Undrained cyclic shear characteristics of silt and sand mixtures

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ABSTRACT

In most design codes, soils are classified as either sand or clay, and appropriate design equations are used to represent their behaviour. For example, the behaviour of sandy soils is expressed in terms of the soil's relative density, whereas consistency limits are often used for clays. However, sand-clay mixtures, which are typically referred to as intermediate soils, cannot be easily categorized as either sand or clay and therefore a unified interpretation of how the soil will behave at the transition point, i.e., from sandy behaviour when fines are few to clay behaviour for high fines content, is necessary. In this paper, the cyclic shear behaviour of sand-silt mixtures was investigated by considering variations in fines content and compaction energy, while paying attention to the void ratio expressed in terms of sand structure. Then, by using the concept of equivalent granular void ratio, it was noted that the contribution of silt on the cyclic shear strength of the soil was about 43% of that of sand.

1 INTRODUCTION

In designing soil structures, soil is classified for simplicity as either sand or clay based on fines content or the particles dominating the soil structure. For cohesionless soils, such as sands and silts, the forces between the soil particles arise from friction between these particles as they make contact with one another. For clayey soils, on the other hand, the main forces arise from the electric repulsion through the adsorbed water layer existing between clay particles. Thus, because the states of forces transmitted between soil particles in sand, silt and clay are different, the state of packing (or density) is important for sand, while consistency limits affect clay behaviour.

However, most soils in natural state are generally composed of combinations of sand, clay, silt, etc. It is hard to classify such soils into either sand or clay, because they possess both properties of sand and fines. These kinds of soil are called *intermediate soils*. The properties of intermediate soils change in various ways due to the effect of density or fines content. Therefore, difficulties arise in understanding their dynamic characteristics and liquefaction potential. The objective of this research is to demonstrate the monotonic and cyclic shear strength characteristics of intermediate soils.

Kim et al. (2006) reported the undrained monotonic shear behaviour of sands mixed with active clays while Hyodo et al. (2006) discussed their cyclic shear characteristics. In this paper, non-plastic silt and sand are mixed together at various proportions, and a wide range of soil structures, ranging from one with sand dominating the soil structure to one with silt controlling the behaviour, was prepared by varying the amount of fines. Then, using the concept of granular void ratio, undrained cyclic shear tests on sand-silt mixtures were performed.

2 MATERIALS AND METHODS

In the experiments, non-plastic silt (Tottori silt) and silica sand were mixed at various proportions. The specimens of sand-silt soil mixtures were prepared by moist tamping method with initial water content set at w=11%. The soil mixtures were placed in a mold in 5 layers with each layer compacted using a steel rammer at a prescribed number of blows. The compaction energy, E_c, was calculated as follows (Adachi et al., 2000):

$$Ec = \frac{W_R \times H \times N_L \times N_B}{V} \quad (1)$$

In the above equation, W_R is the rammer weight (=0.00116 kN), H is the drop height (m), N_L is the number of layers (=5), N_B is the number of blows per layer, and V is the volume of mold (m^3). In the experiment, various compaction energies, E_c, were obtained by changing H and N_B. In the tests presented herein, two levels of compaction energy were used: E_c=22 kJ/m^3 and E_c=504 kJ/m^3.

A series of cyclic triaxial tests was conducted on these specimens with effective confining pressure σ_c'=100 kPa and loading frequency f=0.02 Hz using an air pressure controlled-type cyclic triaxial test apparatus. Figure 1 shows the void ratio, e, vs. fines content, Fc, curves. The ranges of void ratios (maximum and minimum) obtained from tests are also indicated in the figure. It is observed that for the initial condition, the void ratios of specimens with E_c=22 kJ/m^3 are larger than those at E_c=504 kJ/m^3 for all fines contents. After consolidation, however, the trend changed at Fc>20%. This is because during the saturation process, remarkable volumetric compression occurred on specimens with high silt content or with loose initial density. The void ratio of both specimens after consolidation is the lowest at Fc=20~30%.

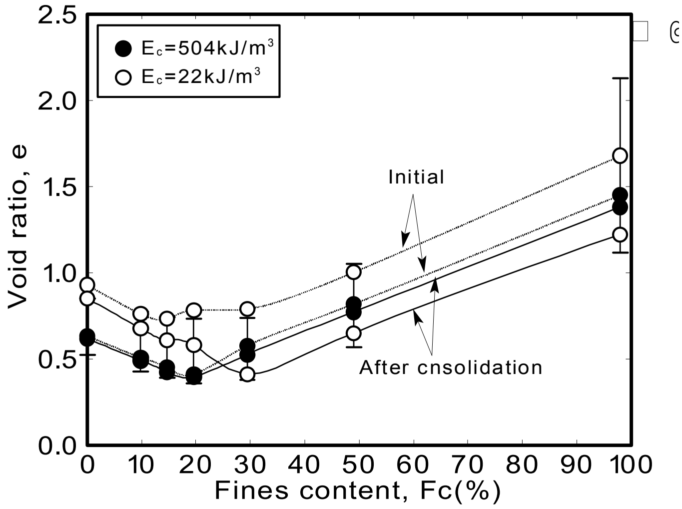

Figure 1: Void ratio vs. fines content curves

Since the structure of the coarse-grained soil greatly influences the strength characteristics of the soil mixture, it is believed that it is more appropriate to pay attention to the specimen density rather than the amount of fines. Therefore, it was attempted to understand the state of the sand structure formed within the soil mixture with only the coarse particles by considering the fines as voids and using the concept of granular void ratio, which is defined as (Mitchell, 1976, Kenney, 1977)

$$e_g = \frac{V_w + V_{sc}}{V_{ss}} \quad (2)$$

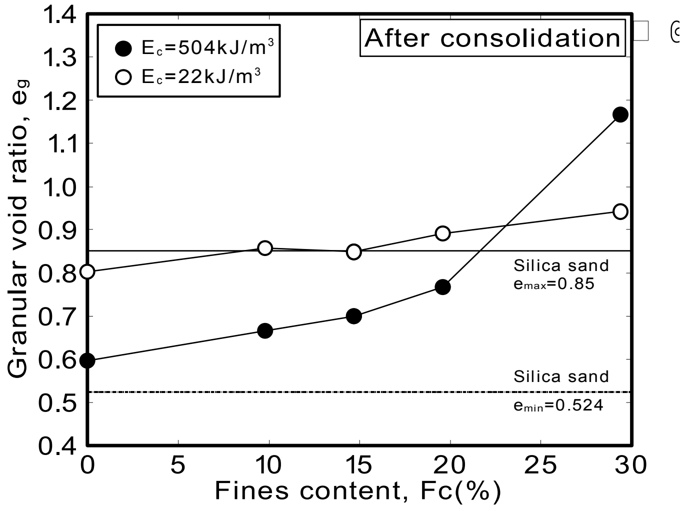

Figure 2: Relation between granular void ratio and fines content for samples after consolidation

where e_g is the granular void ratio of the sand-clay mixture, V_w is the volume of water, V_{ss} is the volume of the coarser grains and V_{sc} is the volume of the finer grains.

Figure 2 provides the relation between granular void ratio, e_g, and fines content, Fc, after consolidation. From the figure, the change in e_g for samples prepared under E_c=22 kJ/m^3 with the increase in Fc is small, while for E_c=504 kJ/m^3, the increase in e_g with increase in Fc is large. However, the silt in fact provides some positive contribution to the strength because of its structure. It is necessary therefore to evaluate how much the fines affect the strength with respect to the coarse particles. For this purpose, the concept of equivalent granular void ratio, e_{ge}, is used. The equivalent granular void ratio is calculated as follows (Thevanayagam et al., 2002)

$$e_{ge} = \frac{e + (1-b)Fc}{1-(1-b)Fc} \tag{3}$$

In the above equation, e is the void ratio, Fc is fines content (%), and b is defined as the portion of fines that contributes to the active inter-grain contacts. Thevanayagam et al. (2002) proposed that b should be $0 \leq b \leq 1$. When b=0, the fines act exactly like voids and when b=1, the fines are indistinguishable from the host sand particles.

3 TEST RESULTS

Figure 3 illustrates the relation between cyclic shear strength ratio required to cause double amplitude axial strain ε_{DA}=5% and number of cycles, N, for soil specimens with constant compaction energy E_c=22 and 504 kJ/m^3. From the figure, although the difference between the liquefaction curves of various samples are small for specimens prepared under E_c=22 kJ/m^3, there is a tendency for the liquefaction strength to increase with increase in fines content. On the other hand, for specimens prepared with E_c=504 kJ/m^3, the liquefaction strength increases as the fines content increases until Fc=14.7%, after which it decreases.

Next, the cyclic shear strength corresponding to 20 cycles is read from the above curves and regarded as cyclic shear strength ratio, $R_{L(\mathrm{N=20})}$. In Figure 4, the cyclic shear strength ratio, $R_{L(\mathrm{N=20})}$ is plotted against fines content, Fc. For specimens prepared under low compaction energy (E_c=22 kJ/m^3), any increase in fines content results in increase in liquefaction strength.

In contrast, for specimens prepared under high compaction energy (E_c=504 kJ/m^3), liquefaction strength shows complex change as the fines content increases.

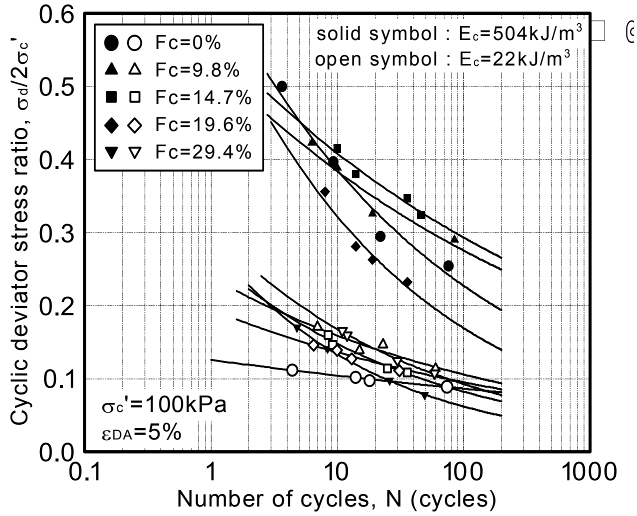

Figure 3: Cyclic shear strength curves

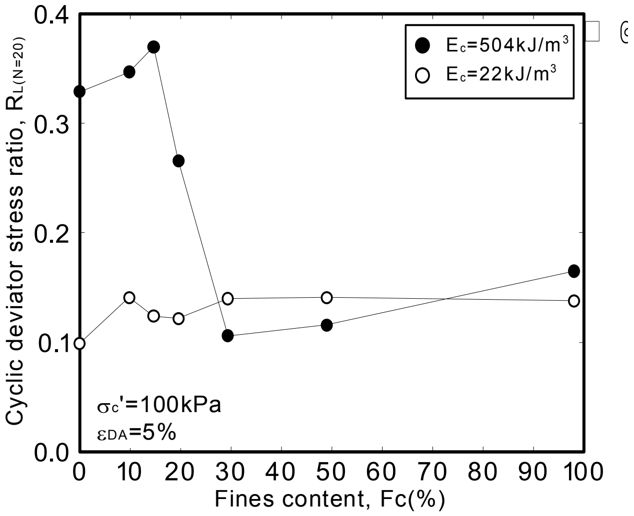

Figure 4: Cyclic shear strength ratio plotted against fines content

Figure 5 is the plot of cyclic shear strength ratio, $R_{L(\mathrm{N}=20)}$ with granular void ratio, e_g, focusing on Fc=0~29.4%. It can be seen that although the granular void ratio increases, the cyclic shear strength also increases when Fc=0~14.7%. In contrast, after Fc=14.7% the cyclic shear strength decreases as the fines content increases.

To derive a more consistent relation, the concept of equivalent granular void ratio was introduced and the appropriate value of the parameter b was analysed by least square method. In Figure 6, the variation in cyclic shear strength, $R_{L(\mathrm{N}=20)}$ with respect to equivalent granular void ratio, e_{ge} is illustrated. As a result of changing the parameter b, good correlation between $R_{L(\mathrm{N}=20)}$ and e_{ge} was obtained at around b=0.43. This observation suggests that for Fc<29.4%, the effect of silt on the cyclic shear strength is about 43% of that of sand.

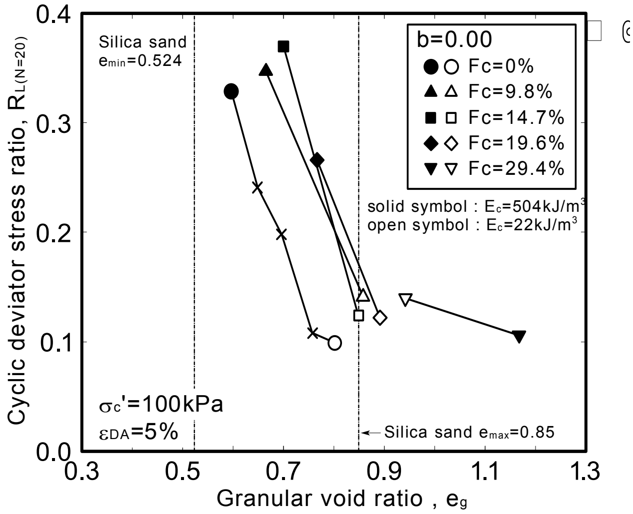

Figure 5: Relation between cyclic shear strength ratio and granular void ratio

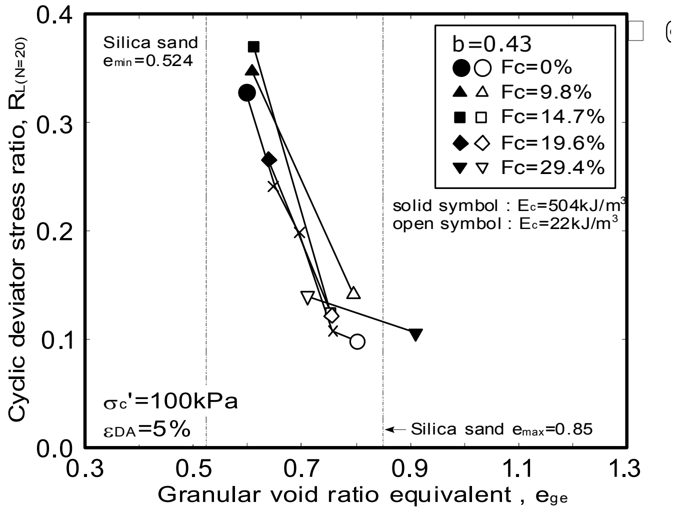

Figure 6: Variation of cyclic shear strength ratio with equivalent granular void ratio

4 CONCLUSIONS

Undrained cyclic shear tests were performed on silt-sand mixtures with various fines contents. Based on the test results, the following conclusions were reached.

(1)For specimens prepared under E_c=22 kJ/m^3, any increase in fines content scarcely affected the undrained cyclic shear strength of soil mixture.

(2)For specimens prepared under E_c=504 kJ/m^3, the undrained cyclic shear strength underwent complex change with increase in fines content.

(3)Regarding the coarse-grained soils comprising the soil mixture, cyclic shear strength ratio had a good relation with void ratio just until about *Fc*=15%; on the other hand, it had a good relation with granular void ratio only after *Fc*=15%. Based on the concept of equivalent granular void ratio, there was good correlation between cyclic shear strength ratio and equivalent granular void ratio at around *b*=0.43. It meant that the contribution of silt on the cyclic shear strength of the soil was about 43% of that of sand.

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