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PILE SHAFT RESISTANCE PARTIALLY OVERLAPPING WITH BACKFILLED SOILS OF VARIOUS PROPERTIES

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ABSTRACT: When a building is reconstructed, many existing piles that support the building are removed. After removing large-diameter piles, the holes are backfilled with various materials. In this study, vertical loading model tests were conducted for the case where a newly constructed pile overlaps clayey backfill columns whose properties differ from those of the surrounding sandy soil. The influence of the variation of the unconfined compressive strength of clayey soil backfill on the shaft resistance of a pile was investigated. The results indicated that the process of soil failure up to the maximum pile shaft resistance varies with the strength of the clayey soil backfill. The maximum shaft resistance of the pile was calculated based on the failure criteria of the soil and the cavity expansion theory, and the strength effect of the clayey soil backfill was estimated. For the pile shaft resistance overlapping partially clayey backfill columns in sandy soil to be equal to the pile shaft resistance in a homogeneous sandy soil, the shear strength of the clayey soil backfill must be higher than the pile shaft resistance in a homogeneous sandy soil.

Keywords: New pile, Pile shaft resistance, Clayey soil backfill, Sand, Loading test

1. INTRODUCTION

High-rise buildings and many buildings on soft ground are supported on pile foundations. The vertical load of the building acting on the pile head is transmitted to the ground by two resistance components, namely the pile tip and the shaft. The vertical bearing capacity of a pile is affected not only by the ground conditions and the pile size, shape, and installation method but also by adjacent construction and construction history. The additional displacement and pile stresses caused by the penetration of piles newly constructed in the ground have been studied using numerical analysis and in-situ measurements to examine the interaction between piles and the ground during adjacent construction [1-3].

The consideration of existing foundations and piles has become increasingly important in the reconstruction of buildings. Old piles can be either removed, reused, or retained. The reuse of existing foundations has been studied. Centrifuge loading tests were conducted under the assumption that new piles were installed between existing piles [4,5]. The results showed that the effect of the existing piles on the vertical bearing capacity of the new piles was small. Studies on the removal of existing piles and the effect of the material used to backfill the holes on settlement have been conducted based on field measurements and numerical analyses of subsidence caused by pile removal [6-8]. It has been reported that backfill soil is heterogeneous and differs from the surrounding soil. Therefore, the present authors previously conducted compressive loading model tests under the assumption that a newly constructed pile overlaps the backfill sand where an existing pile was removed [9,10]. In the tests, the effect of the density of the backfill sand on the pile shaft resistance was investigated. The backfill sand was used the same way as the surrounding soil.

However, the backfill soil is not only sandy soil (e.g. soil used at construction sites or recycled sand), but also liquefied stabilized soil and cement milk. Backfilling processes for liquefied stabilized soil and cement milk include methods using tremie tubes and mixing by screw auger etc. In a previous study, to investigate pile shaft resistance, the shear strength between clay and steel was experimentally determined using a direct shear test apparatus [11]. The results indicated that the failure mode (complete slip at the material boundary, shear failure in clay, or a mixture of these modes) depended on the surface roughness of the steel. The effect of surface roughness at the boundary on shear behavior has also been investigated in large-scale shear tests that focused on the shear strength between clay and concrete [12]. Recently, the influence of temperature on the cyclic behavior at the clay-concrete interface [13] and the influence of surface roughness and boundary conditions (constant normal load, constant normal stiffness, constant volume) on the shear behavior between sand-clay mixture and pile material have been investigated [14]. The stress-strain behavior and behavioral characteristics of improved soils, in which cement or gypsum are added to clayey soils, have also been studied [15-17]. These studies considered only the mechanical properties of the soil material and the frictional behavior along the pile shaft under axisymmetric conditions. Considering actual projects, it can be inferred that it is very difficult to carry out the backfilling process with the same geomaterials according to the strata in the case of soils with alternating layers of sandy and cohesive soils.

In the present study, loading model tests were conducted for a pile that overlaps a clayey backfill column whose mechanical properties are different from those of the surrounding sand soil. The effect of various strengths of the clayey soil backfill on the pile shaft resistance is investigated. The effect of different soil types in the backfill material on the pile shaft resistance characteristics is also discussed.

2. RESEARCH SIGNIFICANCE

When a building is reconstructed, new piles are installed relatively soon after the existing piles are removed and the holes are backfilled. In some projects, the newly constructed piles overlap the backfill soil. The backfill soil is composed of various materials (e.g. recycled construction sand, mountain sand, fluidized soil, cement milk) and its properties differ from those of the surrounding soil. Therefore, it is very important to clarify the effects of the mechanical properties and variation of backfill materials on the vertical bearing capacity of new piles to improve building safety.

3. METHODOLOGY

3.1 Outline of Model Tests

A schematic diagram of the experimental equipment used in this study is shown in Fig. 1.

A cylindrical chamber with a diameter of 500 mm and a depth of 600 mm was used to prepare the model soil. This chamber provides vertical and horizontal pressure on the model soil. Tohoku quartz sand number 6 (see Table 1) was used to create the model sand soil. The model soil was medium sand with a relative density, D_r, of 60%. It was deposited using the air pluviation method.

A clayey backfill column half overlapped the model pile, as shown in Fig. 2. The clayey backfill column was prefabricated before the model soil was made to adjust its shape and strength. It was prepared using the procedure described below. The half-overlapping clayey backfill column and the pile were set up after 300 mm of the soil had been deposited.

The model pile was made of an aluminum pipe with a diameter of 30 mm. The outer surface of the pile was roughened using thermal spray aluminum to simulate a cast in-situ pile. The pile tip was inserted into a cylindrical jig to eliminate pile tip resistance. The pile was set into the model soil with an embedded length of 280 mm.

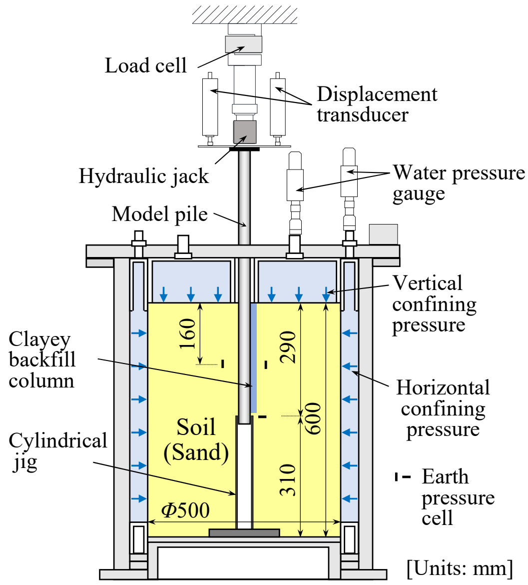

Fig. 1 Setup of experimental apparatus

Table 1 Physical and mechanical properties of Tohoku quartz sand

Parameter	Value
Maximum density, ρ_{dmax} (g/cm^3)	1.712
Minimum density, ρ_{dmin} (g/cm^3)	1.397
Mean grain size, D_{50} (mm)	0.32
Internal friction angle, φ (deg.) *	32.1
Dilatancy angle, ψ (deg.) *	2.1

*Note: *The relative density of sand, D_r, is 60%.*

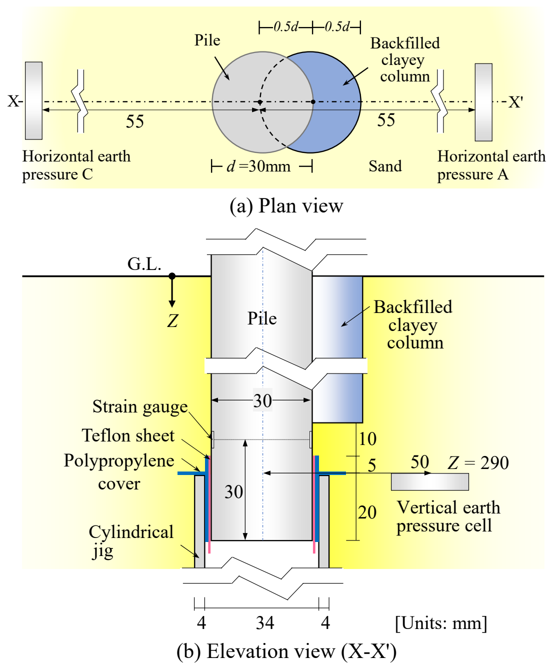

Fig. 2 Layout of pile and clayey backfill column

3.2 Preparation of Clayey Backfill Columns

In this test, relatively low-strength clayey soil with added cement was prepared instead of the liquefied stabilized soil for the fabrication of the test specimens. To create clayey soil backfills, a slurry containing ordinary Portland cement was mixed with kaolin clay shown in Table 2.

Table 2 Physical properties of clay and cement

Parameter		Value
Clay (Kaolin: New snow fine)	Specific gravity, G_s	2.71
	Liquid limit, *LL* (wt. %)	50.9
	Plastic limit, *PL* (wt. %)	18.4
	Plastic index, *PI* (wt. %)	32.5
Ordinary Portland cement	Specific gravity, G_s	3.16

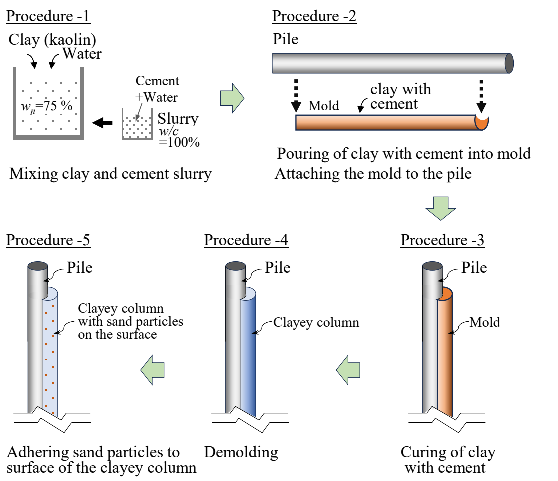

Fig. 3 Schematic diagram of preparation of clayey backfill column

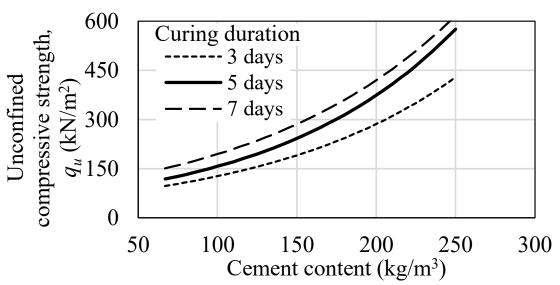

Fig. 4 Relationship between cement content and unconfined compressive strength of clayey soil

The preparation procedure used for the clayey backfill columns is shown in Fig. 3. The clayey soil backfill was obtained by adding a cement slurry to kaolin clay with a water content of 75%, concerning previous studies [15]. The strength of the clayey soil backfill was adjusted (based on a preliminary study) by adding cement to achieve the specified strength in the period between the mixing of the materials and the pile loading tests (5 days of curing), as shown in Fig. 4. In consideration of actual construction conditions, the sand particles were attached to the outer surface of the clayey column to prevent slipping at the boundary between the clayey column and the soil.

3.3 Test Conditions

The test cases are shown in Table 3. The target uniaxial compressive strength, q_u, of the clayey backfill column was set to 150 or 400 kN/m^2. The value of q_u accounted for the shaft resistance of the pile in the sand without backfill soil, and the variation in the strength of the clayey soil backfill. The confining pressures (σ_{v0}, σ_{h0}) on the soil were set assuming a depth of 6.5 or 13 m from the ground line (model names ending in *l* and *h*, respectively). The loading tests of the pile were conducted under six conditions: four conditions with overlapping clayey backfill columns and two conditions with no backfill soil for comparison.

Table 3 Test cases

Model name	MM-l	qu150-l	qu400-l	MM-h	qu150-h	qu400-h
D_r (%)	60	60	60	60	60	60
q_u (kN/m^2)	–	150	400	–	150	400
σ_{v0} (kPa)	100	100	100	200	200	200
σ_{h0} (kPa)	50	50	50	100	100	100

q_u is the target unconfined compressive strength of clayey backfill column, σ_{v0} and σ_{h0} are vertical and horizontal confining pressure on the soil surface, respectively.

Compressive loading tests of piles were conducted. A load was applied to the pile head by hydraulic pressure. During the loading tests of the pile, the compressive load on the pile head, settlement of the pile head, and confining pressure on the model soil were measured by load cells, displacement transducers, and a water pressure gauge, respectively, as shown in Fig. 1. The earth pressure was measured by placing earth pressure gauges within the section at a depth of 160 mm from the model soil surface, and below the clayey backfill column as shown in Fig. 2.

After the loading test, the model pile and clayey backfill columns were removed from the soil to verify the failure condition and measure the thickness of the remaining clayey soil backfill (shear band) on the pile surface. Furthermore, uniaxial compression tests on specimens that were cured in the same way as the clayey backfill columns were carried out to confirm the mechanical properties of the clayey soil. The results are shown in Table 4 and Fig. 5. The compressive strength of the clayey soil generally matched the target value, but Young's modulus varied slightly due to *bedding error*.

Table 4 Results of unconfined compression test of clayey soil

Model name	qu150-l	qu400-l	qu150-h	qu400-h
ρ_c (g/cm^3)	1.59	1.53	1.58	1.54
q_u (kN/m^2)	144.4	447.3	154.0	399.1
E_{50} (MN/m^2)	24.1	46.8	20.1	29.5

ρ_c, E_{50} are density and Young's modulus of clayey soil.

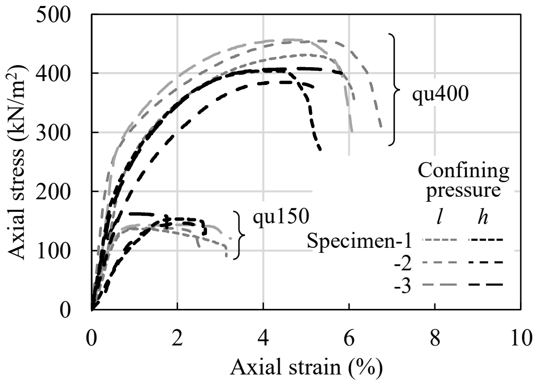

Fig. 5 Stress-strain curves for uniaxial compression tests on clayey soil

4. RESULTS

4.1 Pile Shaft Resistance and Earth Pressure Around Pile

The relationship between the pile shaft resistance and the pile head settlement is shown in Figs. 6 and 7. At the beginning of loading, the initial stiffness of the shaft for a given confining pressure is similar for all models. The pile shaft resistance reaches its maximum value, $f_{s,\,max}$, when the pile head is around 2 mm, regardless of the test conditions. Compared to the results for the soil without clayey backfill columns, the maximum value of the pile shaft resistance showed changes of -7% to +63% (q_u150-l ≈ MM-l < q_u400-l) under restraining pressure *l* and -42% to -2% (q_u150-h < q_u400-h ≈ MM-h) under restraining pressure *h*. The difference in the maximum shaft resistance of the pile is related not only to the uniaxial compressive strength of the clayey soil backfill, but also to the amount of change in earth pressure, as discussed below.

The initial stiffness, K_l, calculated from the relationship between the pile shaft resistance and the pile displacement is shown in Fig. 8. For qu400, the initial stiffness increases with increasing compressive strength. This tendency is more pronounced for the cases with lower confining pressure on the sand soil.

The relationship between the change in horizontal earth pressure and the settlement of the pile head is shown in Figs. 9 and 10. The earth pressure measured by earth pressure gauges A and C tended to change gradually until around the maximum shaft resistance of the pile and then remained almost constant. In the sand soil without backfilled columns, the earth pressure increased with increasing confining pressure. This is because a higher confining pressure leads to a greater initial shear stiffness of the sandy soil and thus a more pronounced volume change (dilatancy) of the sand as the pile is pushed. On the other hand, the earth pressure was suppressed in the soil with clayey backfill columns even when the surrounding soil was sand. This is likely due to the clay having very low dilatancy.

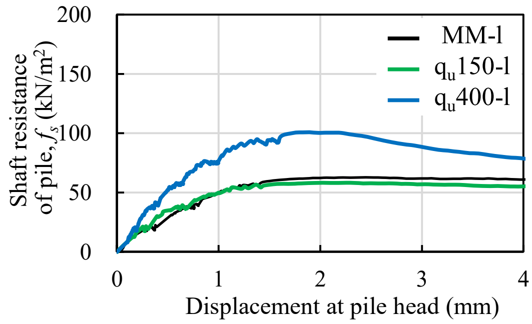

Fig. 6 Relationship between shaft resistance of pile and displacement at pile head (σ_{v0}=100 kPa, σ_{h0}=50 kPa)

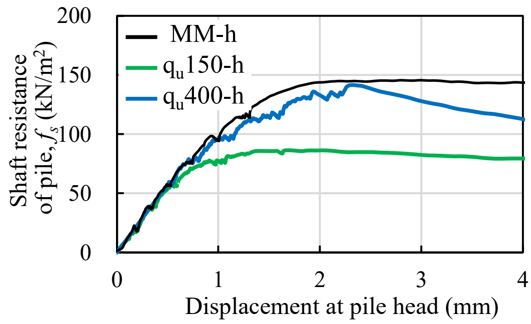

Fig. 7 Relationship between shaft resistance of pile and displacement at pile head (σ_{v0}=200 kPa, σ_{h0}=100 kPa)

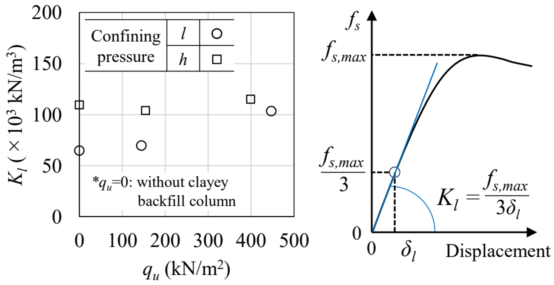

Fig. 8 Effects of compressive strength of clayey backfill column on initial stiffness of relationship between pile shaft resistance and displacement

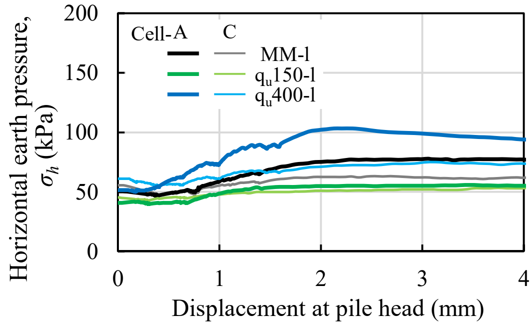

Fig. 9 Horizontal earth pressure measured at depth of 160 mm from model sand surface (σ_{v0}=100 kPa, σ_{h0}=50 kPa)

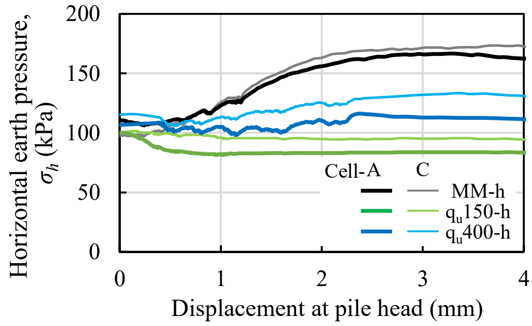

Fig. 10 Horizontal earth pressure measured at depth of 160 mm from model sand surface (σ_{v0}=200 kPa, σ_{h0}=100 kPa)

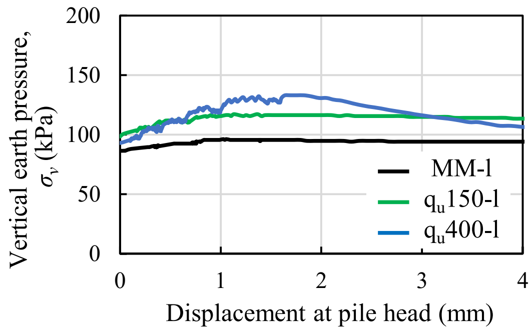

Fig. 11 Vertical earth pressure below clayey backfill column for MM-l, qu150-l, and qu400-l

The vertical earth pressure measured below the clayey column under restraining pressure *l* is shown in Fig. 11. For MM-l and qu150-l, the earth pressure reached its maximum at a pile settlement of about 1 mm and then remained almost constant. For qu400-l, the earth pressure increased gradually up to a pile settlement of about 2 mm, where the pile shaft resistance reached its maximum. After the earth pressure reached its peak, it decreased and remained. This difference in trend is assumed to be due to the different processes leading to the shear failure of the clayey backfill column depending on the uniaxial compressive strength, as discussed below.

4.2 Failure of Clayey Backfill Columns

The failure conditions of the clayey backfill columns after the loading tests are shown in Fig. 12. Vertical cracks were observed in the clayey backfill columns along the entire length of the pile. When the pile was checked after the loading test, a thin layer of clayey soil backfill was found on its surface. Hence, the failure mode was considered to be a shear failure in the clayey backfill columns. The measured thickness of the clayey backfill columns (shear band) was 0.11 to 0.17 mm in all test cases.

A schematic diagram of the shear failure process of a clayey backfill column is shown in Fig. 13. For qu150, it was presumed that the clayey backfill columns and the sand soil near the pile (I + II in Fig. 13) would fail in the same progression because the vertical earth pressure under the soil column reached its peak before the pile shaft resistance reached. For qu400, the clayey soil backfill was stiffer than the sand soil and the clayey backfill column settled with the pile. The vertical earth pressure under the clayey backfill column and the pile shaft resistance reached their peaks at the same settlement (about 2 mm). It was presumed that the clayey backfill column would fail when the sum of the shear strength between the clayey backfill column and the sand (III in Fig. 13) and the subgrade reaction below the column would exceed its shear strength (I + II in Fig. 13).

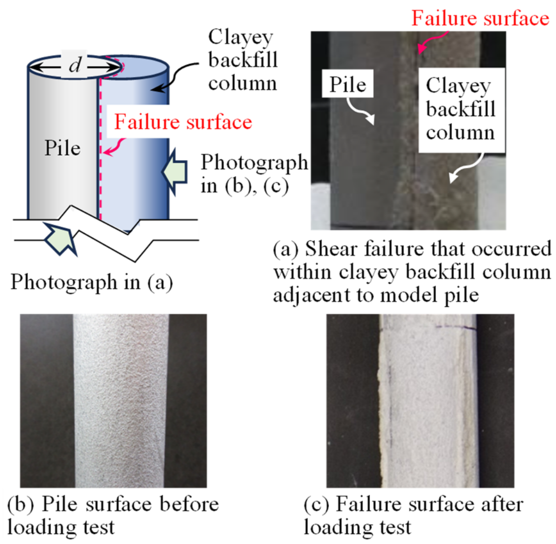

Fig. 12 Failure characteristics in region between clayey backfill column and pile

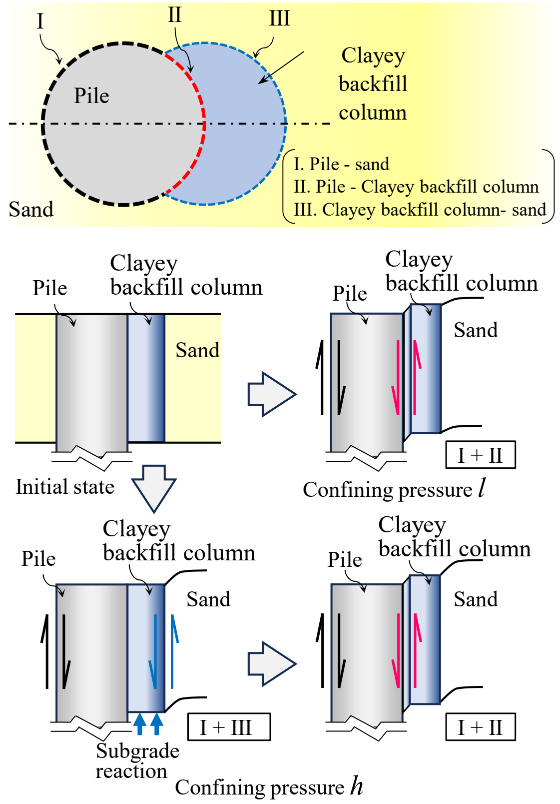

Fig. 13 Schematic diagram of interactions among pile, clayey backfill column, and sand

5. EFFECT OF CLAYEY BACKFILL COLUMNS ON PILE SHAFT RESISTANCE

The effect of the strength of clayey soil backfill on the maximum pile shaft resistance is estimated based on the shear failure criterion of the soils.

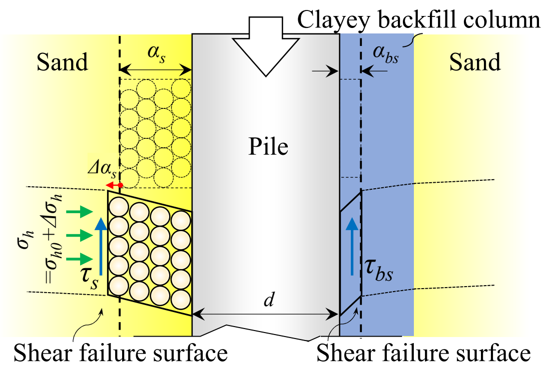

Fig. 14 Schematic diagram of volume change due to shear deformation of sand and clayey backfill column

In this study, the soil types in the surrounding soil and backfilled soil are different. Hence, when the pile shaft resistance is calculated, it is necessary to consider the differences in mechanical properties of the soil, and soil dilatancy shown in Fig. 14. Then, the shear stress of the soil near the pile is considered by dividing it into three zones: a zone that includes the clayey backfill column (Zone A), a zone that of includes the area around the pile (Zone B), and a zone that is symmetrical to the clayey backfill column (Zone C), as shown in Fig. 15.

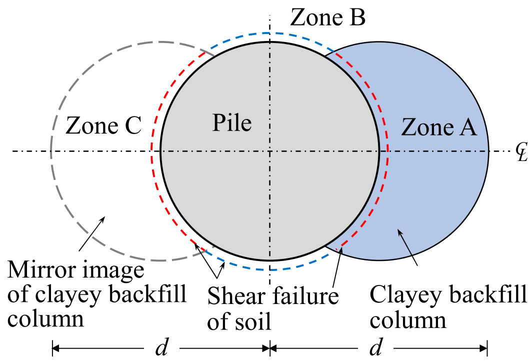

Fig. 15 Schematic diagram of zone for estimating the shear failure in soil near pile

The maximum pile shaft resistance is calculated as the sum of the shear stress at the failure surface considering the ratio of the dominant length at the failure surface, η, as shown in Eq. (1). The location of the shear failure surface is considered the thickness of the shear zone between the backfill soil and the surrounding ground.

$$f_{s,cal} = \frac{d + 2\alpha_{bs}}{d} \cdot \tau_A \cdot \eta_A + \frac{d + 2\alpha_s}{d} \cdot \tau_B \cdot \eta_B + \frac{d + 2\alpha_s}{d} \cdot \tau_C \cdot \eta_C \tag{1}$$

The shear stress, τ_j, in each zone is estimated using Eqs. (2) and (3) based on Coulomb's failure criterion. The mechanical parameters of the soil are set to $c_{u,j} = 0$ for sand, and $\varphi_j = 0$ and $c_{u,j} = q_u/2$ for clayey soil backfill.

$$\tau_j = c_{u,j} + \sigma_{h,j} \cdot \tan\varphi_j' \tag{2}$$

$$\varphi_j' = \tan^{-1}(\sin\varphi_j) \tag{3}$$

The horizontal earth pressure, σ_h, in each zone at the maximum pile shaft resistance is evaluated as shown in Eqs. (4) and (5).

$$\sigma_{h,\mathrm{A}} = \sigma_{h,\mathrm{C}} = \sigma_{h0} + \frac{\Delta\sigma_{h,\mathrm{A}} + \Delta\sigma_{h,\mathrm{C}}}{2} \tag{4}$$

$$\sigma_{h,\mathrm{B}} = \sigma_{h0} + \Delta\sigma_{h,\mathrm{B}} \tag{5}$$

where σ_{h0} is lateral restraining pressure, $\Delta\sigma_h$ is incremental lateral earth pressure due to dilatancy. In Zone A, the soil is clayey soil backfill, so changes in earth pressure are ignored ($\Delta\sigma_{h,A}$ =0). $\Delta\sigma_h$ is expressed based on cavity expansion theory [18], as shown in Eq. (6). The radial expansion displacement, $\Delta\alpha$, is expressed using the pile's settlement and the soil's dilatancy angle shown in Eq. (7).

$$\Delta\sigma_h = \frac{4G}{d+2\alpha} \times \Delta\alpha \tag{6}$$

$$\Delta\alpha = \delta_z \times \tan\psi \tag{7}$$

where G is the shear stiffness of the soil (=0.12G_0; initial shear modulus, G_0, is 1.05×10^5 kN/m^2 for confining pressures h, and 0.74×10^4 kN/m^2 for confining pressures l) and α is the thickness of the shear band in the soil near the pile (α_{bs}=0.15mm for clayey soil backfill, α_s=2.00mm for sand).

The strength effect of clayey backfill columns on the maximum shaft resistance of the pile is shown in Fig. 16. The effect is defined as the ratio of the shaft resistance (f_{s,max_qu}) with backfill clay to the shaft resistance (f_{s,max_MM}) without clayey backfill columns. The value is calculated for each confining pressure applied on the soil. As the surrounding soil is pressure-dependent sand, the shear strength $q_u/2$ of the clayey backfill column must be higher than f_{s,max_MM} in order for the maximum shaft resistance of the pile to be equal to that without backfilling. The required unconfined compressive strength of the clayey soil backfills increases with increasing depth.

The results suggest that it is important to consider the selection of the backfill material and the effect of variations in its properties when designing the vertical bearing capacity of piles that overlap backfill soil after the removal of existing piles.

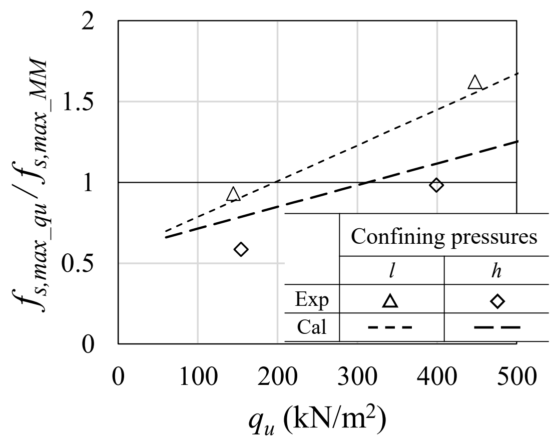

Fig. 16 Effect of clayey backfill columns on maximum shaft resistance of piles (Exp: experimental, Cal: calculated using Eq. (1))

6. CONCLUSIONS

This study conducted vertical loading model tests for the case where a new pile overlaps backfill soil whose mechanical properties are different from those of the surrounding soil. The main results are summarized below.

(1) The shaft resistance of the pile increased with increasing unconfined compressive strength of the clayey backfill columns. The maximum shaft resistance varied from -7% to +63% for low restraining pressures and from -42% to -2% for high restraining pressures compared to the case without backfill soil.

(2) When the soil was clayey soil backfill, the incremental earth pressure in the surrounding soil due to pile pushing was suppressed.

(3) The maximum shaft resistance of the pile depended on the strength of the clayey soil backfill. For high compressive strength, shear failure was assumed to have developed in the clayey soil backfill near the pile after the pile and clayey soil backfill had settled as a single unit.

(4) For the pile shaft resistance overlapping a partially clayey backfill column in sandy soil to be equal to the pile shaft resistance in a homogeneous sandy soil, the strength $q_u/2$ of the clayey backfill column must be higher than f_{s,max_MM}.

7. ACKNOWLEDGMENTS

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