sand as described in "TESTING PROCEDURE", and in this case the soil particles form the single-grained structure.

Consequently, the liquefing strength of disturbed samples will differ only the magnitude of relative density. While, in the undisturbed sample, samples may have had a latent strength such a cementation and may have been subjected to stress history for long term, for example, cyclic stress due to an earthquakes and change of static stress due to the change of topographic features. Consequently, such undisturbed samples have a highly resistance to liquefy and further, liquefing strength is highly complex.

It will be able to consider, from the assumption described previously, that the soil parameters, fine contents, consolidated yield stress and coefficient of deformation, are related to the liquefing strength of the undisturbed samples. Thus, the relationships between such soil parameters and stress ratio were plotted. These results are shown as following.

Fig.21. shows the relationship between the ratio $R_{u.d.}$, (the ratio of cyclic stress ratio for undisturbed sample of diluvium or tertiary ear sands, $R_{undis.}$, to cyclic stress ratio for disturbed sample at the same void ratio in undisturbed sample, $R_{dis.}$, which obtained from cyclic stress ratio-void ratio relationship) and fine contents, F.C., of the sample. In this figure, the results for diluvial sands presented by other investigators are also plotted (2,8,9,). It may be seen from this figure that the value of $R_{u.d.}$ for diluvial sands is almost 1.0 below about 10 percents in F.C. and is somewhat high above 10 percents in F.C., However, for tertiary era sands it have higher value below

10 percents in F.C., especially, and further, the fine contents appear to have an insignificant influence on ratio, $R_{u.d.}$,

Fig.22 shows the relationship between cyclic stress ratio, $(R_{nl=10})_{undis}$, and the consolidated yield stress obtained from consolidation test, P_y, for undisturbed samples tested under the same confining pressure, $\sigma'_0 = 1.0$kg per Sq cm.

From this figuar, it can be seen that this figuar have somewhat sccatter of data points, however, cyclic stress ratio increases with increasing value of consolidated yield stress.

However, as shown in Fig.23 (Fig.23 shows the relationship of stress ratio versus void ratio for disturbed and undisturbed conditions of sample number (M). The cyclic stress ratio at confining pressures of 1.0, 2.0 and 3.0kg per Sq cm for undisturbed condition are plotted with (●), (▲) and (■)) the value of cyclic stress ratio for undisturbed sample tested under the initial confining pressure of 2.0 and 3.0kg per Sq cm are apparently small as compaired with the result at confining pressure of 1.0kg per Sq cm and undisturbed sample at confining pressure of 3.0kg per Sq cm and disturbed sample have almost the same value in cyclic stress ratio. This finding shows that the value of cyclic stress ratio tested under the initial confining pressure of 1.0kg per Sq cm, Fig.20, 21 and 22, are the results under over consolidation conditions, because the consolidated yield stress of the all undisturbed samples (sample number (A) expected) have higher values more than 1.0kg per Sq cm. Consequently, it can be concluded that the undisturbed samples have a highly resistance to liquefy influenced by the over consolidation ratio.

To investigate the influence of over consolidation ratio on cyclic stress ratio of undisturbed samples, the relationship between the cyclic stress ratio, $(R_{nl=10})_{undis.}$, and P_y/σ'_0 is shown in Fig.24. The length of straight line of data points in this figure indicates the magnitude of scatter on test datas. From this figure, it is apparently to see that the undisturbed samples having high over consolidation ratio exhibit high resistance to liquefy.

Fig.25 shows the relationship between cyclic stress ratio causing initial liquefaction at 10 cycles under the initial effective confining pressure of 1.0kg per Sq cm for undisturbed samples, $(R_{nl=10})_{undis}$, and modulus of deformation obtained by unconfined compression test, E_{50},. It may be seen from this figure that the test data protted in this figure fall within somewhat narrow scatter band compared with the

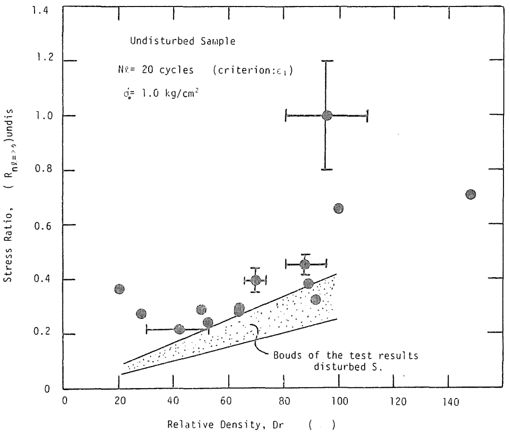
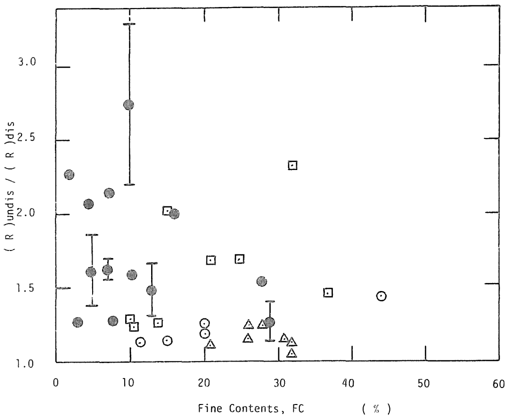


Fig. 20. Stress ratio required to cause initial liquefaction in 20 cycles for undisturbed samples VS relative desity



	Source of Data	Geology	Specimen	Nℓ	σ0'
●	Author	Tertiary. S	Sedimentation	10	1.0
△	Watanabe,et al.	Diluvial. S	Sedimentation	15 ~ 35	1.0
⊙	Mulilis,et al.	Diluvial. S	Moist Tamping	10	-
⊡	Yasuda, et al.	Diluvial. S	-	20	-

Fig. 21. Ratio of stress ratio of undisturbed to disturbed sample VS fine contents

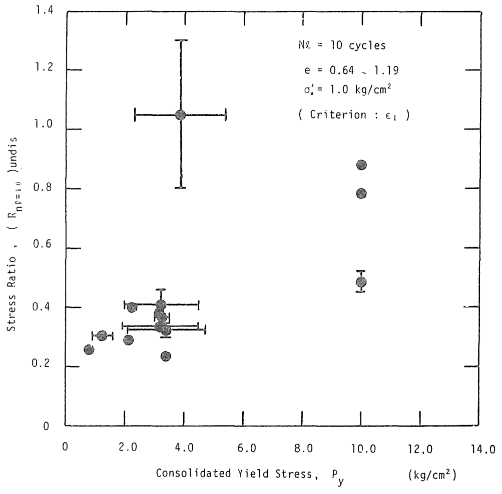


Fig. 22. Stress ratio VS consolidated yield stress

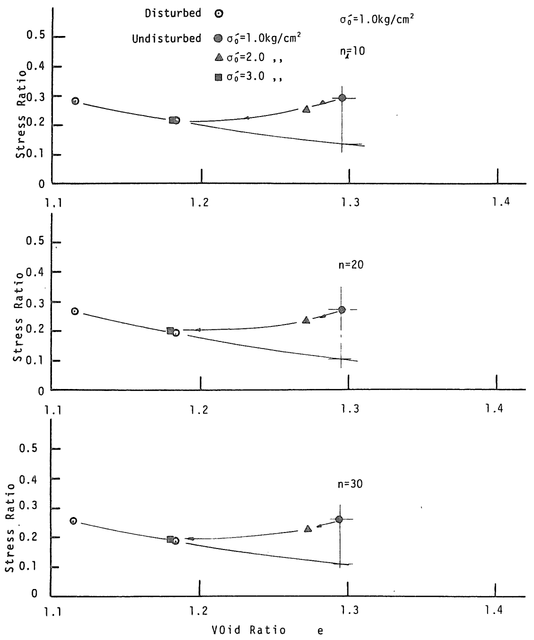


Fig. 23. Stress ratio in disturbed and undisturbed states for sample No. (M) VS void ratio

results shown in Fig.24 and the value of $(R_{nl=10})_{undis}$ increases proportionally with increasing value of E_{50}. It is necessary to note that the test data shown in Fig.25 are the result under the initial effective confining pressure of 1.0kg per Sq cm and if the initial confining pressure varies, the relationship shown in Fig.25 will change due to the influence of over consolidation ratio.

The relationship shown in Fig.25 may be used together with the results shown in Fig.26 and 27, (Fig. 26 shows the relationship of unconfined compression strength, q_u, versus modulus of deformation, E_{50}, for undisturbed samples used in this investigation, and Fig. 27 shows the relationship of E_{50} versus N-Value for many kinds of soils suggested by many investigators), to estimate the magnitude of cyclic stress ratio for undisturbed samples subjected to cyclic shear stress applications.

SUMMARY AND CONCLUSIONS

The investigation described herein are on the liquefaction characteristics of disturbed and undisturbed saturated sands under cyclic loading triaxial compression conditions. Based on the aforementioned experimental findings, the following conclusion are drawn:

1. The cyclic stress ratio obtained by using this apparatus have somewhat small value as compared with the value obtained by many other investigators.

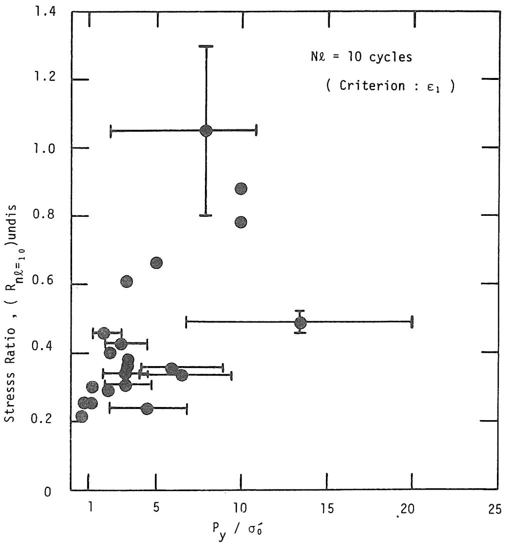


Fig. 24. Stress ratio VS P_y/σ_0'

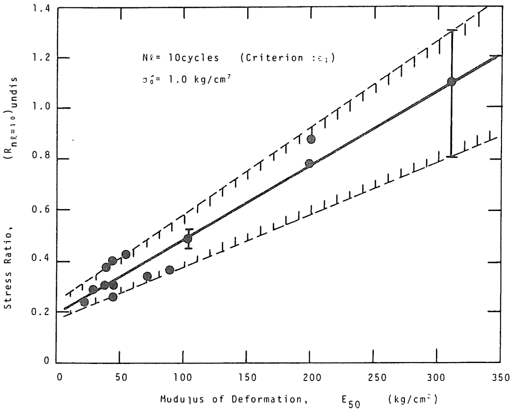


Fig. 25. Stress ratio VS modulus of deformation

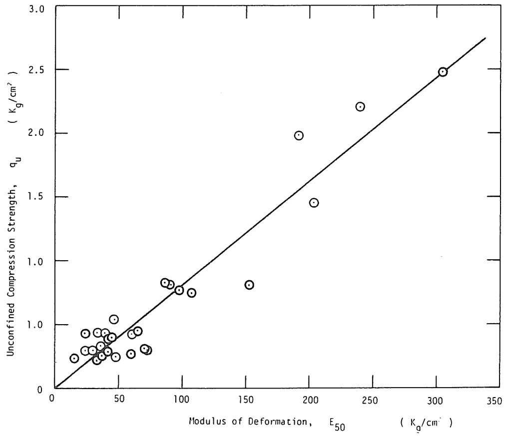
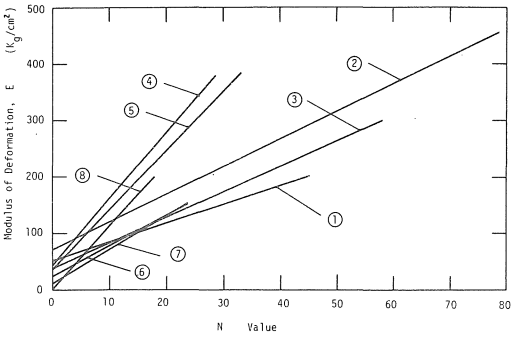


Fig. 26. Relationship between unconfining compression strength and modulus of deformation, unsaturated samples



① $E=52+3.3\,N$, Fine Sand (SW), $d=0.2\sim0.06$, $C_u=2$, $S_r<50\%$, $r=0.758$

② $E=71+4.9\,N$, Fine Sand (SW), $d=0.2\sim0.06$, $C_u=2$, $S_r<100\%$, $r=0.900$

③ $E=39+4.5\,N$, Sand (SW), $d=1.5\sim0.1$, $C_u=2$, $S_r<50\%$, $r=0.954$

④ $E=43+11.8\,N$, Sand with Gravel (SP-GP), $d=12\sim0.15$, $C_u=4$, $S_r<50\%$, $r=0.886$

⑤ $E=38+10.5\,N$, Gravel (GP), $d=63\sim0.06$, $C_u=60$, $S_r<50\%$, $r=0.783$

⑥ $E=24+5.3\,N$, Silty Sand (SM), $d=2.0\sim0.02$, $C_u=8$, $S_r<85\%$, $r=0.764$

⑦ $E=12+5.8\,N$, Silt, Fine Silty Sand (CL), $d=0.1\sim0.002$, $C_u=60$ $S_r<85\%$, $r=0.904$

⑧ $E=4+11.5\,N$, Silt, Clayey Silt (CL), $d=0.1\sim0.001$, $S_r<85\%$, $I_p>15$, $r=0.924$

(NOTE) C_u : Coefficient of Uniformity
S_r : Degree of Saturation
r : Correlation Coefficient
d : Grain Size (mm)

Fig. 27. Modulus of deformation VS N-value

This difference of test data lies in ranging from 0.03 to 0.09.

2. On the criterion of initial liquefaction, the procedures using pore-water pressure development and axial strain development have the same value of cyclic stress ratio. This finding observe in either samples, disturbed or undisturbed sample. The stress ratio decided at initial liquefaction state has a value of about 30 percents lower in maximum difference as compared with the results decided at complete liquefaction state.

3. The difference of sample preparation procedure influences on soil skeleton, interrocking between soil particle and antecedent stress.

4. Cyclic stress ratio on undisturbed samples such as tertiary era sands and diluvium sands have high values affected by cementation between soil particle and over consolidation. Further more, the relationship of cyclic stress ratio against relative density does not have any tendency for disturbed samples.

5. In case of showing the liquefaction resistance for undisturbed sample as a parameter of E_{50}, there is an almost linear relationship between cyclic stress ratio and E_{50}. For practical purpose, this experimental result indicates the possibility of estimating in-situ cyclic stress ratio from N-Value.

6. In comparison with the liquefaction charactor of disturbed samples, the mechanism of pore-water pressure development and magnitude of axial strain at onset of liquefaction for undisturbed sample differ by the influences described in 4.

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