An FEM simulation on the seismic induced deformation of SCP improved ground considering the inter-pile ground conditions.

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SUMMARY

The SCP method is one of the most popular soil improvement methods of soft clay layers. In the numerical simulation for seismic design (seismic performance evaluation), the complicated structure of sand pile and inter-pile soft clay ground is often ignored for the case with high replacement ratio (As=70%). In other word, uniform simple element with adequate value of stiffness and strength is used for the improved area. However, the deformation behaviour of the composite ground of SCP and clay layer depends on the characteristics of sand pile, clay layer and the distance of piles in reality. Especially, the stiffness of clay layer between pile and the distance of piles (inter-pile ground conditions) are key parameters.

In this study, a series of effective stress analysis was performed to capture the deformation characteristics of the composite ground with SCP and clay layer. The research result is beneficial for the adequate parameter identification for the seismic design of a structure on SCP improved ground.

Keywords: Sand compaction pile method, FEM, composite material

1. INTRODUCTION

In most of the cases to construct a structure on a soft clay ground, it is necessary to improve the ground to achieve the stability of the structure. The soil improvement methods for this purpose increase the bearing capacity of the ground and mitigate the settlement due to consolidation. Although there are various types of soil improvement method have been proposed, the most popular one is the sand compaction pile (SCP) method[1)]. In this method, compacted sand piles are installed in the ground to increase the shear strength of the ground. Note, the SCP method can be applied for both clay ground and sand ground. In this paper, the SCP method for clay ground is focused.

In the case of the SCP method for clay ground, the original ground shall be improved as a kind of hybrid structure of sand pile and original clay ground. The shear strength of the ground shall be increased with the large shear strength of the sand piles. In addition, by the drainage effect through the sand pile, the consolidation of the original clay ground shall be accelerated and the shear strength of the original clay ground should be increased.

In this study, a series of effective stress analysis was performed to capture the deformation characteristics of the composite ground with SCP and clay layer. The research result is beneficial for the adequate parameter identification for the seismic design of a structure on SCP improved ground.

2. THE OUTLINE OF THE SAND COMPACTION PILE (SCP) METHOD

The SCP method has been developed in Japan. It was originally developed as a countermeasure for liquefaction phenomenon of sandy ground. However, it is often used for clayey ground to improve the bearing capacity and mitigate the settlement due to consolidation. The machine for SCP installation has been well developed. It can be used both inland and waterways including deep water depth area. The total length of sand piles installed in Japan up to the year of 2001 is 350,000 km (in terms of ϕ 700 mm piles).

There are two types of construction procedure for sand compaction piles. One is the method with vibration to install the pile, and the other is the method with static installation. Followings are the procedure of construction with static installation of sand compaction pile.

a) Place the casing pipe at the appropriate position. Put the sand in it with a hopper.

b) Rotate and push the casing pipe into the ground.

c) The casing pipe reached to the bottom of the improvement area.

d) Pull out the casing pipe to some extent. The sand is ejected from the bottom of the casing pipe.

e) Push back the casing pipe to broaden the diameter of inserted sand pile. With this process, the sand pile is compacted.

f) Pull out and Push back shall be repeated to the top surface of the improvement area.

The improved area becomes a kind of composite structure with sand piles and original ground. Basically, the sand piles are stiffer than the original ground. The piles have higher shear strength than that of the original ground. Thus, the deformation behaviour of the improved ground is rather complicated. It is necessary to clarify the deformation characteristics of the improved ground for the evaluation of the seismic performance of the structure with SCP method. In addition, not only natural sand but also steel slags are expected to be used as the material of piles. In the case of slag material, pile will be solidified, and the shear deformation characteristics might be different from usual sand case. A method to consider the effect of pile stiffness on the deformation characteristics is strongly expected.

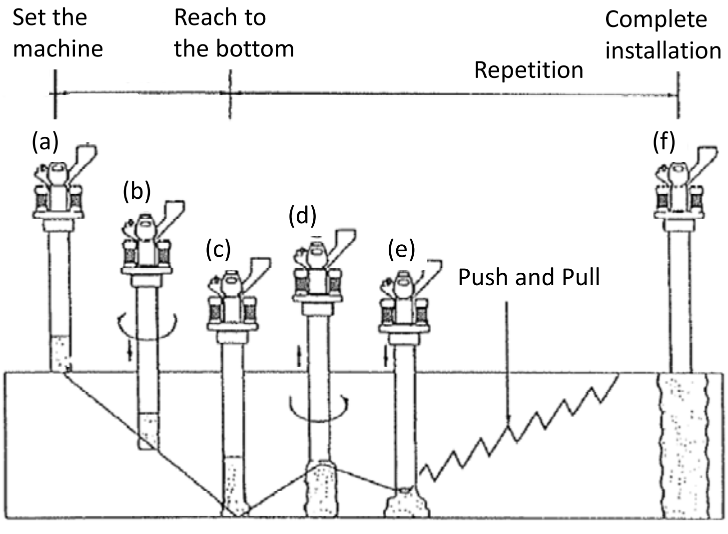


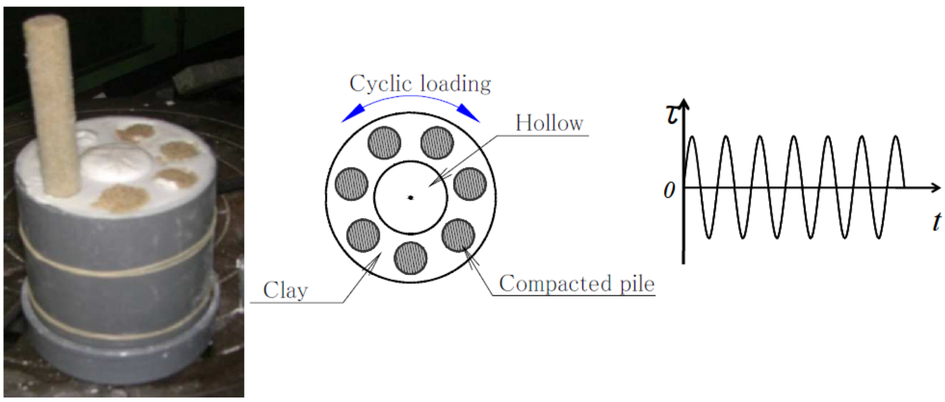
Figure 1. The procedure to construct SCP

3. THE PREVIOUS STUDIES FOR DEFORMATION CHARACTERISTICS OF SCP IMPROVED GROUND

3.1. A series of small model tests with torsional hollow cylinder test apparatus

Some of the authors have been worked to clarify the shear deformation characteristics of pile-clay composite structure using torsional hollow cylinder test apparatus. The size of the specimen is 3 cm in inner diameter, 7 cm in outer diameter, and 8 cm in height. The test was conducted for both sand pile case and slag pile case. The concept of the model test is schematically shown in Figure 2. And an example of the test results is shown in Figure 3.

In Figure 3, the time history of shear strain and pore water pressure for the case of sand pile are shown. In the time history of pore water pressure, up and down of the pressure due to cyclic mobility is observed. The pore water pressure has been build up due to cyclic shear; however, effective stress has been recovered under large shear deformation. It indicate that the sand pile show a typical behaviour of compacted granular material even when the pile was surrounded by clay. Therefore, it is important to consider the dilatancy characteristics of sand to evaluate the cyclic behaviour of pile-clay composite ground.



(a) Model preparation (b) Plan view of the specimen (c) Applied cyclic shear

Figure 2. A schematic illustration for the cyclic test using torsional hollow cylinder test apparatus

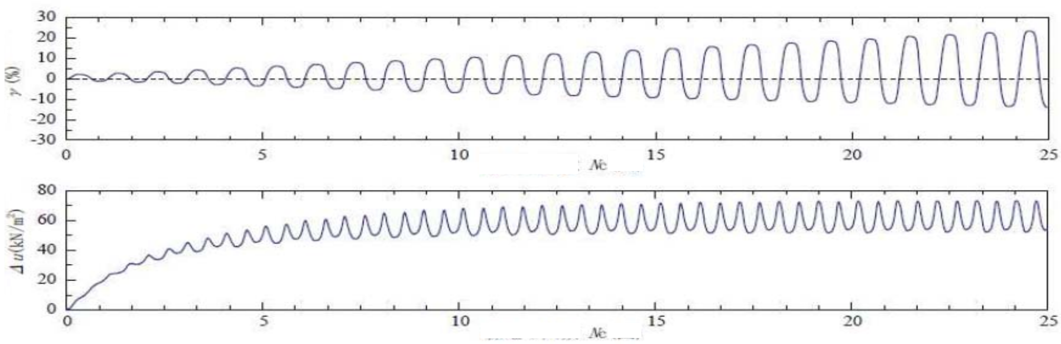


Figure 3. Time histories of shear strain and excess pore water pressure (Toyoura sand case)

3.2. Centrifuge model test for the composite ground beneath caisson walls

Takahashi et al. reported the centrifuge test results for the composite ground of sand pile and clay ground. The outline of the model test is shown in Figure 4. The model ground is consolidated in 50 g at first. Then, reclamation is given to the model ground till the deformation is observed.

The typical test result is as follows. Figure 5 shows the deformation of sand piles. The improved area deformed toward seaside, and the amount of deformation is larger in shallower depth. The clay ground of inland side deformed upward, and the clay ground of seaside deformed downward. It indicates the sand piles bended. This research results implies the importance to consider the difference of stiffness of sand piles to evaluate the shear deformation of improved ground. In addition to the static case of Takahashi et al., the behaviour in dynamic loading is also reported by Kinoshita et al. A similar result was obtained in the dynamic case either.

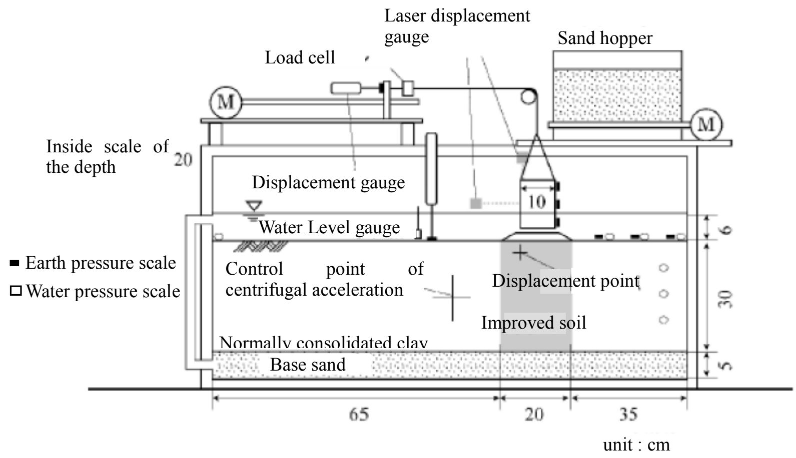


Figure 4. The outline of the centrifuge test model in a previous study

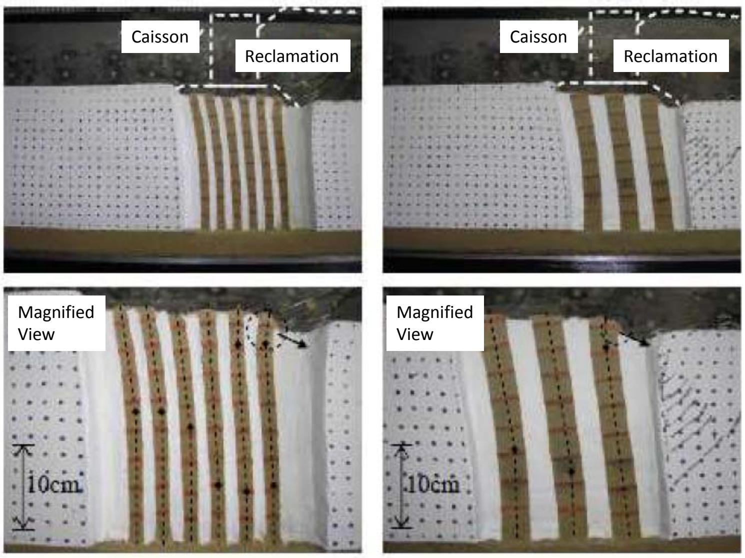


Figure 5. The observed deformation pattern of sand piles in centrifuge tests

4. FEM ANALYSIS OF SAND PILE – CLAY COMPOSITE GROUND

4.1. The analysis model

Figure 6 shows the FEM model for the shear test of sand pile-clay composite ground. The one seventh (1/7) of the specimen used for the torsional hollow cylinder apparatus is modelled in 2D domain. Circular boundary condition is utilized to obtain the same behaviour both in the left and right side boundary. The width of the sand pile is adjusted to give the same improve rate of the real 3D situation. The difference of the Type-1 model and Type-2 model is the aspect ratio (Height/Width) of sand pile. Type-2 model gives the same aspect ratio of the test specimen.

The effective stress-based FEM program 'FLIP' was used. In this program, the effect of dilatancy characteristics can be considered. The parameters for the sand piles are given by the simplified parameter setting procedure. Note, the equivalent SPT N values (N values corrected for 65 kPa overburden pressure) for sand piles were assumed as 20 based on case histories. 3 patterns in the parameters for clay are considered as a parametric study. The summary of material parameters are shown in Table 1 and Table 2.

In the analysis, both the cases in drained condition and in undrained condition are conducted to consider the effect of dilatancy on the behaviour of the ground. Note, in this program FLIP, the existence of pore water is completely ignored in the analysis in drained condition, and the decrease of effective stress is evaluated by the Iai model in the analysis in undrained condition.

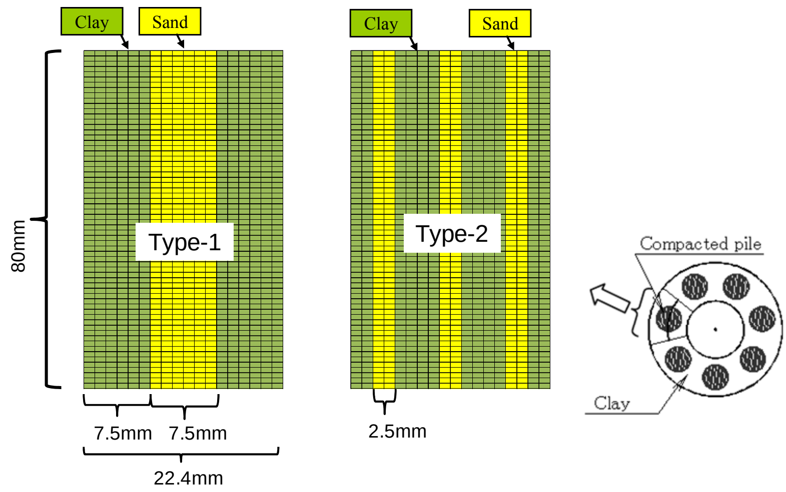


Figure 6. The FEM mesh model

Table 4.1. The material parameters for the analysis (Dynamic characteristics)

Material	Unit weight γ_t (kN/m^3)	Reference mean effective stress σ'_{ma} (kN/m^2)	Initial shear modulus G_{ma} (kN/m^2)	Initial bulk modulus K_{ma} (kN/m^2)	Poisson's ratio	Unconfined compression strength q_u (kN/m^2)	Cohesion C (kN/m^2)	Internal friction angle ϕ (deg.)
Sand pile	20.0	98	131300	342400	0.33	-	0	41.4
Clay (soft)	16.5	98	1462	3813	0.33	50	16.9	6.5
Clay(medium)	16.5	98	2924	7625	0.33	98	32.7	12.4
Clay(hard)	16.5	98	4386	11438	0.33	147	49	18.3

Table 4.2. The material parameters for the analysis (Dilatancy characteristics)

Material	Phase transformation angle fp (deg.)	s1	w1	p1	p2	c1
Sand pile	28	0.005	10.86	0.5	0.807	4.888

4.2. Analysis conditions

In the analysis, there are two phases to simulate the test procedure. The first phase is the simulation of isotropic consolidation of the specimen with 74 kPa. The second phase is the loading shear to simulate torsional shear loading of the specimen in the test. The schematic illustration of the loading is shown in Figure 7. The side boundaries of the analysis domain are set as the circular boundary condition, giving the same value for both sides in displacements.

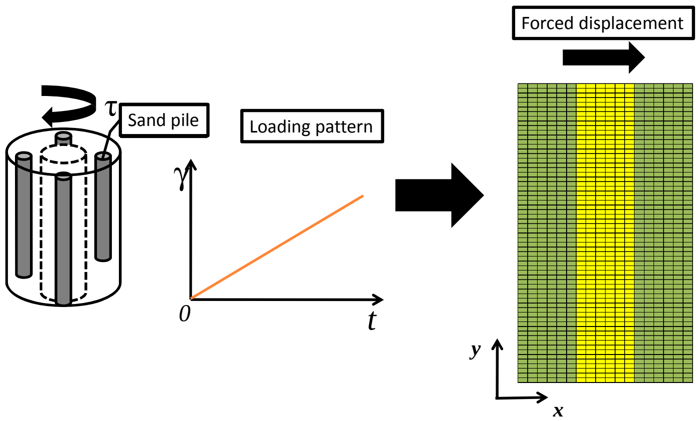


Figure 7. The loading pattern in the test and analysis

4.3. The results of the analysis

The residual deformation and the distribution of the strain after the simple shear loading are shown in Figure 8 and Figure 9, respectively. These are the results for the soft clay case. The red squares in Figure 8 correspond to the sand piles. Although there is no significant difference between the sand pile part and clay part in Figure 8, the large difference can be observed in Figure 9. The sand piles are bended and no major shear strain beyond 1 % level is observed. On the other hand, the clay elements between piles show more than 3 % level of shear strain. Thus, the deformation mode in sand piles and clay part are quite different.

At the Type 1 mesh in drained condition, the shear strains of clay elements are smaller than that in undrained condition. This might be because the dilatancy effect is not observed here due to drained condition. In undrained condition, the stiffness of sand pile can be increased due to the dilatancy effect, and the ratio of axial load on sand pile also increases. But, in drained condition, the stiffness of sand pile is almost constant, and the clay between piles also supports the axial load.

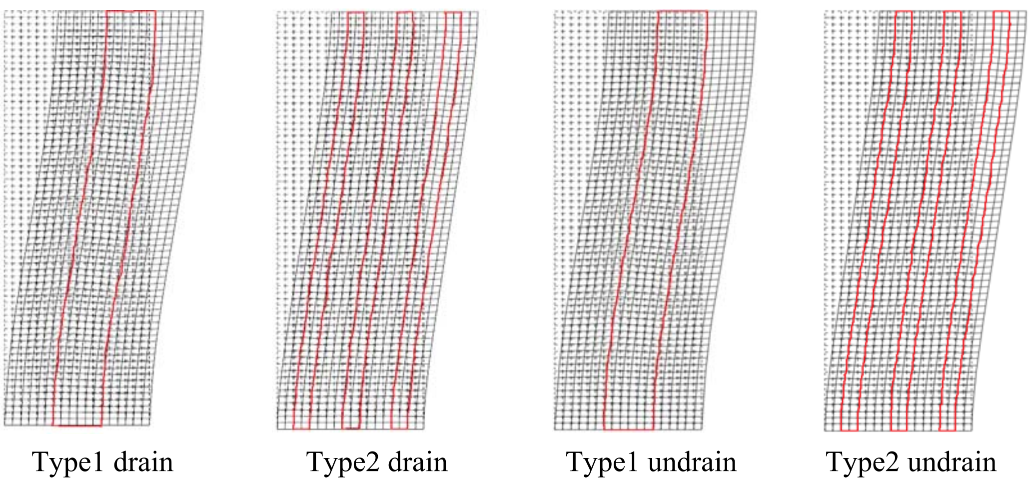


Figure 8. Residual deformation

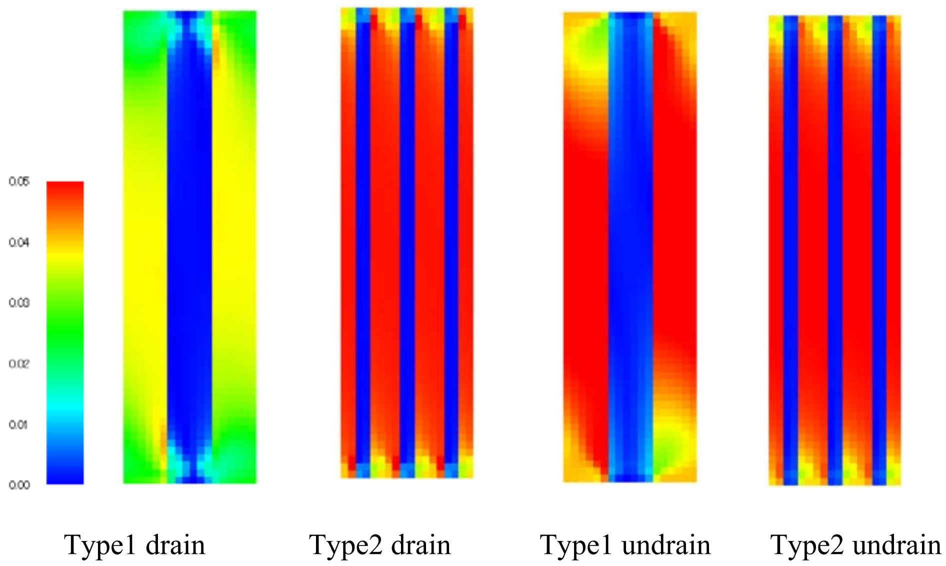


Figure 9. Distribution of computed strains

5. THE EFFECT OF THE VARIATION IN THE STRENGTH OF INTER-SAND PILE CLAY

5.1. Stress and strain in overall model of composite ground

The strain in overall model γ_A is defined as shown in Figure 10, and the average shear stress in the elements at the top of the model is defined as τ_A. And the relationship between τ_A and γ_A are shown in Figure 11 and Figure 12, focusing on the aspect ratio of sand pile and drainage conditions, respectively. With the increase of the strength of clay, the shear stress in each level of shear strain tends to increase.

In drained condition shown in Figure 11, the relationship between τ_A and γ_A for Type 1 and Type 2 are almost same. This is because there is no dilatacy effect, and the behaviour is dependent on the contrast of bending stiffness of sand pile and clay between piles. In Type 1 and Type 2, the replacement ratio is same. Thus, the bending stiffness is almost same and the behaviour is also almost same. On the other hand, in undrained condition, the shear stress in Type 1 is larger than that in Type 2 in same strain level. This is the effect of aspect ratio of sand piles.

Figure 13 shows the stress paths of the element in sand piles. In the tension side of the pile, effective stress increase due to the dilatancy effect. Thus, the shear stress also increased. As shown in Figure 8, the piles bended. Thus, with less aspect ratio (Type 1), the decrease of axial load (σ_y) is more significant, and more compression shear is observed. This corresponds to the difference of drained condition and undrained condition in Figure 12. In Figure 12, the difference due to the difference in dranage condition is more significant in Type 1 than the case in Type 2. In less aspect ratio (Type 1) case, larger compression shear was applied and the effect of dilatancy is increased. Thus, aspect ratio of sand pile has an effect of the behaviour of composite ground.

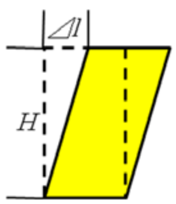


Figure 10. The definition of the overall shear strain in the model

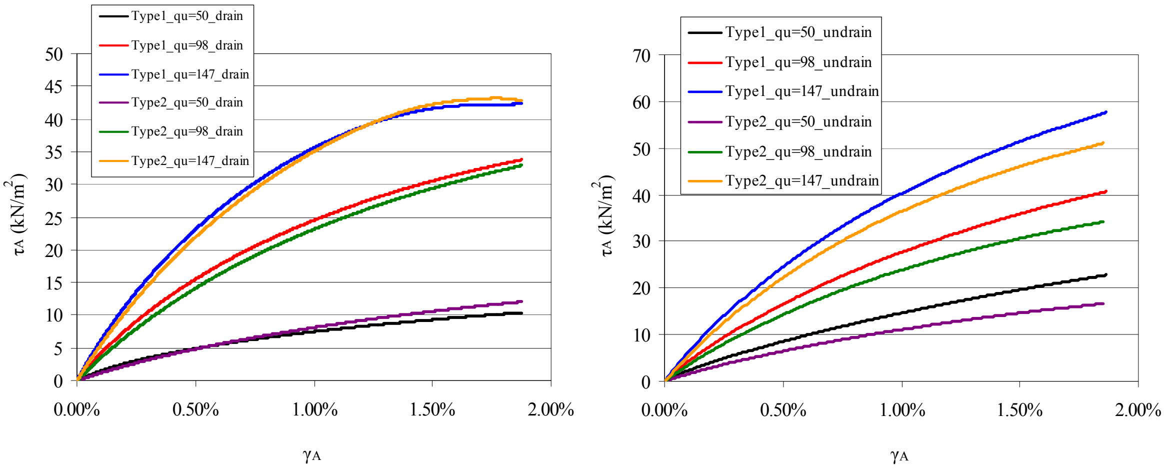


Figure 11. The relationship between τ_A and γ_A (focused on the aspect ratio of sand piles)

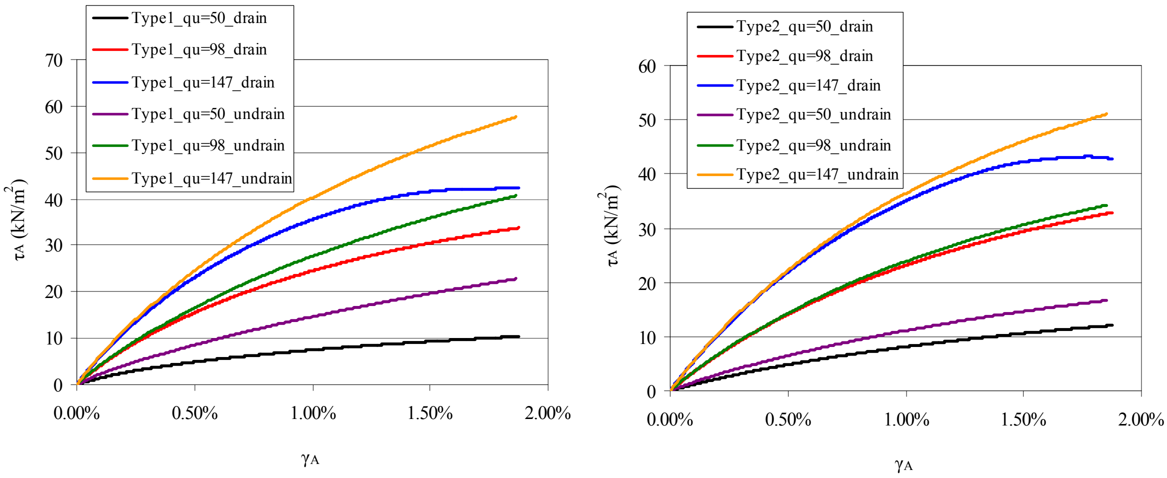


Figure 12. The relationship between τ_A and γ_A (focused on the drainage condition)

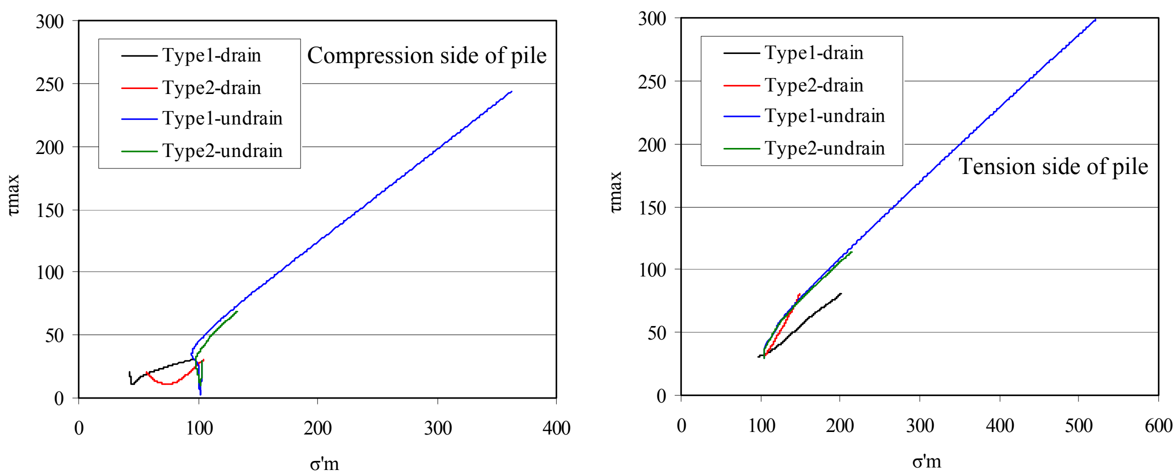


Figure 13. Effective stress paths (soft clay case)

5.2. The effect of the strength of clay between piles

As shown in Figure 9, large shear strain is observed in the clay between sand piles. This is because the shear stiffness and shear strength of clay is far smaller than that of sand piles. As a result, bending type of deformation is observed at the sand pile, and shear stress is not completely transmitted between piles. If the shear stiffness and strength of the pile is increased, this bending type of deformation is restricted, and overall shear strength might be improved. And it can be happen when the consolidation is fully completed with the drainage through the sand piles.

When the shear stiffness and strength of the clay between piles increase to the level of sand pile, the behaviour of the composite ground might be same with the simple sand model. Table 3 shows the ratio of shear strain in composite ground to the shear strain in simple sand model. The strain is given to the shear stress of τ_A = 10 kN/m^2. Note the value is converted to the 70 %, which was the replacement ratio. For the soft clay case, the difference between the composite ground and the simple sand case is large. This difference is smaller in undrained case than in drained case. When the strength of the clay becomes larger, the effect of drainage condition on the difference becomes smaller. This can be confirmed in the Figure 14.

Table 5.1. The ratio of shear strain in composite ground to the shear strain in simple sand model

Case	Drainage condition	Unconfined strength of clay q_u (kN/m^2)	Shear strain for 10 kN/m^2 shear stress (%)	Raito to simple sand case
Simple sand (converted to 70%)	Drained	-	0.019	1.000
Soft clay	Drained	50	1.280	67.368
Medium clay	Drained	98	0.283	14.895
Hard clay	Drained	147	0.175	9.211
Simple sand (converted to 70%)	Undrained	-	0.019	1.000
Soft clay	Undrained	50	0.613	32.263
Medium clay	Undrained	98	0.269	14.158
Hard clay	Undrained	147	0.175	9.211

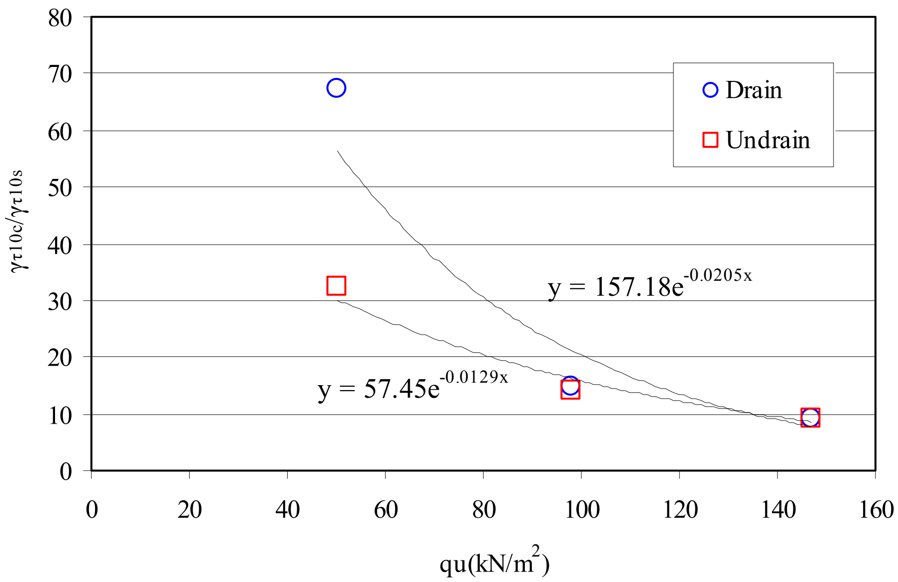


Figure 14. The effect of the strength of clay between sand piles

6. CONCLUSIONS

In this study, a series of effective stress analysis was performed to capture the deformation characteristics of the composite ground with SCP and clay layer. Following conclusions are obtained.

1) Although there is no significant difference between the sand pile part and clay part in the deformation pattern, the large difference can be observed in the distribution of shear strain. The deformation mode in sand piles and clay part are quite different.

2) In less aspect ratio (Type 1) case, larger compression shear was applied and the effect of dilatancy is increased. Thus, aspect ratio of sand pile has an effect of the behaviour of composite ground.

3) If the shear stiffness and strength of the pile is increased, this bending type of deformation is restricted, and overall shear strength might be improved. And it can be happen when the consolidation is fully completed with the drainage through the sand piles.

4) For the soft clay case, the difference between the composite ground and the simple sand case is large. This difference is smaller in undrained case than in drained case. When the strength of the clay becomes larger, the effect of the difference in drainage condition on the difference becomes smaller.

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