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Comparison of Bearing Capacity Characteristics of Sand and Gravel Compaction Pile Treated Ground

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Abstract

Sand compaction piles which are constructed as compacted sand piles in the weak soils are used frequently in Korea as one of the soil improvement techniques. But it is necessary to find a new technique which can replace sand compaction piles because of decrease of sand resources and increase of sand cost. In this study, load tests of model composite ground which is composed of gravel compaction piles and clays are executed in order to find out bearing capacity characteristics and the same load tests of sand compaction piles are executed to compare the results of both load tests. Area replacement ratios of gravel compaction piles and those of sand compaction piles are 30%, 40%, and 50%, respectively and a model composite ground is preconsolidated by centrifugal consolidation apparatus before the load test. And the bearing capacities from test results are compared with those predicted by existing formulas.

Keywords: *bearing capacity, centrifugal consolidation apparatus, composite ground, gravel compaction pile, sand compaction pile*

1. Introduction

Recently, demands for industrial area and residential area are increasing more and more whereas, construction sites which have a good soil condition are extremely limited. Therefore need for improvement of soft soil is required frequently. Soft soils are characterized as low shear strength and large settlement thus soft soil improvement is necessary for safe uses of structures. Sand compaction piles (SCP) and gravel compaction piles (GCP) can be installed in the loose sand or soft clay by compacting sand and gravel and thus the strengths of soft soils are increased. For sandy soils, the advantages of using the SCPs or GCPs are increasing of density of soils, prevention of liquefaction and increase of lateral resistance of ground and for clay soils, those of using the SCPs or GCPs are increase of shear strength and bearing capacity of soil, reducing lateral displacement of soft soil and decrease of consolidation settlement (Barksdale, 1981; Enoki *et al*., 1991). SCP method was first developed in 1955 and has experienced consistent progress and can be used for improvement of soft soils existing onshore and offshore. Although some studies on SCP have been conducted in South Korea (Kim *et al*., 2002; Yoon *et al*., 2004), those are not sufficient for specifying design criteria of SCP and thus, more studies on SCP are required.

GCP method was first used for improvement of high organic soils in France, 1830 and has been widely used for soft soil improvement since 1950. In America the use of GCP has been increasing since 1976 (Barkdale *et al*., 1983). In South Korea, SCP method has been frequently used for onshore and offshore construction whereas, GCP method has ever been used. In the case of SCP, the price of sand has been increasing, and the need for new material that can substitute of sand has been in demand. Therefore study for the GCP is necessary as an alternative soil improvement method. In this study, model load tests were executed on composite grounds which include SCP and GCP and characteristics of bearing capacities were investigated and compared. Also, ultimate bearing capacities computed by existing formula were compared with those obtained from the model tests.

2. Failure Mode and Bearing Capacity Formula of Composite Ground

Failure mode of composite ground which include SCP or GCP varies with the pile length. Failure modes of homogeneous soft soil reinforced with SCP or GCP are classified into bulging failure, shear failure and punching failure. Bulging failure occurs when the pile length is greater than 2~3 times of pile diameter as shown in Fig. 1(a) and shear failure occurs near ground surface when short piles are supported by firm bearing strata as shown in Fig. 1(b). Punching failure occurs when short piles are installed in the homogeneous soft ground as shown in Fig. 1(c). Ultimate bearing capacity formula available on composite ground where bulging failure and shear failure is expected are shown in Table 1. As shown in Table 1, ultimate bearing capacity depends on shear strength of cohesive soil and internal friction angle of sand or gravel.

3. Model Test

3.1. Physical Properties of Test Soils

The test soils were clay and sand obtained from the coast of

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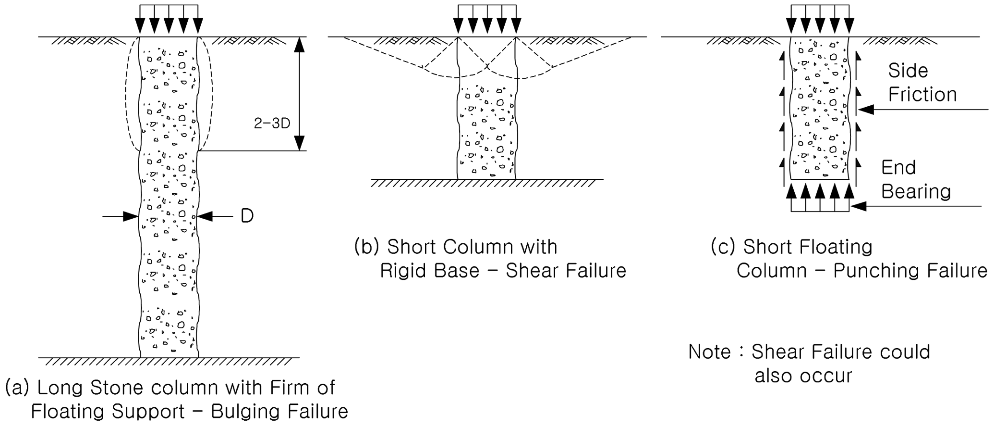


Fig. 1. Failure Modes of SCP and GCP (Barkdale *et al.*, 1983)

Table 1. Suggested Formula to Compute Bearing Capacity of Composite Ground

	Suggested formula	Failre mode
Greenwood (1970)	$q_{ult} = (\gamma_c \cdot z \cdot K_{pc} + 2c_u\sqrt{K_{pc}})\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right)$	Bulging failure
Vesic (1972)	$q_{ult} = (c_u F'_c + qF'_q)\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right)$	
Hughes & Withers (1974)	$q_{ult} = (4c_u + \sigma'_{ro})\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right)$	
Hansbo (1994)	$q_{ult} = (\sigma'_{ro} + 5c_u)\left(\frac{1+\sin\phi_s}{1-\sin\phi_s}\right)$	
Wong (1975)	$q_{ult} = (K_p q_o + 2c_u\sqrt{K_{pc}})\left(\frac{1}{K_A}\right)$: applied when settlement is small $\left(q_0 = 1.5c_u,\ K_A = \frac{(1-\sin\phi_s)}{(1+\sin\phi_s)}\right)$ $q_{ult} = 2\left\{(K_{pc}q_o + 2c_u\sqrt{K_{pc}}) + 3aK_A\gamma_c\left(1-\frac{3a}{2H}\right)\right\}\left(\frac{1}{K_A}\right)$: applied when settlement is large $\left(q_0 = 3c_u,\ K_A = \frac{(1-\sin^2\phi_s)}{(1+\sin^2\phi_s)}\right)$	Shear failure
Madhav & Vitkar (1978)	$q_{ult} = c_u N_c + \frac{1}{2}\gamma_c B N_\gamma + D_f \gamma_c N_q$	

where, γ_c: unit weight of clay, z: depth of composite ground, c_u: undrained shear strength of cohesive soil, ϕ_s: internal friction angle of sand(gravel), q: average stress at equivalent failure depth, K_{pc}: passive earth pressure coefficient of cohesive soil, F'_c, F'_q: coefficient of cavity expansion with pile diameter, σ'_{ro}: initial lateral stress, a: pile diameter, K_A: active earth pressure coefficient of pile, H: pile length, q_0: vertical stress at soil surface, B: loading width, D_f: embedded depth of foundation, N_c, N_γ, N_q: bearing capacity factor

Pusan city area, and gravel obtained from a nearby mountain. Clay is classified as CH from USCS (unified soil classified system). The sand and gravel materials were poorly graded having a maximum size of 2 mm and 25 mm, respectively. The physical properties of the test soils are summarized in Table 2, and the particle size distribution is shown in Fig. 2.

3.2. Centrifuge Facility

Specially designed and fabricated centrifuge facility used in order to consolidate clay prior to preparing composite ground. Fig. 3 shows the facility, which has 8 axi-symmetrical arms of 50 centimeters long, 7.5 HP of capacity, and the maximum value of 1400 rpm. Test soil bin is made of acrylic in order to minimize wall friction and its size is 25 cm×10 cm×25 cm (transverse×width×height) as shown in Fig. 4. Exterior wall of soil bin was reinforced with steel frame to bear centrifugal force.

3.3. Determination of Consolidating Pressure

Consolidating pressure induced from centrifuge test was determined by Eq. (1) and (2).

Table 2. Physical Properties of Test Soils

Soil Type	Specific Gravity, G_s	Atterberg Limit		Coefficient of Consolidation, c_v (cm^2/sec)	Max. Dry Unit Weight, $\gamma_{d\max}$ (t/m^3)	Min. Dry Unit Weight, $\gamma_{d\min}$ (t/m^3)	Shear Strength Parameter ϕ(°)	USCS
		LL(%)	PI(%)					
Clay	2.72	63.8	35	1.3×10^{-4}	-	-	-	CH
Sand	2.68	-	-	-	1.817	1.257	38.3	SP
Gravel	2.65	-	-	-	1.704	1.325	49.1	GP

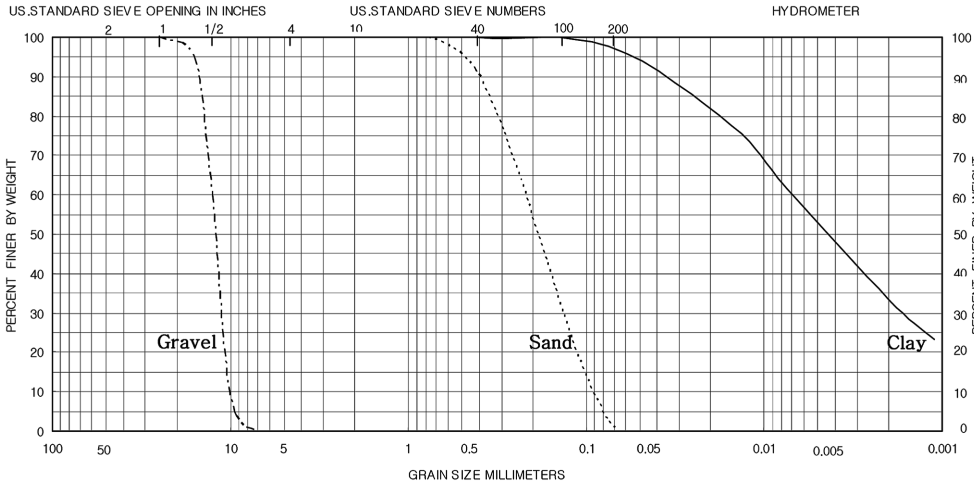


Fig. 2. Grain Size Distribution Curve

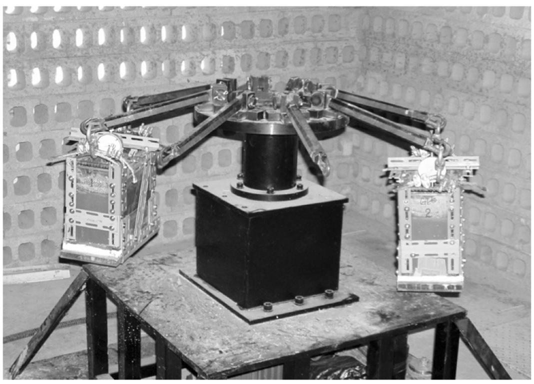


Fig. 3. Centrifuge Facility for Consolidation

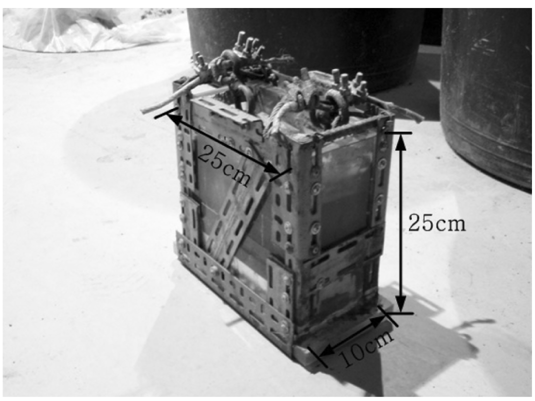


Fig. 4. Soil Bin Configuration

$$1\text{rpm} = \frac{\text{rev.}}{\text{min}} \times \frac{2\pi rad}{1\text{rev.}} \times \frac{1\text{min}}{60\text{sec}} = \frac{\pi}{30}\text{rad/sec} \quad (1)$$

$$F = ma = \frac{W}{g}\omega^2 r \quad (2)$$

Moment arm(r) was considered as the distance from the center of rotation to the mid point of soil bin and the weight of specimen(W) was considered as the half of the total weight of specimen. Consolidating pressures were calculated as the values of centrifugal force(F) divided by the area of soil bin. The reason why mid point of soil bin was considered is linear distribution of centrifugal force along the soil specimen.

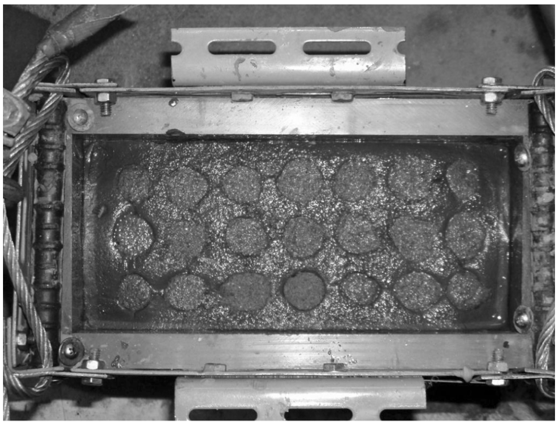
3.4. Testing Procedures

Composite ground which include SCP or GCP was modeled as plain strain problem and bearing capacity measurement test was conducted for two steps, which mean consolidation step and load test step. Test procedures are as follows.

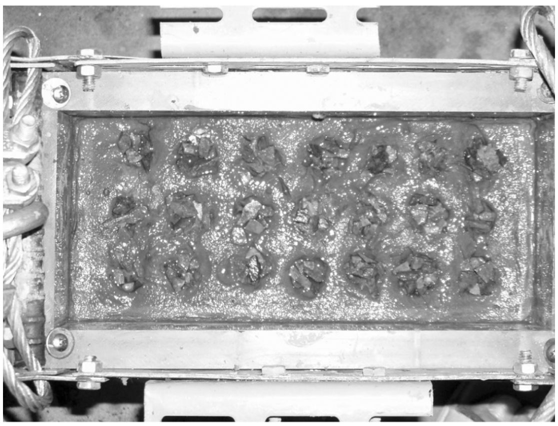
(1) Carefully place a sand mat at the bottom of the soil bin and pour clay slurry of water content over the liquid limit into the soil bin. Water content of the slurry is 90±1.5%.
(2) Place a weight of 3.6 kg on the top of the soil sample and preconsolidate the prepared slurry in soil bin at 150 rpm for 24 hours, resulting in preconsolidating stress of 50 kPa and water content of 60±2.0%.
(3) Insert casings into the preconsolidated samples and build up compaction piles, considering the area replacement ratios, which are 30%, 40%, and 50% as shown in Table 3. Compaction piles installed in the composite ground show rectangular array (Fig. 5). Frozen piles were used and its diameter and length were 2.2 cm and 15 cm respectively.
(4) Centrifuge again the composite ground at 250 rpm. Angular velocity of 250 rpm is equivalent to consolidating pressure of 150 kPa.
(5) Conduct vane shear tests before and after pile installation in order to find out undrained shear strength (c_u) of soil sample.

Table 3. Test Conditions

Area replacement ratio, a_s (%)	Center to center pile spacing (cm)	Arrangement	Applied Improvement Method	Consolidating pressure (kPa)
0	-	Rectangular	SCP GCP	150 kPa
30	3.56			
40	3.08			
50	2.76			



(a) SCP 40 %



(b) GCP 40 %

Fig. 5. Composite Ground

Fig. 6. Loading Plate and Loading Cap

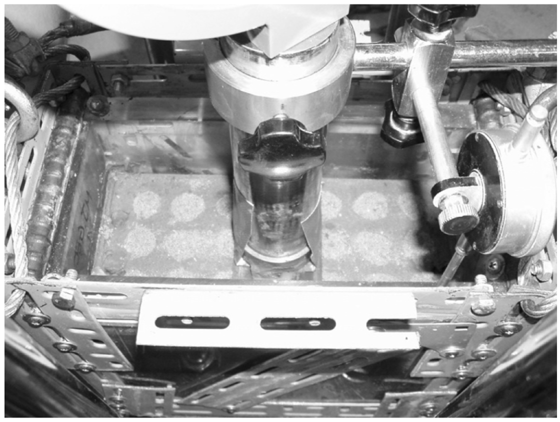


Fig. 7. Load Test on Composite Ground

(6) Place loading plate on the top of soil sample after executing vane shear test and conduct load test as shown in Fig. 7. The size of loading plate shown in Fig. 6 is 10 cm×6 cm×0.5 cm (length×width×thickness). Load increment considered in the load test was 1 kgf and settlement corresponding to the respective load increment was measured. Load test was conducted by load control method.

4. Analysis of Test Results

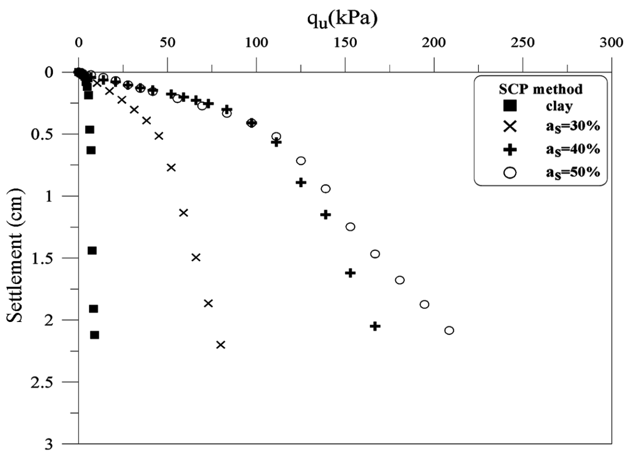
4.1. Test Results

Load-settlement curves obtained from the load tests are shown in Fig. 8. As shown in Fig. 8, load-settlement curve can be approximated as two linear sections and ultimate bearing capacity was determined as the load value corresponding to the intersection of two linear sections. Ultimate bearing capacities obtained from load tests are shown in Table 4 and Fig. 9. As shown in Fig 9, ultimate bearing capacity of SCP and GCP composite ground increases as the area replacement ratio increases but the increasing ratio of ultimate bearing capacity corresponding to GCP composite ground with area replacement ratio is greater than that of SCP composite ground.

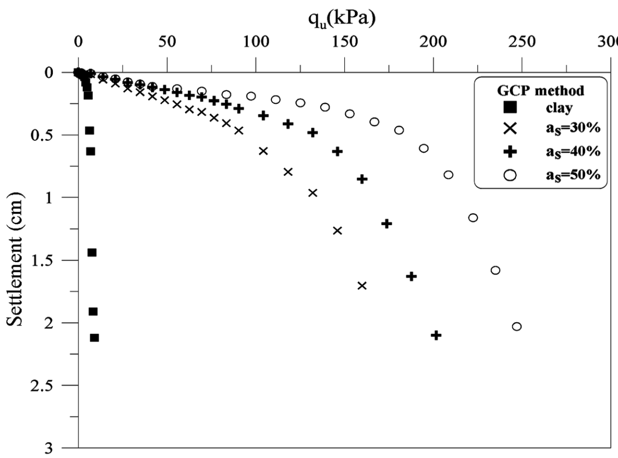
Ultimate bearing capacities of SCP and GCP composite ground were greater than those of non-treated ground by 30~40 times at area replacement ratio of 50%. Ultimate bearing capacities of GCP composite ground were greater than those of SCP composite ground by 1.25~2.6 times. Vane shear tests were conducted in order to measure undrained shear strengths of clay before and after installing piles and those values are shown in Table 5. As shown in Table 5, due to the role of vertical drainage of piles installed in the composite ground, undrained shear strengths of SCP and GCP composite ground were greater than those of non-treated ground. Regression analysis was executed based on the data shown in Table 5 and analyses results are shown in Fig. 10. As shown in Fig. 10, undrained shear strength increases linearly as the area replacement ratio increases.

4.2. Comparison of Ultimate Bearing Capacity from Theoretical Formula and Model Test

Ultimate bearing capacity of SCP and GCP composite ground can be determined by the equations shown in Table 1. Although the pile (pile length/diameter ≈ 6.8) used in this study is classified



(a) SCP composite ground



(b) GCP composite ground

Fig. 8. Load-settlement Curves

Table 4. Ultimate bearing Capacities Obtained from Load Tests

Area replacement ratio (a_s, %)	Ultimate bearing capacity (kPa)	
	SCP composite ground	GCP composite ground
0	5	5
30	60	130
40	120	155
50	135	210

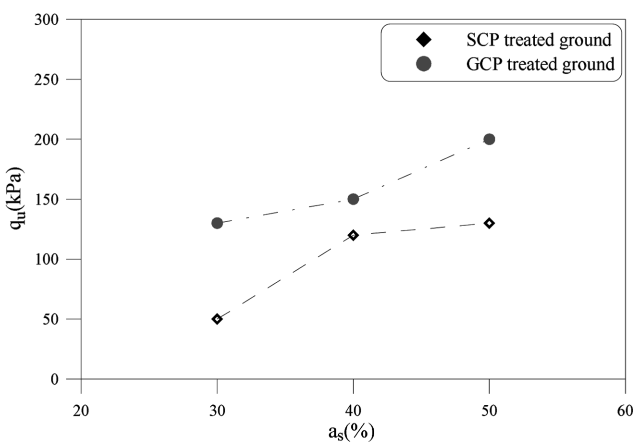


Fig. 9. Measured Ultimate Bearing Capacities with Area Replacement Ratio

Table 5. Results of Vane Shear Tests

Area replacement ratio (a_s, %)	Undrained shear strength (kPa)			
	SCP composite ground		GCP composite ground	
	c_u	Δc	c_u	Δc
0	12.0	8.3	12.0	8.3
30	18.1	14.5	17.2	13.6
40	20.7	17.1	18.6	14.8
50	21.7	18.0	20.5	16.6

note: 1) Initial water content of slurry (w_0) is 90.5%
2) Average undrained shear strength (c_0) before pile installation is 3.7 kPa
3) Δc : increased undrained shear strength

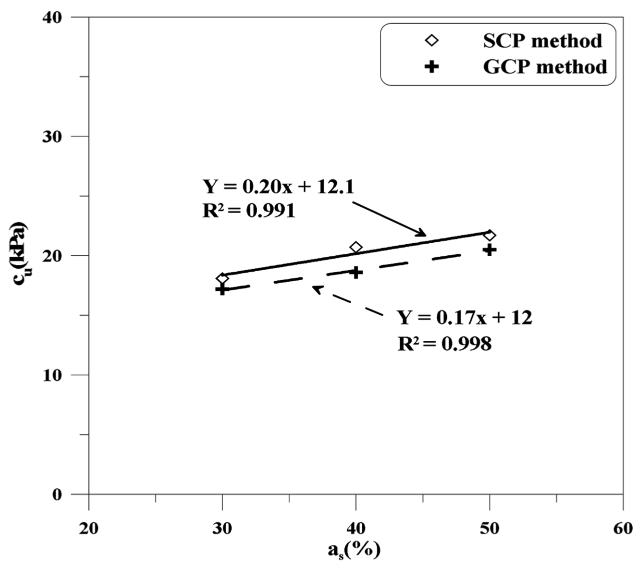


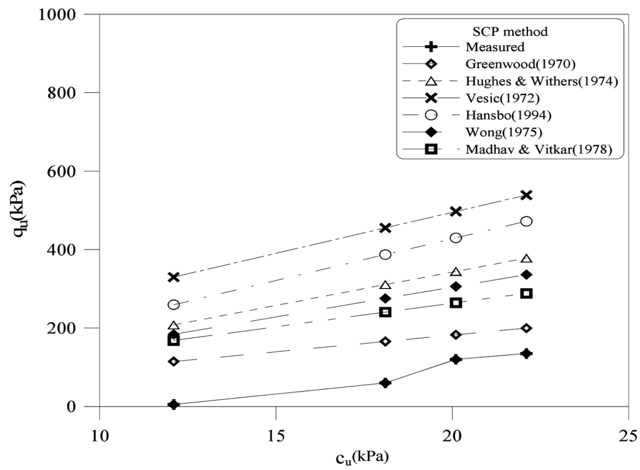
Fig. 10. Relationship between c_u and area Replacement Ratio

into long pile and bulging failure is expected, ultimate bearing capacity from theoretical formula assuming shear failure was also calculated considering configuration of group pile and boundary condition of soil bin. In order to compute ultimate bearing capacity, unit weight of clay of 18 kN/m^3, depth of bulging failure of 2.2 cm (1D) and depth of composite ground (z) of 15 cm were applied in the formula. Also Poisson's ratio (ν) of 0.5 assuming undrained condition and earth pressure coefficient of clay of 1 were considered in accordance with ϕ_u=0 concept. Undrained shear strength of clay was determined by the equation resulted from regression analysis shown in Fig. 10. Internal friction angles of sand and gravel are 38.3^o and 49.1^o respectively. Computed results of ultimate bearing capacities are shown in Table 6 and Fig. 11.

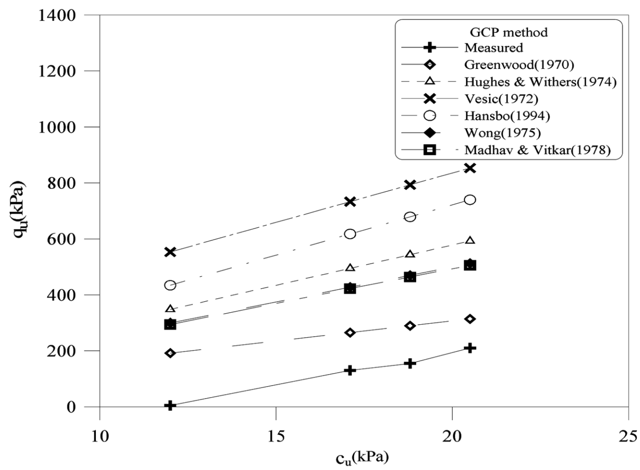
As shown in Fig. 11, ultimate bearing capacity computed from theoretical formula increases linearly with undrained shear strength (c_u). This trend of linear increase results from the fact that area replacement ratio and diameter of pile are not considered in theoretical formula. As shown in Table 5 and Fig. 11, ultimate bearing capacities are much greater than those measured from load tests. Ultimate bearing capacities computed from the theoretical formula suggested by Vesic (1972) show greatest deviation from those obtained from model tests and those computed from the theoretical formula suggested by Greenwood (1970) show smallest deviation from those obtained

Table 6. Ultimate Bearing Capacities Computed from Theoretical Formula

Ultimate bearing capacity (kPa) / Author	SCP composite ground				GCP composite ground			
	c_u=12.1 kPa (a_s=0%)	c_u=18.1 kPa (a_s=30%)	c_u=20.1 kPa (a_s=40%)	c_u=22.1 kPa (a_s=50%)	c_u=12.0 kPa (a_s=0%)	c_u=17.1 kPa (a_s=30%)	c_u=18.8 kPa (a_s=40%)	c_u=20.5 kPa (a_s=50%)
Greenwood (1970)	114.6	165.7	182.8	199.8	191.9	265.3	289.8	314.2
Hughes & Withers (1974)	207.9	310.8	344.2	378.2	347.9	494.6	543.5	592.4
Vesic (1972)	329.8	455.3	497.1	538.9	553.1	733.1	793.1	853.1
Hansbo (1994)	259.4	387.2	429.8	472.0	434.3	617.6	678.7	739.8
Wong (1975)	184.1	275.4	305.9	336.3	300.0	427.5	470.0	512.5
Madhav & Vitkar (1994)	168.0	240.6	264.4	288.2	294.0	422.3	464.0	505.6



(a) SCP composite ground



(b) GCP composite ground

Fig. 11. Ultimate Bearing Capacity with Undrained Shear Strength

from model tests. It is thought that difference between the values from theoretical formula and those from model tests is due to size effect and disregarding area replacement ratio in theoretical formula. Comparing ultimate bearing capacities calculated from theoretical formula with those measured from model tests, new formula that properly estimate ultimate bearing capacity of composite ground is required.

5. Conclusion

In order to find out ultimate bearing capacity of SCP and GCP composite ground, centrifuge tests were conducted on model ground whose area replacement ratios are 30%, 40 %, 50%. Ultimate bearing capacities obtained from model tests are compared with those calculated from theoretical formulas. Conclusions from this study are as follows.

(1) Ultimate bearing capacity of SCP and GCP composite ground increases as the area replacement ratio increases but the increasing ratio of ultimate bearing capacity corresponding to GCP composite ground with area replacement ratio is greater than that of SCP composite ground.

(2) Ultimate bearing capacities of SCP and GCP composite ground were greater than those of non-treated ground by 30~40 times at area replacement ratio of 50%. Especially, ultimate bearing capacities of GCP composite ground were greater than those of SCP composite ground by 1.25~2.6 times.

(3) Comparing ultimate bearing capacities measured on SCP and GCP composite ground, GCP composite ground shows greater bearing capacity than SCP composite ground.

(4) Ultimate bearing capacities calculated from theoretical formula are much greater than those measured from load tests. It is thought that difference between the values from theoretical formula and those from model tests is due to size effect and disregarding area replacement ratio in theoretical formula. Comparing ultimate bearing capacities calculated from theoretical formula with those measured from model tests, new formula that properly estimate ultimate bearing capacity of composite ground is required.

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