Evaluation of Mechanical Properties of SCP Composite Ground

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ABSTRACT:Sand Compaction Pile (SCP) method has been used widely in Japan as a countermeasure against liquefaction. Its effectiveness has been confirmed at many sites that have suffered large earthquakes in the recent past. In the current evaluation method, the SPT-*N* value at a midpoint of a rectangular area surrounded by four adjacent sand piles, at which liquefaction resistance is considered to be the smallest, is set as a criterion for the degree of compaction. The main drawback of this method is that the characteristics of SCP composite ground are not considered. For that reason, in the research reported in this paper, a new evaluation method was developed for SCP composite ground: Parallel Element Test (PET). This testing method uses two elements to represent the sand pile and its surrounding improved ground. The characteristics of SCP composite ground under undrained cyclic shear loading were evaluated using a multiple series of PETs with two hollow cylindrical torsional shear apparatus. The results of the subsequent tests reveal that the most influential factor for evaluating SCP composite ground at a low area replacement ratio is the coefficient of total lateral pressure.

Key Words:

1 INTRODUCTION

Compaction methods have been widely used in Japan as a countermeasure against liquefaction. Sand Compaction Pile (SCP) method is considered an extremely reliable method; it has been used extensively to ameliorate liquefaction resistance of loose sandy deposits. Its effectiveness has been confirmed at many sites that have suffered powerful earthquakes in the recent past (Ohbayashi et al., 1999; Okamura et al., 2003). Increased soil density and lateral effective stresses are considered to enhance the liquefaction resistance of foundation soils.

In current practice, the general aspects of the execution of SCP are done using an empirical chart, which illustrates the relationship between area replacement ratio and SPT-*N* values measured between sand piles. The chart was established using numerous observed SPT-*N* values of improved ground samples. It uses the SPT-*N* value at a midpoint of a rectangular area surrounded by four sand piles, under which conditions, liquefaction resistance is considered to be the smallest. That value is set as a target criterion for the degree of compaction.

Some limited information isavailable regarding examination of the empirical method mentioned above. Few related research projects have been undertaken within the last few years, despite laboratory studies and instrumented field applications. Ohbayashi et al. (1999), using data from 16 improved ground samples, showed that the Swedish Weight Sounding method(N_{sw}) decreases with horizontal distance from the sand pile. In contrast, Okamura et al. (2003) found that SPT-*N* values and Rotary Ram Sounding testvalues (RRS-N_d) at all depths show violent fluctuation. A clear tendency of a decrease in SPT-*N* values and RRS-N_d values over horizontal distance is undetectable. These observations imply that the improved ground is highly heterogeneous and that SPT-*N* values or RRS-N_d values are distributed randomly in a horizontal plane. Furthermore, liquefaction resistance, obtained from laboratory experiments are randomly distributed in the improved ground at all depths, is consistent with observations of SPT-*N* values and RRS-N_d values (Okamura et al., 2003). Therefore, the SPT-*N* value at a midpoint of a rectangular area surrounded by four adjacent sand piles does not always provide a conservative evaluation of the liquefaction resistance of improved ground. In addition, stability and deformation characteristics of SCP composite ground under earthquake-induced motion were not elucidated sufficiently both experimentally and numerically. Therefore, it is important at present to develop a more rational evaluation method considering the characteristics of SCP composite ground.

To overcome the shortcomings described above, the author developed a new methodto evaluate the liquefaction resistance of SCP composite ground under undrained cyclic shear deformation. This testing method is the Parallel Element Test (PET), which includes two elements to represent the sand pile and surrounding improved ground. Element tests, which are

conducted to find densification characteristics resulting from SCP installation, are described first. Stress state and volume changes at a point between sand piles during SCP installation are evaluated using a cyclic shear triaxial apparatus. The coefficient of lateral pressure (K), which is defined as the ratio of effective horizontal stress over effective vertical stress, has been used as a parameter to indicate the effects of improvement of horizontal stress on liquefaction resistance. Further effects of fine contents on improvement of densification were evaluated quantitatively. PET was conducted based on preliminary element test results using a two hollow cylindrical torsional shear apparatus. Basic concepts of the proposed method and laboratory experimental arrangements are described. Finally, test results are used to verify the applicability of the proposed method for assessing liquefaction resistance of the SCP composite ground.

2 STRESS STATE AND VOLUME CHANGE DURING SCP INSTALLATION

It is usually believed that effectiveness of the SCP method is dependent on the contribution of reconstitution of sand particles and the densification of sand piles as a result of casing driving and compaction (pile-making process). To elucidate the improvement of densification attributable to SCP installation in surrounding ground, a soil element between sand piles was considered and stress path changes in SCP installation were reproduced using a cyclic shear triaxial apparatus.

2.1 Materials and Methodology

Actual soil often comprises loose sandy soil with a certain fraction of silt and clay; it was expected that compaction occurrence might vary depending on fine content (FC). To test that inference, artificial materials with different fine contents were prepared; tests using different types of such materials were carried out. Toyoura sand was mixed with Soma sand to create soil specimens having fine contents by weight of 0, 10, 20, 30, and 40 %.

Sand specimens which represent the unimproved (original) ground before SCP installation were prepared using air-pluviation method to obtain low relative density of about 20 %. This method leads to formation of inherent anisotropy in the specimen with the bedding plane in the horizontal direction. In triaxial tests, soil specimens were prepared in a 5 cm diameter and 10 cm high cylindrical mold. After the sets of apparatus were prepared, specimens were preconsolidatedisotropically under 20 kPa and were saturated by circulating carbon dioxide and de-aired water.Saturation was completed with back pressure of 100 kPa. The specimens were accepted as fully saturated for this research study when Skempton's B parameter became greater than 0.95.

Saturated samples were consolidated anisotropically by opening the drainage valve. The volume decreased because of pore water drainage. Anisotropic consolidation was carried out with deviator stress (q) of 50 kPa and cell pressure of 50 kPa, implying that K = 0.5. After completion of consolidation, axial-strain controlled testing was performed at a frequency of 0.1 Hz using a sinusoidal waveform, as shown in Fig. 1. The cyclic loading history, which gradually increased the axial strain (ε_a) amplitude, requires six steps, each of which consists of 10 cycles with constant amplitude. The step amplitudes were 0.1, 0.2, 0.4, 0.8, 1.6, and 3.2 %, respectively, in steps 1, 2, 3, 4, 5, and 6. The purpose of selection of this type of cyclic loading history was to obtain a large area replacement ratio to simulate the actual SCP installation. In other words, to obtain large volumetric strain (ε_v) with reconsolidation, a sufficient strain history was applied to test specimens under the undrained condition. It was assumed that ε_v is equal to the area replacement ratio (a_s) because no ground upheaval was detected under laboratory conditions (Priyankara, 2006).

The results of the triaxial tests were measured in terms of $q$andε_a, and converted to shear stress (τ = 0.5q) and shear strain (γ = 1.5ε_a) using the conventional theory of elasticity under an undrained condition. Typical test results of stress-strain path and effective stress path are shown in Fig. 2, as obtained from FC = 20 %. In this case, the initial shear stress was 25 kPa, as implied by the initial vertical stress σ_v= 25 kPa and K = 0.5. As loading proceeds, keeping the horizontal stress (σ_h) constant, the generated pore pressure was increased to the level of confining pressure (horizontal stress); reduction of mean effective stress occurs, which is accompanied by softening of the specimen (σ_h= σ_v= 50 kPa). Fig. 3(a) depicts the relationship between $\sigma_v$and σ_h, whereas Fig. 3(b) illustrates the relationship betweenε_v and mean effective stress (σ_m'). As shown in Fig. 3(a), before opening the valve of the drainage system, σ_v was increased to its initial value (σ_v=100 kPa) from the isotropic state to maintain vertical displacement equal to zero by controlling the cell pressure. As a result, the K value was increased because of the increase ofeffective horizontal stress (σ_h'). Then the drainage system valve was opened to allow

pore pressure to dissipate by keeping the σ_v constant as its initial value by increasing the cell pressure (Fig. 3(a)). Pore water was drained at an average volumetric strain rate of 0.05 %/min. Particular attention is given to the influence on the amount of shear strain imposed on the soil specimen on the drain volume changes. After pore water pressure was fully dissipated, the drainage system valve was closed, thereby doubling the amplitude of axial strain; the same test procedure was repeated until excessive deformation occurred (specimen failed).

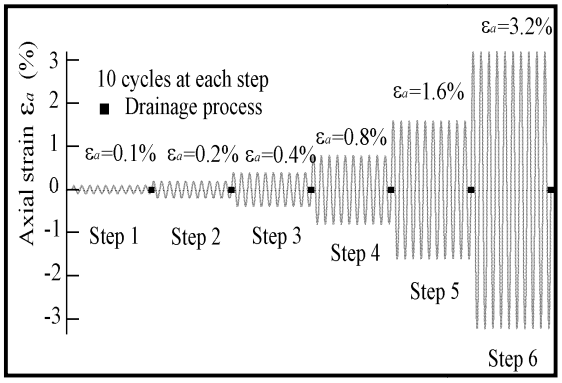

Fig 01Loading history of axial strain applied to the specimen

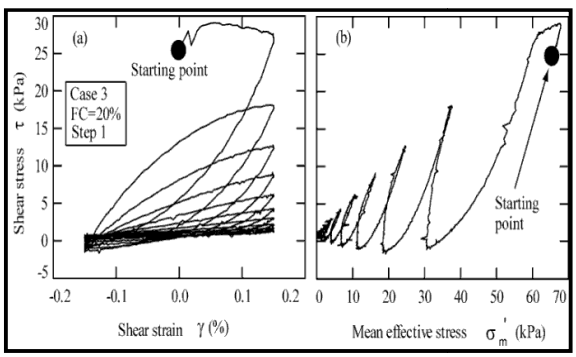

Fig 02Axial strain controlled cyclic shear triaxialtest under undrained condition ((a) stress- strain relationship (b) effective stress path)

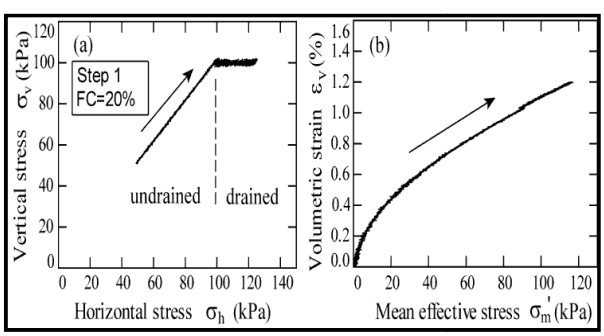

Fig 03Normal stress changes and volume changes during drainage

2.2 Relationship between ε_v and K

Densification characteristics attributable to SCP installation can be described in terms of reconsolidation volumetric strain followed by liquefaction. It was found that ε_v increased gradually with accumulated shear strain, irrespective of fine content. Similarly,K value increases considerably over small accumulated shear strain, irrespective of fine content. It achieves a constant value at higher strain levels (Priyankara, 2006).

As explained above, ε_v can be replaced by a_s if ground upheaval during SCP installation is neglected. A plot of K values with a_s, as shown in Fig. 4, shows that the K value increases sharply with small a_s; it increases gradually with larger a_s. The data depicted in Fig. 4 are used to conduct PET, as described in subsequent sections.

3 CYCLIC SHEAR DEFORMATION BEHAVIOUR OF SCP COMPOSITE GROUND

To overcome the shortcomings in the conventional evaluation method of SCP improved ground, the author propose a new method to evaluate SCP composite ground characteristics under earthquake-induced cyclic shear motion. Two soil elements, which respectively represent the sand pile and surrounding improved ground, were used in this study. It is conservative to assume thattotal σ_v in the sand pile and surrounding improved ground are equivalent at a certain depth (Priyankara, 2006), $i.e. \sigma_{v(im)} = \sigma_{v(scp)}$, where $\sigma_{v(im)}$ and $\sigma_{v(scp)}$ respectively represent the total σ_v in the surrounding improved ground and in the sand compaction pile.

The author introduce a new parameter, designated as the coefficient of total lateral pressure (K_T), to identify characteristics of SCP composite ground during cyclic shear motion. Where σ_h represents total horizontal stress and σ_v is the total vertical stress, K_T is defined as σ_h / σ_v . It was assumed that K_T values in the sand pile and surrounding improved ground change identically according to cyclic loading. This presumption implies that σ_h in SCP ground is equal in both media because the ground behaves as one unit after SCP installation, $i.e. \sigma_{h(im)} = \sigma_{h(scp)}$, where $\sigma_{h(im)}$ is the total horizontal stress in the surrounding improved ground and $\sigma_{h(scp)}$ is the total horizontal stress in the sand pile.

Fig. 5 shows the possibility of defining SCP composite ground using two elements based on the above assumptions and explanations. The element that represents the sand pile has a higher relative density, whereas the element that represents the surrounding improved groundhas a lower relative density depending on a_s. Because SCP-improved ground behaves as composite ground, it was assumed that shear strain that occurred because of earthquake-induced motion in the sand pile (γ_{scp}), and that the shear strain in the surrounding improved ground (γ_{im}) at a particular depth is equal (Priyankara, 2006), i.e. $\gamma_{scp} = \gamma_{im} = \gamma_{com}$, where γ_{com} is the shear strain in the composite ground. Because of application of shear strain in the SCP improved ground, the shear stress developed separately in the sand pile and in the surrounding improved ground according to their physical properties. The shear stress in the composite ground can be expressed as

$$\tau_{com} = \tau_{scp} a_s + \tau_{im}(1 - a_s) \qquad (1)$$

where τ_{com} is the shear stress in the composite ground, τ_{scp} is the shear stress in the sand compaction pile, τ_{im} is the shear stress in the surrounding improved ground.Eq. (1) was used as the basic equation in this concept. Two soil elements with different physical properties were subjected to equivalent shear strain at the same time. This is the usual procedure of PET.

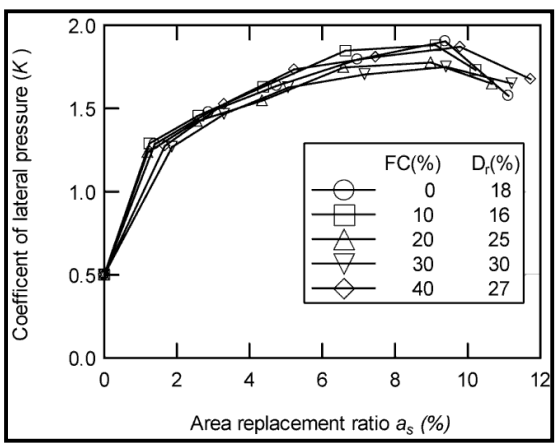

Fig 04 Relationship between K and a_s

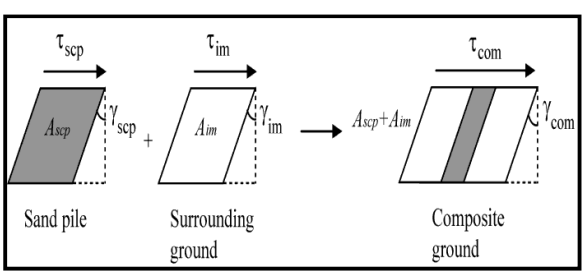

Fig 05Shear deformation behaviour of SCP composite ground

Test equipment and initial conditions

To elucidate characteristics of the SCP composite ground, multiple PET series were conducted using a two hollow cylindrical torsional shear apparatus. The complete experimental setup is depicted in Fig. 6.

Relative density (D_r) and K after SCP installation are given as initial conditions for all PET test cases. The values of K and D_r corresponding to $a_s$were obtained from the preliminary element tests based on characteristics of the volume change during SCP installation (Fig. 4). TheD_r and K values in the SCP improved surrounding ground were changed according to a_s, as listed in Table 1. In a rest condition, *i.e.*, after SCP installation,(K_T) was assumed to be equal to K. Also, $a_s = 0\,\%$ indicates the unimproved ground whereas a_s = 3, 6 and 10% indicate the SCP improved surrounding ground.

The $D_r$of the sand pile was assumed to be 90 %; $D_r$of the unimproved ground was assumed to be 20 % based on field data published by Okamura et al. (2003). The relative density of the surrounding improved ground ($D_{r(im)}$)is obtainable from the preliminary test data.

3.1 Specimen preparation and test procedure

All specimens except specimens witha_s=0 %were prepared using dry-rodding method (lightly in a circular pattern) with a 6 mm diameter rod. Sand specimens with a_s=0 %were prepared using air-pluviation to achieve low $D_r$of about 20 %. Soil specimens with a_s= 3, 6 and 10 % were prepared in five layers toachieve a more uniform density. Reconstituted specimens were compacted in the hollow cylindrical mold: 7 cm outer diameter, 3 cm inner diameter, and 10 cm height. Each layer was compacted to the required height.

Soil samples representing the sand pile and surrounding improved ground were saturated using the method described above. Saturated samples were consolidated anisotropically to achieve the required value of K_T, maintaining total σ_v constant at 100 kPa.

Equivalent cyclic shear strain was applied to both specimens maintaining total σ_v the same as the initial value σ_v=100 kPa by controlling the cell pressure. Consequently, K_T values in both specimens were changed. Then, K_T in the specimen with surrounding improved ground was calculated. To maintain the equivalent K_T value in both the surrounding ground soil specimen and sand pile soil specimen, the axial displacement of the sand

pile soil specimen was controlled. Furthermore, equal water pressure was maintained in outer and inner cells of both torsional shear systems. Subsequently, shear stresses in both samples (τ_{scp} and τ_{im})were measured and the shear stress in the SCP composite ground was calculated using Eq. (1). Using this method, a computer analysis cyclic loading, which calculates the composite shear stresses, was combined with a computer online data processing system. It was assumed thatinitial liquefaction occurred when the double amplitude of shear strain (DA) reached 5 %.

Table 1 shows 12 test cases that were conducted under PET. In this study, $a_s$and the amplitude of the composite shear stress (τ_{amp}) were selected as parameters. The case name displayed in the table implies a combination of $a_s$and(τ_{amp}). For example, Case 3-20 indicates the composite shear stress amplitude of 20 kPa with a_s of 3 %. In that table, $D_{ri(scp)}$ and $D_{ri(im)}$ respectively indicate the initial $D_r$of the sand pile soil specimen and the surrounding improved ground soil specimen.

4 DISCUSSION ON RESULTS

Typical test results are shown in Fig. 7; they were obtained from a test with a_s of 10 % and composite shear stress amplitude of 40 kPa (Case 10-40). This graph indicates the variation of shear strain (γ), σ_h, and σ_v with respect to time. The cyclic shear strain in both samples was equal. In other words, similar distortion occurred in both the sand pile and the surrounding improved ground, hence same distortion in the SCP improved composite ground. Furthermore, σ_h and σ_v of both samples were equal and total σ_v was maintained at 100 kPa. As shown in the graph, $\sigma_h$gradually decreased from 190 kPa (K_T = 1.9) under cyclic shear loading and finally reached the value of total σ_v, *i.e.* an isotropic state.

A typical stress-strain relationship and the effective stress path of the sand pile element and surrounding improved ground element for Case 10-40 is shown in Fig. 8. As that figure shows, the mean effective stress path of the sand pile element decreased gradually with cyclic loading (Fig. 8(c)). However, this decreasing tendency became gradual when mean effective stress approached 70 kPa. A further increase of cyclic shear loading caused dilation of the sand pile element. Consequently, the mean effective stress increased to 280 kPa, whereas shear stress increased to 220 kPa. Soil element dilation was accompanied by the generation of negative pore water pressure. On theother hand, for the surrounding improved ground element, the mean effective stressreachedzero,illustratingstrainsoftening (Fig. 8(d)), which clearly indicates the effect of densification on cyclic shear loading.

Fig 06Complete experimental setup

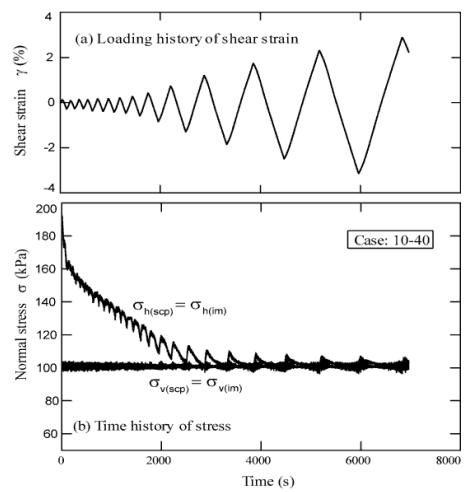

Fig 07Strain history and normal stress variation

Table 01 Test cases

Case Name	a_s(%)	τ_{amp} (kPa)	K_T	$D_{ri(scp)}$ (%)	$D_{ri(im)}$ (%)
Case: 0-10	0	10	0.5	-	15
Case: 0-15	0	15	0.5	-	27
Case: 0-20	0	20	0.2	-	15
Case: 3-15	3	15	1.5	85	34
Case: 3-20	3	20	1.5	85	32
Case: 3-25	3	25	1.5	84	32
Case: 6-20	6	20	1.7	82	45
Case:	6	30	1.7	85	58

6-30					
Case: 6-35	6	35	1.7	84	49
Case: 10-30	10	30	1.9	86	67
Case: 10-35	10	35	1.9	84	65
Case: 10-40	10	40	1.9	85	65

Liquefaction resistance of composite ground

Liquefaction resistance of the SCP improved composite ground is defined in terms of the shear stress ratio (τ_{amp}/σ'_{v0}), which is obtained by normalizing the composite shear stress amplitude (τ_{amp}) with respect to the initial effective vertical stress (σ'_{v0}) of100kPa. The relationship between the stress ratio of composite ground and the number of loading cycles (N_l) required to cause the double amplitude of shear strain (DA) equal to 5 % is shown in Fig. 9. The greater the value of a_s, the greater the resistance to liquefaction.

Usually, liquefaction resistance (R_l)is defined in terms of the stress ratio at the equivalent number of cycles (N_e) equal to 20; this condition is typically used for design. Therefore, to obtain a clear perspective of liquefaction resistance with respect to a_s, shear stress ratios corresponding to N_e = 20 were obtained from Fig. 9 and plotted in Fig. 10. The liquefaction resistance R_{lv} has a linear relationship with a_s. The term R_{lv} is used to represent the liquefaction resistance because the stress ratio is normalized with respect to the initial effective vertical stress.

Laboratory investigations and field observations have shown that the liquefaction potential of a soil deposit to earthquake-induced motion depends on the soil characteristics, initial stresses acting on the soil, and earthquake characteristics. The significant factors include soil type, $D_r$or void ratio, initial confining pressure, intensity of ground shaking, and duration of ground shaking. Reportedly, liquefaction resistance in SCP improved ground depends on many factors. For simplicity, three main factors are considered in this discussion: (1) $D_r$of the surrounding improved ground (effect of a_s), (2) Coefficient of total lateral pressure (effect of K_T), and (3) Sand pile rigidity (effect of composite ground).

It is a general feature that some factors are combined with others. For example, a change of $D_r$in the surrounding ground occurs concomitant with the change of K_T. In other words, it is difficult to isolate the influence of one factor on liquefaction resistance. However, the following is anattempt todetermine the influence of K_T on liquefaction resistance.

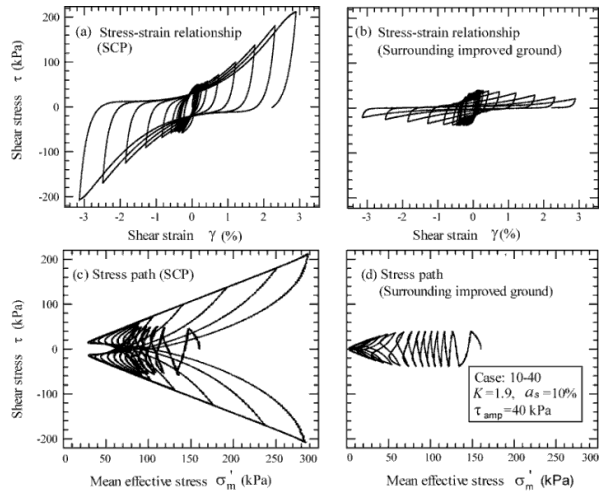

Fig08Typical test results of the stress-strain

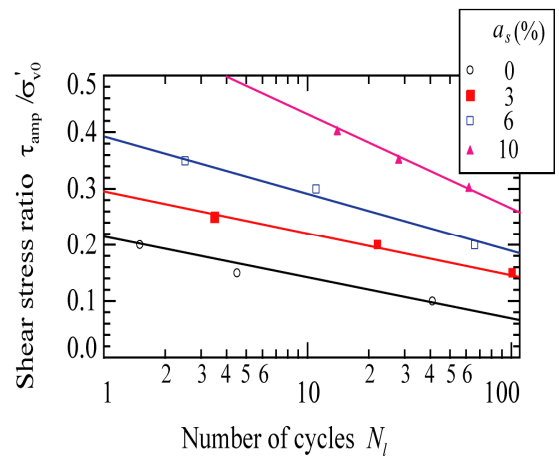

Fig 09Relationship between stress ratio of τ_{amp}/σ'_{v0} and the number of cycles

4.1 Effect of K_T on liquefaction resistance

It is believed that the relationship shown in Fig. 9 depends mainly on the above three factors. To examine the effect of initial K_T conditions, the term initial effective vertical stress (σ'_{v0}) in the definition of stress ratio was converted to the initial mean effective confining stress (σ'_{m0}) where, $\sigma'_{mo} = \frac{1+2K_T}{3}\sigma'_{v0}$. If the cyclic stress ratio of τ_{amp}/σ'_{v0} in the ordinate of Fig. 9 is changed to the cyclic stress ratioof τ_{amp}/σ_{m0}, the test data canberearrangedasshownin Fig. 11. Data of a_s =0, 3 and 6% under different $K_T$conditions are correlated uniquely (behave as one group) with the number of cycles. However, the shear stress ratio of a_s=10% deviates from the above uniqueness.

Similar to the R_{lv}, liquefaction resistance is

defined as $N_e = 20$ and is shown in Fig. 10 together with R_{lv} . In this case, the liquefaction resistance is denoted by R_{lm} because the stress ratio was normalized using initial effective mean confining pressure (σ'_{m0}). The figure shows that $R_{lm}$is almost constant at a_s =0, 3 and 6 %. However, the R_{lm} value increased remarkably at a_s = 10 %. Using σ'_{m0} to normalize the shear stress (τ_{amp}/σ'_{m0}), the effect of the K_T value is omitted. Therefore, the dominant factor at low a_s is K_T.

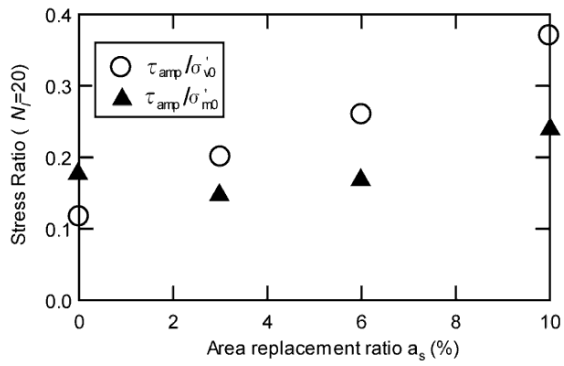

Fig10Close association of Rlv, Rlm and a_s

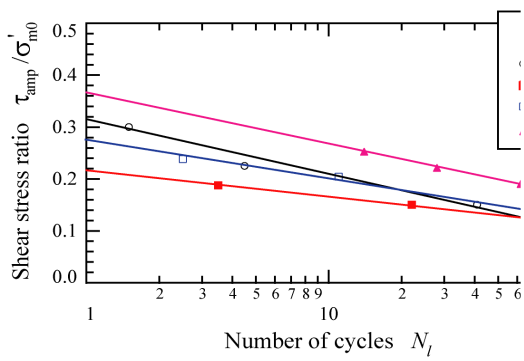

Fig11Relationship between stress ratio of τ_{amp}/σ'_{m0} and the number of cycles

Shear stress distribution in SCP composite ground

When a load is applied to the SCP composite ground, it is known to be distributed to the sand pile and to the surrounding improved ground. Methods of analyzing the shear stress distribution in SCP improved ground against the possibility of earthquake-induced liquefaction can be represented by normalizing Eq. (1) with respect to the shear stress of the composite ground as $\frac{\tau_{scp} a_s}{\tau_{com}} + \frac{\tau_{im}(1-a_s)}{\tau_{com}} = 1$. The first term represents the stress concentration ratio in the sand pile ($\alpha_{scp} = \tau_{scp} a_s / \tau_{com}$), whereas the second term represents the stress concentration ratio in the surrounding improved ground ($\alpha_{im} = \tau_{im} a_s / \tau_{com}$). Then, equation can berewritten as $\alpha_{scp} + \alpha_{im} = 1$. The peak values of α_{scp} were plotted against the ratio of N/N_l , where N is the number of loading cycles and N_l is the accumulated number of loading cycles required to cause liquefaction (Fig. 12). As shown in Fig. 12, all curves illustrate the same tendency in variation of α_{scp}, in which α_{scp} remains constant for a particular cyclic number ratio (N/N_l); it is particularly sensitive to the number of cycles, provided that there is some residual margin of safety against liquefaction with the use of SCP. A clear tendency of variation of α_{scp} with respect to the amplitude of composite shear stress (τ_{com}) is visible in this figure. This peculiar shape is similar for different a_s; in fact, a comparison of different α_{scp} values is shown in Fig. 13. There is no marked increment in α_{scp} in the case of $a_s = 3\%$ when compared with $a_s = 6$ and 10 %, implying that, because of the low a_s, the shear stresses do not transfer adequately to the sand pile during cyclic loading. The larger the a_s, the greater the maximum α_{scp} at N/N_l = 1.0. This tendency implies that the portion of the shear stress occupied by the sand pile during cyclic loading depends mainly on the a_s; the larger the a_s, the greater the portion of shear stress is occupied by the sand pile. α_{scp} starts to increase at lower N/N_l ratio if a_s is large. In other words, whena_s is large, the earthquake-induced cyclic shear stress quickly transfers to the sand piles and the sand piles function appropriately to mitigate liquefaction. On the other hand, when a_s is small, before transfer the cyclic load to the sand piles, large ground deformationsmight occur. Consequently, the sand pile can not show its function sufficiently in this situation.

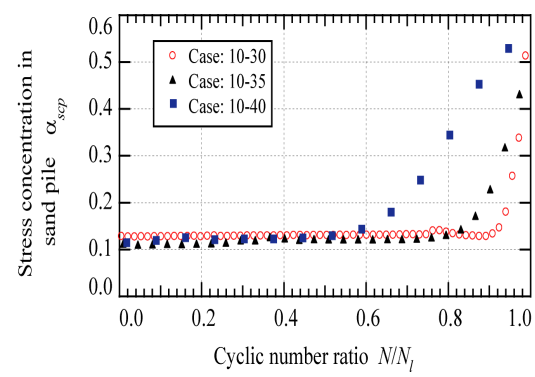

Fig12Illustration of stress concentration in sand pile

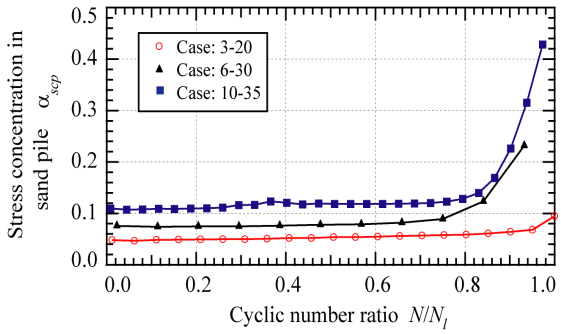

Fig13Effect of area replacement ratio on stress concentration in sand pile

5 CONCLUSIONS

The authors proposed a new method to evaluate SCP composite ground using PET. Densification characteristics were evaluated using the simulated SCP installation procedure. Salient conclusions of this study are summarized as follows:

1) The relationship between K and the a_s, obtained from cyclic shear triaxial tests simulating SCP installation processes, represent the effects of densification attributable to SCP installation.
2) The PET was introduced by defining a new parameter: the total coefficient of lateral pressure ($K_T = \sigma_h / \sigma_v$). The concept of PET is that shear strain in SCP composite ground is equivalent until liquefaction.
3) Test results revealed that, when a_s is small, the dominant factor of liquefaction resistance is K_T. Similarly, when a_s is large, rigidity of the dense sand pile and the $D_r$of surrounding improved ground more dominated liquefaction resistance than K_T.
4) Stress concentration in the sand pile (α_{scp}) depends strongly on a_s. When a_s is large, earthquake-induced cyclic shear stress is transferred quickly to the sand pile and the sand pile functions properly to mitigate liquefaction. On the other hand, if a_s is small, before transfer of the cyclic load to the sand pile, the surrounding ground might incur large deformation. Therefore, the sand pile cannot mitigate liquefaction sufficiently.

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