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Full-Scale Experimental Study on
the Single and Group of Soil-
Cement Columns under Vertical
Load Applying for Buildings

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ABSTRACT

This article presents the results of in-situ research on the soil-cement (SC) column as the foundation of buildings. A physical model was constructed consisting of two single SC columns, one group of three SC columns (G1), and another group of five SC columns (G2) with a diameter of 0.6 m and a length of 7.5 m. Among ten experimental columns, four SC columns were instrumented by strain gages to determine the load transfer and analysis of the skin and toe resistance distributions along the depth of the SC columns. These columns were constructed by the wet mixing method according to Japanese technology with two static blades to increase the quality of the mixture. The bearing capacity of the single SC column was measured as 1,180 kN, and the top and toe displacements were measured to be approximately 36.6 mm and 27.7 mm, respectively. For the group of SC columns, the skin resistance of the center and outer columns decreased by 4.17 % and 16.16 %, respectively, in comparison to the single column. The toe resistance of the SC column in the groups was significantly lower than that of the single column, from 45.10 % for the group G1, and up to 60.78 % for the group G2. The effect of the group of SC columns was also determined from the experiment with the group coefficients around 0.664 for group G1 and 0.554 for group G2. The research results from the full-scale model are essential in evaluating the group effects of the SC columns, especially in applications for the foundation of buildings.

Keywords

soil-cement column, foundation, load transfer, wet mixing method, top displacement, toe displacement, group coefficient

Nomenclature

- A = cross-section area
- D = distance between two columns
- D = diameter
- E = elastic modulus
- P_i = load at the i th elevation
- P_{max} = maximum load test
- Q_u = ultimate bearing capacity
- ΔP_i = load increment
- η = group coefficients

Introduction

The soil-cement (SC) column was studied in the 1970s in Sweden and Japan. The SC column has been applied popularly throughout the world because of its many advantages, such as using local soil material, improving soft-soil, reducing total settlement and time-dependent settlement, increasing the bearing capacity of the ground, and reducing construction time and environmental impact. The SC column has been mainly applied for three areas, including soft-soil treatment, deep excavation, and foundation of constructions.

Research on the responses of a single SC column and a group of SC columns in ground improvement has been demonstrated in some publications. Firstly, Broms (1999) showed his research with full theoretical contents for calculating bearing capacity, shear resistance, stability problems of embankment blocks on soil-lime, and SC columns. In 2006, Filz and Navin published a study on the stabilization of embankment blocks on the SC columns with the practical and numerical models (Filz and Navin 2006). Bouassida, Jellali, and Porbaha (2009) presented the theory of foundations based on SC ground under the effect of the vertical load.

Furthermore, the centrifugal model, the physical model, and full-scale experiments have been applied to observe the change in strength, load distribution, and the load-strain relationship along the SC columns (Bell, Baka, and Galagoda 2007; Chai, Liu, and Du 2002; Dehghanbanadaki, Ahmad, and Ali 2016; Frikha et al. 2017; Shirvani and Shooshpasha 2015; Sukpunya and Jotisankasa 2016; Taheri and Tatsuoka 2015). The results of the research on the destructive mechanism, bearing capacity, and stability of the SC ground were also summarized by Bruce (2000) and Kitazume and Terashi (2013). These studies mainly focused on using SC mix to improve the bearing capacity of the ground. Otherwise, there have been a few studies on the SC column applied in the sandy ground as the foundation of structures.

In the suitable ground, the SC column could be applied as the foundation of buildings. For example, the SC columns constructed in sandy and clay-sandy soils on the Central Coast of Vietnam, with a cement content from 15 to 20 %, the unconfined compressive strength (q_u) of SC material was measured from 4 to 10 MPa (Do 2012; Do and Nguyen 2013). The bearing capacity, load transfer, and mobilization of skin resistance of a single SC column and a group of SC columns will be affected by the stiffness of SC material, the type of surrounding soil, and the effects of the group of columns.

This article presents the research results on the full-scale model of the SC columns with a diameter of 0.6 m and a length of 7.5 m by the wet mixing method. Some columns in the experiment were instrumented by strain gages to observe the load and strain transfer mechanisms along the single SC column and groups of SC columns. Core drilling experiments, unconfined compression test, and axial static load tests for the single SC column and columns in the groups were conducted. These test results were used to analyze the load transfer mechanism, toe and skin resistance, and the group effect of the SC columns so that it could be applied as the foundation of buildings.

Soil Profile

The site is located at the construction of Tuyen Son entertainment area, Hoa Cuong Bac Ward, Hai Chau district, Da Nang city, Vietnam. Field investigation results of soil are shown in [Table 1](#). In addition, cone penetration tests (CPTu) were also conducted at the site area with the results presented in [figure 1](#).

It can be seen that the toe resistance (q_t) value and the skin friction (f_s) distributions obtained from the CPTu tests are quite consistent with the field investigation shown in [Table 1](#). From 0 to 2 m, the first soil layer is the clayey sand with the standard penetration test (SPT) of about 9 and q_t value around 10 MPa. The groundwater is found at a depth of 1 m from surface ground level. From 2 to 4 m, the sandy clay layer shows low resistance values

TABLE 1

Soil parameters

Items	Unit	Depth Below Original Ground Surface						
		1.0 m	3.0 m	5.5 m	7.3 m	9.2 m	11.3 m	13.5 m
Water content, W_n	%	23.6	35.3	25.4	23.8	23.7	26.5	31.2
Saturated density, ρ_{sat}	kN/m ³	18.9	18.1	19.1	19.7	18.7	18.8	18.1
Void ratio, e_0	...	0.76	1.03	0.77	0.73	0.70	0.83	0.91
Liquid limit, LL	%	...	32.5	...	...	...	39.5	41.3
Plastic limit, PL	%	...	20.3	...	...	...	23.1	23.3
Clay fraction	%	...	20	...	...	...	28	35
Silt fraction	%	6	49	25	5	33	25	21
Sand fraction	%	94	31	75	95	67	47	44
Friction angle, φ	(°)	27.3	4.3	26.2	30.2	31.2	15.1	13.9
Cohesion, c	kPa	1.2	9.2	1.30	1.05	0.8	24.6	22.7
SPT blow count	bl/0.3 m	9	2	5	14	30	11	8

Note: SPT = standard penetration test.

FIG. 1 CPTu results at the site.

(SPT = 2, $q_t = 0.3$ MPa, and $f_s = 6.0$ kPa). From 4 to 11 m, the soil layers contain more than 95 % sand with high friction angle ($\varphi > 30^\circ$), the q_t value is approximately 20 MPa, and the f_s reaches 80 kPa. Below a depth of 11 m, the toe resistance and the f_s values drop to 1.5 MPa and 25 kPa, respectively. It can be seen that the ground contains a high proportion of sand with low bearing capacity; hence, the deep mixing method was applied to build 1,300 SC columns as the foundation of the construction.

SC Column Installation

FULL-SCALE MODEL DESIGN

The experimental model includes two single columns with a diameter of 0.6 m and a length of 7.5 m (C4 and C5), a group G1 with three columns (C1 to C3) covered by a 0.8-m-thick triangle cap block, and a group G2 with five columns (C6 to C10) covered by a square cap block. The locations of the experimental columns, geological drilling holes, and CPTu tests are shown in [figure 2](#). The distance between the two columns in the group was chosen as 0.9 m. The distance between the single columns (C4 and C5) and the groups of columns (G1 and G2) are 6 m and 8.5 m, respectively.

INSTRUMENTATION OF SC COLUMNS

In the experiment, the strain gages (Geokon 4200) were used to observe the strain and load distribution along the depth of the single column (C5) and the columns in the groups (C3, C7, and C8). Location details and elevations of the strain gages (GL1 to GL4) are shown in [figure 3](#). These strain gages were connected to wires with two distortion cores to measure the displacement and two temperature cores to measure temperature change as shown in [figure 4](#). The operation of strain gages was checked before drilling and after inserting to the columns. The strain gages were glued in a D22-mm steel tube in the right positions. They were covered by epoxy for waterproofing. The steel tube D22 had a diameter of 22 mm and a length of 7.4 m and covered their toes with plastic buttons. The tubes were inserted in the center of the fresh SC columns when the mixing process was finished. After 24 h, a steel bar (tale-tell bar) was used to push the plastic button out of the tube. Then, the steel bar could

FIG. 2 The layout of the single SC columns and groups of SC columns.

FIG. 3

Details of the instrumented SC columns.

FIG. 4

Strain gages Geokon 4200.

contact the toe of the columns, so the displacement of the column toe could be measured through the movement of the tale-tell bar during the static load test.

CONSTRUCTION OF SC COLUMNS

A drilling rig was designed with three mixed blades and two static blades. The static blades did not rotate during the drilling process so that the quality of SC material was more uniform. The rig was made in a workshop with a diameter of 0.6 m; the static blade has a length of 0.7 m and an entire height of 1.12 m as shown in [figure 5](#).

Dry cement in a tank was pumped into the mixing tank with the water-cement ratio of 0.6. Cement content of 350 kg/m^3 was applied for the first layer (0–2 m) and the sand layers (4–7.5 m). For the sandy clay layer from 2 to 4 m, 400-kg/m^3 cement was used because this layer contains a high amount of clay and shows low bearing capacity. Therefore, it is necessary to increase the cement content for this layer to make sure that the strength along the column is not different.

Cement and water were weighed through the automatic weighing system. The SC columns were drilled by two cycles. In the first cycle, a mortar was injected and mixed with a pressure of 20 MPa and a drilling speed of 0.5 m/min. The second cycle with a drilling speed of 0.7 m/min was applied to increase the uniformity of SC

FIG. 5

The drilling mixer.

mortar in the SC columns. After drilling, D60-mm pipes and D20-mm steel tubes with glued strain gages were pressed into the SC columns as shown in [figure 6](#). The SC columns then were cleaned of the extra mixture on the top surface. The operation of strain gages was also double-checked. After 21 days, the 0.8-m-thick caps were built by 35-MPa concrete. The sizes of these caps are shown in [figure 2](#).

Unconfined Compression Strength Tests and Axial Static Load Tests

UNCONFINED COMPRESSION STRENGTH TESTS

After 21 curing days, a core drilling experiment was conducted to evaluate the quality and uniformity of SC material. The samples ([fig. 7](#)) were drilled at the core of the columns C3, C5, C7, and C8 with a diameter of 70 mm and a height of 140 mm (the height-to-diameter ratio was 2). These samples were cured and the unconfined compression test was conducted at 28 days using ASTM [D1632-96](#), *Standard Practice for Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory*. The load increased with the speed of 1–2 mm/min until the sample was destroyed. For each test, three SC samples were tested, and the average values of them were used for further analysis. The unconfined compressive strength (UCS) results along the depth of columns are shown in [figure 8](#). It is clear that the strength of SC material along the column and among the experimental columns was constant and fluctuated from 4.6 to 6.8 MPa.

FIG. 6

Installing D20-mm steel tubes with strain gages.

FIG. 7

Coring samples from the SC columns.

FIG. 8 The UCS results along the depth of SC columns: (A) C3, (B) C5, (C) C7, and (D) C8.

AXIAL STATIC LOAD TEST

The static load test by a standard ASTM D1143 was conducted after 28 days. The axial load increment (ΔP_i) for the single columns and the group of SC columns were applied in one process and were stopped when fast displacement occurred. The diameter of the load plate is 0.6 m.

For the single columns (C4 and C5), ΔP_i was 80 kN and the maximum load test (P_{max}) was 1,120 kN. For each interval, the ΔP_i was maintained for 10 min and the displacements and strain were recorded at 0, 5, and 10 min. The highest load was maintained for 24 h.

For the group G1, the ΔP_i was 200 kN and the P_{max} was 2,800 kN, whereas group G2 required higher values with a ΔP_i of 300 kN and a P_{max} of 4,500 kN. Figures 9 and 10 illustrate the instrumentation and test weights for

FIG. 9 Axial static load tests for the single SC column: (A) equipment for the single SC column; (B) static load test for the single SC column.

FIG. 10 Axial static load tests for the group of SC columns: (A) equipment for the group of SC columns; (B) static load test for the group of SC columns.

the single columns and the groups of SC columns. Reading data were used to interpret the distribution of axial load transfer at the column toe and along the column skin for the single columns and the groups of columns as well.

Experimental Results

LOAD TRANSFER PRINCIPLE

The load at the i th elevation along the column is calculated by equation (1).

$$P_i = \Delta\mu\epsilon_i \cdot E \cdot A \quad (1)$$

where E is the modulus of elastic material of pile; A is the horizontal cross section of the pile; and $\Delta\mu\epsilon_i$ is the deformation value of the experimental columns at the i th level, determined by equation (2).

$$\Delta\mu\epsilon_i = (R_0 - R_j) \cdot G \quad (2)$$

where R_0 is the initial deformation number; R_j is the deformation reading at the j th load level; the unit of the reading number is micro strain ($\mu\epsilon$); and G is the correction factor for the strain gage.

Mobilize friction (f_i) values in the column between the soil layers are determined by equation (3).

$$f_i = \frac{P_{i-1} - P_i}{U \cdot l_i} \quad (3)$$

where P_i and P_{i-1} are the initial deformation numbers; l_i is the deformation reading; and U is the circumference of the column.

LOAD BEARING CAPACITY

The limited bearing capacity of the columns could be determined based on the following methods.

De Beer (1972)

The bearing capacity is determined from the head displacement (Δ) of the column as equation (4).

$$\Delta = 2.5\% \times D \quad (4)$$

where D is the diameter of the column, $D = 600$ mm.

So, $\Delta = 2.5\% \times 600 = 15$ mm

SNiP (TCVN 10304:2014, *Pile Foundation-Design Standard*)

The bearing capacity is determined from the settlement (S) of the column as shown in equation (5).

$$S = \xi \times S_{gh} \quad (5)$$

where S_{gh} is the limited settlement of the foundation, and for normal building structure, $S_{gh} = 80$ mm; and ξ is the factor that considers the effect of time factor ($\xi = 0.2$).

So, $S \times \xi \times S_{gh} = 0.2 \times 80 = 16$ mm

Davisson (1973)

The bearing capacity is determined from the settlement (S) of the column as shown in equation (6).

$$S = \frac{QL_p}{EA} + 0.0038 + \frac{D}{120} \quad (6)$$

where Q is the vertical load; L_p is the length of the column; A is the cross-section area of the column; E is the elastic modulus of material; and D is the diameter of the column.

Chin Extrapolation (1971)

According to Chin (1971), the limited bearing capacity is determined by equation (7).

$$Q_u = \frac{1}{C} \quad (7)$$

where C is the angle ratio of the line between the settlement (S) and the ratio of the settlement and the load (S/p).

THE EXPERIMENT RESULTS OF THE SINGLE SC COLUMNS

Figure 11 shows the curves of the load against displacement on the top and the toe of the column C4. It could be seen that, at the load of 1,120 kN, the displacement at the column top is around 40 mm, and the toe displacement obtained from the tell-tale bar is 25 mm. After reducing the load, the residual displacements at the top and the toe of the column are 30 mm and 21 mm, respectively.

FIG. 11

Load versus
displacement curves of
the column C4.

For the column C5, **figure 12** shows that the displacements at the top and the toe of the columns are 36.6 mm and 27.7 mm. The residual displacements after decreasing the load to zero are 27.5 mm and 23 mm. In addition, it shows that the total bearing capacity of the single SC column reaches over 1,160 kN.

Data obtained from the strain gages along the columns are calculated based on equations (1) and (3), and the results are shown in **figures 13** and **14**. The load versus f_s at soil layers are different from each other. **Figure 13** shows that the ultimate f_s occurs at the sand layer (GL1), about 62 kPa when the load increases to 1,000 kN, then that value slightly decreases to the next increment loads. The f_s value on the sandy-clay layer (GL2) is low, only 26 kPa. When the load is increased, the f_s values at the lower layers (GL3 and GL4 elevations) are increased, up to 57 kPa as shown in **figures 13** and **14**. In summary, the total skin resistance is about 638 kN (55 %), and the toe resistance is 582 kN (45 %) of the ultimate bearing capacity (Q_u).

THE EXPERIMENT RESULTS OF THE GROUP SC COLUMNS

Group G1 (Three SC Columns)

Figure 15 shows the curves of the load against displacement on the top surface of the cap block of the three SC column group G1 and the toe of the column C3 in this group. The highest applied load for this group is 2,800 kN, and the displacement at the top surface of the cap block is around 56.7 mm, whereas the toe displacement of the column C3 in the group G1 is 40.5 mm. The residual displacements at the top surface and the toe of columns when the load decreases to zero are 47.4 mm and 35.8 mm, respectively.

FIG. 12

Load versus
displacement curves of
the column C5.

FIG. 13

Load versus f_s curves of the column C5.

FIG. 14

Load distribution along the depth of the column C5.

Figure 16 illustrates the skin and toe resistances curve of the SC column C3. It is clear that the skin resistance reached the ultimate value (653.35 kN) and then decreased in the next increment loads. On the other hand, the toe resistance of the column C3 is only 290 kN at the highest applied load, and it tends to increase to a higher value if the increment load is applied.

FIG. 15

Load versus displacement curves of the group of three SC columns (G1).

FIG. 16

Skin and toe resistances of the column C3 in the group G1.

Figure 17 shows the strain and skin resistance of the soils along the column C3 at strain gage elevations. The f_s results at the first and second layers (at GL1 and GL2) are similar to the single columns (C4 and C5). However, the highest resistance along the lower layers is 54 kPa, which is lower than the value in the single column C4 (62 kPa). That could be the effect of the group of SC columns. Figure 18 illustrates the load distribution along the

FIG. 17

Load versus f_s curves of the column C3 in the group G1.

FIG. 18

Load distribution along the depth of the column C3 in the group G1.

depth of the column under different increment loads during the static load test. It shows that the total load is 930 kN, in which the toe resistance is 290 kN and skin resistance is around 653 kN.

Group G2 (Five SC Columns)

Figure 19 shows the curves of the load against displacement on the top surface of the cap block of the five SC column group G2 and the toe of the column C7 in that group. The highest applied load for the group G2 is 4,500 kN, the displacement at the top surface of the cap block is around 56.6 mm, and the toe displacement of the column C7 in the center of the group G2 is 40.5 mm. This result is similar to that of the group G1.

Figure 20 illustrates the skin and toe resistances of the columns C7 and C8 in the group G2. It is clear that the skin resistance does not reach the ultimate value as the column C3 in the group G1, and it stops at 535.03 kN at the highest load. The toe resistances of the column C7 is only 305 kN, and it is on the trend of the increment process. For the column C8, the skin resistance is 611.52 kN, which is higher than that of the column C7.

Figures 21 and 22 show the load distributions along the center and outer columns (C7 and C8) of the group G2. **Figures 23 and 24** illustrate the load versus f_s under the experimental loads. The f_s decrement along the columns in the group G2 is much higher than the group G1. The skin resistance of the sand layer always reaches the ultimate value, whereas that resistance of the lower layers slightly increases initially and increases to a higher value at the next increment loads, as shown in **figures 13, 17, 23, and 24**.

FIG. 19

Load versus displacement curves of the group of five SC columns (G2).

FIG. 20

Skin and toe resistances of the columns C7 and C8 in the group G2.

Figures 25 and 26 summarize the change of the skin and the toe resistances of the experimental columns (C3, C5, C7, and C8). For the skin resistance, the single column (C5) shows the higher values in comparison to the columns in the groups (C3 of the group G1, C7, and C8 of the group G2). The reducing values are shown in Table 2. Figure 25 shows that the skin resistance (R_s) is decreased with the order from the single SC column C5, the column C3 of the group G1, the center column C7 of the group G2, and the outer column C8 of the group G2.

To illustrate the strength reduction, a ratio of the skin resistance reduction of the SC columns (C3, C7, C8) in the groups and the skin resistance of the single column (C5) was calculated. Table 3 shows that the reduction ratios of the skin resistance of the outer columns (C3 and C8) are not much (1.98–4.17 %); however, for the center column (C7), the skin resistance decreased significantly to 16.16 %.

The vertical displacement to maximize the skin resistance to the single SC column obtained from figure 25 is 20–22 mm, corresponding to 3.5 % of the column diameter. For the columns in the group, the displacements are 30–35 mm, corresponding to about 5 % of the diameter of the SC column. These displacement values are higher than 8 mm, according to Coyle and Reese (Randolph 1983). The SC column C7 at the center of the group G2 shows the smallest skin resistance that is due to the most affected of the group effect.

Figure 26 shows that the toe resistance of the columns in the groups is all lower than the single column. In detail, the toe resistance of the column C3 in the group G1 is 290 kN, which is approximately 50 % lower than the value of the single column (C5). In the group G2, the toe resistances of the columns C7 and C8 are 47 % and 60 % in comparison with the single column. As shown in figure 26 and Table 3, in the same group (G2), the toe

FIG. 21

Load distribution along
the depth of the column
C7 in the group G2.

resistance of the center column (C7) is higher than that of the outer column (C8) (304.97 kN and 228.48 kN, respectively).

ANALYSIS OF ULTIMATE BEARING CAPACITY AND GROUP COEFFICIENT

The bearing capacities of the single SC column and columns in the groups G1 and G2 were calculated by equations (4)–(6) according to the extrapolation methods of De Beer, Davisson, and Chin Konder. The results are shown in figures 27 and 28. The values of the Q_u of the SC columns are summarized in Table 4, in which Q_u values are the column C3 in the group G1, the column C7 in the group G2, and the average values of the single SC columns (C4 and C5). Table 4 shows that the bearing capacity of the SC column (C3 and C7) in the groups is lower than that of the single columns.

To evaluate the effect of the group, a group coefficient factor (η) is applied. The coefficient of the group of the SC columns is determined by dividing the load-bearing capacity of the column into the groups (G1 and G2) to the experimental single SC column value (C4 and C5), which is taken as the average value of the columns C4 and C5. The group coefficient of the groups G1 and G2 are shown in Table 5. The results from the ultimate loading capacity of De Beer and Davisson methods are approximately similar. The results determined by the method of Chin Konder extrapolation are much higher. In practice, the real working load of the SC column is usually in the range of 40–60 % of the ultimate load, and the displacement of the SC column is small. Hence, the Q_u value determined by Davisson is chosen to calculate the group coefficient. The group coefficient for concrete pile could be calculated by the following methods.

Feld's Method

The bearing capacity of the group of piles takes into account the coefficient of the group as shown in equation (8) (Bowles 1996).

FIG. 22

Load distribution along the depth of the column C8 in the group G2.

FIG. 23

Load versus f_s curves of the column C7 in the group G2.

$$Q_{ag} = \eta \times n \times Q_u \quad (8)$$

where Q_{ag} is the allowable bearing capacity of the pile group; η is the coefficient of pile group; n is the number of piles in the group; and Q_u is the allowable bearing capacity of the single pile.

This method is simple; the Q_{ag} is determined according to the fixed map from 2 to 9 piles. This method is only applied for small section piles and according to the sample diagram.

FIG. 24

Load versus f_s curves of the column C8 in the group G2.

FIG. 25

Skin resistances of the experimental columns.

FIG. 26

Toe resistances of the experimental columns.

TABLE 2

Skin resistance from the static load tests

Depth, m	C5 (kPa)	C3, kPa	C7, kPa	C8, kPa
0–2.0	62.12	62.02	63.62	62.99
2.0–4.0	26.79	25.79	24.44	26.39
4.0–5.5	53.25	51.0	41.25	49.50
5.5–7.5	57.75	54.0	50.25	51.00

TABLE 3

Comparison of the skin and toe resistances

SCC Resistance	C5	C3	C7	C8
R_s , kN	638.17 (0 %)	625.5 (–1.98 %)	535.03 (–16.16 %)	611.52 (–4.17 %)
R_p , kN	582.6 (0 %)	290.4 (–45.1 %)	304.97 (–47.66 %)	228.48 (–60.78 %)

FIG. 27

Ultimate bearing capacity (Q_u) by De Beer and Davisson.

FIG. 28

Ultimate bearing capacity (Q_u) by Chin.

TABLE 4Ultimate bearing capacity (Q_u) of the SC columns

SC Column/Method	De Beer	Davission	Chin Konder
C4	950	1,060	1,428.5
C5	875	1,110.6	1,428.5
Average of C4 and C5	912.5	1,085.3	1,428.5
C2 in G1	625	720.5	1,250
C7 in G2	555.4	601.5	1,111.1

Note: Q_u of the group of SC columns represents the average values.

TABLE 5Calculation results for the group coefficient (η) from the experiment

Group	De Beer	Davission	Chin Konder
Group G1 (03 SC columns)	0.685	0.664	0.875
Group G2 (05 SC columns)	0.608	0.554	0.778

TABLE 6

Comparison of the group coefficient from the experiment with theory formula

SC Columns	Experiment	Feld	Terzaghi and Peck	Convese-Labarre	Coduto
Single	1.0	1.0	1.0	1.0	1.0
Group G1	0.664	0.87	0.75	0.69	0.595
Group G2	0.554	0.80	0.688	0.56	0.455

Terzaghi and Peck's Method

According to Terzaghi and Peck (Bowles 1996), the pile group coefficient is determined by equation (9).

$$\eta = \frac{Q_{ug}}{n_1 \times n_2 \times Q_{us}} \quad (9)$$

where Q_{ug} is the axial load bearing capacity of the group of piles; Q_{us} is the axial load bearing capacity of the single pile; n_1 is the number of a vertical row of piles; and n_2 is the number of horizontal row of piles.

The progress of this method is to take into account the mechanical properties of the soil layers through the piles and the soil under the piles in the coefficient of the piles.

Convese-Labarre's Method

According to Convese-Labarre (Bowles 1996), the pile group coefficient is given by equation (10).

$$\eta = 1 - \frac{\theta}{90} \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{n_1 \times n_2} \right] \quad (10)$$

where θ is arctan (D/d) (degree); D is the diameter of the pile; d is the distance between the two piles; and n_1, n_2 are the number of vertical and horizontal row of piles.

This equation is used quite widely. The limitation of this method is not to consider the pile length and ground around the pile; it is suitable for foundation piles in the uniform ground.

Coduto's Method

Another approach is to compare the mechanism of destructive pile groups, including individual failure and block failure. Partial damage means that the soil surrounding the piles will be damaged and that local piles will be

pushed down. Vandal sabotages both earth and pile in the block that is considered to be a massive mass shifted down to sabotage. Block destructive patterns are used if the total perimeter of the piles is greater than the perimeter of the pile group, and the pile coefficient is determined by equation (11) (Coduto 2001).

$$\eta = \frac{2d(n_1 + n_2) + 4D}{\pi \times n_1 \times n_2 \times D} \quad (11)$$

where D is the diameter of the pile; d is the distance between the two piles; and n_1, n_2 are the number of a vertical and horizontal row of piles.

Table 6 presents the group coefficient obtained from Feld, Terzaghi, and Peck; Convese-Labarre (Bowles 1996); and Coduto (2001). The group coefficient which calculates by Feld's method is based on the predetermined diagrams, regardless of the impact of the column's diameter and ground. Terzaghi and Peck's method takes into account the properties of soil, so the difference between group coefficients is about 10 %. Coduto's method assumes the destruction of the block, including the SC columns and soil in the foundation, resulting in a much smaller aggregate. This is a problem in the point of view that considers the foundation works as a block that previous studies have used in calculating the weak ground treatments. Convese-Labarre's method gives the smallest difference, as it takes into account the distance and the number columns in the horizontal and vertical direction. Therefore, it is considered appropriate with the model condition in the research.

Conclusions

The full-scale model for the single columns and group of columns was examined with the cement content of 350 kg/m³ in sand layer and 400 kg/m³ in sandy-clay soils by the wet mixing method, and the UCSs are 4.78–6.88 MPa. So, they are considered as semi-rigid columns to support the high building instead of treating the soft-soil ground.

The axial static load test using strain gages could determine the load distribution along the single column and columns in the group. The column's f_s value of the sand layer always reaches the ultimate results, whereas the friction of the lower layers slightly increases initially and increases in later increment loads.

The displacements to maximize the skin resistance for the single SC column are 20–22 mm, corresponding to 3.5 % of the column diameter. For the columns in the groups, the displacements are 30–35 mm, corresponding to about 5 % of the diameter. These displacement values are higher than 8 mm, according to Coyle and Reese.

The bearing capacity of the single SC column reaches over 1,160 kN, in which the skin resistance is about 55 % and the toe resistance is 45 % of the Q_u .

As the effect of the group, the decrement of skin resistance is about 1.98 % (the group G1 of 3 columns) and 4.17 % (the group G2 of 5 columns). The column in the middle of the group lost 16 % of its strength. At the same time, mobilization of the toe resistance at the center columns was higher than the outer columns (C8).

The group coefficients (η) are determined as 0.664 for the group G1 (three SC columns) and 0.554 for the group G2 (five SC columns). The results are close to Convese-Labarre's method. The research results explain the load transmission mechanism in the single SC column and the columns in the group.

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