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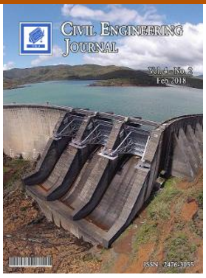

Investigation of Performance of Soil-Cement Pile in Support of Foundation Systems for High-Rise Buildings

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Abstract

This paper presents the experimental study of Soil-Cement Pile (SCpile) by wet mixing method in sandy soils, with the typical project at An Trung Complex apartment, Da Nang city, Vietnam. With the characteristic of soil layers is sandy soil, the strength of laboratory stabilized soils with the amount of cement from 150÷300 kg/m^3 was determined. Simultaneously, the authors also performed the experiments of 20 test piles collected from the site which has cement content about 280 kg/m^3 and the unconfined compressive strength q_u= (4.5÷6.0) MPa. After that, a full-scale model static axial compressive load tests of two single piles and a group of four piles with diameter 800 mm and 12 m length were also conducted. The experiment results show that the bearing capacity of every single pile is 1.200 kN with settlement 6.93 mm and the group of four CSpiles is 3.200 kN with settlement 5.03 mm. The results presented in the paper illustrate that SCpile is the suitable solution for foundation construction process with low cost and saving time for high rise buildings. The result shows a capable application of soil cement piles for support of high-rise buildings.

Keywords: Soil-Cement Pile; Sand Soil; Bearing capacity; Laboratory Tests; Full-Scale Experiment.

1. Introduction

Apartment complex construction for infrastructure projects has increased considerably during the past few decades in coastal and lowland regions where soft clay is popular. Geotechnical engineers dealing with these activities in such site conditions face a real challenge due to the low strength and compressibility characteristics of soft clay. A wide range of ground improvement techniques has been developed to increase the bearing capacity of soft ground and thereby to increase the use of the soft ground for the construction activities. Although solutions based on rigid-piles are available for the soil to increase the bearing capacity which yields uneconomical design. As a result, soil-cement pile (SCpile) are used as a hardening pile. This technique is an economical alternative compared to the rigid pile or inclusions. However, their performance under loading in ultimate limit state or serviceability conditions has so far received only limited investigation.

The soil-cement piles have been investigated and applied into practice from the past four decades, starting in Sweden and Japan is now an established and increasingly popular technique [1-3]. There include two following conventional methods such as the dry method and wet method. The methods in which dry binder is blown pneumatically into the ground are called the dry process of deep mixing. In contrast, the techniques in which binder-water slurry is pumped into the ground are generically called the wet method of the deep blend. In general, there is a wide range of applications can be used by SCpiles such as piled embankment, support for deep excavation, maybe even for the foundation of high rise buildings [4-5].

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During the past four decades, a variety of results on soil-cement piles published and released. In summary, the following two different approaches are more common: The first method is based on the physical characteristics of materials. Donal A. Bruce (2000) [1], Ajorloo A.M (2010) [4] presented the study results for various soils in France and America. Jacobson et al. (2003) [5] investigated the performance of lime-cement columns in support of the dam. Kitazume et al. (2013) [6] came out many precise results that are involved in the strength of soil-cement piles.

The second way is developed to consider the performance of soil-cement piles in applying to foundation for high-rise building which consists taking into account the mechanical properties of single pile and pile group, for instance, Broms (1999) [7], Japan (2001) [8], Jie Han (2004) [9], Bouassida et al.(2009) [10]. Some authors have also been investigated based on the centrifuge models, such as Kitazume and Maruyama (2007) [11], Abbas & Tatsuota (2015) [12], Jian-Hua & Zhen Fang (2010) [13]. As well as the top of this way, many experiments have been conducted in situ to discover an increase in understanding of interaction mechanism between soil-pile such Banverket (2009) [14].

By the extensive laboratory tests on a variety of sands, it was found that most of sands easily gained strength of the order of 5MPa to 10MPa regarding unconfined compressive strength. Therefore, the soil-cement piles can be used for the foundation of high-rise buildings in the role of bearing capacity piles. In practice, many successful case histories, both in Vietnam and abroad, have been reported in the literature over recent years [15-21]. Authors proposed and applied the soil cement piles for the foundation of some buildings with the height of about 7 to 17 stories in Vietnam, successfully. With the valuable experiences in practice, these results will re-interpreted here. This paper aims mainly is to investigate and present the potential of SCpiles in using the foundation of high-rise buildings that yields an economical design. The laboratory was performed to recognize the essential physical characteristics of piles, and then field experiments for the single pile and pile groups were conducted and measured. The primary results will be interpreting in this paper.

2. Research Methodology

2.1. Geotechnical Test

In this study, the geotechnical tests consist of laboratory and field experiments were conducted to investigate the main problems of the paper. The laboratory tests were carried out to elucidate the relationship between the undrained compressive strength with other involved factors. Meanwhile, the full-scale field loading test program has the performance to study the load transfer mechanism and the soil-structure interaction. The results will be interpreted in terms of strength and resistance which allow taking into account the bearing capacity of the soil-cement pile.

2.2. Generic Design Case Considered

In this project, the author performed experiments on SCpile that applied for the foundation of An Trung complex apartment in Da Nang city, Vietnam, the location of the project is shown in Figure 1. This project was designed as a twelve-stories building to provide the services to low-income people. Some requirements were given in design process, included reducing the cost of the project in general and foundation structure in particular. Based on the previous studies including the pros and cons of SCpile, the method using SCpile was chosen for support to the foundation structure of the projects.

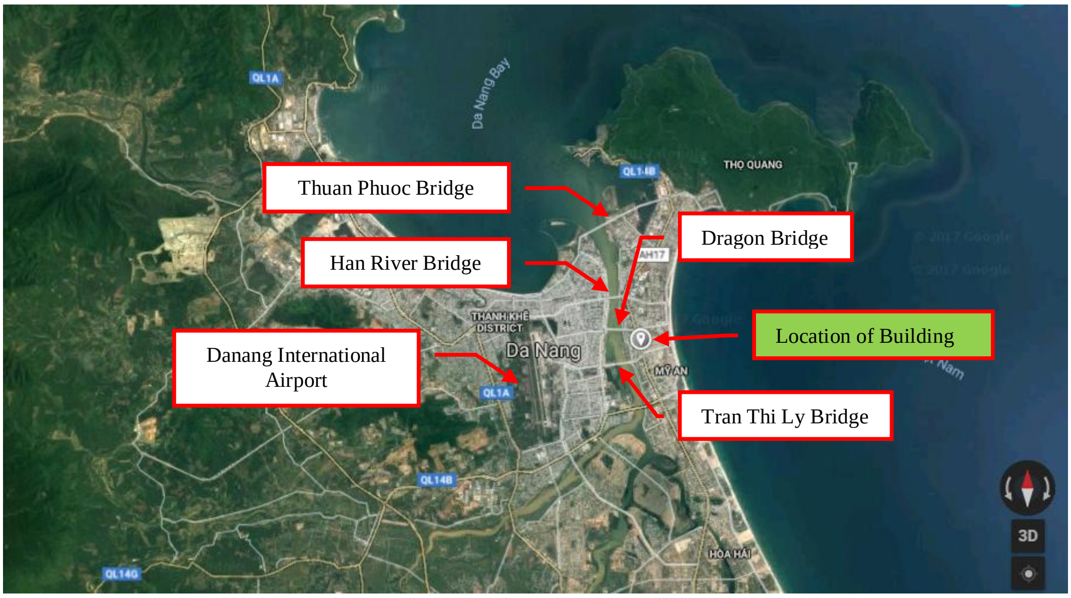
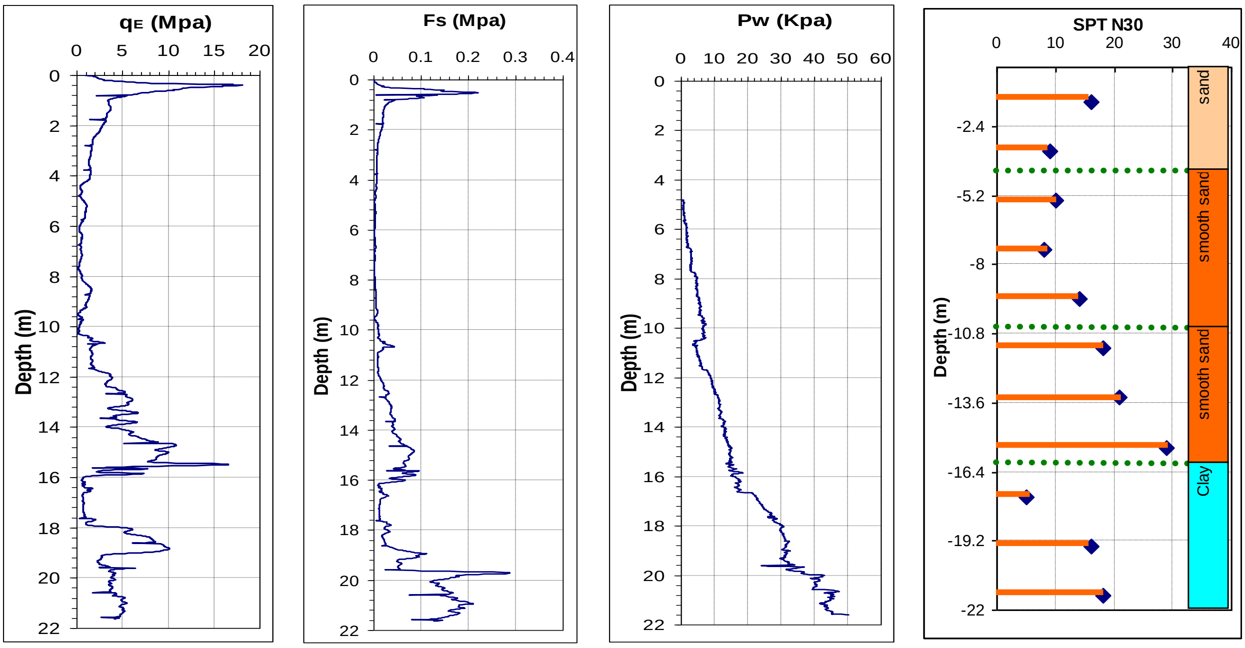


Figure 1. Location of project under the satellite system

2.3. Site Condition

From soil testing and Cone penetration test (CPTu) shown in Figure 2 (a,b,c). The properties of soil layers can be described briefly as follows: Layer 1: 0-1.1 m is crust; Layer 2: 1.1-4.0 m is small-grained sand soil, $\overline{q_E} = 0.7\ MPa$, and $\overline{F_s} = 0.0044\ MPa$; Layer 3: 4.0-10.2 m is fine-grained sand soil with loose state, $\overline{q_E} = 0.7\ MPa$, and $\overline{F_s} = 0.0044\ MPa$; Layer 4: 10.2-16.0 m is fine-grained sand soil with dense state, $\overline{q_E} = 4.87\ MPa$, and $\overline{F_s} = 0.038\ MPa$; Layer 5: 16.0-22.0 m is clay with plastic state, $\overline{q_E} = 3.32\ MPa$ and $\overline{F_s} = 0.073\ MPa$. The result of Standard Penetration Test (SPT) from bore hole is presented in Figure 2d.



a) b) c) d)

Figure 2. Measured data from CPTu test and SPT test

2.4. Structure of Foundation

The load transfer below structure is around about (3.800-4.500) kN per structure building column, the space between building columns is 6.3 m. The rectangular combined footings above have the width of 3.2 m, the length of 20.8 m and the height of 0.7 m. The solution was initially suggested by using reinforced concrete piles. However, the disadvantage of this technique is slow construction, the over-high cost. To accelerate the process of construction and economic costs, the SCpiles has been proposed and recommended to the contractors instead of concrete piles. The basic geometry parameters of the SCpiles as follows: Diameter is 800 mm, length of the pile is 12 m, and pile spacing is 1m. The design capacity of a single pile is 400 kN. Cement content is about 280 kg/m^3, design compressive strength F_c=2.0 MPa, allowable compressive strength f_c=1.0 MPa, allowable tensile strength $f_t = 0.15.f_c = 0.15$ MPa.

3. Result Analysis

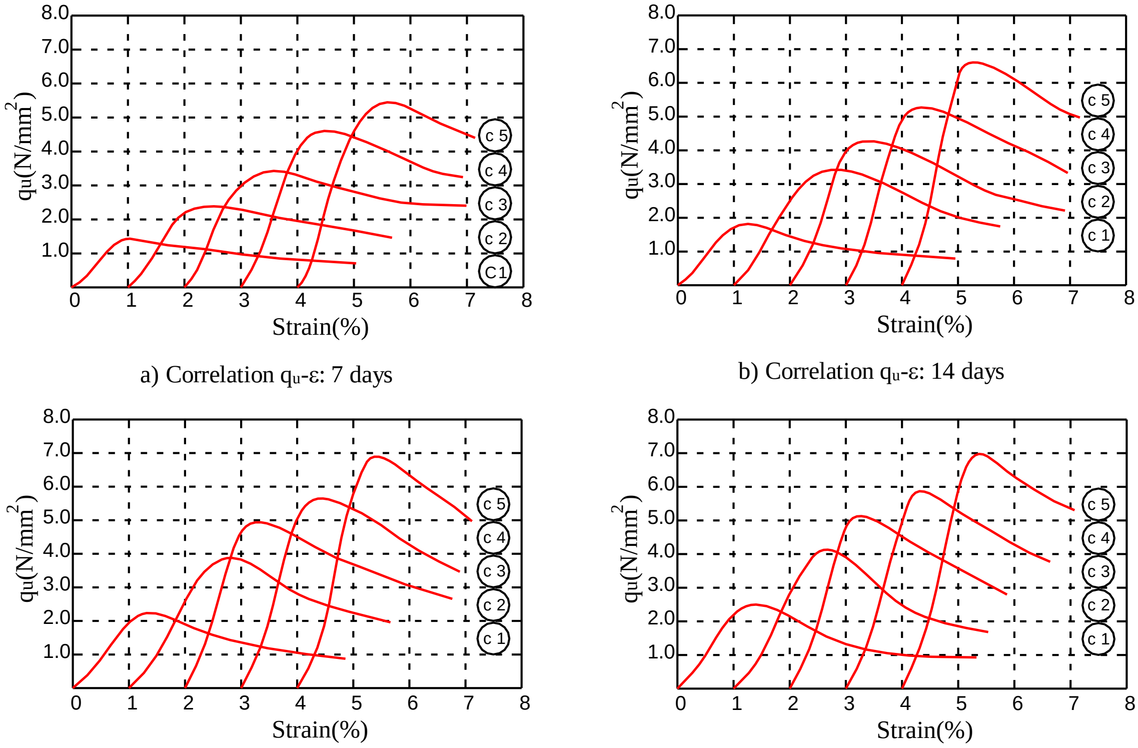
3.1. Material Strength

3.1.1. Laboratory Test

At the first step, the soil sample has got from the field through the process of drilling, then mixing soil with cement carried out at the laboratory. There are five cases corresponding to the different cement content: 150 kg/m^3 (C1), 200 kg/m^3 (C2), 250 kg/m^3 (C3), 300 kg/m^3 (C4), 350 kg/m^3 (C5). The water to cement ratio is remained at about W/C = (0.6-0.8). The dimensions of a stabilized soils samples are 70.7 × 70.7 × 70.7 mm followed Vietnamese code [19]. These samples were cured and measured the unconfined compressive strength among 7-14-21-28-56 days in table 1. The results of stress-strain curve from Trapezium 2.0 software are presented in Figure 3.

Table 1. Relationship between strength increase and curing period

Days	Case/q_u (N/mm^2)				
	C1	C2	C3	C4	C5
7	1.46	2.35	3.35	4.68	5.42
14	1.85	3.15	3.98	5.22	6.4
28	2.45	3.98	4.85	5.66	6.8
56	2.5	4.02	5.02	5.85	7



a) Correlation q_u-ε: 7 days

b) Correlation q_u-ε: 14 days

c) Correlation q_u-ε: 28 days

d) Correlation q_u-ε: 56 days

Figure 3. Unconfined compressive strength - strain of laboratory stabilized soil

Figure 3 shows that the strain of the sample increases and reach to the maximum value at the somewhere between 1.35 % and 1.75 %, and the failure of the sample take up to occur during this period. The softening-strain behavior then occur later as the strength decreases while the strain continually increases. According to the figures, it has been observed further that the tendency of strength increase is more similar for all five cases, especially, soar within seven days to 14 days. At this period, the strength has obtained a relative degree of strength at 28 days, with q_{u7days} = (0.60-080) $q_{u28days}$ and $q_{u14days}$ = (0.76-0.94) $q_{u28days}$ respectively. The latter period of day 28, however, seems to be stable in this figure, with the additional increase only about between 2 % and 4 %.

The unconfined compressive strength of material increased with curing time-based on the results of laboratory tests illustrated in Figure 4. The line of regression for the strength increase with time for all various cases are shown in Figure 4.

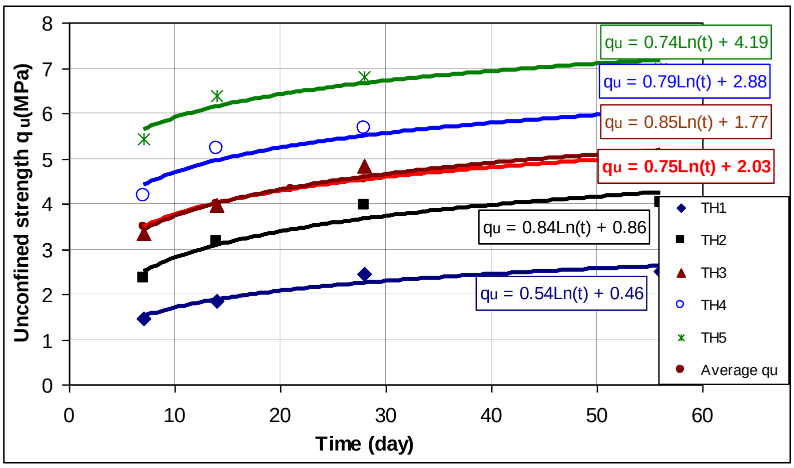


Figure 4. Increase of unconfined compressive strength with time

The strength increase with curing period which has obtained in this study compared to the results of some authors in the literature recently, as shown in Table 2. For the cases corresponding to 7 days and 14 days, the strength increase observed in this study is higher significantly compared to that of Kitazume et al. (2007) [11], Helen (2006) [16]. Due to the clay degree placed in the sand soil sample is very small can be considered as an explanation of why the strength of soil sample is quickly reached to the asymptote at day 28. This is more different to the strength growth of clay soil when its strength may increase supplement from 4 % to 25 % after 28 days.

Table 2. Comparison of testing results with that of some other authors (for soft soil)

This study	Helen et al. [16]	Kitazume [11]	Sweden [17]
$q_{u7} = (0{,}6\text{-}0{,}8)q_{u28}$	$q_{u14} = (0{,}43\text{-}0{,}67)q_{u28}$	$q_{u7} = (0{,}63\text{-}0{,}694)q_{u28}$	$q_u = 0{,}58q_{u28}$
$q_{u14} = (0{,}76\text{-}0{,}94)q_{u28}$	$q_{u21} = 0{,}56q_{u28}$	$q_{u21} = 0{,}56q_{u28}$	$q_{u14} = 0{,}79q_{u28}$
$q_{u56} = (1{,}02\text{-}1{,}04)q_{u28}$	$q_{u56} = (1{,}04\text{-}1{,}25)q_{u28}$	$q_{u56} = (1{,}04\text{-}1{,}25)q_{u28}$	$q_{u56} = 1{,}208q_{u28}$

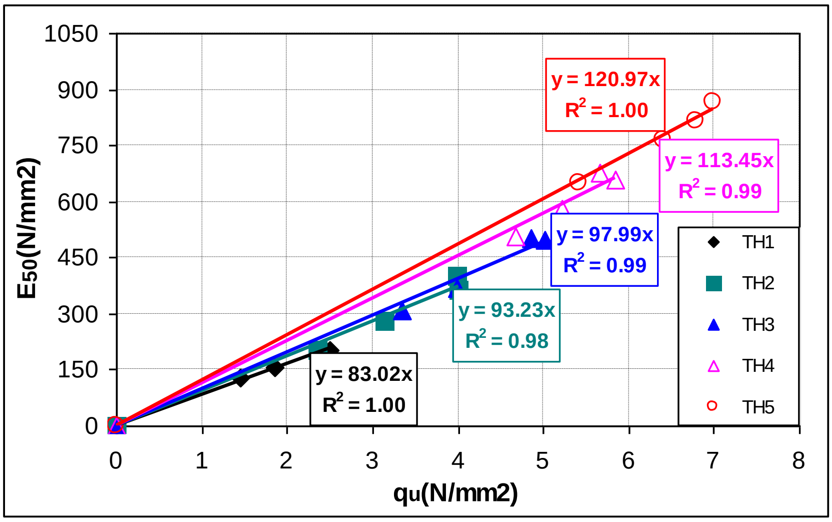


Figure 5. Relationship between secant elastic modulus and value q_u

From the graph q_u-ε of the different cases corresponding to various curing periods, the secant elastic modulus E_{50} is determined by the ratio between strength and strain respectively. Figure 5 demonstrates the relationship between the secant elastic modulus E_{50} and the unconfined compressive strength q_u for the various amount of cement of five cases in this study. According to the presented data, the correlation between elastic modulus and unconfined compressive strength is determined by the following approximate equation:

$$E_{50} = (83.016\text{-}120.97)\ qu \tag{1}$$

There are existing several of the correlation equations between the secant elastic modulus and unconfined compressive strength which have been proposed as follows:

Kitazume et al., 1977 [11]: $E_{50} = (75.5\text{-}1000)\ qu$ (2)

Saitoh, (1985) [16]: $E_{50} = (350\text{-}1000)\ qu$ (3)

Jie Han, (2004) [9]: $E_{50} = (50\text{-}150)\ qu$ (4)

FHWA-RD-99-138 [1]: $E_{50} = (100\text{-}500)\ qu$ (5)

Comparison of the proposed equation with the results of some other authors shows that there has a broad range of prediction to the secant elastic modulus. It is also evident that the result of this study agrees well or equally well with that of J. Han (2004) [9].

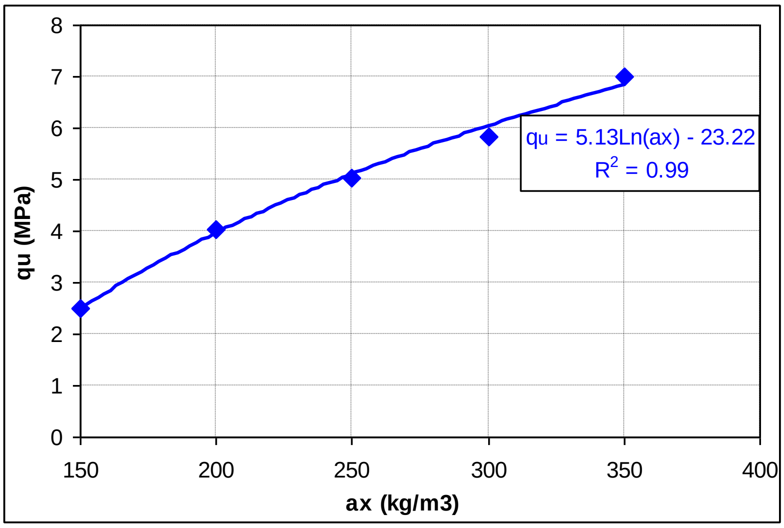


Figure 6. Relationship between binder content and q_u value

Figure 6 demonstrates the influence of binder content to the unconfined compressive strength of cement treated soil on day 28 from the beginning the curing period. As can be seen that the unconfined compressive strength is involved in the binder content by the following equation:

$$q_u = 5.13Ln(ax) - 23.22 \text{ (MPa)} \tag{6}$$

This equation may contribute a specific role in estimating the amount of cement that need to use to obtain the design strength without further tests.

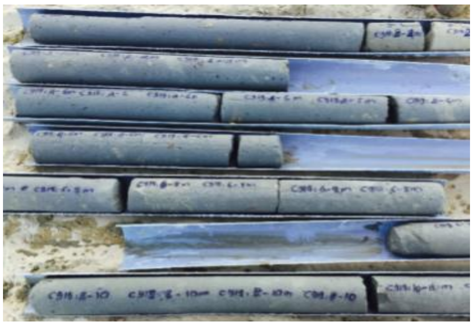
3.1.2. Full-Scale Field Experiments

Soil-cement pile is established at the site as follows: piles are constructed by deep mixing method that based on Japanese technology. The ratio of water/cement W/C = 0.6-0.8, penetration speed 0.5 m/min, the rotation of mixing blades 30-35 rev/min. When the drill reaches a design depth, the cement slurry is injected from the outlets near the mixing blades and is mixed with the soil. The procedure constructs a stabilized soil with rectangular parallelepiped shape as shown as in Figure 7.

a. SCPile Machine **b. The Blade** **c. SCPile after drilling**

Figure 7. SCpile construction process at si-tu

The geometry and design parameters for soil-cement pile in this study as follows: diameter – 800 mm, length – 12 m, cement content - 280 kg/m^3, curing period - 21 days. Figure 8 illustrates the samples for testing that took place in situ and laboratory.



a) Core drilling samples

b) Compression test samples

Figure 8. Core drilling in situ and compression test in laboratory

The results of the unconfined compressive strength measured through experiments in situ are shown in Figure 9. It has been observed that the average value of q_u is 4.3 MPa for all samples and this result is much higher than design strength value, about 2.0 MPa. It can also be seen that at the locations of soil with high density, the strength will increase highly, at 5.5 MPa in this study. The minimum strength, meanwhile, is required so that reaching to the material capacity of SCpile is mere 2.72 MPa.

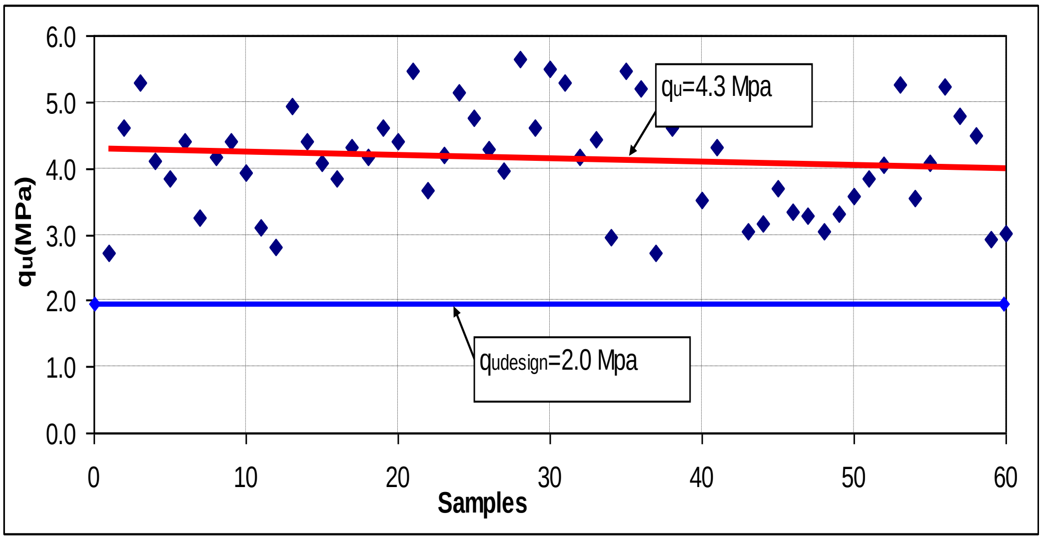


Figure 9. The distribution spectrum of strength among testing samples

3.2. Bearing Capacity of Single and Group of Soil-Cement Piles

3.2.1. Design and Testing Program

In this study, two separate SCpile with similar parameters in the previous section are selected for testing, denoted by TP01 and TP02, and a group consists of 4 SCpile, denoted is TP3. The detailed scheme of the test is shown in Figure 10.

These SCpiles experimented under static axial compressive load based on standards on ASTM D1143 [4]. A group of four SCpiles with the space of 1m was built to prepare for testing. The design load of each group SCpile is 1.60 kN and applying the pressure in increments of 25 % of the group design load. Some images of the testing process shown in Figure 11.

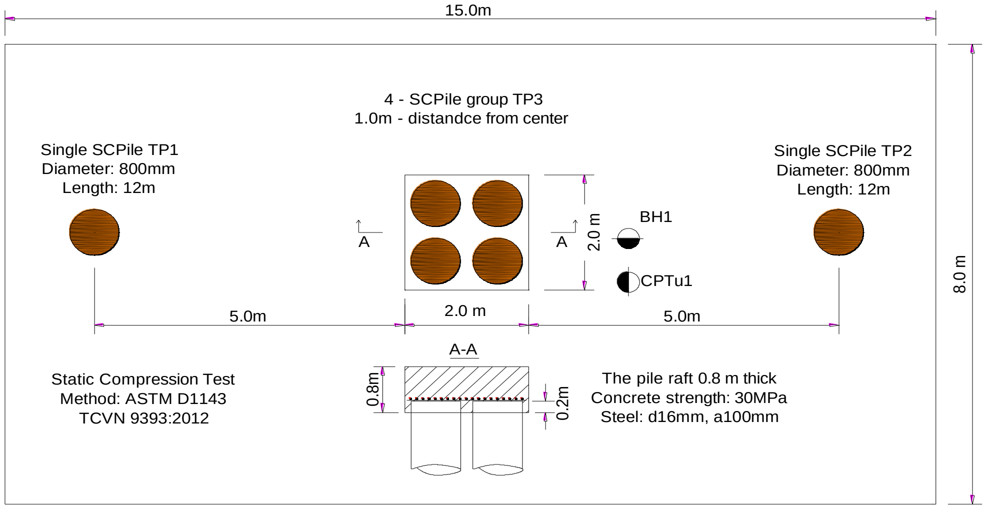


Figure 10. Schematic presentation of the test geometry

a) Static load test for group SCpile

b) Equipment for load test

Figure 11. Arrangement for applying load in an axial compressive test

3.2.2. Interpretation of Results

Figure 12 shows the relationship between loads and settlement for individual SCpile TP01, the loading conducted to a value is 1200 kN, correspond to 300 % of design load. As can be seen that, the total settlement observed at this value is 6.93 mm. Compared to allowance settlement value at 10%. D=80 mm, the bearing capacity of the pile is evidently much higher than requirements. Therefore, one can be concluded that the SCpile has able to mobilize the high resistance in order to ensure the capacity of a high-rise building.

An experiment has been performed similarly for the SCpile, TP02, and the results drawn in Figure 13. However, the maximum value for the loading, in this case, is 1.360 kN. It can be seen that the total settlement, in this case, is 7.75 mm, the material of pile head was damaged, and the Q-s graph is plunged, as shown in Figure 13. At the pile head where endured, a locally high-stress state can be considered as the reason of why the damage took place in this area. However, the pile may reach a higher bearing capacity if the strength of the material at pile head is improved.

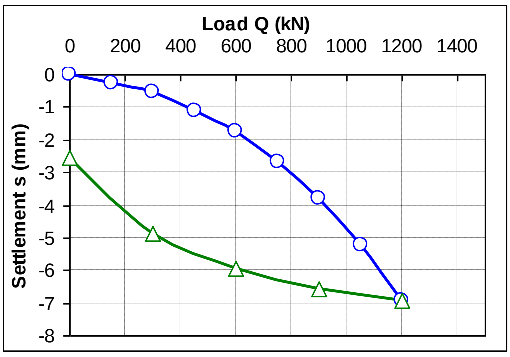


Figure 12. Result of load-Settlement for TP01

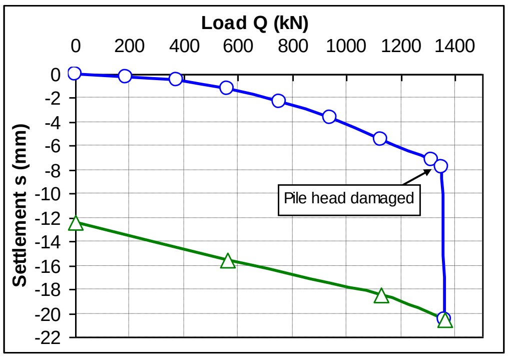


Figure 13. Result of load- Settlement for TP02

A static load test is conducted for the group of 04 SCpiles, and results are shown in Figure 14. In this presented case, the maximum load imposed at 3.200 kN, double the design load. The figure shows that total settlement is 5.03

mm. Consequently, with the designed charge for the work, the foundation of soil-cement piles can carry the very high load and ultimately meets the requirements of the project. Figure 15 shows the results comparison between the single pile, TP1 and per a single pile in the pile group, TP3, in which the load acting on the group shall be divided uniformly to 04 SCpiles. The graph proves that the load carrying capacity of the single pile is much more than that of the group at the same movement value. The cause for this phenomenon is due to the influence of group effect. When the piles are placed close to each other, a reasonable assumption is that the stresses transmitted by the piles to the soil will overlap a much larger area and extend to a greater depth than that of a single pile. Reducing the load-bearing capacity of the piles is therefore occurs.

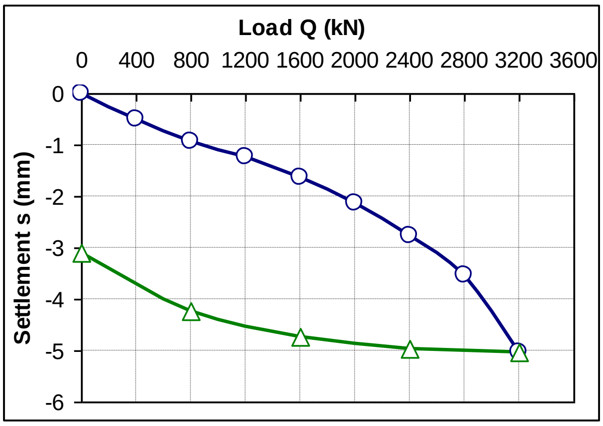


Figure 14. Result of load- Settlement for group TP3

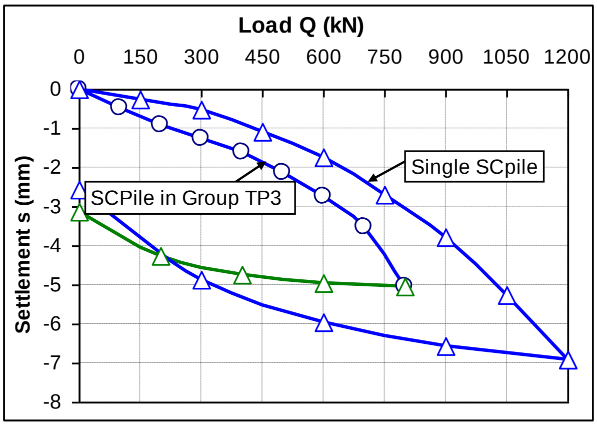


Figure 15. Comparison between single and pile group

3.2.3. Evaluation of Group Coefficient

In order to increase the reliability of results, the well-known formula proposed by Converse **-** Labarre will be used to compare to the experiment results in this study.

$$\eta = 1 - \frac{\theta}{90}\left[\frac{(n_1-1)n_2+(n_2-1)n_1}{n_1.n_2}\right] \tag{7}$$

Where: $\theta = \tan^{-1}(D/d)$ (degree); D – pile diameter; d – Center to center pile spacing; n_1, n_2– the number of piles in vertical and horizontal axis respectively.

The result has got from this equation is $\eta_{CL} = 0.55$, while the result calculated by the experiment data at same settlement valua at 5.03 mm is $\eta_{test} = 0.76$. Hence, group efficiency factor based on the result of the field investigation is higher than that of the theoretical equation. The higher interaction of soil-structure in SCpile through the shaft surface compared to other rigid piles can be the reason for this difference.

3.2.4. Analysis of the Bearing Capacity

The pile end bearing stress from CPT has been observed to vary from about $0.4q_E$ to $2q_E$, where q_E is the cone tip resistance. The determination of q_E from cone penetrometer results is controversial, and several methods have been suggested. The difference in the ways come from the influence zone over which the cone values are extracted and the averaging procedures used. One method, proposed by Eslami and Fellenius (2009) [18], is to consider the cone resistance over an influence zone eight pile diameters above the base and four piles diameter below the base-a total of twelve pile diameters- for piles penetrating a weak soil and resting on a dense ground.

$$(q_E)_{ag} = (q_{E1}q_{E2}q_{E3\ldots\ldots}\, q_{En})^{1/n} \tag{8}$$

Where q_{E1} to q_{En} are discrete cone resistance over a distance twelve piles diameter or two piles diameter depending on the soil layering, and n is the number of q_E values.

For the skin friction, f_s, in the CPT is a measure of the skin or shaft frictional stress. The relative density of the soil and soil compressibility affect the sleeve resistance while the relative density of the soil, method of installation, soil compressibility, pile geometry, and surface roughness affect skin frictional stress on a pile. An estimate can be made from one of several equations proposed in the literature. In this study, the one of Eslami and Fellenius (2009) is chosen to introduce for computation here.

$$f_s = Cs.q'_{Es} \tag{9}$$

With C_s - the coefficient of shaft friction resistance: Cs=0.015 for sand and Cs=0.018 for smooth sand, q'_E – average effective toe resistance of cone penetration testing CPTu: $q'_E = (q_E\text{-}u)$.The details of the calculation are presented in Table 3.

Table 3. Distribution of shaft friction resistance along single SCPile TP1

Depth (m)	q_E (MPa)	fs (0.01MPa)	p_w (0.01MPa)	q'_E (MPa)	Cs	f_s (kN/m^2)	Q_f (kN)	Q_{fz} (kN)
0	0	0	0.00	0.000				0
1	7.255	6.958	0.00	7.255	0.015	108.83	273.38	273.38
2	3.376	2.033	0.00	3.376	0.015	50.64	127.21	400.58
3	2.047	1.086	0.00	2.047	0.015	30.70	77.11	477.70
4	1.479	0.719	0.00	1.479	0.015	22.19	55.74	533.44
5	0.728	0.470	0.065	0.727	0.018	13.09	32.89	566.33
6	0.847	0.372	0.170	0.845	0.018	15.21	38.21	604.53
7	0.474	0.302	0.295	0.471	0.018	8.49	21.31	625.85
8	0.434	0.240	0.476	0.429	0.018	7.72	19.40	645.25
9	1.253	0.477	0.60	1.247	0.018	22.44	56.37	701.62
10	0.512	0.732	0.70	0.505	0.018	9.10	22.85	724.47
11	1.421	1.659	0.52	1.415	0.018	25.48	64.00	788.47
12	2.469	1.189	0.92	2.460	0.018	44.28	111.24	899.70

Figure 16 demonstrates the distribution of the pile end bearing stress q_E (MPa) along the depth, the results shown under unit 0.01 MPa in order to ensure the convenience for assessing the end bearing resistance of the SCpile. The pore-water pressure also considered and illustrated on the graph under unit 0.01 MPa, take up at level 4 m below from the ground.

Figure 17 show the shaft resistance that accumulated among all the length of pile, the interpretation based on Eslami and Fellenius's methods is presented in Table 3. According to the data, the end bearing capacity or toe resistance of SCpile is Q_f = 899.7 kN. The curves Q_1 and Q_2 represent vertical loading inside the pile, the load descends from the pile head because of shaft friction absorption. The remaining load acting on the pile toe is Q_r = 300.3 kN (25%.Q_1) and Q_r = 460.3 kN (33.8%.Q_2) for TP1 and TP2 respectively. For the case of pile group, TP3, the total resistance distributed mainly by the component of shaft resistance, and the end bearing resistance, therefore, are approximately zero which means that with this load, the pile toe does not have to work. It can also be realized that the resistance of pile group can reach the higher value than what measured in the field experiments here.

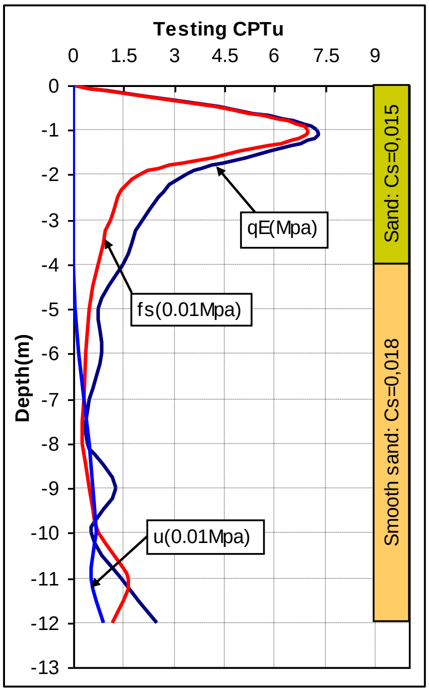


Figure 16. Data from CPT test

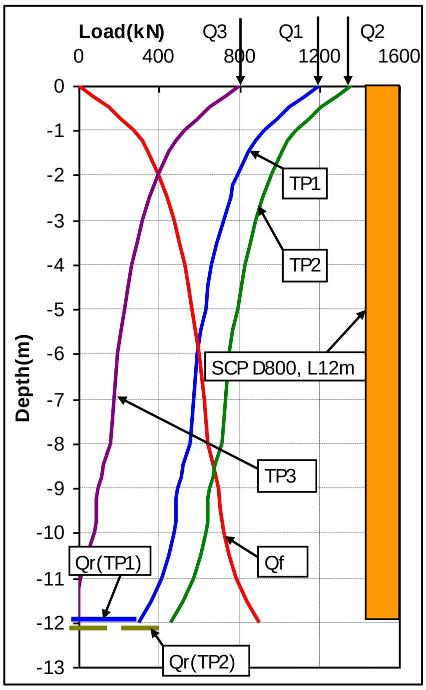


Figure 17. Distribution of frictional resistance

4. Conclusion

The problems relate to the increase of material strength as well as the complex mechanisms of interaction between SCpile-soil have been mostly investigated using the laboratory test and field experiments. The findings are summarized based on the experimental analyses as follows:

The results found on the laboratory test for five cases with various cement content shown that the trend of strength increase develop exponentially and reach the approximating value at day 28. The increase of unconfined compressive strength is involved with the time by the following equation: $q_u = 0.75\ln(t) + 2.03$.

The value of unconfined compressive strength can determine the secant elastic modulus of the stabilized soil sample. According to the experimental results, one proposed here is $E_{50} = (83.016\text{-}120.97)q_u$, and strictly to the study of J. Han. Also, the relationship between the value of unconfined compressive strength and binder content is $q_u = 5.13\text{Ln}(ax)\text{-}23.22$ (MPa),

The unconfined compressive strength of the material of SCpile gain q_u = (2.72-5.8) MPa with 280 kg/m^3 cement content, it is higher than cemented in soft soil. This results proved that SCile possible used for the foundation of a high-rise building, which leads to economical design than using concrete piles or bored piles.

Load bearing capacity of single SCpile reached a relatively high value, around between 1.200 kN and 1.360 kN, which exceeded 300 % of the design load. As a consequence, SCpile is more efficient to use for load bearing foundation.

The efficiency factor of SCpile determined from the experimental study was about 0.76, higher than that of the theoretical method proposed by Converse - Labarre is 0.55. The reason for this difference is due to the complex interaction between structure-soil. The friction distributed along the shaft of SCpile might be better than other types of piles.

The analysis of resistance of the pile from CPTu test shows that for the single pile, the end bearing resistance and shaft resistance are mobilized at the same time with the different magnitude, in which the pile toe resistance is accounting for 25 % to 33.8 % of the ultimate bearing capacity. However, for the case of the pile in the group, only partial shaft friction is mobilized while the toe resistance is approximately zero. Therefore, one can be concluded that the full end bearing is not assembled at the same displacement as the whole skin fiction-the total skin fiction is mobilized about one-tenth the movement required to mobilize the entire end bearing resistance

The results show that soil cement piles might be sufficient to apply for high rise buildings with more prominent advantages such as cost savings, fast progress and reducing environmental pollution. This is an improvement over conventional applications using soil cement piles to treat soft soils.

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6. References

[1] Bruce, Donald A. An introduction to the deep soil mixing methods as used in geotechnical applications. No. FHWA-RD-99-138, 2000.

[2] Åhnberg, Helen. Strength of Stabilised Soil-A Laboratory Study on Clays and Organic Soils Stabilised with different Types of Binder. Avd. för Byggnadsmekanik/Department of Construction Sciences, LTH, 2006.

[3] Kitazume, Masaki, and Masaaki Terashi. The deep mixing method. CRC Press, 2013.

[4] Ajorloo Ali Mohammad, Characterization of the Mechanical Behavior of Improvement Loose Send for Application in Soil-Cement Deep Mixing, httpdoc.Univ Lille 1.fr, (2010).

[5] Jacobson, Jesse R., George M. Filz, and James K. Mitchell. Factors affecting strength gain in lime-cement columns and development of a laboratory testing procedure. Virginia Center for Transportation Innovation and Research, 2003.

[6] Terashi, Masaaki. “The Deep Mixing Method” (February 21, 2013). doi:10.1201/b13873.

[7] Broms, BengtB. "Keynote lecture: Design of lime, lime/cement and cement columns." In International Conference on Dry Mix Methods: Dry Mix Methods for Deep Soil Stabilization, pp. 125-153. Balkema, Rotterdam, 1999.

[8] Building center Japan, A design and quality control indicator of the improvement foundation for a building, Japan, (2001).

[9] Jie Han, Deep Mixed Columns, FHWA NHI-04-001, (2004).

[10] Bouassida, Mounir, Belgacem Jellali, and Ali Porbaha. “Limit Analysis of Rigid Foundations on Floating Columns.” International Journal of Geomechanics 9, no. 3 (May 2009): 89–101. doi:10.1061/(asce)1532-3641(2009)9:3(89).

[11] Kitazume, Masaki, and Kenji Maruyama. "Centrifuge model tests on failure pattern of group column type deep mixing improved ground." In The Seventeenth International Offshore and Polar Engineering Conference. International Society of Offshore and Polar Engineers, 2007.

[12] Taheri, Abbas, and Fumio Tatsuoka. “Small- and Large-Strain Behaviour of a Cement-Treated Soil During Various Loading

Histories and Testing Conditions." Acta Geotechnica 10, no. 1 (December 17, 2014): 131–155. doi:10.1007/s11440-014-0339-7.

[13] Yin, Jian-Hua, and Zhen Fang. "Physical Modeling of a Footing on Soft Soil Ground with Deep Cement Mixed Soil Columns under Vertical Loading." Marine Georesources & Geotechnology 28, no. 2 (May 20, 2010): 173–188. doi:10.1080/10641191003780872.

[14] Banverket, D2.2.5 – Subgrade reinforcement with columns Part 1 Vertical columns, Part 2 Inclined columns, INNOTRACK Project no. TIP5-CT-2006-031415, (2009).

[15] TCVN 9403:2012. Soft soil Reinforcement - Soil-cement method, Construction Publishing House, Hanoi, Vietnam, (2012).

[16] Latifi, Nima, Ahmad Safuan A. Rashid, Aminaton Marto, and Mahmood Md Tahir. "Effect of Magnesium Chloride Solution on the Physico-Chemical Characteristics of Tropical Peat." Environmental Earth Sciences 75, no. 3 (January 25, 2016). doi:10.1007/s12665-015-4788-6.

[17] Shirvani, Reza Alijani, and Issa Shooshpasha. "Experimental Study on Load-Settlement Behaviour of Cement Stabilised Footing with Different Dimensions on Sandy Soil." Arabian Journal for Science and Engineering 40, no. 2 (December 6, 2014): 397–406. doi:10.1007/s13369-014-1444-5.

[18] Bengt H. Fellenius. Basic of Foundation and Design, Electronic editor. www.Fellenius.net, 346pp, (2009).

[19] Frikha, Wissem, Habib Zargayouna, Samia Boussetta, and Mounir Bouassida. "Experimental Study of Tunis Soft Soil Improved by Deep Mixing Column." Geotechnical and Geological Engineering 35, no. 3 (January 3, 2017): 931–947. doi:10.1007/s10706-016-0151-2.

[20] Yi, Yaolin, Songyu Liu, Anand J. Puppala, and Peisheng Xi. "Vertical Bearing Capacity Behaviour of Single T-Shaped Soil–cement Column in Soft Ground: Laboratory Modelling, Field Test, and Calculation." Acta Geotechnica 12, no. 5 (April 27, 2017): 1077–1088. doi:10.1007/s11440-017-0555-z.

[21] Tuan, Pham Anh. "Analysis and Numerical Modelling of a Piled Foundation Reinforced Geosynthetics to Support for Full-Height Bridge Abutments Constructed through Soft Soil." Global Journal of Research In Engineering (2016).