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IMPROVED SOIL-PILE INTERACTION OF FLOATING PILE IN SAND WITH SHAFT TREATMENT

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ABSTRACT: The present study explored the efficacy of pile shaft surface treatment on the improvement of soil-pile frictional resistance for floating or friction piles. 4 surface conditions of model piles measuring 200 mm long and 20 mm in diameter were examined, namely smooth (control), roughened, fishbone and checked shafts. A pile cap made of plywood was fixed to the top of the pile with 10 mm embedment depth, leaving 190 mm clearance for installation in the sand bed. The test chamber was a see-through glass tank with a footprint of 100 mm x 200 mm and 300 mm height. Coarse sand of D_{50} = 1.5 mm were loosely placed in the chamber by layers up to 200 mm height before the piles were installed either in single or triple group formations. The incremental load test of conducted via application of dead load ranging between 0.01-0.08 kPa on the pile cap, and the corresponding settlement was recorded. The test results revealed settlement to be reduced by the piles in the order of roughened > fishbone > checked > smooth for the single pile configuration, with maximum reduction of 40 % recorded by the roughened pile. As for the pile group, settlement reduction of the piles with surface treatment clearly outperformed the control pile by almost 50 %, though differences between the former were marginal with seemingly overlapping stress-strain plots. All in all the surface treatment of pile shaft enhanced the shaft friction for the piles installed in sand, but field implementation would require further examination of the pile-driving efficiency as the improved piles could cause additional resistance during installation.

Keywords: Friction / Floating pile, Settlement reduction, Contact surface, Frictional resistance, Sand

1. INTRODUCTION

Piles fall under the category of deep foundation, which function to receive and transfer load imposed by superstructures through weak or compressible layers of soil to a firm strata [1]. Nonetheless, it is well established that a pile carries an applied load effectively either by end-bearing in a hard layer or by frictional shaft resistance, hence 'floating' in an adequately strong soil layer. Therefore in cases of friction piles, as long as the pile is able to counter the applied load with an upwards resistance derived from the surrounding enveloping soil around the pile shaft, it is not necessary to adopt long piles to reach the hard layer. This could result in significant savings in cost and time, where the number of piles and splicing work could be reduced. Comparable to reports by Sun et al. [2] in their work on large diameter pipe piles installed for offshore structures, pile-splicing incurs extra construction time while disturbing the pile driving process, resulting in intrinsic changes of the soil fabric with rapid dissipation of pore pressure [3], leading to a surge in shear strength of the soil resisting the driving force and signaling premature penetration refusal [4].

The analysis of soil-pile interaction has been rather extensively studied by various researchers. For pile groups in clay, the summation of individual pile capacity of the group may not equal the group load-bearing resistance; while in sand, the capacity of individual pile is taken as an isolated pile capacity via a pile-to-pile interaction factor [5]. Perhaps drawn by the appeal of virtual simulations, soil-pile interaction has also been popularly investigated using numerical modelling. The entire soil-pile system can be simulated and analysed with Finite Element or Boundary Element Methods, as can be found in works by Padron et al. [6], Wang et al. [7] and Sheil, et al. [8], among others. These analytical works mainly revolve around the assumed premise of the soil-pile system as one composite continuum, taking into consideration complex influencing factors, such as pile-soil-pile interaction, 3D pile group configurations and soil anisotropy [9].

Piles in earthquake region have also received much attention, especially for port facilities where socio-economic impact of catastrophes could be tremendous for the long term [10]. Displacement-based design approach is probably the most widely adopted (e.g. 11 & 12), with non-linear static analysis as the basis to make predictions of the displacement demand for design earthquake motions. Other dynamic analysis were conducted for fully embedded single pile or pile groups in single-phase elastic soil (e.g. 13 & 14), though partially embedded piles were more common and realistic in practice with significant differences for the fully embedded counterparts (e.g. 15 & 16).

Other recent interesting reports on soil-pile interaction were found in work by Inazumi et al. [17], who studied the removal effect of relic piles for land reutilization and redevelopment projects, particularly in the vicinity of the pull-out hole when earthquake strikes. On a separate note, Halling et al. [18] conducted impact load tests on pile groups in soft clay and monitored the resulting soil-pile system's response at low strain levels. Load field tests were also carried out by Dezi et al. [19] to determine the pile-soil-pile dynamic interaction of a 3-pile system vibro-driven into nearshore soft clay deposit. In the examination of interaction between floating screw piles and the surrounding soil, Akopyan & Akopyan [20] found the helical shaft of the pile to be completely covered with a cylindrical cut-out soil mass, where bearing capacity afforded by the pile was in excess of 1.3-1.7 times that of the original ground.

The above literature clearly outlined the importance of identifying mechanisms of soil-pile interaction for effective load-bearing and settlement control of piles, especially for the floating or friction ones. It follows that improved interface frictional resistance between the pile shaft and surrounding soil is crucial in enhanced pile performance as mentioned earlier. The paper describes the settlement reduction of 3 model floating piles with various surface treatment, i.e. roughened, fishbone and checked, benchmarked against the smooth shaft control pile in load tests conducted in a lab-scale maintained load test setup.

2. MATERIALS AND METHODS

2.1 Model piles

A total of 4 model pile designs were examined in the present study, which included a control pile with a smooth shaft. The model piles were made of PVC pipes of 20 mm diameter with a 30 mm long pointed tip attached. The top 10 mm of the pile was inserted into a premade slot in the 100 mm x 100 mm plywood which served as the pile cap.

As shown in Fig. 1, the piles with surface treatment had their shaft scarified to various patterns and named roughened, fishbone and checked respectively. The roughened surface was formed by approximately 200 small punctures, but care was taken to avoid making through-and-through holes on the shaft. The fishbone shaft had sets of shallow inverted V-shape lines cut into the shaft at 60° inclination to the vertical (total of 64 inclined lines). A total of 40 1.5 cm x 1.5 cm square incisions were made on the pile shaft to form the checked shaft. Note that all the patterned surfaces of the pile shafts were created using a heated soldering hand tool on pre-drawn lines or points on the shaft. The piles were also arranged in a triangular formation as a pile group for the load tests (Fig. 2), where the pile-to-pile distance was kept at 60 mm.

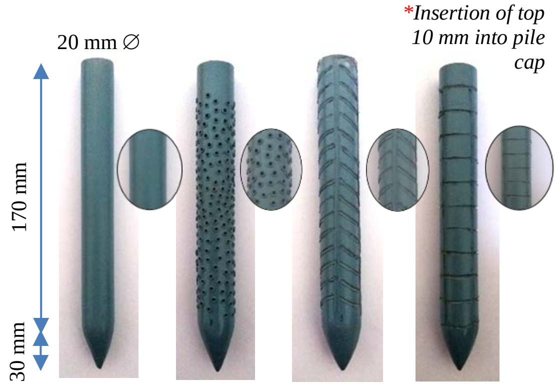

Fig. 1 Model piles for the study.

2.2 Test chamber

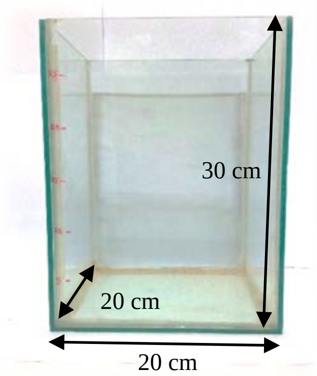

Fig. 2 Load test chamber (final sand bed measured 200 mm thick).

Fig. 3 Laying the sand bed layer by layer via the 'sand raining' method.

The test chamber measured 200 mm x 200 mm x 300 mm height (Fig. 2) and dead weights were used to apply vertical stress on the piles. Note that the piles were kept at distance of at least twice the pile's diameter from the boundary of the chamber. Coarse sand (D_{50} = 1.5 mm) were spread in layers in the chamber via the 'sand raining' method from a funnel (Fig. 3). The funnel was kept 15 am above the soil surface and slowly moved wall-to-wall repeatedly to lay the sand uniformly. Adoption of the seemingly tedious 'sand raining' technique was necessary to prevent non-uniform distribution of the sand particles when transferred to the chamber, i.e. dense pockets in the loose sand layer due to over-dosage. The loose sand layers would eventually form a 200 mm thick sand bed for installation of the piles.

2.3 Load test

Referring to Fig. 4, the load test was conducted by applying dead weights on the pile cap. The pile or pile group was first pushed into the sand bed up to 100 mm depth to secure the shafts prior to test. The dead weights were placed incrementally from 0.01 to 0.08 kPa in sequence of 0.01 kPa per increment. The vertical stress was derived simply by dividing the load applied with the surface area of the pile cap to maintain the same stress application for both single and pile group tests. Nonetheless it should be noted that the actual downward imposed load on the shaft surface can be easily calculated by taking into account the pile-soil contact area.

Settlement induced by each load application was recorded when no further change was observed. Then only was the next load increment applied. Care was taken to ensure vertical alignment of the piles to avoid inclined and non-uniform load distribution. Clearance of 70 mm between centre of the pile and inner wall of the test chamber was considered sufficient to avoid boundary effects on the load-settlement mechanism. In addition, the dry sand bed provided an undrained environment for the loading condition in the short term.

Fig. 4(a) Single pile load test.

Fig. 4(b) Pile group load test.

3. RESULT ANALYSIS AND DISCUSSIONS

3.1 Load-Settlement relationship

The load-settlement plots for both single (S) and group (G) formation of the piles are compiled in Fig. 5. Looking at the plots for the single piles, it is apparent that all the piles with treated surface outperformed the control pile to various degrees, in the ascending order of settlement reduction, Checked (S) < Fishbone (S) < Roughened (S). The Roughened and Fishbone piles effectively reduced the final settlement by 35-45 %, while the Checked pile produced a more modest 10 % reduction. This is suggestive of more pronounced frictional resistance afforded by the Roughened and Fishbone patterns, where the coarse surface of the former could have had a larger contact interface with the surrounding soil during load application compared to the Fishbone design. The equally spaced slanting ridges of the Fishbone pile, on the other hand, would have had additional contact surface with the soil within the ridges. As for the Checked pile, the equal vertical and horizontal ridges presumably in some way cancel out the enhanced frictional contact with the soil. This could be explained by the vertical ridges allowing sand particles to slide up the pile shaft while the horizontal ridges acted as a barricade against the downward movement of the pile. Combined the force vector could be one that was marginally higher than the applied load.

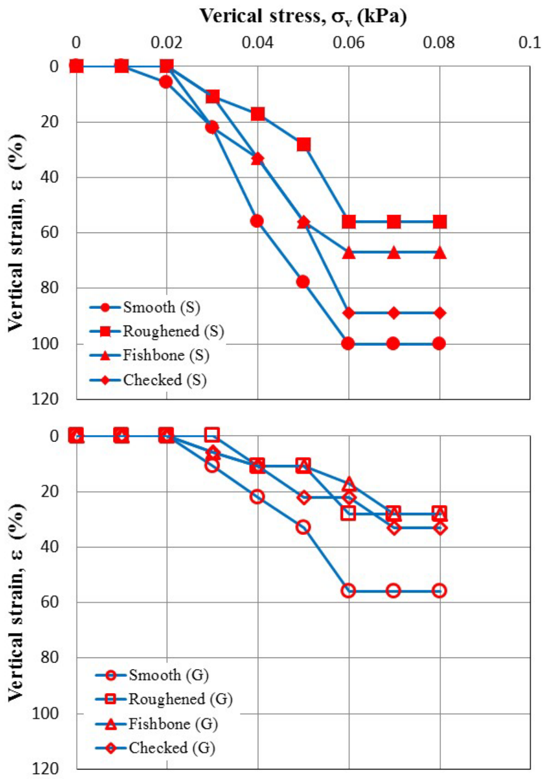

Fig. 5 Settlement plots for single piles and pile groups.

In the same figure for the pile groups, settlement reduction was surprisingly similar for all pile types, i.e. settlement was reduced by about 50 % compared to the Smooth pile group. Nonetheless the 3-pile group formation did provide better load-bearing capacity, even with the control piles, as can be observed from the approximately 40 % improvement in reduced settlement from the single pile. The seemingly independence of the settlement from the shaft treated surface type is indicative of the piles acting as a block than individuals in bearing the applied load. The piles would react like a raft foundation, where the effective frictional soil-pile interface would equal an area derived from multiplication of the perimeter of the pile group with the embedment depth. In other words, the intermediate soil-pile interaction was negligible and considered non-existent for load-bearing purposes, resulting in uniform vertical displacement of the piles as a single unit.

3.2 Settlement and stress range

Referring to Fig. 5, the stress-strain plots depict 3 distinct phases per the stress range. Divided into categories of low (0-0.03 kPa), middle (0.02-0.06 kPa) and high (0.06-0.08 kPa) range, the corresponding average vertical strain (ε_{AVG}) for each stress range was extracted and plotted in Fig. 6. Only the single control pile showed notable settlement in the low stress range, highlighting the limited frictional resistance mobilized between the smooth shaft surface and the loose sand particles. This was apparently overcome when the smooth piles were installed in a group. Nevertheless the control piles showed the most significant vertical displacement in all cases, be it in the single or group formation.

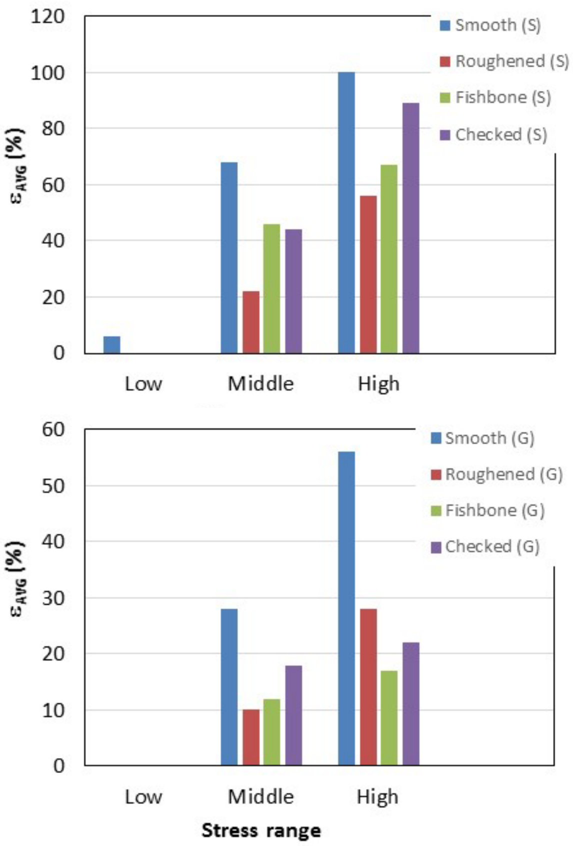

Fig. 6 Strain limits for different stress range.

In the middle stress range, the Roughened pile underwent the least settlement in both single and group formation the magnitude of approximately 20 and 10 % respectively. In the same stress range, the single Fishbone and Checked piles recorded similar settlement but Fishbone fared better in the group performance. This could be due to the greater vertical strain necessary to be endured before the maximum shear resistance was mobilized for the Fishbone single pile compared to its group counterpart. Note too apart from the Fishbone piles, all other pile groups displayed approximately half the settlement of the single pile, i.e. the Fishbone pile group gained better traction with the surrounding soil in the middle stress range.

For single piles in the high stress range, Roughened outperformed Fishbone and Checked by about 18 % and 64 % respectively in terms of settlement reduction, but the pile groups showed Fishbone more effective than Roughened and Checked by 56 % and 22 % respectively. Interestingly both Fishbone and Checked had about 73 % settlement reduction between the single and group formation, whereas Roughened only recorded 49 % reduction. It is worth noting that Roughened plotted a plateau in the stress-stain curve in the high stress range (Fig. 5), pointing to attainment of the maximum shear or frictional resistance at the onset of the high stress range. This could account for the lower settlement reduction percentage mentioned earlier.

3.3 Correlation between stress and strain

Fig. 7 relates the strain and stress as recorded during the pile load tests. Clearly greater settlements were accompanied by higher stresses, as the piles approached the maximum shear resistance and failure with increased loading. Also, settlement of the pile groups can be observed to be about 75 % more significant than the single piles. However this only holds true for the mid-range stresses and above, where displacement of the single piles were no more than 20 %. This suggests ability of the single piles to sustain smaller loads without much deformation, i.e. adequate frictional contact between soil and pile.

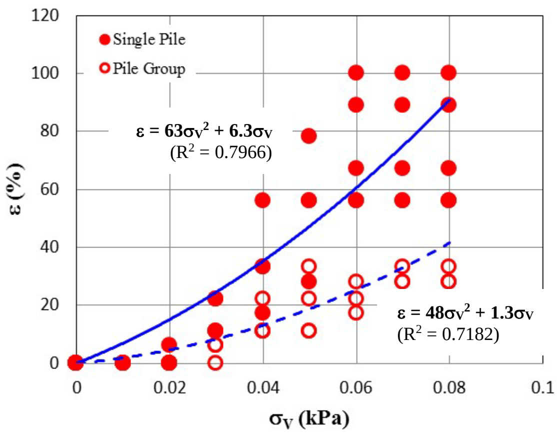

Fig. 7 Strain (ε) - stress (σ_V) relationship for all samples.

In addition, the rather steep rise of settlement with stress increment beyond the middle range is associated with the abrupt turns in the stress-strain plots in Fig. 5 which signify the approach of maximum shear forces and yield of the foundation. The markedly reduced settlements also corresponded with much higher stiffness of the pile group, contributing towards improved compressibility control for the foundation system. For instance, approximation of the gradient of the stress-strain plots in Fig. 5 would give a general idea of the stiffness comparison between the single and group piles.

The stiffness could be differentiated by taking into account the pre- and approaching yield stages mentioned earlier. For the pre-yield stages, the stiffness was derived to be 0.12 and 0.25 kPa per unit vertical strain for the single and group piles respectively. As with the approach of yield, stiffness increased to 0.075 for the single pile and 0.12 kPa per unit vertical strain for the pile group. The stiffness comparison is remarkable, where the pile group provided twice the resistance per unit strain pre-yield and 1.5 times thereafter. Besides, compared to the control piles, i.e. 0.04 (single pile) and 0.07 (pile group) kPa per unit vertical strain approaching failure, the stiffness improvement is even more encouraging at about 1.75 times for both the single and group piles. This corresponds well with earlier discourse on the efficacy of the pile shaft surface treatment on overall settlement reduction.

3.4 Estimation of frictional resistance

Table 1. Derivation of β-values.

Method	β-value
NAVFAC DM7.2 (1984) [21]	0.41
McClelland (1974) [22]	0.25
Meyerhof (1976) [23]	0.32
Kraft & Lyons (1990) [24]	0.40

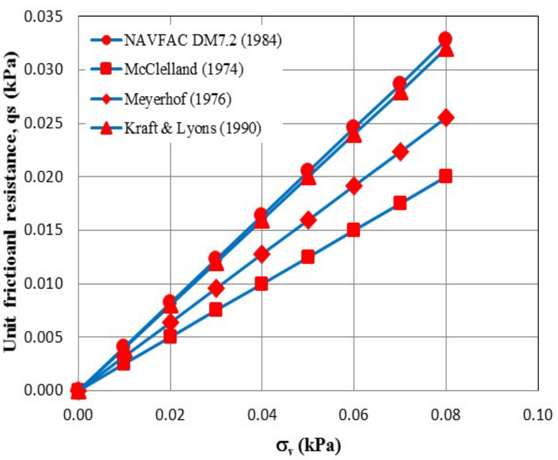

Fig. 8 Unit frictional resistance (q_s) vs. vertical stress (σ_v) plots.

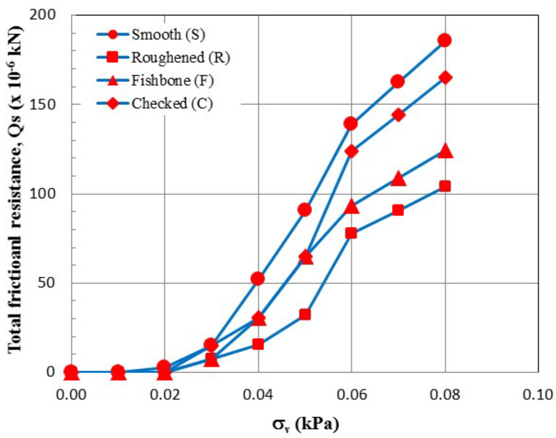

Fig. 9 Total frictional resistance (Q_s) derived with the maximum β-value vs. vertical stress (σ_v) plots.

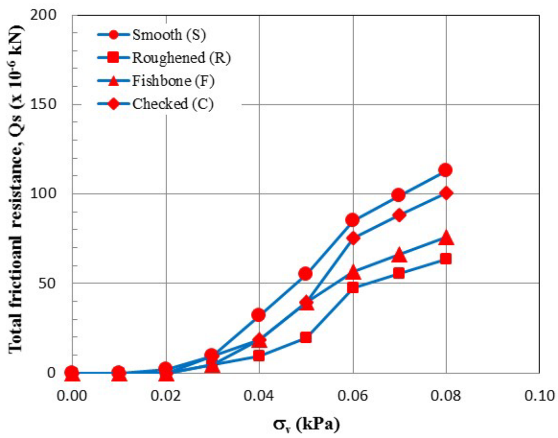

Fig. 10 Total frictional resistance (Q_s) derived with the minimum β-value vs. vertical stress (σ_v) plots.

Based on long term drained analysis as per the β-method, multiplication of the friction factor (β) and the effective lateral stress (σ_h') would give the unit frictional or skin resistance (q_s).

$$q_s = \mu\sigma_h' = \beta\sigma_v' \quad \text{Eq. 1}$$

The factor β can be derived from a number of established empirical approaches, as summarized in Table 1. Note that the β-values were calculated based on certain assumptions for the loose sand bed and piles: c' = 0, ϕ' = 30º (as common for loose sand), c_u = 25 kPa, the model piles were driven and that they were compression piles.

With the β-values in Table 1, the resulting unit frictional resistance (q_s) is illustrated in Fig. 8 as a plot against σ_v ($\sigma_v = \sigma_v$' as the loose sand is a quick-draining medium, i.e. effective stress analysis is valid). It is apparent that the shaft resistance was no more than 41 % of the vertical stress applied in all cases. By multiplying q_s with the soil-pile contact surface computed from the settlement data, the total frictional resistance (Q_s) can be obtained. As shown in Fig. 9 & 10, Q_s for the single piles was derived using the maximum and minimum β-values, i.e. the NAVFAC DM7.2 (1984) and McClelland (1976) methods respectively.

Governed by the settlement which determined the contact surface area between soil and pile, the plots understandably follow the same pattern as the settlement plots. Q_s increased with higher σ_v, but significant increment in Q_s was noted from the 0.03 kPa onwards, corresponding with the mid-range stress application. It is also apparent that the maximum frictional resistance was not attained when the load test was terminated, as indicated by the rising plots.

Nevertheless, it is important to note that the friction angle (ϕ') has a tendency to reduce with depth, and that the skin friction equation loses accuracy at high stress levels due to rearrangement of the sand particles. Besides overall local skin friction is reportedly reduced with increased pile depth. Examination on the critical depth where maximum skin friction is attained can be conducted in future work.

4. CONCLUSIONS

Model pile load tests were conducted on friction piles embedded in loose sand deposits to ascertain the extent of performance enhancement in terms of settlement control. The piles with shaft surface treatment were found to have significant positive influence in settlement reduction, i.e. 35-45 % less settlement compared to the Control in single pile cases, and 40 % settlement reduction in the pile groups. On the other hand, the different patterned shafts did not seem to make much difference in the pile groups' settlement reduction, as demonstrated in the load-settlement plots. Further work is recommended for pile driving resistance with the treated shafts as part of the feasibility studies for field implementation.

5. ACKNOWLEDGEMENT

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