# Static Behavior of Strain Gauge Instrumented Continuous Flight Auger (CFA) Piles

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## ABSTRACT

The practice of using Continuous Flight Auger (CFA) piles in the Middle East construction industry has increased considerably in the last few decades. The rapidity and economy of environment-friendly CFA pile construction can offer added benefits to infrastructure projects in the shallow ground conditions. In this article, the static behavior of strain gauge instrumented CFA pile in compression has been studied in the Jebel Ali Container Terminal, Dubai to understand the piles performance and verification of the design. The test was carried out for four 1000mm diameter instrumented piles (namely, PTP1, PTP2, PTP3 and PTP4) with a load ranging from 6010kN to 8820kN in dense sand, sandstone and calcareous siltstone formations. Vibrating wire waterproof robust sister bar strain gauges of 914 mm length, consisting of four units at each level, were attached to the rebar of the test piles. The load transfer, skin friction and settlement analysis were carried out to understand the soil-pile interaction performance of the piles. Reassessment of the original pile design using strain gauge instrumented static load tests demonstrated that the pile lengths can be further optimized to provide a cost-effective unequivocal solution for the shallow foundation construction industry in the region.

**KEYWORDS:** CFA piles, Skin friction, Strain gauges, Settlement, Static behavior.

## INTRODUCTION

Foundation using piles helps support structures through the transfer of loads to deeper soil and rock layers. For more than 4000 years, piles have been used in foundation by the Ancient Greeks and Romans (Fleming et al., 1992). Pile foundations are executed to transfer loads from the structure to the deeper layers when embedded in soil or weak rock stratum (Sekhri et al., 2020). Contemporary piling techniques are assorted and the term 'pile' is used to define an extensive variety of columnar load-transferring components in a foundation (Tomlinson, 1994). They are considered a suitable design choice for sites with weak and shallow soil/rock layers to support heavy structures. From a piling perspective, the optimal piling technique for shallow foundation adoption would be Continuous Flight Auger (CFA), as this technique offers the most cost-effective piling methodology. Continuous-flight-auger (CFA) piles have been used in the Gulf construction sector for the last few decades. However, their use in infrastructure projects has been limited to secondary shallow structures that exert very limited bearing loads. At present, CFA piles have been used extensively in the UAE for infrastructure and low-rise building projects to support large bearing loads and are more economical to install than competing foundation systems, especially in soils and soft rocks. Hence, this study will give awareness of the performance of axially loaded CFA piles to understand the soil-pile interaction to benefit future designers in the foundation industry.

CFA piles differ from other bored piles in their mode of construction and have been described by various authors (Derbyshire, 1984; Brown, 2005). They appear to be fairly uniform with a typical diameter up to 1000mm and a 30m length. The static behavior of bored and CFA piles has been explored by several authors (Albuquerque et al., 2005; Farrell and Lawler, 2008; Gavin et al., 2009; Gavin et al., 2013; Ismael, 2001). The

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adequate behavior of CFA pile mostly depends on the proper installation, the geometry of the pile and the properties of the prevailing soil stratum (Massarsch et al.,1988; Van and Peiffer, 1997; Van, 1988; Viggiani, 1993). Various studies show that the ultimate static capacity of CFA piles was high as compared to that of bored piles (Farrell and Lawler, 2008; Albuquerque et al., 2005). Hence, a precise understanding of the static behavior of CFA piles and their design parameters is required to execute a cost-effective project. CFA piles are constructed by rotating a continuous-flight, hollow-shaft auger into the ground to a predetermined tip elevation. Uninterrupted injection of grout through the auger shaft is carried out as the auger is being withdrawn and then, a reinforced steel cage is inserted into the grout after the auger is completely removed from the borehole. Some of the unique advantages of CFA piles are: being an economic alternative compared to other pile types, mainly because material costs are relatively low and installation is fast, suitability for projects where a large number of piles are required, vibrations and noise levels are low, they can be constructed in limited access conditions where conventional driving equipment cannot be operated and premanufactured piles cannot fit geometrically (Wang, 2008). But, CFA piles are not suitable in variable, hard and unstable geotechnical conditions. The design, execution and interpretation of the instrumented pile load tests are critical and important to any foundation design process.

Dubai Ports, including Jebel Ali Port and Port Rashid, are the busiest ports in the Middle East. Dubai Ports experienced rapid development in their history and has already become an integral part of UAE cargo transport. Jebel Ali Port is situated 35 kilometers southwest of Dubai. Today, Jebel Ali Port is the world's biggest artificial harbor and a leading port in the Middle East as well as being the largest container port between Rotterdam and Singapore. Due to the rapid cargo volume growth in Dubai Ports and the closure of Port Rashid for renovation, Jebel Ali Port will not cater the needs to handle containers and other cargos in the future. Thus, this container terminal expansion (Latitude: 25.0222; Longitude: 55.0576) is essential to cater the handling capacity requirements in Jebel Ali Port (Fig.1). The main objective of this article is to understand the significance and response of strain gauge instrumented prototype CFA piles to static load and thereby to a reappraisal of the initial design to aid future shallow foundation projects.


**Figure (1): Location map of the study area (Wang, 2008)**

## GEOLOGY OF THE AREA

The geological setting of the United Arab Emirates and the Arabian Gulf area has been substantially influenced by the deposition of marine sediments due to sea-level changes in the recent geological time (Poulos, 2018). The near-surface geology is dominated by Quaternary to Late Pleistocene age, mobile aeolian dune sands and sabkha/evaporites deposits. The geologically stable Arabian Plate is separated from the unstable Iranian Fold Belt by the Arabian Gulf (Sujatha and Kontopoulos, 2018). It is believed that a tilting of the entire Arabian Plate occurred during the early Permian period, resulting in an uplift in southern Yemen and depression to the northeast. Crustal deformations and igneous intrusions occurred in the northeast as a result of this movement. Subsequent tectonic movements, peripheral to the folding of the Iranian Zagros Range, during the Plio-Pleistocene epoch, resulted in the formation of the Arabian Gulf depression and the mountainous regions in the United Arab Emirates and Oman.

Dubai's near-surface geology is dominated by Aeolian dune sand deposits of the Holocene to Pleistocene age. From the soil investigation data, it is identified that these deposits typically comprise fine-grained, dense silty variable cemented calcareous sand. The degree of cementation generally increases with depth and grades to primarily calcareous sandstone. Occasionally, very silty, gypsiferous sabkha and evaporate layers are identified within the Aeolian sand deposits (Material Lab., 2012). Even though surficial sabkha deposits are found throughout the coastal belt of the Arabian Gulf, they are not common in the Dubai region and these superficial deposits were underlain by siliceous calcarenite, calcareous sandstone, siltstone and conglomerates (Cherian, 2018; 2022). The subsurface strata observed from boreholes with depth (DMD - Dubai Municipality Datum) are comprised of dense to medium-dense calcareous sand with occasional organic content, few corals and rare cemented sand fragments. The rock encountered consists of slightly weathered, medium bedded and thinly laminated, light brownish grey, fine to coarse-grained, very weak sandstone. This layer is followed by calcareous siltstone and conglomerate with sandstone was encountered till the termination of boreholes. The general UCS (Uniaxial Compressive Strength) plot (Fig. 2) and the SPT N value obtained from the geotechnical investigation report (Material Lab., 2012) were used for calculating the unit shaft friction values. The unit shaft friction is calculated from UCS and SPT N value using the equation given by Horvath and Kenney (1979) and Clayton (1995) (Table 1).

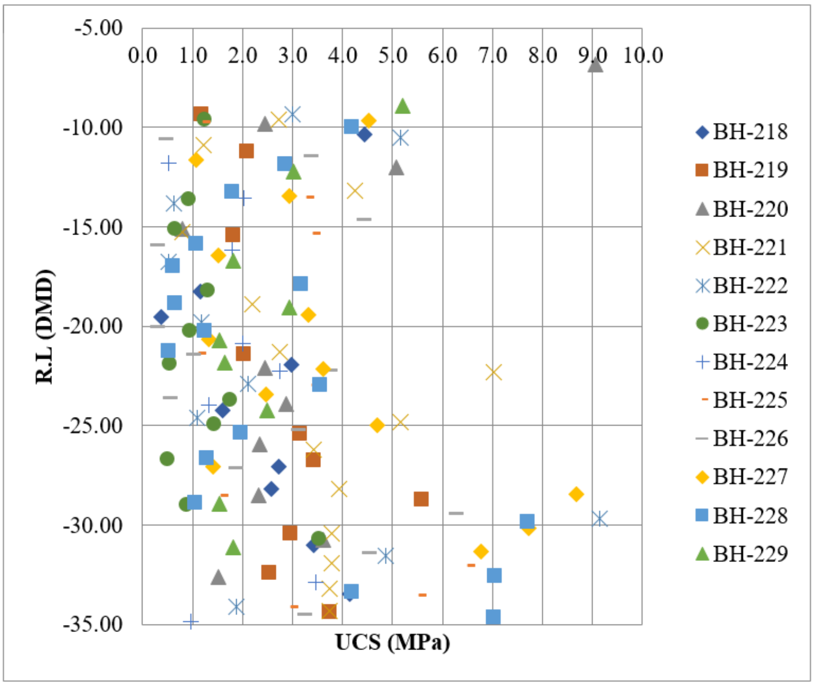


**Figure (2): UCS values with depth (Material Lab., 2012)**

**Table 1. The general soil and rock profile (Material Lab., 2012)**

| Strata | Depth (m DMD) | UCS (MPa) | SPT N | Ultimate Skin Friction (kPa) |
|---|---|---|---|---|
| Dense to medium-dense calcareous sand | +3.50 to -9.30 | - | 12 | 36 |
| Sandstone | -9.30 to -11.20 | 1.20 | - | 274 |
| Calcareous siltstone | -11.20 to -15.40 | 2.10 | - | 362 |
| Conglomerate/sandstone | -15.40 to -35.00 | 3.00 | - | 433 |

## MATERIALS AND METHODS

The area of works was segregated from other works to ensure that other personnel is at a safe distance from our piling operators. The auger string was centralized over the pile position and vertically set. The hollow stem of the auger at the base of the digging head was closed off with an expandable cap or hinged flap. The pile was bored down to the required depth, observing both the rate of penetration of the auger string and the hydraulic demand of the drilling motor. It was ensured that the auger string remains in-line and the mast remains plumb during boring.

Upon achieving the required depth, the auger string was withdrawn and the concrete pumped to allow the expandable cap to be blown clear of the digging head. Concrete was then pumped through the hollow stem to the base of the auger and the pile constructed as the auger was withdrawn at a controlled rate. Where necessary, the auger was cleaned with the aid of a mechanical auger cleaner. The pile cage was lifted and lowered vertically into the fluid concrete. The lifting slings were released from the top of the cage when the cage was embedded in the concrete. The steel cage was lowered centrally to the fresh concrete by its own weight down to the remaining few meters above the toe, then by putting an additional weight to the cage and using a vibrator attached to a crane. Spoil remnants were cleared away from the upper surface of the concreted pile bore and were then barricaded for protection. The piles were instrumented with vibrating wire sister bar strain gauges at different levels (PTP1, PTP2 and PTP3) and four different levels (PTP4) with four strain gauges in each level (Table 2). One 1000-ton compression load cell was installed to measure the load applied to each test pile. Four numbers of 100-mm range displacement transducers were installed between the reference beam and the test pile cap at 90 degrees apart for the pile head settlement measurement. The load and unit skin friction distribution along the pile length were calculated from the strain readings.

**Table 2. Test pile details**

| Pile no. | Pile diameter (mm) | Cut-off level (m DMD) | Toe level (m DMD) | Pile length (m DMD) | Working load (kN) | Test load (kN) | Strain gauge levels (mDMD) |
|---|---|---|---|---|---|---|---|
| PTP1 | 1000 | +3.20 | -13.00 | 16.20 | 2404 | 6010 | -1.00, -5.00 -9.00, -10.50 and -12.70 |
| PTP2 | 1000 | +3.20 | -13.00 | 16.20 | 2850 | 7125 | -1.00, -5.00 -9.00, -10.50 and -12.70 |
| PTP3 | 1000 | +2.73 | -15.30 | 18.03 | 3207 | 8018 | 0.00, -5.00 -10.80, -13.00 and -15.00 |
| PTP4 | 1000 | +2.73 | -13.70 | 16.43 | 3528 | 8820 | 0.00, -5.00 -10.80 and -13.00 |

The pile test has been performed following the maintained load procedure as per ICE (2007). The test consists of three cycles from zero up to 100%, 200% and a maximum test load of 250%, followed by unloading in each cycle. The load was applied in terms of pressure using a hydraulic jack and measured by a compression load cell. A data acquisition system with a laptop was used to monitor and process the strain gauge, load cell and displacement transducer raw data. The installation and test setup are provided in Pictures 1 and 2.

**Picture (1): CFA pile installation**


**Picture (2): CFA pile test setup**

**Ultimate Compressive Resistance**

The introduction of Eurocode in the foundation testing allows for the application of modern unified technological tools for the design of structures and different combinations of loads with the application of all types of construction materials. Eurocode 7 (2004) describes the procedures for obtaining the characteristic compressive resistance of a pile:

a) Directly from the static load test;

b) By calculation from soil profiles.

Eurocode 7 provides correlation factors to convert the measured pile resistances or pile resistances calculated from profiles of test results into characteristic resistances. The design pile resistances derivation requires applying the resistance partial factors to the characteristic values (Cherian, 2021). The combinations of partial factor values that should be used for Design Approach I are as follows:

DA1.CI: A1 + M1 +R1

DA1.C2: A2 + M1 or M2 + R4

where:

A1/A2 denotes partial factors on action.

M1/M2 denotes partial factors for soil parameters.

R1/R2/R3/R4 denotes partial factors for resistance.

The partial factor combination used for Design Approach 2 are as follows:

DA2: A1 + M1 + R2

whereas Design Approach 3 (the shear strength parameters are divided by partial factors and the effect of load is multiplied by load factor) is computed as:

DA3: A1 + M2 + R3

## RESULTS AND DISCUSSION

The mobilized unit skin friction calculated from strain gauge readings and average settlement values obtained during the tests are presented in Tables 3 to 6. The load transfer to the soil between two successive strain gauge levels was calculated as the difference in loads at these levels. From that, the unit skin friction was calculated by diving the surface area. For each pile test, the maximum applied load was 250% (6010 kN for PTP1) of the working load (100% of 2404 kN for PTP1). No failure was reached at the maximum applied load and pile head movement ranging from 4.72 mm (for pile PTP1) to 8.20 mm (for pile PTP4). Table 7 summarizes the calculated unit skin friction values at maximum applied load for the tested piles in this study together with the corresponding pile head displacements.

**Table 3. Unit skin friction and pile head displacement values (PTP1)**

| Skin Friction (kPa) | | | | |
|---|---|---|---|---|
| Strain gauge level (DMD) | 1202 kN | 2404 kN | 4808 kN | 6010 kN |
| | 50% | 100% | 200% | 250% |
| -1.00 to -5.00 (1 to 2) | 9 | 12 | 30 | 32 |
| -5.00 to -9.00 (2 to 3) | 17 | 27 | 42 | 44 |
| -9.00 to -10.50 (3 to 4) | 43 | 94 | 167 | 204 |
| -10.50 to -12.70 (4 to 5) | 111 | 247 | 492 | 704 |
| Pile head displacement (mm) | 1.62 | 2.33 | 3.23 | 4.72 |

**Table 4. Unit skin friction and pile head displacement values (PTP2)**

| Skin Friction (kPa) | | | | |
|---|---|---|---|---|
| Strain gauge level (DMD) | 1425 kN | 2850 kN | 5700 kN | 7125 kN |
| | 50% | 100% | 200% | 250% |
| -1.00 to -5.00 (1 to 2) | 11 | 22 | 46 | 50 |
| -5.00 to -9.00 (2 to 3) | 25 | 38 | 62 | 79 |
| -9.00 to -10.50 (3 to 4) | 49 | 84 | 150 | 193 |
| -10.50 to -12.70 (4 to 5) | 84 | 193 | 407 | 532 |
| Pile head displacement (mm) | 2.00 | 3.09 | 5.35 | 7.12 |

**Table 5. Unit skin friction and pile head displacement values (PTP3)**

| Skin Friction (kPa) | | | | |
|---|---|---|---|---|
| Strain gauge level (DMD) | 1604 kN | 3207 kN | 6414 kN | 8018 kN |
| | 50% | 100% | 200% | 250% |
| 0.00 to -5.00 (1 to 2) | 11 | 14 | 33 | 42 |
| -5.00 to -10.80 (2 to 3) | 18 | 28 | 74 | 94 |
| -10.80 to -13.00 (3 to 4) | 57 | 129 | 210 | 287 |
| -13.00 to -15.00 (4 to 5) | 79 | 171 | 321 | 417 |
| Pile head displacement (mm) | 1.48 | 2.59 | 5.55 | 7.18 |

**Table 6. Unit skin friction and pile head displacement values (PTP4)**

| Skin Friction (kPa) | | | | |
|---|---|---|---|---|
| Strain gauge level (DMD) | 1764 kN | 3528 kN | 7055 kN | 8820 kN |
| | 50% | 100% | 200% | 250% |
| 0.00 to -5.00 (1 to 2) | 7 | 13 | 25 | 57 |
| -5.00 to -10.80 (2 to 3) | 32 | 49 | 139 | 166 |
| -10.80 to -13.00 (3 to 4) | 101 | 249 | 407 | 480 |
| Pile head displacement (mm) | 2.60 | 4.17 | 7.27 | 8.20 |

**Table 7. Unit skin friction with depth at maximum applied load for the tested piles in this study**

| Pile no. | Pile diameter (mm) | Test load (kN) | Strain gauge levels (mDMD) | Unit skin friction at test load (kPa) | Pile head displacement at test load (mm) |
|---|---|---|---|---|---|
| PTP1 | 1000 | 6010 | -1.00 to -5.00<br>-5.00 to -9.00<br>-9.00 to -10.50<br>-10.50 to -12.70 | 32<br>44<br>204<br>704 | 4.72 |
| PTP2 | 1000 | 7125 | -1.00 to -5.00<br>-5.00 to -9.00<br>-9.00 to -10.50<br>-10.50 to -12.70 | 50<br>79<br>193<br>532 | 8.28. |
| PTP3 | 1000 | 8018 | 0.00 to -5.00<br>-5.00 to -10.80<br>-10.80 to -13.00<br>-13.00 to -15.00 | 42<br>94<br>287<br>417 | 7.18 |
| PTP4 | 1000 | 8820 | 0.00 to -5.00<br>-5.00 to -10.80<br>-10.80 to -13.00 | 57<br>166<br>480 | 8.20 |

**Axial Pile Performance (t-z Curves)**

The axial pile performance was identified using the relationship between mobilized soil- pile shear transfer and pile vertical deflection/settlement (t-z curve). There are numerous different methods for interpreting this kind of axial load transfer and pile displacement curves (API, 2007). The most common approach is by demonstrating the mobilization of shaft friction displayed as a set of springs distributed along the pile shaft and the axial elastic stiffness of the pile (Karlsrud, 2014). The distribution of stiffness of both the pile and soil, pile geometry and soil distribution are all influential factors. The construction of t-z analysis of the static load test data describes the soil-pile behavior over the entire length of the pile. In order to estimate the load transfer mechanism, calculation of pile settlement can be made using plots of unit skin friction *versus* pile settlement. The plots obtained for the four tested piles are shown in Figures 3 through 6.

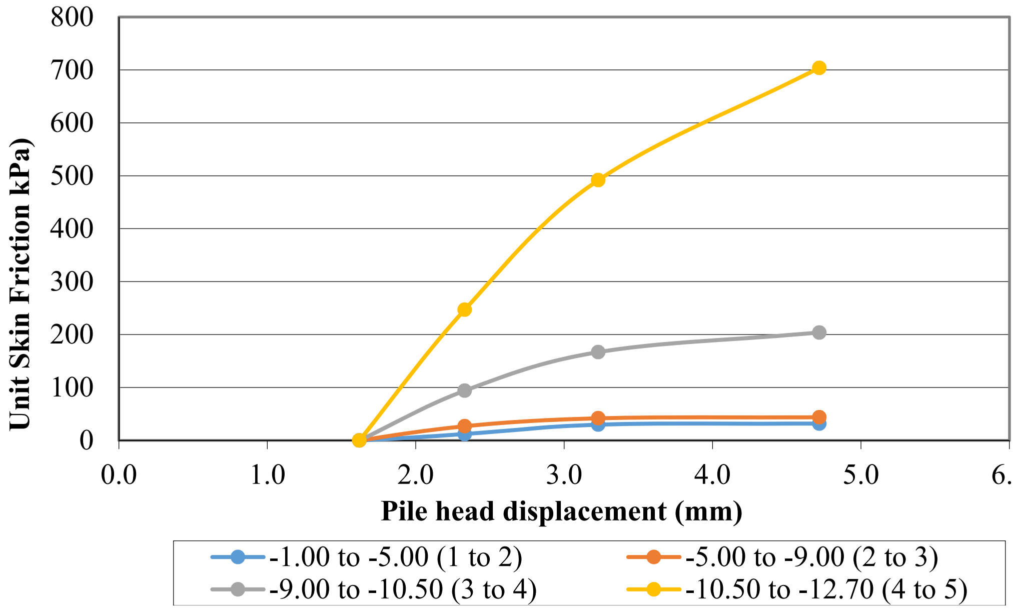


**Figure (3): Pile head displacement – unit shaft friction plot for PTP1**

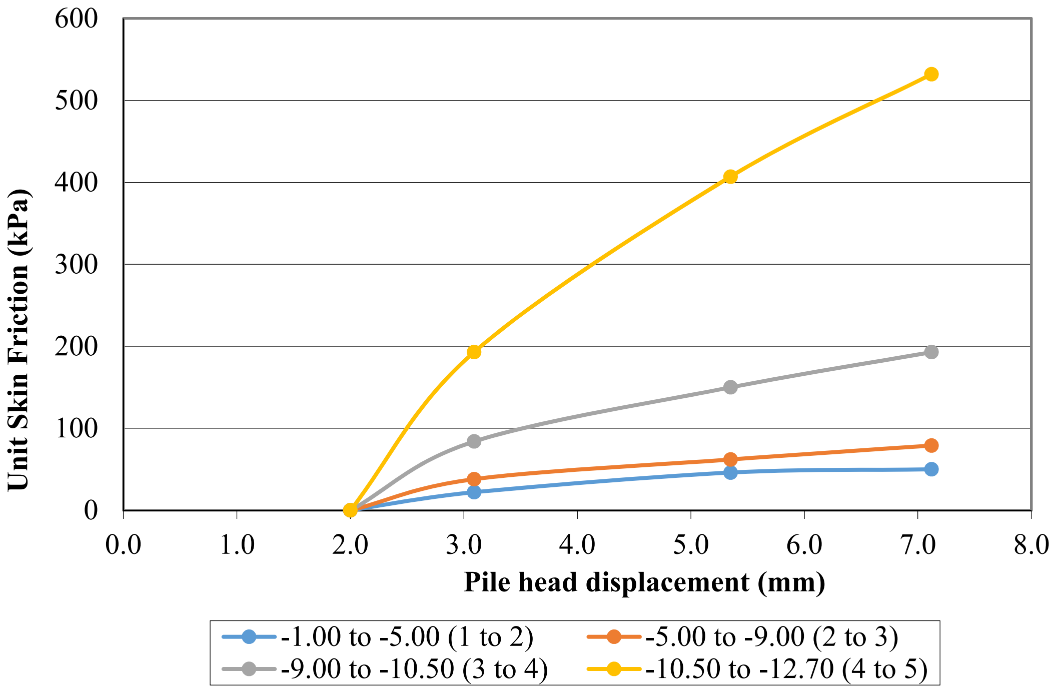


**Figure (4): Pile head displacement – unit shaft friction plot for PTP2**

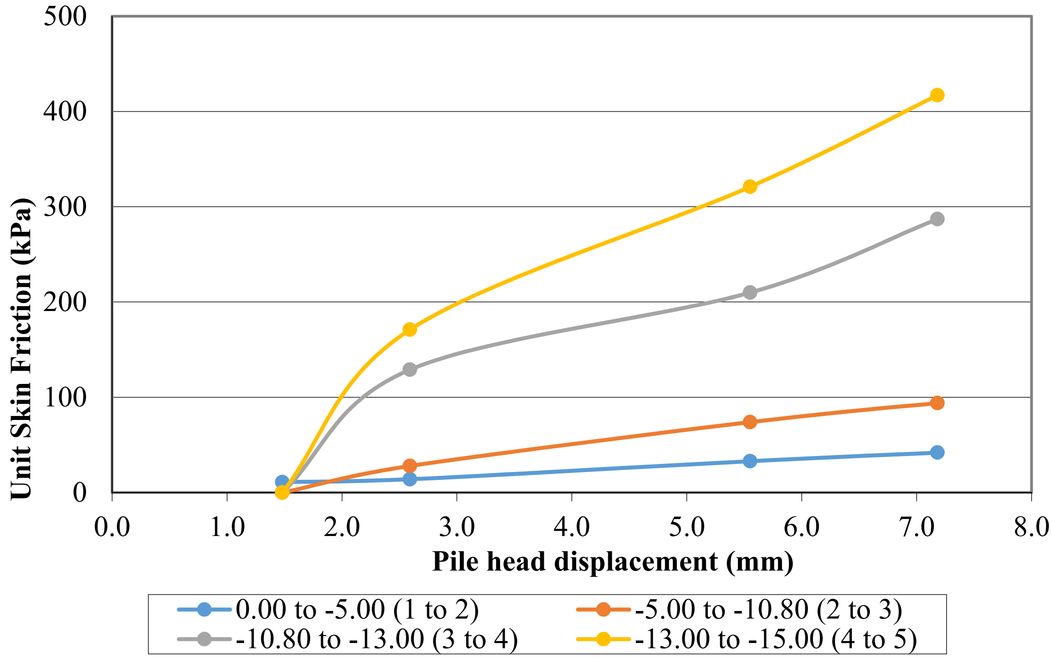


**Figure (5): Pile head displacement – unit shaft friction plot for PTP3**

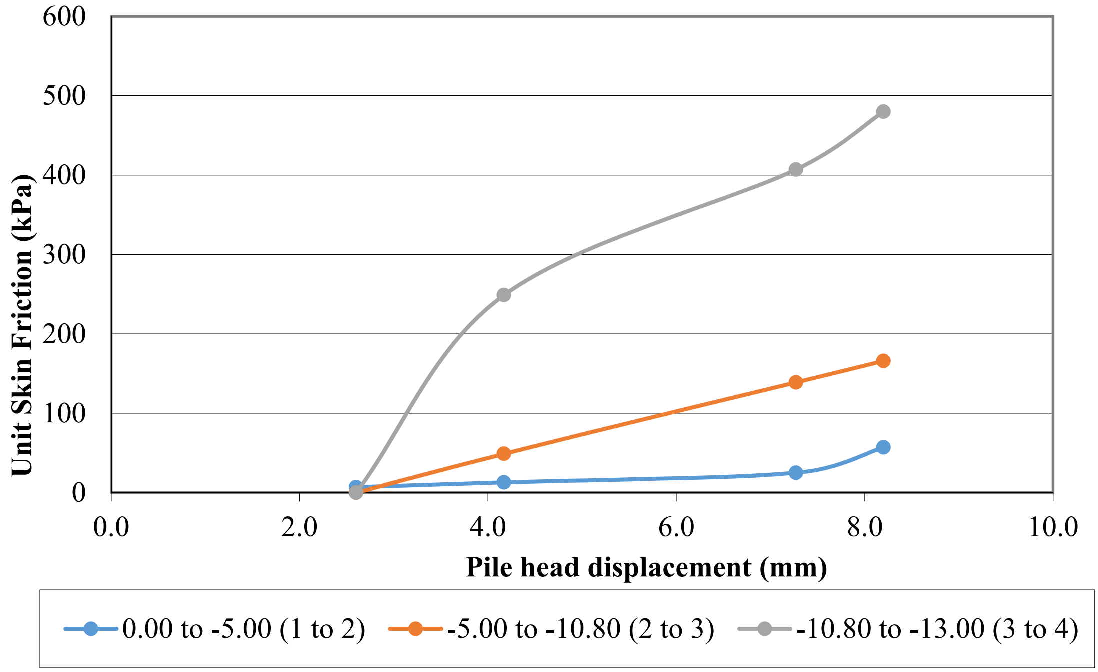


**Figure (6): Pile head displacement – unit shaft friction plot for PTP4**

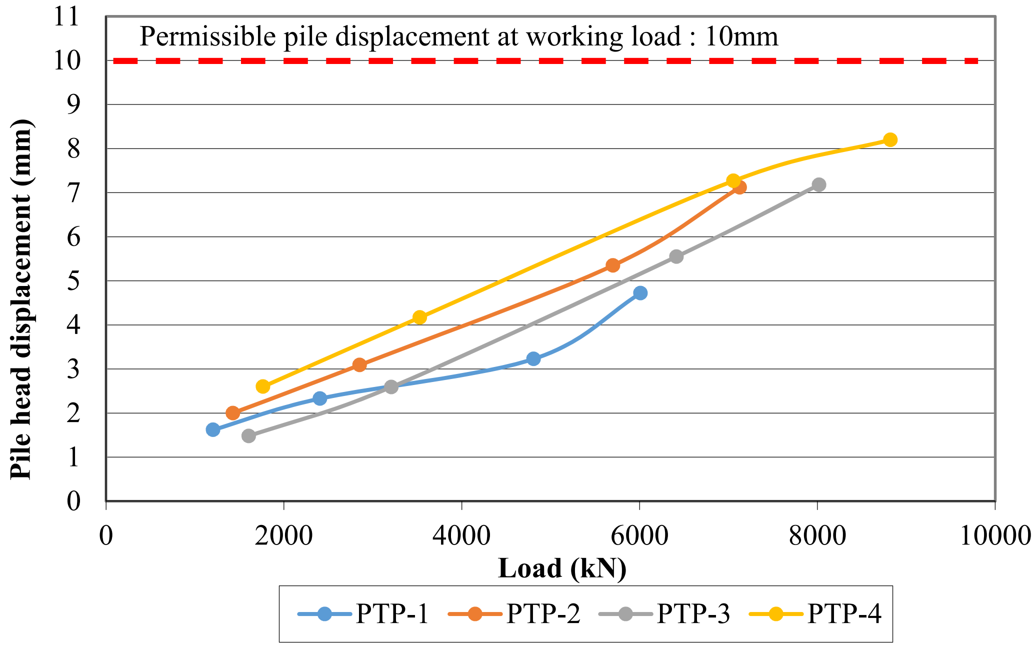


**Figure (7): Load - pile head displacement plots from pile load tests**

The unit skin friction *versus* average pile head displacement plot (Figs. 3 to 6) shows an increasing trend in all the tests. This can be attributed to the soil densification and induced confining pressure around the pile from the pressurized concrete, rapid mode of pile installation without any noise or vibration, pile diameter and pile surface roughness.

The mobilized average unit skin friction at maximum applied load ranges from 32 to 704 kPa at a normalized pile head displacement of less than 1 % of the pile diameter. The CFA pile shaft friction was kept increasing linearly and did not reach its maximum stress and geotechnical failure during the test. Based on the ideal hyperbolic t-z curves, it is suggested that plastic deformation is not reached along the pile shaft at different loads. This can be due to the fact that the majority of the load is consumed by the surrounding and deeper stiff layers as well as pile surface roughness. This behavior can be a result of a stiff pile and the phenomenon is partly dependent on a very small pile settlement as observed (less than 10mm at ultimate test load) from the load tests (Table 7). This indicates that the ultimate capacities of the tested piles are not fully mobilized and the pile head displacement is within the general permissible limit of less than 1% of the pile diameter at working load (Cherian, 2020). In order to assess the performance of the test piles further to derive suitable shaft friction design values, optimization of pile design was carried out using Eurocode 7 procedure (2004) before execution of production piles of the project.

**Optimization of Pile Design**

The characteristic resistance is calculated by applying a certain correlation factor and partial factor. In this analysis, we have considered only the reaction of a pile; namely, the skin friction and the partial factors on the reactions of the pile. Based on Eurocode 7, unit shaft resistance values obtained from load test results are compared with those obtained from the ground test (Table 8). The unit skin friction values are increasing linearly and do not show evidence of developing geotechnical failure. The unit shaft resistance was mobilized with a maximum settlement of 8.20mm. This indicates that the pile can be still loaded to mobilize ultimate skin friction resistance and settlement along the complete shaft length. It can be concluded that the load test outcomes can appropriately represent the characteristics of soil strata and the side resistances determined are higher than the design values adopted. Recommended design shaft friction values consider only the derivation from load test and ground tests are calculated by subtracting a half of the standard deviation from the average load test and ground test results (Table 8).

**Table 8. Recommended unit skin friction values**

| Elevation in m DMD | Soil / Rock | Soil Friction | | Ground test based Eurocode | Recommended Value |
|---|---|---|---|---|---|
| | | $f_s$ (Comp) | Load test | | |
| | | kPa | kPa | kPa | kPa |
| 0.00 to -5.00 | Sand | 15 | 57 | 22 | 28 |
| -5.00 to -10.8 | Sanstone | 110 | 166 | 165 | 165 |
| -10.80 to -13.0 | Siltstone | 145 | 704 | 217 | 289 |
| -13.00 to -15.0 | Siltstone | 145 | 417 | 217 | 247 |

The recommended values range from 28 to 289 kPa was used for the production of pile design. The revised pile capacity values adopted for the execution of piles in the project site are presented in Table 9.

**Table 9. Revised pile capacity adopted**

| Pile diameter (mm) | C.O.L. (m DMD) | Toe level (mDMD) | Effective length (m) | Pile capacity (kN) |
|---|---|---|---|---|
| 1000 | 2.730 | -11.80 | 14.53 | 3433 |
| 1000 | 2.730 | -14.00 | 16.73 | 4830 |

As per the analysis, the revised pile capacity adopted shows a reduced pile length. Load bearing characteristics of piles may also slightly increase because of the rough interface created by the boring process. The mobilized shaft resistances obtained during the instrumented static load test aided a more precise evaluation of pile capacity. This study indicates that the piles can achieve the required capacity with a 10% reduction of length, thereby allowing cost and time savings. Hence, instrumented static loading tests can be used as a valuable tool to determine the settlement and load-carrying capacity.

## CONCLUSION

This paper investigates the performance of four instrumented axial load tests to verify the initial pile design to execute an environment-friendly cost-effective project. The load test provides data about the behavior of individual piles and the group analysis subsequently shall take into account the foundation interaction to arrive at the optimum grouping of foundations. CFA pile load test results demonstrated a reliable and safe performance as well as higher skin friction and load-carrying capacity, allowing the designer to quantify the geotechnical parameters precisely. This study of the instrumented pile load response with depth makes the results of the tests applicable to other piles at the project site in a much more economical, reliable mode than directly assessing capacity and applying a factor of safety to the pile capacity value. It is suggested that further study of the static effect of instrumented CFA pile in soils and soft rocks with strict quality control is required to understand the ground influence on pile behavior.

**Conflict of Interest**

The author declares that there is no conflict of interest regarding this work.

**Data Availability Statement**

All data and related information used during the study appear in the submitted article.

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