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Effect of performance method of sand compaction piles on the mechanical behavior of reinforced soft clay

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Abstract. Sand Compaction Piles (SCPs) are constructed by feeding and compacting sand into soft clay ground. Sand piles have been installed with irregular cross-sectional shapes, and mixtures of both sand and clay, which violate the design requirement of circular shape according to the replacement area ratio due to various factors, including side flow pressure. Therefore, design assumptions cannot be satisfied according to the conditions of the ground and construction and the replacement area ratio. Two case histories were collected, examined, and interpreted in order to study the effect of the shape of SCPs. The effects of the distortion of SCP shape and the mixture of sand and clay were studied with the results of large direct shear tests. The design internal friction angle was secured with the irregular cross-sectional sand piles regardless of the replacement area ratio. The design internal friction angle was secured regardless of mixed condition when the mixture of sand and clay was higher than the replacement area ratio of 65%. Therefore, systematic construction management is recommended with a replacement area ratio below 65%.

Keywords: sand compaction pile (SCP); replacement area ratio; composite soil; mixed soil; large scale direct shear test

1. Introduction

The Sand Compaction Pile (SCP) method is a ground improvement method that involves constructing compacted sand piles in soft ground by feeding sand or similar material such as polymer (Arasan *et al.* 2016), gravel (Fattah *et al.* 2015), and mixture of lime-soil (Malekpoor and Poorebrahim 2014) into soft ground through a casing pipe and compacting it with the vibrating compaction technique. Shin *et al.* (2004) studied the strength properties of SCPs at a replacement area ratio (a_s, sectional area of sand piles/cross-sectional area of clay ground and sand piles) of less than 30% using triaxial compression tests. The stress concentration ratio increased with the increasing replacement area ratio and with the increasing consolidation at a replacement area ratio of less than 40% in soft clay ground. The stress concentration ratio decreased under the rigid loading condition rather than the flexible loading condition. In other words, the efficiency of SCPs decreased under the rigid loading condition (Bae *et al.* 2006). Chu *et al.* (2012) recommended a replacement area ratio ranging from 30% to 40% for a bottom ash mixture compaction pile due to cost effectiveness. Jorat *et al.* (2013) studied the strength and compressibility characteristics of the peat model ground reinforced with sand columns. The sand area of 30% led to the highest shear strength in the range from 20% to 40% for the peat model ground reinforced by the sand columns.

Park *et al.* (2000) studied the variation of both stress concentration ratio and settlement reduction effect at a replacement area ratio ranging from 20% to 60% in soft clay ground based on model tests using a soil tank, and the shear strength characteristics of composite ground according to a replacement area ratio ranging from 20% to 46% in large-scale direct shear tests. Ahn and Kim (2012) studied the consolidation behavior of SCPs and recommended the concentration ratio of 3 with a finite element analysis using the Modified Cam-Clay model. As the sand content of mixed soil increased to 50%, liquid limit, plastic limit, cohesion, and undrained shear strength measured by performing both vane shear tests and direct shear tests decreased, while internal friction increased (Park and Nong 2014). As water content increased, the undrained shear strength of the remolded specimen rapidly decreased and gradually decreased at a certain water content called the Flow Water Content. The Flow Water Content decreased with the increasing sand content.

Gutub and Khan (1998) studied the shear strength behavior of soft Madinah clay including sand drains by comparing the undrained direct shear test results with the theoretical values of the composite soil to analyze the positive effect of the sand drain system reinforcing the shear strength properties of soft Madinah clay. Dafalla

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(2013) performed direct shear tests to find the general trends in the behavior of clay-sand mixtures according to variable clay and moisture contents. The increasing water caused a drop in both cohesion and internal friction angle and the change was dependent on the density state of clay-sand mixtures. Andreou *et al.* (2008) compared the behavior of reinforced soil with that of unreinforced soil by performing tri-axial compression tests on composite soil specimens of soft kaolin clay, which was reconstituted from slurry with a compacted reinforcement column. While the increasing confining pressure decreased the strength of the reinforced soil, the soil improvement was more pronounced for the soil reinforced with a sand column than for a gravel column at high confining pressure. Li *et al.* (2013) studied the effects of test conditions on shear behavior of composite soil specimens which were differently mixed with kaolin and glass beads by performing 35 direct shear tests at different shearing rates under different normal stresses. They found a correlation between volume change and deviation of water content between the shear- and outer-zones, and a relatively small void ratio in the shear zones under high normal stress and low shearing rate.

You *et al.* (2006) studied the stress sharing behavior of composite soil and its mechanism during consolidation at a low replacement area ratio in soft clay ground by a numerical analysis. Based on centrifuge model tests for the failure behavior of clay ground improved by SCPs, the settlement reduction ratio increased with the increasing penetration ratio regardless of the replacement area ratio. The settlement rapidly decreased at the replacement area ratio of 80% (Jeong *et al.* 2012). Juneja and Ahmed (2012) investigated the effect of the smear zone on SCP by measuring the pore pressure during consolidation and the undrained shear strength of the composite sample. Juneja *et al.* (2013) showed that the smear zone reduced the permeability of composite soil by 20% by using scanning electron microscope (SEM) images. Pore pressure induced by the reduction of the permeability changed the undrained shear strength of the composite soil. O'Kelly (2013) proposed an equation showing the inverse relationship between relative water content and relative strength.

In previous studies, the shear strength parameters of composite soil and mixed soil have not been compared and analyzed according to a wide range of replacement area ratios with the variation of undrained shear strength parameters of original clay ground and the variation of the density of sand. Moreover, the shape effects of mechanical behavior of ground reinforced by SCPs have not been analyzed according to the replacement area ratio. However, sand piles constructed in soft clay ground have irregular cross-sectional shapes unlike the original circular shaped design during the installation of sand piles according to the replacement area ratio due to 5 factors to be discussed in this section. In addition, there are many cases wherein silts instead of sands from sand piles have been sampled by SPT at Japanese SCP construction sites consisting of marine clay ground, because the sand piles were mixed with marine clay during the installation of SCPs. The following case histories were collected, examined, and interpreted in order to study the effect of the shape of SCPs.

The test construction report of Kansai International Airport among test construction cases in Japan, where marine SCPs have been constructed for decades, shows information obtained from slant-drilled boreholes to check the status of the SCPs (Takai *et al.* 1989). It was assumed that the SCPs with diameters of 2000 mm maintained the circular cross-sectional areas during the installation of SCPs. A replacement area ratio of 70% was calculated with the circular cross-sectional areas. However, the cross-sectional shape of sand piles became irregular and ameba-like, not circular, under both the large diameter and high replacement area ratio of sand piles. It was verified with inclined boring at the test construction site that the cross-sectional shapes of sand piles were not circular. In addition, lateral displacement occurred during the construction of SCPs. Most importantly, the bearing ends of sand piles moved considerably, and the centers of sand piles moved

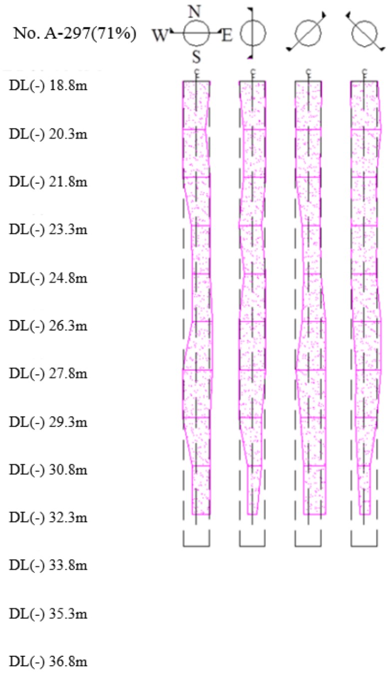


Fig. 1 Shape of SCP unveiled by lattice boring (Im 2003)

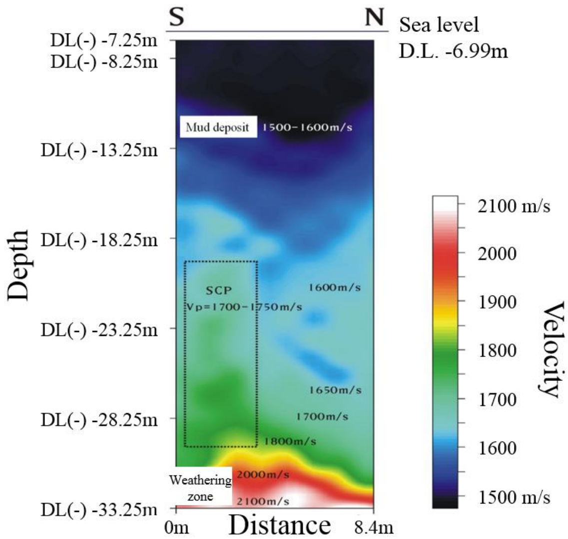


Fig. 2 Shape of SCP unveiled by elastic wave tomography (Im 2003)

about 50 cm at the middle level. Thus, the cross-sectional shapes of sand piles were irregular, not circular, in the ground improved by the SCP method. In addition, the sand piles were not located at the original construction location, showing significant lateral displacement according to the replacement area ratio of sand piles.

Furthermore, partially mixed material of original ground soil and sand was observed in the boundary area of sand piles distorted by the intrusion of clay materials in the private dock stage 1-1 of Busan Newport (Im, 2003). The irregular cross-sectional shape of sand piles and the mixture of sand and original ground material could be found in SCP construction sites. Therefore, the conditions of ground and construction and the replacement area ratios of 40% and 71% could not meet the design assumptions that sand piles were constructed in intact cylindrical shapes and located in the exact coordinates. The stability of reinforced ground could not be guaranteed due to the disagreement between design assumptions and actual sand piles. The results of quality assurance audit were reviewed and analyzed in order to check the shape of sand piles in the private dock stage 1-1 of Busan Newport. Approximately 22,000 sand piles were constructed at the site, and check boring was performed on one in every 500 sand piles. The diameter and length of sand piles were 2.0 m and 20-40 m, respectively. Lattice boring consisting of nine boreholes and seismic exploration were performed to investigate the shape of sand piles. As shown in Fig. 1, the shape of sand piles unveiled by lattice boring were irregular, and a mixture of sand and clay was found on the boundary of the sand piles. Identifying the shape of the sand piles was not possible with the result of seismic exploration, because the wave velocity of sand piles was similar to that of clay ground, as shown in Fig. 2.

The cause of the irregular shape based on the soil mechanics can be explained by the following statements (Im 2003).

1) The sand could flow into the weak side of the ground when compacting sand piles to increase pile diameter due to the deviations of the confining pressure and modulus of deformation in the ground caused by the heterogeneity of the ground.

2) When the sand piles are constructed near existing sand piles, the resisting stiffness of the existing sand piles could cause the sand to flow into the weak side of the ground, expanding the diameter of sand piles.

3) The lateral displacements of sand piles have been found at construction site of high replacement area ratios, as described in a Japanese test construction case. Most importantly, the bearing ends of sand piles moved significantly, and the centers of sand piles moved about 50 cm at the middle level. Therefore, the position error could occur due to the lateral displacement of sand piles at different levels.

4) The rod of SPT could bend or deviate from the course during sampling in which the N-value of the sand pile is about 50 and the depth is about 30 m.

5) The deviation could occur due to the shaking of casing according to weather and sea conditions in construction.

Thus, different types of model ground, such as composite soil, mixed soil, and ameba-like cross-sectional composite soil, were prepared and tested based on the results collected from the SCP construction sites in Korea and Japan. It is because there is no absolutely perfect cylindrical sand pile without the mixture of sand and clay (best case) at the SCP construction sites. On the other hand, sand will never completely mix with clay homogeneous (worst case) in nature. However, partially mixed material of clay and sand was observed at sand pile-clay interface. It was difficult to quantify the mixed extent of partially mixed material at construction site. Therefore, the completely mixed soil was tested and discussed from a conservative point of view in this study. The composite soil and the mixed soil could show both extreme ends as the best and the worst of the sand piles, respectively. Composite soil, mixed soil, and ameba-like composite soil are defined as follows.

Composite soil: Cylindrical sand piles without the intrusion of clay or mixture of sand and clay

Mixed soil: A homogeneous mixture of sand and clay

Ameba-like composite soil: Irregular cross-sectional sand piles without the intrusion of clay or mixture of sand and clay

The mechanical behaviors of the different types of model ground were analyzed according to the replacement area ratio in order to suggest a standard for the quality assurance of SCPs.

2. Model test according to SCP shape

2.1 Types of model tests

An experimental program was designed based on the shape, undrained shear strength parameters, and density of sand in composite soil and the replacement area ratio. The different types of model ground (i.e., composite soil, mixed soil, and ameba-like composite soil) were prepared in order to analyze their mechanical behaviors according to the replacement area ratio of sand, based on the field examples for the shape of SCPs. The mechanical characteristics of the different types of model ground were analyzed with the variation of strength parameters that were obtained by large direct shear tests. The model tests of six groups were performed as described for the content and purpose of the tests in Table 1. The shear strength parameters of the composite soil, mixed soil, and ameba-like composite soil were compared and analyzed according to the replacement area ratio with three groups of tests (see No. 1-3 in Table 1).

Table 1 Description and analysis type of model test groups

No.	Description	Analysis type
1	Variation of shear strength according to replacement ratio of composite soil	Comparison of shear strength parameters of composite soil, mixed soil, and ameba-like composite soil
2	Variation of shear strength according to mixed rate of mixed soil	
3	Variation of shear strength according to ameba-like shape of composite soil	
4	Variation of shear strength of composite soil according to undrained shear strength of original soil	Analysis of how undrained shear strength parameters of original ground influence shear strength parameters of composite or mixed soil
5	Variation of shear strength of mixed soil according to undrained shear strength of original soil	
6	Variation of shear strength of composite soil according to density of SCP sand	Analysis of how density of sand influences shear strength parameters of composite soil

In addition, they were analyzed, with two groups of tests, to determine the influence of the undrained shear strength parameters of original ground on the shear strength parameters of the composite or mixed soil (see No. 4 and 5 in Table 1). Lastly, they were analyzed with a group of tests to determine how the density of sand influenced the shear strength parameters of the composite soil (see No. 6 in Table 1).

2.2 Large direct shear tests

Large direct shear tests were performed in order to analyze the mechanical characteristics of the different types of model ground according to the replacement area ratio of composite soil, the mixture ratio (same amount of sand as the replacement area ratio of composite soil) of mixed soil, and the cross-sectional shape of sand piles. The boundary conditions of the shear tests should be similar to the field conditions in order to obtain the strength parameters. As shown in Fig. 3, the suitable shear tests are described on the location along a failure line underneath a footing. The direction of the failure plane could be the plane of maximum stress obliquity (A and C of Fig. 3) and zero extension direction (B of Fig. 3), depending on the underground location.

In this study, the plane strain state is assumed for the SCP constructed underneath an embankment. Performing tests under the plane strain condition is difficult; therefore, a triaxial test, as an alternative to the plane shear test, is generally performed. However, accurate strength parameters cannot be obtained from the triaxial test due to the deviation of the boundary conditions. Moreover, it is not known which one is a failure line among the plane of maximum stress obliquity, the plane of the zero extension direction, and the plane between the two planes. However, a failure line occurs in the zero extension direction when the failure of ground cannot be achieved in the vulnerable direction. In other words, the ground underneath an embankment will not fail in the direction restraining the deformation of the ground. The shear failure line, which is generated inward in ground improved with SCPs, is close to the plane of the zero extension direction. The plane systemically governs the global stability of the reinforced ground. The shear strength parameters can be estimated by performing direct shear tests or simple shear tests.

If the failure line is the same as the plane of maximum stress obliquity, the shear strength parameters can be obtained by performing triaxial compression tests or triaxial extension tests, according to the location of the failure line. The shear strength parameters at the failure line of the SCP ground consisting of composite soil or mixed soil are different from the shear strength parameters obtained by triaxial tests using the boundary condition of axis symmetry. It seems the boundary condition of plane strain is more reasonable. Therefore, the “accurate” strength parameters in the embankment case should be obtained by a test performed under the plane strain condition.

The shear strength parameters of composite soil, mixed soil, and irregularly shaped composite soil were obtained by performing large direct shear tests according to ASTM D 3080 (1998), and the properties were compared and analyzed. A shear test apparatus was designed especially for the large direct shear tests. The vertical load is loaded on the specimen by using a moment loading method. The lateral load as a shear force is loaded with a motor that is controlled by a gear speed reducer. The strain rate of 1mm/min is maintained during shearing time, and the shear forces under the normal stress of 35 kPa, 71 kPa, and 106 kPa are measured with a load cell with a capacity of 5000 kN at the same time.

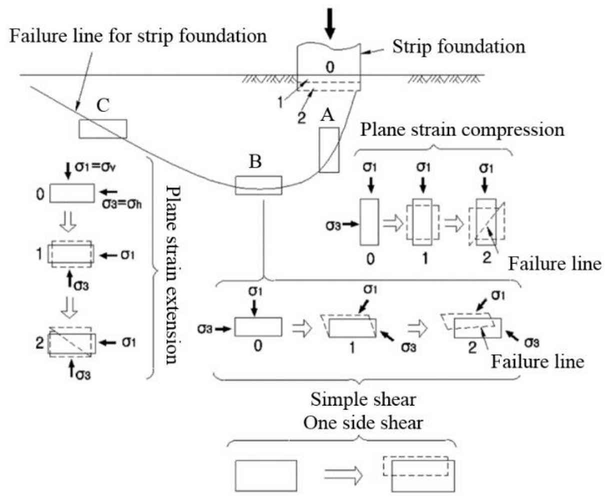


Fig. 3 Conceptual drawing describing shear deformation of soil in plane strain condition

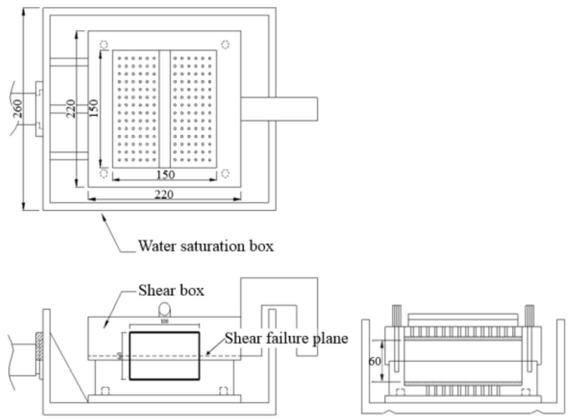


Fig. 4 Schematic drawing of shear box

The size of the shear box is 150 mm (L)×150 mm (W)×60 mm (H), as shown in the schematic drawing in Fig. 4. Bearings are installed in both the upper and the lower shear boxes in order to minimize the mechanical friction. One side of the shear box has a transparent acrylic resin wall, through which the behavior of the model ground can be observed. Moreover, a membrane sheet with lattices is attached to the transparent acrylic resin of the shear box in order to observe the movements of the model ground. The behaviors of the ground can be analyzed by taking pictures before, during, and after shear testing. The experimental procedure is described in section 2.4 in detail.

2.3 Material properties of model tests

Marine clay from Gamcheon Port, in Busan, and Jumunjin standard sand were used as model materials for

Table 2 Properties of marine clay of Gamcheon port

Description	USCS	Specific gravity	Natural moisture content (%)	Liquid limit (%)	Plasticity index (%)	Compression index	Coefficient of consolidation (cm^2/sec)
Property	CL	2.67	82.5	46.0	18.2	0.45	2.14×10^{-4}

Table 3 Properties of Jumunjin standard sand

Description	Max. void ratio	Min. void ratio	Max. dry density (Mg/m^3)	Min. dry density (Mg/m^3)	Specific gravity	Water content (%)	D_{50} (mm)	D_{10} (mm)	Uniformity coefficient	Coefficient of curvature	USCS
Property	0.923	0.604	1.652	1.378	2.65	0.30	0.595	0.443	1.402	0.912	SP

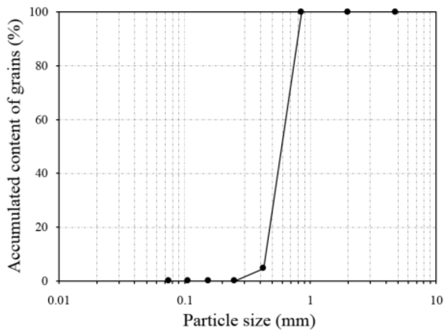


Fig. 5 Grain-size distribution curve of Jumunjin standard sand by sieve analysis

the ground and the SCPs, respectively. The clay used in this study is marine clay obtained from the construction site of the public fish market at Gamcheon Port in Busan. The marine clay of Gamcheon Port is CL, according to USCS. The properties of the marine clay are tabulated in Table 2. Jumunjin standard sand was used as the material of the SCPs. The physical properties and grain-size distribution curve of the Jumunjin standard sand are shown in Table 3 and Fig. 5, respectively.

2.4 Materials and test methodology

Three types of model ground were prepared for the large direct shear tests according to the replacement area ratio of composite soil, the mixed ratio of mixed soil, and the cross-sectional shape of sand piles. The preparation of the different types of model ground and testing procedure are described in the following sections. Preliminary large direct shear tests were performed on reconstituted clay samples after they were consolidated to fit the undrained shear strengths of 7 kPa, 16 kPa, and 33 kPa. The required pressures and consolidation time to fit the shear strengths were obtained from the preliminary tests. The purpose of the initial consolidation is to simply gain the initial shear strength of the clay ground in order to simulate underground stress in the model test. The increment rate of loading was not concerned and considered for the initial consolidation. All reconstituted clay samples were prepared after consolidation to desired undrained shear strength by 1-D consolidation, out of which consolidated direct shear test samples were prepared. The undrained shear strengths of the original clay ground were adjusted to fit 7 kPa, 16 kPa, and 33 kPa, which were measured by the large direct shear tests. The undrained shear strength values were corrected based on an equation due to the dispersion of the measurement in composite soil. The densities of sand piles were adjusted to fit 1.4 Mg/m^3, 1.5 Mg/m^3, and 1.6 Mg/m^3. The replacement area ratio and mixed ratio were changed for the two variances.

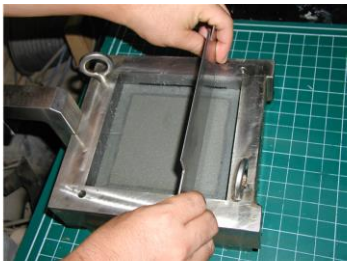


Fig. 6 Molding of disturbed model clay ground

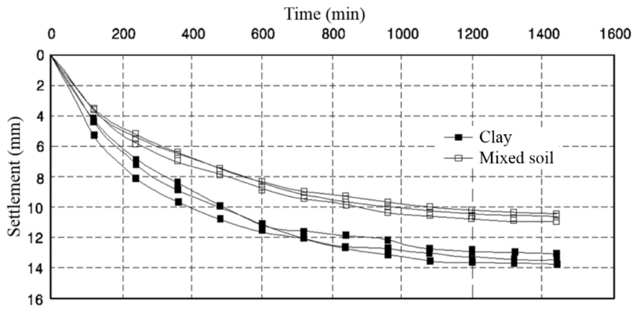
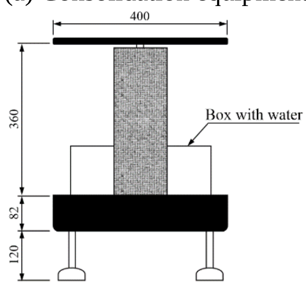
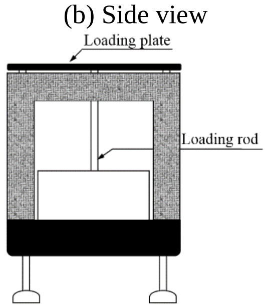


Fig. 7 Time-settlement curve for clay and mixed soil (preparation of saturated model ground)

(a) Consolidation equipment



(b) Side view



(c) Front view

Fig. 8 Saturated clay ground consolidated by applying point load

Fig. 9 Auguring of model clay ground consolidated to desired undrained shear strength

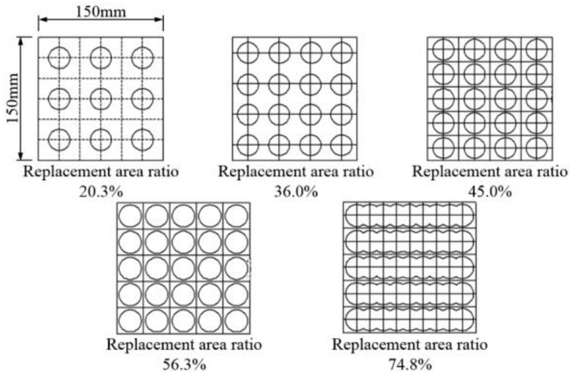


Fig. 10 Illustration for installing sand piles with various replacement area ratios

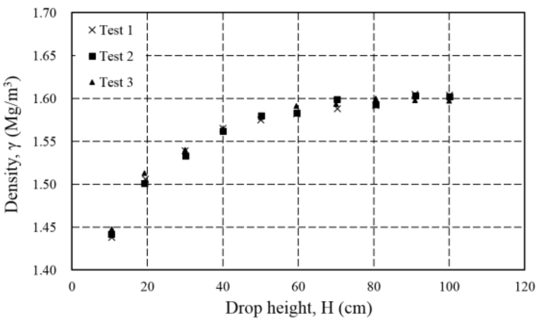


Fig. 11 Relation between dry density and drop height of Jumunjin standard sand

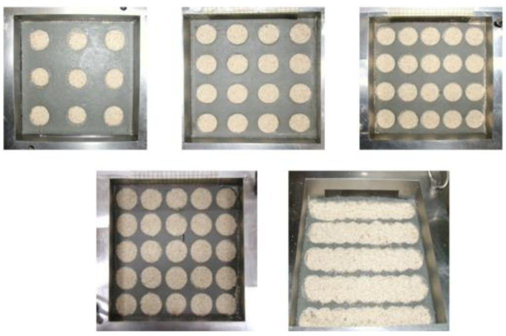


Fig. 12 Model sand piles constructed in consolidated clay ground

Table 4 Average values of undrained shear strength of original clays

Undrained shear strength (kPa)	7	16	33
Average undrained shear strength (kPa)	7.4	18.6	36.1

2.4.1 Preparation of composite soil

Model clay ground disturbed with an agitator was filled and scraped in the shear box, as shown in Fig. 6. The natural moisture content of the clay was 82.5% before initial consolidation of 24 kPa to fit the undrained shear strength of 7 kPa. The water content after the initial consolidation should be lower than liquid limit of 46% because SCP casing holes were sustained without a casing. As a result, the water contents of all model clay grounds should be lower than the liquid limit of 46% to fit the undrained shear strengths of 7 kPa, 16 kPa, and 33 kPa. To minimize the time gap between the clay ground of undrained shear strength and composite ground, the composite ground was prepared and tested right after the clay ground of undrained shear strength was prepared. The saturated clay ground of undrained shear strength of 7 kPa was prepared by applying pressure of 24 kPa to the disturbed soft clay model ground in the shear box for 24 hours, as shown in Fig. 7. Clay samples whose water content became lower than liquid limit of 46% after completion of consolidation were prepared for direct shear tests. The saturated clay ground with undrained shear strengths of 16 kPa and 33 kPa were prepared by applying additional pressure of 20 kPa and 40 kPa for an additional 24 hours to the saturated clay ground of 7 kPa, respectively, as shown in Fig. 8. The static load consisting of various weights delivered by a crane was transferred to the upper plate through a steel bar in order to distribute the point load to the model ground evenly.

Circles of 25.4 mm were marked on cross stripes in square pattern to achieve 5 target replacement area ratios in the clay ground area (150 mmx150 mm) as shown in Fig. 10. Acrylic plates of 150 mmx150 mm guide a casing (see Fig. 9). Then, a circular cross section shape casing of 25.4 mm was pressed in the clay ground before the clay in the casing was augured, as shown in Fig. 9. Smear effect was not expected while inserting casing into the sample for construction of SCP because the casing holes for SCP were sustained without a casing. Sand of a certain density (density of 1.4 Mg/m^3, 1.5 Mg/m^3, and 1.6 Mg/m^3) was filled in the casing before pulling the casing out. Circular SCPs of 25.4 mm were located in square pattern in the clay model ground of the square shape according to the replacement area ratio (20.3%, 36.0%, 45.0%, 56.3%, and 74.8%), as shown in Fig. 10. The replacement area ratio of 45.0% was not perfectly uniform but it was practically accepted as a uniform arrangement as shown for the other area ratios. Furthermore, as shown in Fig. 14, the uniformity of replacement area ratio did not influence shear strength parameters such as cohesion and internal friction angle. The sand piles were constructed by raining the sand through equipment with a 2 mm-wide slot at heights of 6 cm, 20 cm, and 90 cm in order to fulfill the target density of 1.4 Mg/m^3, 1.5 Mg/m^3, and 1.6 Mg/m^3 in the model ground, respectively, as shown in Figs. 11 and 12. The dry density of the sand increased with the drop height, but the density converged with a drop height of 0.9m or more as shown in Fig. 11. After the load corresponding to the normal stress of 35 kPa, 71 kPa, and 106 kPa was loaded on the model ground, direct shear testing was performed on the square shape model ground. The drain condition of the direct shear testing may not be clear, as the clay ground and sand piles

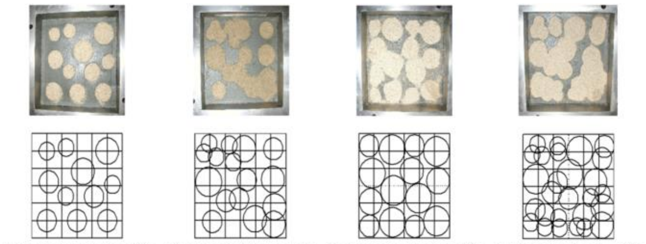


Fig. 13 Installation of ameba-like sand piles with various replacement area ratios

should be in the undrained and drained conditions, respectively, at the construction sites of SCPs. It seems the undrained condition is more reasonable than the drained condition for the direct shear testing of this study.

It is an inevitable consequence that the values of the undrained shear strength of the original ground are somewhat dispersed for all the direct shear tests performed with the composite soil of a certain replacement area ratio. Thus, the undrained shear strengths of the original clay ground were back calculated by using the cohesions of composite soil obtained from the large direct shear testing and Eq. (1), which was developed by using the inverse relationship between the cohesion and the replacement area ratio in this study. The inverse relationship is shown in Fig. 14(a). While the replacement area ratio of 1.0 means composite soil consisting of 100% sand, the replacement area ratio of zero means composite soil consisting of 100% clay. The average value of the back-calculated undrained shear strengths was used as the undrained shear strength of the original clay ground for all the direct shear tests

$$c_m = c_u(1 - a_s) \tag{1}$$

where c_m is the cohesion of composite soil, c_u is the undrained shear strength of the original clay ground, and a_s is the replacement area ratio (sectional area of sand pile/cross-sectional area of clay ground and sand pile). The average undrained shear strengths are tabulated in Table 4. Hereafter, the variation of the internal friction angle according to the replacement area ratio was calculated with the average undrained shear strengths.

2.4.2 Preparation of mixed soil

The amounts of sand and clay which had the same area percentage corresponding to the replacement area ratio of the composite soil were completely mixed by using an agitator. The model ground consisting of mixed soil was consolidated (the undrained shear strength of 7 kPa, and 16 kPa) by applying the same pressure as that applied to the original clay ground of composite soil for 24 hours in the shear box. The direct shear testing was performed with the normal stress of 35 kPa, 71 kPa, and 106 kPa for the model ground.

2.4.3 Preparation of ameba-like composite soil

As shown in Fig. 13, the ameba-like sand piles, according to the replacement area ratio, were installed in the model clay ground in order to analyze the properties of the shear strength parameters. The disturbed soft model clay ground was consolidated up to the undrained shear strength of 7 kPa by applying the pressure of 24 kPa for 24 hours in the shear box. The density of the Jumunjin standard sand was 1.5 Mg/m^3, which was the relative density of 50%.

3. Results and analysis of model tests

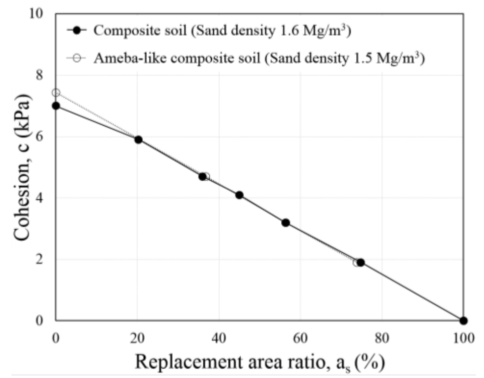
3.1 Test results and analysis for shear strength of composite soil and ameba-like ground

The composite soil of model ground consisted of the original clay consolidated by the desired undrained shear strength and the sand with a density of 1.6 Mg/m^3. Large direct shear tests were performed on the model ground with various replacement area ratios. The centers of sand piles were not located in the same position due to the various replacement area ratios. Preliminary tests were performed, and the inflection points of shear stresses were found before the horizontal displacement of 10 mm. The relationship between stress and strain was plotted and analyzed up to the horizontal displacement of 10 mm to be consistent in arranging the test results.

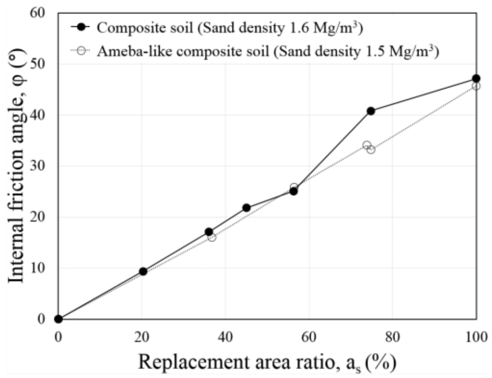
The shear strength parameters relative to the replacement area ratios in the composite ground were plotted, as shown in Fig. 14. The peak value of the internal friction angle increases with the increasing replacement area ratio, while cohesion decreases with the increasing replacement area ratio. It is a general tendency, as the amount of sand activating the internal friction angle increases with the increasing replacement area ratio. Likewise, the direct shear tests were performed with irregularly shaped sand piles, which did not have the intrusion of clay or the mixture of sand and clay in order to analyze the characteristic of shear strength. The shear behavior of the ameba-like composite soil is similar to that of the composite soil as shown in Fig. 14. Therefore, the distorted shape of cylindrical sand piles does not influence the shear strength of model ground with the same cross-sectional area of cylindrical sand piles unless sand piles are mixed with native soil.

3.2 Test results and analysis for shear strength of mixed soil

The amount of sand and clay corresponding to the replacement area ratio of the composite soil was completely mixed by using an agitator. The model ground consisting of mixed soil was consolidated by applying the same pressure as that applied to the original clay ground of composite soil for 24 hours in the shear box before large direct shear testing was performed. Cohesion and internal friction angles relative to the replacement area ratios for the average undrained shear strengths of 7.4 kPa and 18.6 kPa are shown in Fig. 15. The cohesion remains constant with the increasing replacement area ratio, but the cohesion significantly decreases after the replacement area ratio of 57.3%. While the internal friction angle of mixed soil is about 25% of composite soil at low replacement area ratios (i.e., less than 57.3%), the internal friction angle of mixed soil rapidly increases after the replacement area ratio of 57.3%. The tendency occurred regardless of the undrained shear strength of the model ground.

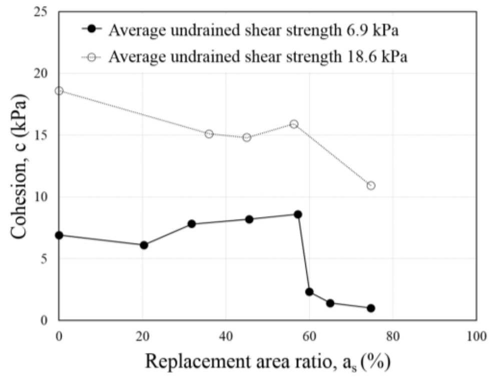


(a) Cohesion

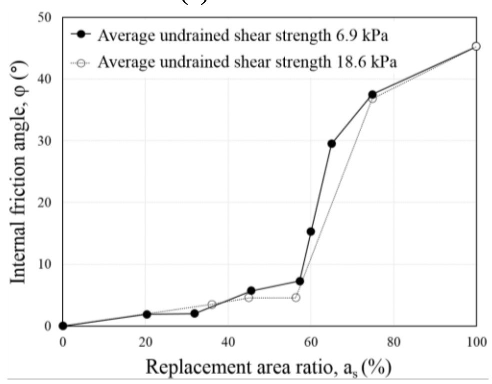


(b) Internal friction angle

Fig. 14 Variation of shear strength parameters with replacement area ratio for composite soil and ameba-like composite soil (undrained shear strength of 7 kPa)



(a) Cohesion



(b) Internal friction angle

Fig. 15 Variation of shear strength parameters with replacement area ratio for mixed soil (undrained shear strengths of 7 kPa and 16 kPa)

Table 5 Variation of shear strength parameters with undrained shear strength for composite soil

Type	Undrained shear strength (kPa)	Replacement area ratio (%)	Cohesion (kPa)	Internal friction angle (°)
Composite soil	7	20.3	5.9	9.4
		36	4.7	17.1
		45	4.1	21.8
		56.3	3.2	25.1
		74.8	1.9	40.8
	16	36	12.0	16.6
		56.3	8.2	25.2
		74.8	4.7	40.6
	33	36	23.1	14.8
		56.3	15.8	24.9
		74.8	9.1	38.8

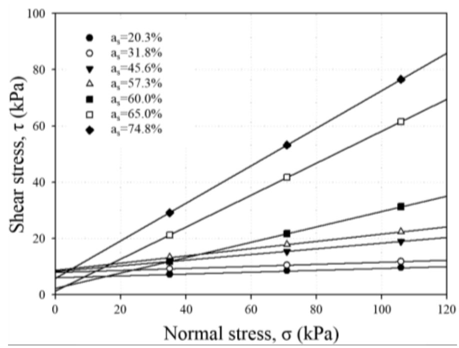
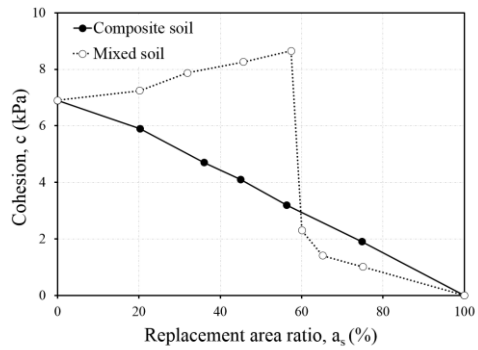
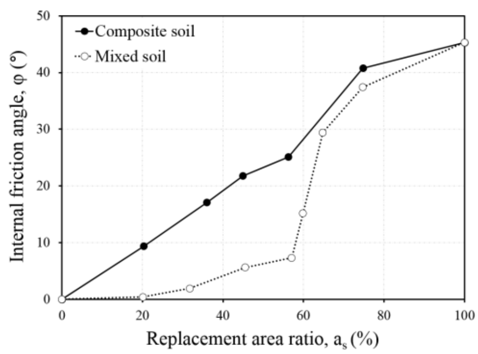


Fig. 16 Relation between maximum shear stress and normal stress for mixed soil (average undrained shear strength of 7.49 kPa)



(a) Cohesion



(b) Internal friction angle

Fig. 17 Comparison of shear strength parameters of composite soil and mixed soil

3.3 Comparative analysis for shear strength of composite soil and mixed soil

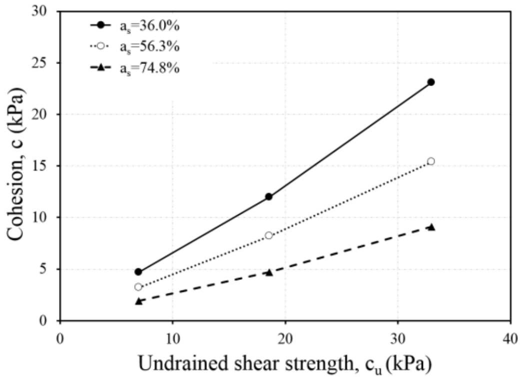
The results of the large direct shear tests were compared and analyzed for the composite soil and mixed soil. Most importantly, the relationship between the maximum shear stress and the normal stress was compared and analyzed with the increasing replacement area ratio for composite soil and mixed soil, as shown in Table 5 and Fig. 16, respectively. While the slope of the Mohr-Coulomb failure criteria envelope steadily increases with the increasing replacement area ratio for composite soil, the deviation of the slope rapidly increases at the replacement area ratio ranging from 60 to 65% for mixed soil.

The shear strength parameters relative to the replacement area ratio for both composite soil and mixed soil are shown in Fig. 17. While the internal friction angle of composite soil increases linearly with the increasing replacement area ratio, that of mixed soil does not clearly increase at replacement area ratios lower than 60% but rapidly increases at the replacement area ratio of 65%, showing a value similar to that of composite soil. Thus, if the amount of sand corresponding to the minimum replacement area ratio of 65% is secured, one can have the same shear resistance as composite soil, regardless of the shape of sand piles. Likewise, if the amount of sand corresponding to the minimum replacement area ratio of 65% is secured, the cohesion of mixed soil would behave as that of composite soil, as shown in Fig. 17(a). It is because the soil type suddenly changed at the replacement area ratio ranging from 60 to 70%. The replacement area ratio of 65% appears to be the turning point that converts silt-clay materials to granular materials, as granular materials are defined-soils in which 35% or less pass through a 0.075-mm sieve-by the AASHTO soil classification system.

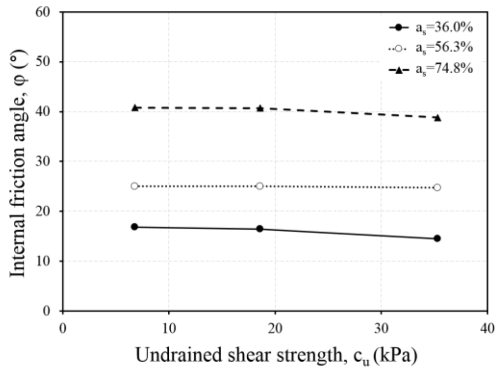
3.4 Test results and analysis for shear strength of composite soil according to undrained shear strength of original clay

Direct shear tests were performed with the composite soil in order to estimate the influence of the undrained shear strength of the original clay ground on the shear strength of composite soil. The characteristics of the shear strength of composite soil were analyzed by changing the undrained shear strength of the original clay ground, while the density of sand and the replacement area ratio were fixed. The original clay ground was consolidated by the undrained shear strengths of 7 kPa, 16 kPa, and 33 kPa. The direct shear testing was performed with the composite soil by changing the replacement area ratio while the sand piles with a density of 1.6 Mg/m^3 were installed into the clay ground.

The shear strength parameters of composite soil relative to the undrained shear strength of the original clay ground were tabulated and plotted, as shown in Table 5 and Fig. 18. While cohesion increases linearly with the increasing undrained shear strength, as shown in Fig. 18(a), the internal friction angle does not show a clear change, as shown in Fig. 18(b). The internal friction angle remains constant with the increasing undrained shear strength, regardless of the replacement area ratio. The internal friction angle at the undrained shear strength of 7 kPa decreases to about 0.86 times that of 33 kPa for the replacement area ratio of 36%. The ratio of the decreased internal friction angle ranges from 0.98 to 1.0 in most cases. This tendency occurs in the low replacement area ratio and the high undrained shear strength of the original clay ground. The internal friction angle of sand may not be fully activated before the shear failure of model ground, likely because the cohesion of original clay ground has a larger influence on the shear resistance of the composite soil with the higher undrained shear strength of original clay ground.

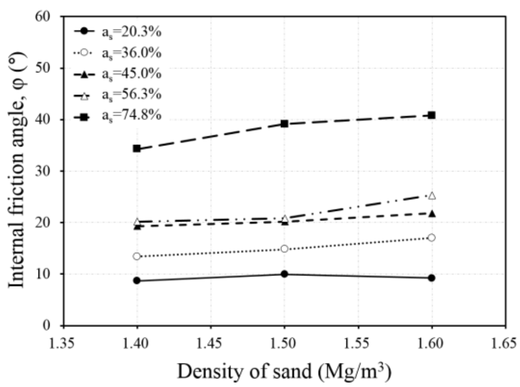


(a) Cohesion



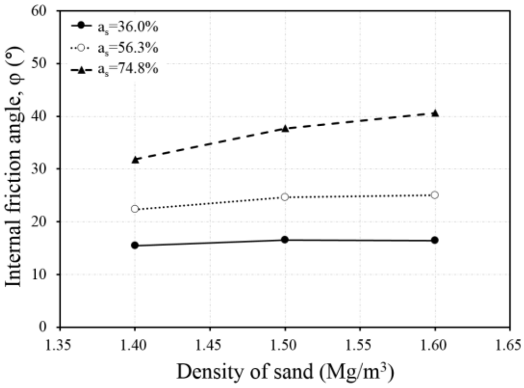
(b) Internal friction angle

Fig. 18 Variation of shear strength parameters with undrained shear strength for composite soil

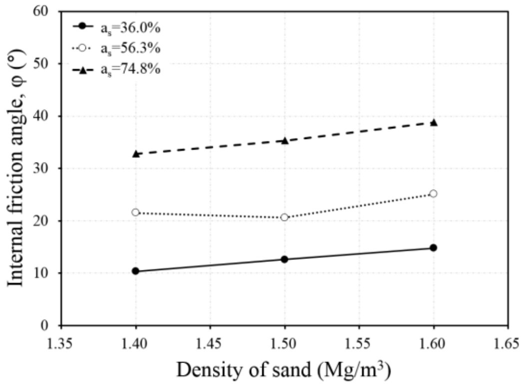


(a) Undrained shear strength of 7 kPa

Fig. 19 Variation of internal friction angle with density of sand in original clay grounds of various undrained shear strengths



(b) Undrained shear strength of 16 kPa



(c) Undrained shear strength of 33 kPa

Fig. 19 Continued

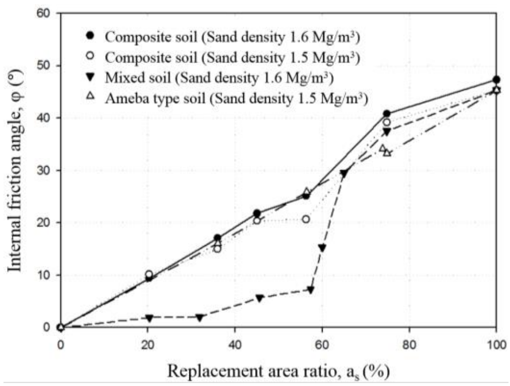


Fig. 20 Relation between internal friction angle and replacement area ratio for composite, mixed, and ameba-like composite soil

3.5 Test results and analysis for shear strength of composite soil according to density of sand

The direct shear tests were performed by changing the density of sand in order to analyze how the density of sand influences the shear strength of composite soil. The sand piles were constructed with the densities of 1.4 Mg/m^3, 1.5 Mg/m^3, and 1.6 Mg/m^3 for the tests after the original clay ground was consolidated by the undrained shear strengths of 7 kPa, 16 kPa, and 33 kPa. Fig. 19 shows the internal friction angle relative to the density of sand while changing the replacement area ratio and undrained shear strength of the original clay ground. Regardless of the undrained shear strength of the original clay ground, the internal friction angle increased with the increasing density of sand at high replacement area ratios (i.e., over 56.3%). In Fig. 19(a), there was very small difference between the results a_s=45% and 56.3% because the values of the undrained strength of the original ground were somewhat dispersed for all the direct shear tests performed with the composite soil of a certain replacement ratio as discussed in section 2.4.1. The internal friction angle decreased by about 0.9 and 0.8 times for the sand densities of 1.5 Mg/m^3 and 1.4 Mg/m^3, respectively, than for the sand density of 1.6 Mg/m^3 at the replacement area ratio of 74.8%.

3.6 Test results and analysis for shear strength according to cross-sectional shape of SCP

Large direct shear tests were performed with composite soil, mixed soil, and ameba-like composite soil while changing the replacement area ratio. The test results were compared and analyzed. The internal friction angle of ameba-like composite soil was plotted with that of composite soil (i.e., highest quality SCP) and that of mixed soil (i.e., lowest quality SCP), as shown in Fig. 20. The design internal friction angle was secured, regardless of the shape of sand piles or the mixture of sand and clay, when the replacement area ratio was higher than 65%. The replacement area ratio of 65% appears to be the turning point that converts silt-clay materials to granular materials, as granular materials are defined-soils in which 35% or less pass through a 0.075-mm sieve-by the AASHTO soil classification system. In other words, the quality control procedure of the SCP method could be secured and simplified by measuring the input of sand. When the replacement area ratio is lower than 65% and the sand is mixed with clay, the design internal friction angle is not secured. Thus, the continuation and integrity of sand piles should be secured by construction management.

4. Conclusions

It was found that the irregular cross-sectional sand pile with partially mixed material at sand pile-clay interface could be constructed at an SCP construction site. There is no absolutely perfect cylindrical sand pile without the mixture of sand and clay (best case), and complete homogeneous mixture of the clay and sand (worst case) at the SCP construction sites. Therefore, the mechanical behavior of model ground with various SCP shapes was analyzed in order to suggest a standard for the quality assurance of the SCP method. The results of the analyses are summarized in the following statements.

• Large direct shear tests were performed with various replacement area ratios on the composite soil (i.e., the model ground of ideal cylindrical sand piles). The internal friction angle increased with the increasing replacement area ratio, while cohesion decreased with the increasing replacement area ratio. This is a general tendency, as the amount of sand activating the internal friction angle increases from 9.4° to 40.8° with the increasing replacement area ratio ranging from 20.3% to 74.8%. The shear behavior of ameba-like composite soil was similar to that of the composite soil. Therefore, the distorted shape of cylindrical sand piles does not influence the shear strength of model ground with the same cross-sectional area of cylindrical sand piles unless sand piles are mixed with native soil.

• Large direct shear tests were performed in order to analyze the behavior of the shear strength parameters of composite soil according to the undrained shear strength of the original clay ground. While cohesion increased linearly with the increasing undrained shear strength, the internal friction angle did not show a clear change with the increasing undrained shear strength regardless of the replacement area ratio. The internal friction angle of sand may not be fully activated before the shear failure of model ground, because the cohesion of original clay ground has a larger influence on the shear resistance of the composite soil with the higher undrained shear strength of the original clay ground.

• Regardless of the undrained shear strength of the original clay ground, the internal friction angle increased with the increasing density of sand at high replacement area ratios (i.e., over 56.3%). The internal friction angle decreased about 0.9 and 0.8 times for the sand densities of 1.5 Mg/m^3 and 1.4 Mg/m^3, respectively, than for the sand density of 1.6 Mg/m^3 at the replacement area ratio of 74.8%.

• Large direct shear tests were performed on composite soil, mixed soil, and ameba-like composite soil while changing the replacement area ratio. The design internal friction angle was secured, regardless of the shape of sand piles or the mixture of sand and clay, when the replacement area ratio was higher than 65%. The design internal friction angle is not secured when the replacement area ratio is lower than 65% and the sand is mixed with clay. Thus, the continuation and integrity of sand piles should be secured by construction management.

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