Effects of Fines Content on Cyclic Shear Characteristics of Sand-Clay Mixtures

Les effets de la teneur en fines sur les caractéristiques du cisaillement répéte des mélanges de sable et argile

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ABSTRACT: The undrained cyclic shear behaviour of sand-clay mixtures with various fines content and compaction energy, has been investigated with reference to the sand structure void ratio. For a low fines content, sand particles dominate the soil matrix, whereas when the fines content is high, clay particles dominate the soil structure. For sand-clay mixtures, it was observed that the matrix structure of the coarse particles has a large effect on the undrained cyclic shear strength characteristics, and that the density of the sand structure is more significant than the fines content.

RÉSUMÉ : Le comportement au cisaillement cyclique non drainé des mélanges sable-argile avec diverses teneurs en fines etl' énergie de compactage ont été étudiés considerant l'indice des vides de la structure du sable. Pour une faible teneur en fines, les particules de sable dominent la matrice de sol, tandis que, quand la teneur en fines est élevée, les particules d'argile dominent la structure de sol. Pour les mélanges de sable et argile, on a observé que la structure de la matrice des grosses particules exerce un grand effet sur les caractéristiques de résistance au cisaillement répéte, et que la densité de la structure de sable est plus significative que la teneur en fines.

KEYWORDS: sand-clay mixtures, compaction energy, cyclic shear strength

1 INTRODUCTION

The gigantic 2011 Tohoku Earthquake with moment magnitude M_w 9.0, caused extensive damage to life and property in the Tohoku and Kanto regions in eastern part of Japan. In many locations, there was severe damage to residential land developed by cut and fill methods in hilly areas. In the Kanto area, over ten thousand houses suffered from serious settlements or tilting due to soil liquefaction. The foundation soils generally contained a lot of fines.

A study was therefore carried out on natural clay and sand mixed together in various proportions, giving a range of soil structures, ranging from sand to clay dominating the soil matrix. These were prepared by varying the amount of fines added. The undrained cyclic shear behaviour of sand-clay mixtures was then examined based on the concept of granular void ratio, and the variation in cyclic shear strength with increase in fines content was investigated. Based on the test results, a method for a unified evaluation of the cyclic shear strength of soils consisting of wide range of grain sizes (from sand to clay) was proposed.

2 MATERIALS USED AND EXPERIMENTAL PROCEDURE

2.1 Test Materials

Marine clay obtained from Iwakuni Port in Yamaguchi Prefecture (Japan) and Silica sand with adjusted grain size distribution were mixed in various proportions in order to form a wide range of soil types, i.e., from sand to clay. Initially, No. V5, No. R5.5, No. V6 and No. V3 Mikawa silica sands were mixed in 1:2:2:5 ratios by dry weight in order to modify the grain size distribution. The silica sand with adjusted grain size distribution had maximum and minimum void ratios of emax=0.850 and emin=0.524, respectively. For Iwakuni clay, soil particles larger than 0.425 mm were removed by sieving. Next, 8 types of soil mixture were produced by mixing the silica sand with adjusted grain size and sieved Iwakuni clay in different proportions based on dry unit weights. The engineering properties of the samples used are given in Table 1, and the grain size distribution curves are illustrated in Fig. 1. In its natural state, Iwakuni clay has a fines content Fc=98.0% and clay content Pc=38.8%. It has a plasticity index Ip=47.5, indicating a clay with medium plasticity. Since Iwakuni clay has a sand content of 2%, the fines content of each sample of the sand-clay mixture shown in the table is smaller than the Iwakuni clay content. From the table, when the fines content Fc>19.6%, the soil mixture has activity, while when Fc<16.7%, the mixture is considered as non-plastic.

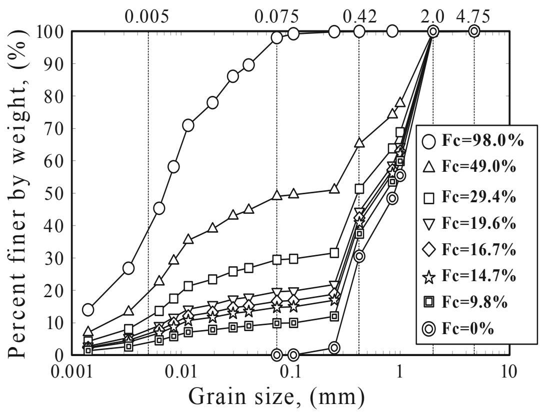


Fig.1: Grain size distribution curves of samples us

2.2 Specimen Preparation

Samples with an Iwakuni clay content of 20% or greater, were mixed at a water content of about twice the liquid limit, after which they was placed in a pre-consolidation cell measuring

Table. 1 Relation between dropping number and compaction energy

E_c (kJ/m^3)	Number of dropping per a layer					Height of dropping weight (m)	Weight of rummer (kN)
	1st	2nd	3rd	4th	5th		
504	60	80	100	120	140	0.184	0.00116
324	40	50	65	75	85	0.184	0.00116
113	14	19	23	25	29	0.184	0.00116
51	4	7	10	13	16	0.184	0.00116
22	5	10	15	25	30	0.050	0.00116

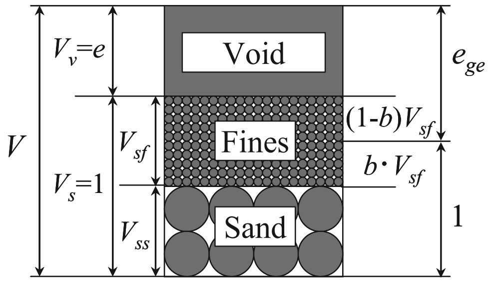


Fig. 2 Phase diagram

50mm in diameter and 200mm in height and subjected to a vertical pressure of 50 kPa. After consolidation, a specimen measuring 5 cm in diameter and 10 cm in height was formed.

To prepare specimen from samples with an Iwakuni clay content 17% or less, the soil mixtures were placed in a mold in 5 layers with each layer being compacted using a steel rammer with a prescribed number of blows. The compaction energy, E_c, was calculated as follows (Adachi et al., 2000):

$$E_c = \frac{W_R \cdot H \cdot N_L \cdot N_B}{V} \tag{1}$$

where, W_R is the rammer weight (=0.00116 kN), H is the drop height (m), NL is the number of layers (=5), N_B is the number of blows per layer, and V is the volume of mold (m^3). Various compaction energies, E_c, were obtained by changing H and N_B. Each soil mixture was thoroughly mixed at an initial water content of w=11%. Given that the initial water content of a compacted soil sample does not change the properties of the clay, the water content was selected to allow the free passage of carbon dioxide. In order to improve the specimen saturation, carbon dioxide was first passed through the specimens before allowing water to percolate.

In this research, soil samples with fines content F_c=0, 9.8, 14.7, 16.7% were formed at constant compaction energies, corresponding to E_c=22, 51, 113, 324, 504, 1008 kJ/m^3, and at constant relative densities of sand structure, i.e., D_{rsc}=0, 30, 50, 65%. (Table.1)Note that for soil samples prepared under compaction energies of E_c=504 and 1008 kJ/m^3, the variation of granular void ratios with fines content is practically similar; therefore, considering the rammer used in the sample preparation, relative densities of the sand-clay mixtures cannot be changed even with higher compaction energies. Thus, the granular void ratios corresponding to these compaction energies can be considered as the minimum granular void ratios in these experiments.

As mentioned earlier, for soil samples with fines contents greater than F_c=19.6%, sample preparation by compaction was not possible; instead, the pre-consolidation method was employed. Depending on the normal consolidation condition, the granular void ratio of the pre-consolidated sample is unique for a given effective confining pressure.

2.3 *Skeletal structure of sand-clay mixture*

A fully saturated sand-clay mixture has a three-phase composition, namely the coarse-grained particles, fine-grained particles and pore water, as shown in Fig. 2. Here, *b* is defined as the portion of fines that contributes to the active intergrain contacts. In Thevanayagam et al. (2002), it was introduced to represent the beneficial secondary cushioning effect of silica silts in silty sand as follows:

$$e_{eq} = \frac{V_v + (1-b)V_{sf}}{V_{ss} + bV_{sf}} = \frac{V_v + (1-b)V_{sf}}{V_s - (1-b)V_{sf}} \tag{2}$$

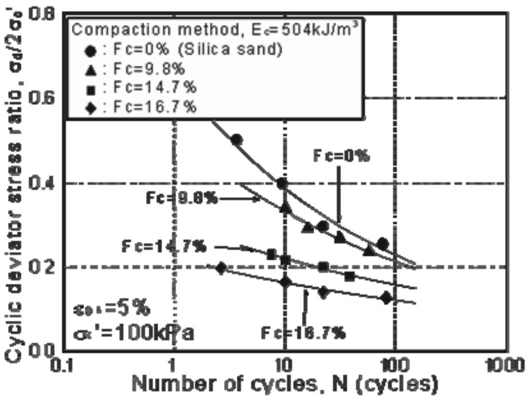
where e_{ge} is the equivalent granular void ratio, F_c is the fines content (in terms of volume) and b denotes the portion of the fines that contributes to the active intergrain contacts. Basically, *b*=0 means that none of the fine grains actively participates in supporting the coarse-grain skeleton (i.e. the fines act exactly like voids); and *b*=1 implies that all fines actively participate in supporting the coarse grain skeleton (i.e. the fines are indistinguishable from the host sand particles). The magnitude of *b* depends on grain size disparity and grain characteristics.

3 UNDRAINED CYCLIC SHEAR PROPERTIES

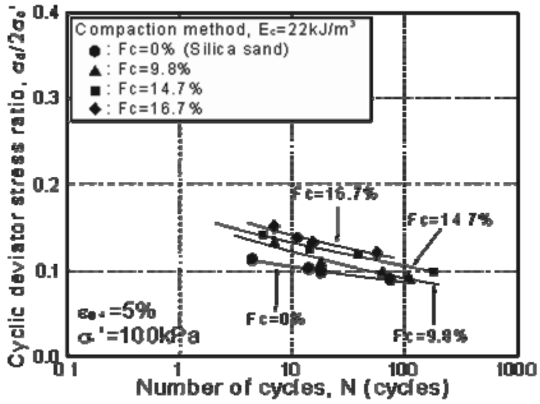
The specimens of sand-clay soil mixtures prepared by compaction and pre-consolidation methods were isotropically consolidated at an effective confining pressure of σ_c'=100 kPa. Then, undrained cyclic triaxial tests were conducted with effective confining pressure σ_c'=100 kPa and loading frequency f=0.02Hz using an air pressure controlled cyclic triaxial test apparatus.

Fig. 3 shows the relationship between the cyclic shear strength ratio (σd/2σc') required to cause double amplitude axial strain ε_{DA}=5% and the number of cycles (N) for soil specimens with fines mixtures (Iwakuni clay). Based on Fig. 3(a), for specimens prepared under high compaction energy E_c=504 kJ/m^3, the liquefaction strength decreases as the fines content increases, with the liquefaction strength of specimens with F_c=14.7% significantly smaller than those of specimens with F_c=0% and 9.8%. On the other hand, for specimens prepared under a low compaction energy E_c=22 kJ/m^3 (Fig. 3(b)), although the difference between the liquefaction curves of various samples are small, there is a tendency for the liquefaction strength to increase with an increase in fines content. Next, the results of specimens prepared under constant fines content F_c=0% (Fig. 3(c)) shows that the liquefaction strength increases as the compaction energy increases, with the liquefaction strength of specimen with E_c=22, 51 kJ/m^3 significantly smaller than those of specimens with $E_c \geq$ 113 kJ/m^3. On the other hand, for specimens prepared under constant fines content F_c=16.7% (Fig. 3(d)), it is also observed that there is increase in strength as the compaction energies increased. However, the liquefaction strengths show almost similar values. Fig. 3(e) shows the result of the pre-consolidation samples for Iwakuni clay mixtures ($F_c \geq$19.6%). The cyclic shear strength ratios increase with fines content. However, in this case the liquefaction strengths show almost similar values ($F_c \geq$29.4%).

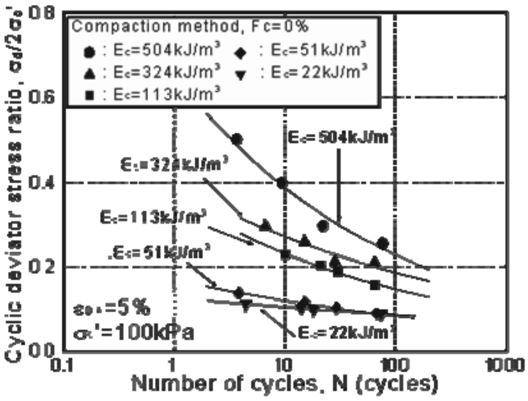
The cyclic shear strength corresponding to 20 cycles (hereinafter referred to as cyclic shear strength ratio, $R_{L(N=20)}$) is read off from the above curves and plotted against the fines content for Iwakuni clay mixtures, as shown in Fig. 4. At constant relative density of sand structure, on the other hand, any increase in fines content results in an increase in cyclic strength. This can be explained as follows. In the case of constant relative density of sand structure, more fines will occupy the voids between the sand particles when the fines



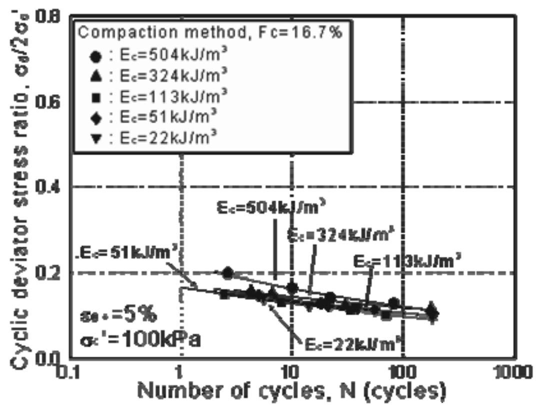
(a) E_c=504kJ/m^3 (at constant compaction energy)



(b) E_c=22kJ/m^3 (at constant compaction energy)



(c) F_c=0% (at constant fines content)



(d) F_c=16.7% (at constant fines content)

content is high, resulting in an increase in density of the specimen leading to an increase in cyclic strength.

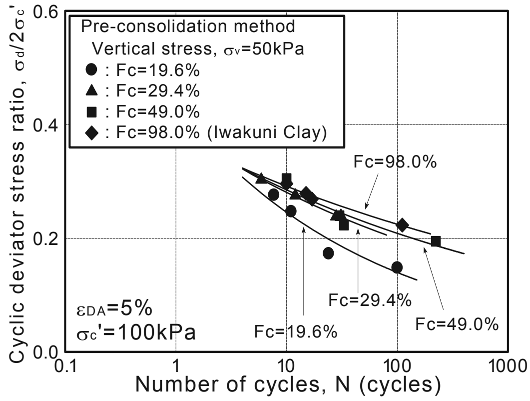
4 RELATIONSHIP BETWEEN SKELETAL STRUCTURE AND CYCLIC SHEAR STRENGTH

Based on the results of the cyclic shear tests discussed above, it can be summarised that the fines content and the resulting skeletal structure have significant effects on the cyclic strength of clay-sand mixtures. By assigning different values of b, the void ratios are re-calculated. From the results, a b value of 0.3 indicates that almost all the data falls into a narrow band that surrounds the data for clean sand (F_c=0%), as illustrated in Fig. 5. This observation suggests that for F_c<20%, the effect of fines on the cyclic shear strength is about 30% of that of sand for Iwakuni clay.

5 EVALUATION OF LIQUEFACTION RESISTANCE BY EQUIVALENT GRANULAR VOID RATIO

It is widely known that soil density has significant effects on the engineering properties of sandy soils. Due to the effects of the size and shape of soil particles and uniformity, however, the compactness of sandy soils can not be evaluated reasonably only by the in-site dry density. Relative density is computed from the maximum and minimum void ratios (dry densities) and in-site void ratio (dry density). The maximum and minimum dry void ratio are determined according to a standardized method (for example, ASTM D4253 and D4254, JIS A 1224). However, it can only be applied to a small fine (F_c<5%) sandy soil. In Fig. 6, the relationship between equivalent granular relative density and $R_{L(N=20)}$ was illustrated for all the soil mixtures used. The equivalent granular relative density D_{rge}, is defined as:

$$D_{rge} = \frac{e_{g\,max} - e_{ge}}{e_{g\,max} - e_{g\,min}} \times 100 \quad (\%) \tag{4}$$



(e) F_c=19.6~98.0% (pre-consolidation specimen)

Fig. 3. Cyclic shear strength curves

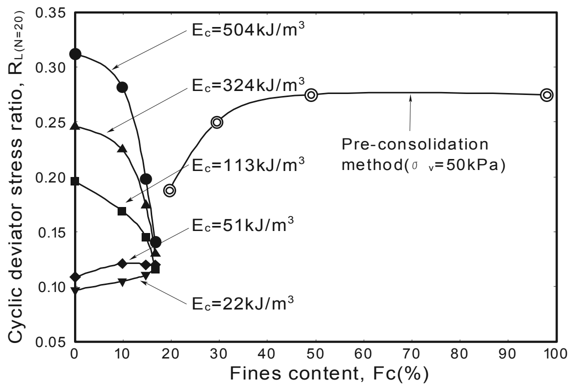


Fig. 4 Correspond to constant compaction energy and pre-consolidation specimens

where, $e_{gmax,}$ and $e_{gmin,}$ are the maximum and minimum void ratios of the host sand (without any fines), respectively, and e_{ge} is the equivalent granular void ratio. In Fig. 6, if the equivalent void ratio in Fig. 5 is re-calculated by assigning a D_{rge} for sand-clay mixture, the results indicated that all the data falls within a small band, including those for clean sand (F_c=0%). This was done for all of the specimens from a clean sand to a sand with less than 20% fines.

As may be seen from Fig. 6, fairly good correspondence is recognized between the predicted and experimental results in spite of various fine contents. It was confirmed that the proposed model is a reasonable method for liquefaction resistance of sandy soils subjected to various magnitudes and equivalent granular relative density.

6 CONCLUSIONS

Undrained cyclic shear tests were performed in order to investigate the effects of fines content on the liquefaction strength of sand-clay soils as foundations of dykes and embankments. The following were the major conclusions obtained in this study.

1. For soil samples with F_c >20%, the strength is governed by the fines, which control the soil matrix.
2. Increasing fines content results in a decrease in cyclic strength for dense soils, while an increase in strength is observed for loose soils.
3. As the fines content exceeds F_c=19.6%, the cyclic shear strength increases rapidly, and when fines content is greater than F_c=50%, the liquefaction strength asymptotically approaches the liquefaction strength of the clay particles. This is because the fines form the structure of the soil matrix as the fines content increases.
4. When sand particles dominate the soil matrix, a unified formula representing the cyclic shear strength and equivalent granular void ratio for the sand-clay mixtures with various sand structures and fine contents is presented.

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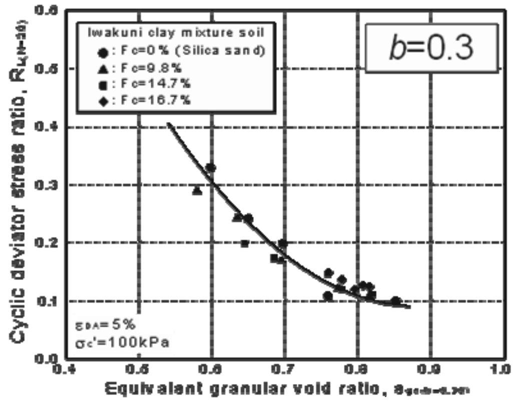

Fig. 5 Relationship between cyclic shear strength ratio, $R_{L(N=20)}$ and Equivalent granular void ratio, e_{ge}

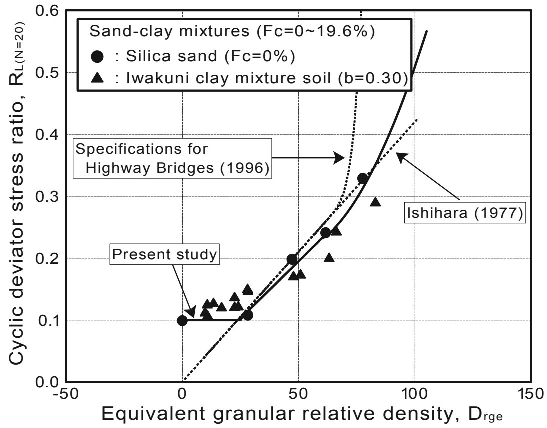

Fig. 6 Predicted and experimental cyclic shear strength ratio, $R_{L(N=20)}$ when equivalent granular relative density, D_{rge}