Effect of Stone Columns on the Behaviour of Single Pile in Soft Clay Soil

Vladimir Znamenskii¹, Mostafa Abdou Abd El-Naiem², Ahmed Rushdy Towfeek³, Osama Hegazy¹,³

¹ National Research Moscow State University of Civil Engineering, Moscow, Russia;
² Department of Civil Engineering, Faculty Engineering, Aussit University, Aussit, Egypt;
³ Department of Civil Engineering, Faculty Engineering, Al-Azhar University, Quena, Egypt;
osamahegazy46@yahoo.com

Abstract: Soft clay is consolidated upon any slight change in the effective stress. Consolidation and settlement of soft clay surrounding a pile usually drag the pile down ward. The down drag movement adds additional loads to the already loaded pile. This force is expressed as a negative skin friction. Negative skin friction starts from the pile head to a neutral depth depending on the interface properties. This depth is referred to as the neutral plan. This paper presents a study on the effect of stone columns on the behaviour of piles during consolidation of soft clay soil. For this goal the experimental tests were done to study effect of stone columns on the behaviour of single pile embedded in soft clay. The study concluded that the neutral plane is located closer to the end of the pile as the end bearing increases. Stone columns are considered one of methods used to reduce the settlement of soft clay soil, and helps to speed up the rate of consolidation by shorting the length of drainage path within the soft clay layers, and then decrease the relative displacement between the pile and the surrounding soft soils. The use of stone columns reduced the amount of negative friction developed along the pile depth.

1. Introduction

Pile foundations are installed in different soil stratification. Usually the penetrated soil stratifications offer considerable resistance for the pile shaft deformation upon loading. This resistance is called skin friction. Soft clay is consolidated upon any slight change in the effective vertical stress. This soil behaviour can be reversed drastically if the penetrated soil encompasses soft clay. Consolidation and settlement of soft clay surrounding a pile usually drag the pile downward. The magnitude of negative skin friction depends on the ultimate resistance and relative displacement between pile shaft interface and the surrounding soft soil, and also the pile toe stiffness. To exactly compute negative skin friction, the subsoil displacement profile, pile shaft stiffness and the pile toe stiffness are required to determine the depth of the neutral plane. The cumulative pile shaft resistance above the neutral plane is a drag load, whereas, positive resistance is found below the neutral plane to resist the total downward loads from negative skin friction and the imposed load at pile top. As indicated by Fellenius [1], the neutral plane is the plane where there is no relative displacement between the pile and the surrounding soil. He also indicated that the greater the pile toe resistance, the deeper the neutral plane, and the larger the drag
load. Johannessen and Bjerrum [2], Fellenius [3,4], Blanchet et al. [5], Bozozuk [6], and Indraratna et al. [7] performed field measurements on instrumented piles.

Stone column technique is one of the most commonly used soil improvement technique for soft clay soils. The technique has been used to increase the load carrying capacity of soft soils and also to reduce the settlement of superstructure constructed on them. Datye and Nagaraju [8] proved that the stone columns reduced the settlement significantly. Recently, Mostafa et al [9], reported that stone columns reduced the settlement of the surrounding soil with 54.68%, as well as increased the resistance capacity of piles. The aim of this study is the distribution of shear stress along pile-soil interaction on single pile constructed in soft clay during soil settlement under surcharge load; a special tank was designed to achieve this goal.

2. Experimental work

2.1 Steel model
A movable steel model was designed to host the bed of soft clay soil and all accessories. The circular model was made of steel plate 3 mm in thickness. The internal dimensions were 520 mm in diameter and 750 mm in height. These dimensions were chosen to eliminate the boundary effects. The model was provided with four plates of steel 20×20×5 mm that allows it to move freely and provided with sheet of Perspex 520×50×5 mm on the outer surface of the tank, to see and adjust the height of the soft clay layers. Figure 1 shows details of the model was suggested by Mostafa et al [9].

![Experimental Model](image)

**Figure 1.** Experimental Model

2.2 Materials
2.2.1 Soft clay
Soft clay of high plasticity was collected from Tahta city in Sohag- Egypt. The soil was subjected to laboratory tests to determine its properties, these tests include: Atterberg’s limits (Liquid and plastic
limits) and specific gravity according the Egyptian code of soil mechanics and foundations [10]. The natural properties of used soft clay are present in Table 1. The results show that the soil is classified as clay of high plasticity.

### Table 1. Physical Properties of soft clay

| Property                      | Value |
|-------------------------------|-------|
| Average Water content (%)     | 58.85 |
| Bulk Unit Weight ($\gamma_d$, kN/m$^3$) | 15.0  |
| Liquid Limit (L.L) %          | 65.55 |
| Plastic Limit (P.L) %         | 36.87 |
| Plasticity Index %            | 28.68 |
| Consistency Index, (C.I)      | 0.766 |
| Degree of Saturation (S_r), (%) | 100  |
| Classification (USCS)         | CH    |
| Specific Gravity (G_S)        | 2.54  |

2.2.2 Crushed stone. The size of the crushed stone particles was chosen in accordance with the size guidelines suggested by Nayak [11] and Al-Shaikhly [12], where the particle size is about 1/7 to 1/9 from the diameter of stone columns opening. The particle sizes of stone chips varying from 3mm to 5 mm. the natural properties of used crushed stone are shown in Table 2. The results show that the stone chips are classified as poorly graded.

### Table 2. Physical Properties of crushed Stones

| Property                      | Value |
|-------------------------------|-------|
| Dry Unit Weight ($\gamma_d$, kN/m$^3$) | 17.0  |
| Specific Gravity (G_S)        | 2.78  |
| $D_{10}$ (mm)                 | 1.1   |
| $D_{30}$ (mm)                 | 1.8   |
| $D_{60}$ (mm)                 | 3.78  |
| Uniformity coefficient (C_u)  | 3.43  |
| Coefficient of curvature (C_c) | 0.77  |
| Classification (USCS)         | GP    |

2.3 Pile model

In this investigation three PVC circular model piles with different diameters of 16, 26 and 32 mm were chosen to model the pile. The modulus of elasticity of the pile material was 1.2 MPa. The surface of the pile was covered with sand paper number 80. Pile alignment device was designed to hold the model pile in vertical position through the holes in upper plate. The pile model was covered at its top by a circular cap of steel to mount the dial gauge on it.

2.4 Installation of stone columns

The position of the stone columns is suggested at a distance equal to three times of the pile diameter. A hollow PVC pipe with external diameter of 32 mm coated with petroleum jelly is pushed down through the soft soil. The stone columns were studied in the end bearing case. Figure 2 shows installations of the stone columns surround the pile. To remove the soil inside the PVC pipe, a hand auger, manufactured for this purpose was used. After that, the PVC pipe was removed carefully while the crushed stone was gradually charged into the hole in five layers and each layer compacted by using 20 mm diameter rod of steel. Spacing between stone columns was chosen to be 64 mm.
3. Model Testing Procedure
The model tests were carried out on original soft clay and soft clay improved with ordinary stone columns. The tank, the top disk and lower perforated disk were cleaned and polished with petroleum jelly to reduce soil adhesion during test. The PVC pile model is provided with strain gauges. Strain gauges were fixed at different depths of the pile to measure the strains that occurred on the pile during consolidation of the soft clay soil. On the other hand, number and location of strain gauges were designed depending on the thickness of the soft clay layer. The pile was fixed vertically in the tank before the placement of soft clay soil. After that, soft clay soils were statically compacted on layers of about 50 mm thickness along the height of the tank. The sand layer was placed over the soft clay soil to allow upper drainage during consolidation. To distribute the pressure uniformly from load to the soil and compact layers of the soft clay soil the circular plate of steel was placed over the soil. One dial gauge with accuracy of 0.001mm was fixed in position to measure the settlement of the upper circular disk due to soil settlement, and another dial gauge was placed in the top of the PVC pile to measure the settlement of the pile during consolidation of the soft clay. Then, the wiring of the strain gauges was connected to the data loggers as shown in Figure 1.

Table 3 illustrates the experimental program. In the present study three groups were studied. In the first case the pile is ended in the soft clay layer. In the second case the pile is ended in the dense sand layer, whereas, in the third case the pile is embedded in soft clay improved by stone columns.

4. Analysis and discussions
The results obtained from experimental tests are discussed. The experimental results include 21 tests. Three cases of tests for three values of \( \frac{L_p}{D} \) ratio (10, 15, and 20) were studied. Each case includes three tests for pile diameters \( D = 16 \text{ mm}, 26 \text{ mm}, \) and \( 32 \text{ mm} \). The monitored data was analyzed to study the behaviour of piles in soft soil.

4.1 Effect of Stone columns on time- strain relationships of pile during soil consolidation
Figure 3 shows the variation of axial strain-time relationships measured by strain gauges for pile model \( (L_p)_{390SC} \). The pile diameter is 26 mm. The investigation included \( (L_p/D) \) ratio equal to 15. It can be seen from the Figure that top, upper middle, lower middle and bottom strains are continually increasing until reaching the peak point after 18 minutes from the beginning tests, below this point the axial strain is reduced with different trends until the end of the test. This may be attributed to stone columns playing two influential roles in the soft clay soil. Firstly, as a part of soil, they carry a part of the load and transfer it to the bottom sand layer. Secondly, stone columns act as vertical drains and thus speeding up the consolidation process during soft clay consolidation.
Table 3. Experimental Program

| Test No | Groups | Pile diameter (mm) | (L_p/D) | Pile length (mm) (L_p) | Clay thickness (mm) | Code | Strain gauge Number |
|---------|--------|-------------------|--------|-----------------------|--------------------|------|---------------------|
| 1       | Case I | 16                | 10     | 160                   | 350                | L_p160EC | 3                   |
| 2       | Case I | 15                | 15     | 240                   | 350                | L_p240EC | 3                   |
| 3       | Case I | 20                | 20     | 320                   | 400                | L_p320EC | 4                   |
| 4       | Case I | 10                | 10     | 260                   | 350                | L_p260EC | 3                   |
| 5       | Case I | 15                | 15     | 390                   | 400                | L_p390EC | 4                   |
| 6       | Case I | 20                | 20     | 520                   | 600                | L_p520EC | 4                   |
| 7       | Case I | 10                | 10     | 320                   | 350                | L_p320EC | 4                   |
| 8       | Case I | 15                | 15     | 480                   | 595                | L_p480EC | 4                   |
| 9       | Case I | 20                | 20     | 640                   | 650                | L_p640EC | 4                   |
| 10      | Case II| 16                | 10     | 160                   | 160                | L_p160ES | 3                   |
| 11      | Case II| 15                | 15     | 240                   | 240                | L_p240ES | 3                   |
| 12      | Case II| 20                | 20     | 320                   | 320                | L_p320ES | 4                   |
| 13      | Case II| 10                | 10     | 260                   | 260                | L_p260ES | 3                   |
| 14      | Case II| 15                | 15     | 390                   | 390                | L_p390ES | 4                   |
| 15      | Case II| 20                | 20     | 520                   | 520                | L_p520ES | 4                   |
| 16      | Case II| 10                | 10     | 320                   | 320                | L_p320ES | 4                   |
| 17      | Case II| 15                | 15     | 480                   | 480                | L_p480ES | 4                   |
| 18      | Case III| 16               | 10     | 160                   | 160                | L_p160SC | 3                   |
| 19      | Case III| 15               | 15     | 240                   | 240                | L_p240SC | 3                   |
| 20      | Case III| 10               | 10     | 260                   | 260                | L_p260SC | 3                   |
| 21      | Case III| 15               | 15     | 390                   | 390                | L_p390SC | 4                   |

Where:
L_p = embedded length of the pile (mm).
EC = Pile ended in soft clay soil, (Case I).
ES = Pile end bearing on sand, (Case II).
SC = Soft clay treated with stone columns, (Case III).
D = diameter of pile (mm)
(L_p/D) = embedded ratio

Figure 3. Time-axial strain curves of pile model, (L_p390SC) and (L_p/D) =15.
4.2 Effect of \((L_p/D)\) ratio on time-strain behaviour of piles during consolidation of soft clay

Figures 4 and 5 shows the axial strains with time of applying the surcharge load. Two embedded ratios 10 and 15 of tested piles are used. The pile diameters are 16 mm and 26 mm. Three cases; I, II and III of tested piles are shown in figures. From figures it can be seen that the monitored strains in the pile increases by increasing the value of \((L_p/D)\) ratio from 10 to 15. In case of pile ended in soft clay stabilized by stone columns Case III the axial monitored strains smaller than Case I and Case II. The subsistence of sand layer below the pile toe increases the axial strain of pile. In addition, stone columns decrease the axial strain of pile. This is adequate for different diameters and lengths of piles.

4.3 Evaluation of Shear Stress due to Consolidation of Soft Clay

The Shear stress \(\tau\) evolution between the pile and soft clay along the interaction length is derived from the normal stresses \(\sigma\) induced in the piles according to Equation. (1), in which \(i\) and \(i-1\) denote the subsequent strain gauges. The difference between the normal strains which are generated in the subsequent strain gauges is transmitted to the soil by shear stress \(\tau\).

\[
\tau = \frac{(P_i - P_{i-1})}{(\pi D \times \Delta X_i)}
\]  

where:

- \(P_i\) = Normal force at the location of strain \((i)\).
- \(i\) = strain gauge number.
- \(P = \sigma \times A\)
- \(\sigma_i = E \times \varepsilon_i\)
- \(E =\) Young’s modulus of pile model
- \(A =\) Cross section area of pile model
- \(D =\) diameter of pile model
- \(\Delta X_i =\) distance between the subsequent strain gages.
Table 4 presents the calculated values of shear stress for pile models (LP390EC, 390ES, 390CS). As can be seen from Table, the shear stress turns from -0.371 kPa, -0.436 kPa, -0.053 kPa at lower middle strains, to 0.852 kPa, 0.571 kPa, 0.246 kPa at bottom strains. This leads to conclude that the neutral plane is located between these two values. In addition, the magnitude of resistance shear stress at the bottom portions depends on the stiffness of the soil at the toe of the pile. Where, it decreases with the increase of soil stiffness at the toe of the piles. On the other hand, the bearing resistance in the case of soft soil stabilized with the stone columns (case III) is bigger than those in case I and case II. Hence, the depth of the neutral plane in case III will be reduced compared with these case I and case II.

Table 4. Axial strain, stress, Force and shear stress on pile model in three cases

| Case   | Depth (mm) | i  | Axial strain (micro) | Stress (kN/m²) | Force (kN) | Shear stress (kN/m²) |
|--------|------------|----|----------------------|----------------|------------|----------------------|
| CASE I |            |    |                      |                |            |                      |
| 0      | 0          | 0  | 0                    | 0              | 0          | 0                    |
| 30     | 1          | 16.232 | 19.4784              | 0.001784       | 0.001784  | -0.428               |
| 140    | 2          | 68.976 | 82.7712              | 0.007427       | 0.007427  | -0.673               |
| 250    | 3          | 87.152 | 104.5824             | 0.009385       | 0.009385  | -0.371               |
| 360    | 4          | 20.325 | 24.4224              | 0.002192       | 0.002192  | 0.852                |
| CASE II|            |    |                      |                |            |                      |
| 0      | 0          | 0  | 0                    | 0              | 0          | 0                    |
| 30     | 1          | 20.232 | 24.2748              | 0.002179       | 0.002179  | -0.533               |
| 140    | 2          | 90.976 | 109.1712             | 0.009769       | 0.009769  | -0.902               |
| 250    | 3          | 115.152 | 138.1824             | 0.010.124      | 0.010.124 | -0.436               |
| 360    | 4          | 70.352 | 84.4224              | 0.007576       | 0.007576  | 0.571                |
| CASE III|           |    |                      |                |            |                      |
| 0      | 0          | 0  | 0                    | 0              | 0          | 0                    |
| 30     | 1          | 16.251 | 19.5012              | 0.00175        | 0.00175   | -0.428               |
| 140    | 2          | 31.051 | 37.2612              | 0.004978       | 0.004978  | -0.188               |
| 250    | 3          | 35.231 | 42.2772              | 0.003794       | 0.003794  | -0.053               |
| 360    | 4          | 15.905 | 19.086               | 0.001713       | 0.001713  | 0.246                |

4.3 Effect of Stone columns on distribution of normal strain along the pile depth

Figure 6 shows that effect of stone columns on distribution of normal strain along the depth of pile for models (LP260EC, LP260ES, LP260SC). The shown strains were monitored at the end of the test. Strain increases until it reaches a peak value at an intermediate depth. Then, it decreases. Strain decrease reflects a decrease in the drag load. That is the developing of positive shear resistance along the pile shaft. Hence, the zone of the peak strain is a transition zone from negative skin friction to positive skin friction. Obviously, the neutral plan is located at this peak point. From the figure, it can be seen that the axial strain developed along the normalized depth of the pile models decreases by using the stone columns (case III). The location of peak point or neutral plane in case III is lesser compared with the same location in case I and case II. This could be attributed to the stone columns transmission part of the load to the bottom sand layer. Hence, it decreases the strain developed along the pile depth.
Figure 6. Axial strain distributions along model piles, (L_p260EC, L_p260ES, L_p260SC and have L_p/D =10).

Figure 7. Shear stress distributions along pile lengths (D= 26 mm, L_p= 260 mm and 390 mm).

4.4 Effect of the embedded mean ratio (L_p/D) on the location of neutral plane
Figure 9 and illustrates relationship of the neutral depth with the pile length to the pile diameter ratio. The pile diameter is 26 mm. Two cases of tested pile are shown in the figure. As can be seen from the Figure, the normalized depth of neutral plane increases with the increase of L_p/D ratio. The explanation
of the effect of the embedded mean ratio $L_p/D$ on the neutral plane is due to the compressibility of the pile length. When negative skin friction is induced in a short pile, most of the drag load is transmitted to the pile tip in the form of penetration to the bearing layer. Whereas, for long pile the drag load is somewhat taken by the pile compressibility, and somewhat transmitted to the toe pile.

4.5 Effect of bearing Stratum stiffness on the location of Neutral plane

Figure 8 depicts distribution of shear stress along the depth of the pile for models (Lp390EC, Lp390ES, Lp390SC), respectively. As can be seen from the figure, the location of the neutral plane is 0.75$L_p$, 0.79$L_p$ and 0.71$L_p$ for cases I, II and III respectively. For a floating pile (case I), the normalized depth of neutral plane is about 75% of the pile length. For bearing pile (case II), the normalized depth of neutral plane is about 79% of the pile length. Neutral plane is determined where shear stress is changing from negative to positive. That is at the intersection of the curve with the vertical axis. On the other side, using of stone columns decreases the neutral plane. It can be referred to that stone columns decrease settlement of soft soil and then decreases the depth of pivot point (neutral plane) compared with case I and case II. Neutral plane is determined where shear stress is changing from negative to positive.

That is at the intersection of the distribution of shear stress- normalized depth curves with the vertical axis. This means that the neutral plane is inappreciably affected by end bearing stratum stiffness, even though the neutral plane moves to the pile toe when the base layer is getting stiffer. This observation can be explained based on based on simple vertical force equilibrium where the sustained top plus the cumulative negative skin friction is equal to the cumulative positive side friction plus the toe end bearing resistance ($\Sigma$NSF = $\Sigma$POS + bearing resistance). Since small bearing resistance is available for case I and III, and then positive skin friction (PSF) should be large enough to resist negative skin friction (NSF). Hence NSF will be reduced with the neutral plane (NP) well above the pile toe. However, for a bearing pile (case II) where end bearing resistance is quite large, PSF is not required, thereby more negative skin friction is developed. Hence, the depth of (NP) in case II increased compared with both case I and case III.

5. Conclusions

This paper presents a study on the effect of stone columns on the behaviour of pile in soft clay. In order to carry out the investigations, experimental program was designated. A special model was designed and constructed for this purpose. Based on test results the findings summarized are given below:
1. The neutral plane is located closer to the end of the pile as the base layer under pile getting stiffer.
2. Stone columns decrease relative movement between pile and surrounding soil. This decreases both, negative skin friction and the drag load created along the pile shaft. Hence, there is reduction in both monitored strain and the normalized depth of the neutral plane, when compared with the same pile models embedded in clay without stone columns, group I and group II.
3. Stone columns accelerate the consolidation process significantly, as the stone columns behave like vertical drains.
4. The normalized depth of neutral plane increases with the increase of the pile length.
5. Stone columns play an effective role in increasing the bearing capacity for the soft clay soil.
6. The normalized neutral depth increases with the increase of embedded meant ratio (Lp/D).
7. Shear stress is initiated from zero value at the ground surface of the soil and increases until it reaches a peak negative value at an intermediate depth. Then, it decreases down to zero at the depth of the neutral plane where the positive skin friction develops.

References
[1] Fellenius, B.H., Negative skin friction and settlement of piles, in: Proceedings of 2nd International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 1984.
[2] Johannessen, I.J., and Bjerrum, L. Measurement of the compression of a steel pile to rock due to settlement of the surrounding clay, Proceedings of 6th International Soil Mechanics and Foundation Engineering, Montreal, Canada, vol. 2, pp. 261–264, 1965.
[3] Fellenius, B.H., Down drag on piles due to negative skin friction, Canadian Geotechnical Journal 9 (4), pp. 323–337, 1972.
[4] Fellenius, B.H., Results from long-term measurements in piles of drag load and down drag, Canadian Geotechnical Journal 43 (4), pp. 409–430, 2006.
[5] Blanchet, R., F. Tavenas, and Garneau, R., Behavior of friction piles in soft sensitive clays, Canadian Geotechnical Journal vol. 17, pp. 203–224, 1980.
[6] Bozozuk, M., Bearing capacity of pile preloaded by down drag, in: Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, vol. 2, Stockholm, Sweden, pp. 631–636, 1981.
[7] Indraratna, B., A.S., Balasubramaniam, Phamvan, P., and Wong, Y.K., Development of negative skin friction on driven piles in soft Bangkok clay, Canadian Geotechnical Journal 29 (3), pp. 393–404, 1992.
[8] Datye, K.R. and Nagaraju, S.S., “Design A pproach and Field Control for Stone Columns” Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, pp. 673–640, 1981.
[9] Abd EL Naiem, M. A., Towfeek, A. R., and Hegazy, O.M.M, “Investigation of the distribution of skin friction on single pile constructed in Natural Soft Clay Soil Treated With stone columns”, International Journal of Engineering Development and Research (IJEDR), ISSN:2321-9939, Vol.3, Issue 3, pp.1-10, 2015.
[10] "Egyptian Code of Soil Mechanics and Foundations Engineering ", Research Center for Housing, Building and Planning, Giza, EGYPT, 2012.
[11] Nayak, N.V., “Recent advances in ground improvements by stone column”, Proceedings of Indian Geotechnical Conference, Madras, Vol. 1, p. V-19,1983.Al-Shaikhly, A.A. (2000), "Effect of Stone Grain Size on the Behavior of Stone Column", M.Sc. Thesis, Building and Construction Engineering Department, University of Technology, Iraq.