Geotechnical and construction considerations of pile foundations in problematical soils

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ABSTRACT

The Continuous Flight Auger (CFA) technology has evolved especially in recent decades. Using of CFA technology has expanded all over the world. The article deals with geotechnical design of pile foundations, especially by static load tests and by calculation using numerical models. Results of static load tests which were performed in and in Slovakia are analysed and compared with numerical modelling. The technology of CFA has been used due to the advantages of this technology in comparison with driven and bored piles in local geotechnical conditions. Plaxis 2011 was used for CFA piles analyses by FEM. Advanced constitutive hardening soil model was used for soil layers. Comparison of load-settlement curves, ratio of pile base and pile shaft resistance and distribution of unit shaft friction are presented in the paper. The best geoengineering solution in this case is the use of new pile technology CFA (continuous flight auger), FDP (full displacement piles), DDS (drilling displacement system), that lead to increased bearing capacity.

Keywords: numerical model, pile foundations, CFA technology, load tests, design methods, FEM analyses

1. INTRODUCTION

Many megaprojects are emerging in the new capital of Kazakhstan – Astana: Khan-Shatyr, Peace Palace – Pyramid, house estate of “Severnoe Siyanie”, Abu-Dhabi Plaza Hotel, New Aktau city near the Caspian Sea and so on, see Fig. 1.

Modern construction puts modern requirements in front of engineers and designers, and so instead of traditional decisions it came to the use of new economical and ecological efficient technologies such as CFA (continuous flight auger and so on. It is well known that pile foundation is one of the most widely used types of foundation at the construction sites of Kazakhstan. Application of pile foundation is explained by necessity of ensuring a high bearing capacity for high-rise buildings.

It has been mentioned previously that existing Kazakhstan standard documentation of pile design is out of date and does not meet the requirements of modern engineering. The standard needs to be revised. Nowadays, conception of pile foundation design is in the process of modernization, as presented in (Zhussupbekov A. et al., 2013).

Fig. 1. Engineering megaprojects in Kazakhstan.

Design of pile foundation includes two critical stages of analysis: bearing capacity and settlement analysis. The preliminary design is performed based on
the engineering and geological investigation of construction site. Accuracy of pile design generally depends on the accuracy of data presented in geological report. Final pile design project is corrected after approval by field tests.

2 DESIGN METHODS FOR PILE FOUNDATIONS

Currently, a number of design methods are known to determine the pile resistance or load-settlement diagram. The choice of design method depends often on the experiences, economy and technology availability in given countries. Bearing capacity of single pile can be determined by the following:

- the results of static load tests carried out on working piles in real scale (experimental resistance),
- the results of static load tests carried out on model (trial) piles (piles of smaller diameter),
- the results of in-situ tests (e.g. CPT tests),
- empirical or analytical calculations based on the results of laboratory and in-situ soil testing to determine the geotechnical parameters of soils,
- calculations which take into account damage of pile body.

Best results can be obtained by static load tests of instrumented piles, which are installed in the same geological conditions, by the same technology and the same dimensions as system piles (Masopust, 1994). Analytical and semi-empirical calculation methods based on the results of site investigation and ground testing are also often used for pile design, especially due to the cost saving. Analytical or empirical calculation methods are also the simplest methods which provide an adequate view of load distribution from pile to the surrounding soils. Using numerical methods (e.g. FEM) for pile design has increased in last years. FEM allows introducing more complex boundary ground conditions and also local inhomogeneity of subsoil, local effects of concentrated load at the pile base (Feda, 1977) and deeper understanding of behaviour of the soil-pile system as shown in many papers.

The pile design approaches are different in our countries. In Kazakhstan, static load tests are used for pile design, while in Slovakia, piles are designed only by analytical methods and static load test are used only in exceptional cases (large constructions and projects) for verification of design resistance and settlement. Results of three static load tests in Slovakia and in Kazakhstan (Fig.2) were compared with the results of numerical analysis.

2.1 Numerical modelling of CFA piles

Numerical modelling often does not take into account the effect of pile installation, because piles are modelled as a "wished-in-place". Modelling of pile installation is needed especially for displacement piles where the influence of technology is more significant than for bored piles.

Using cavity expansion theory published by Mecsi estimated that lateral earth pressure around pile shafts was only slightly increased. This finding has led to the fact that the technology in these cases could be taken into account in numerical models only by using interface elements between Piles and surrounding soils.

CFA piles presented in this article are modelled using the software Plaxis 2011. Tasks were done as axisymmetric ones using 15 node elements. Linear elastic (LE) constitutive model was used for CFA pile body and advanced Hardening Soil (HS) constitutive model was used for soil layers. Hyperbolic relationship between the vertical stress and the deviatoric stress in primary triaxial loading was considered for HS model. Soil shows a decreasing stiffness and simultaneously irreversible plastic strain development during the primary deviatoric loading. Yield surface form in the space a hexagonal pyramid given by Mohr-Coulomb criterion. Hardening parameter is not constant, but varies depending on accumulated plastic strain soil.

2.1.1 Numerical model of CFA pile in Bratislava

Static load test on CFA pile in Bratislava, Slovakia (Fig. 2, over) has been made on system (working) pile of piled raft foundation below high-rise building. Depth of excavation pit is 6.3 m below the original surface. CFA piles are 15.0 m long with diameter of 630 mm. Geological conditions vary along the length of the pile. Quaternary well graded gravels are located in upper...
part of pile environment up to 2.5 m. Free phreatic level has been taken into account in stratum of gravel. Sandy silts and silts with low plasticity are situated below the layer of gravel from 2.5 to 12.5 m. These soil layers have similar geotechnical properties and the ground model of homogenous layer was therefore used. The silty sand with confined groundwater in the depth 12.5 – 13.9 m has been defined with pore pressure equal to 100 kPa. Silts with medium plasticity are located below sandy layer. Geotechnical model of CFA pile is shown in Fig. 3.

2.1.2 FEM analysis of CFA piles in Karagandy

Two static load tests (Fig. 2, under) have been done below a 10th floor residential building in Karagandy. Both test piles were 8.0 m long with diameter of 630 mm. Excavation pit for foundation of building was approximately 3.7 m below the surface. Geological conditions of Karagandy are represented by fine grained soils. This area is formed predominantly by silts to sandy silts with lenses of clay to sandy clay. Geotechnical models of both piles are shown in figure 3. Input soil parameters for HS model are taken into account as: \( E_{oed}^{ref} = E_{50}^{ref} \), \( E_{urref} = 4 . E_{oed}^{ref} \) and \( m = 0.7 \) for fine-grained soils. Effect of excavation was considered using the value of over consolidation ratio in both cases.

Numerical analysis were done in form of a parametric study. Analyses include 24 models for CFA pile in Bratislava and 2 x 18 models for CFA piles in Karagandy. Technology impact, interface, phreatic level and over consolidation were observed. Load-settlement curves obtained from static load tests were compared with load-settlement curves calculated by numerical modelling, as shown in Figure 4 for Bratislava and in Figure 5 and Figure 6 for two tests in Karagandy.

Static load test in Bratislava has been fully instrumented. Distribution of load over the pile has been measured by deformation reading recorders. These results were used for comparison of real pile base and pile shaft resistances with calculated ones. Pile base and pile shaft resistances of CFA piles in Karagandy from numerical modelling, are compared with real measurements included in load-settlement curves.

The impact of CFA technology has a significant influence on the pile resistance, especially on shaft friction. Distributions of unit shaft friction along the CFA pile are shown in Figure 7. It can be seen that in case of CFA pile in Bratislava, unit shaft friction is visibly small in quaternary gravels under the phreatic level. A small decrease is also in layer of silty sand which is caused by the pore water pressure. Distribution of unit shaft friction in cases
K-TP1 and K-TP2 is constant and proportional to the depth due to relatively homogeneous soil profiles.

Fig. 7. Distributions of shaft friction over CFA piles.

3 COMPARISON OF EXPERIMENTAL AND DESIGN DDS BEARING CAPACITY

Experimental and designed values of bearing capacities are presented in Table 1. There are big differences between experimental results and design values of predicted bearing capacities.

Table 1. Experimental and designed values of bearing capacities.

| № | Description of piles | Bearing capacity (kN) | Coefficient \( k = \frac{F_u}{F_d} \) |
|---|----------------------|-----------------------|--------------------------|
| 1 | Pile DDS L=17m \(d=410\ mm\) | №1 2280 1545 | 1,48 |
| 2 | Pile DDS L=17m \(d=410\ mm\) | №2 2150 1545 | 1,39 |
| 3 | Pile DDS L=17m \(d=410\ mm\) | №3 2325 1545 | 1,50 |
| 4 | Pile DDS L=17m \(d=410\ mm\) | №4 2475 1545 | 1,60 |
| 5 | Pile DDS L=17m \(d=410\ mm\) | №5 2200 1545 | 1,42 |
| 6 | Pile DDS L=17m \(d=410\ mm\) | №6 2080 1545 | 1,35 |
| 7 | Pile DDS L=17m \(d=410\ mm\) | №7 2190 1545 | 1,42 |
| 8 | Pile DDS L=2m \(d=600\ mm\) | №1 2700 2110 | 1,28 |
| 9 | Pile DDS L=2m \(d=600\ mm\) | №2 470 272 | 1,73 |
| 10 | Pile DDS L=2m \(d=600\ mm\) | №3 490 272 | 1,80 |
| 11 | Pile DDS L=2m \(d=600\ mm\) | №4 460 272 | 1,69 |

The comparison of SLT and design value of bearing capacity is presented in Fig. 8. All the linear function points are higher than the diagonal ones which means that all experimental values of bearing capacity are higher than the designed ones. Significant differences between experimental and designed bearing capacities of DDS. In case of DDS, piles were surrounded with soil subjected to compaction.

4 COMPARISON OF FEM CFA AND CASING BORING (TRADITIONAL) PILES

The FEM elasto-plastic analysis was provided by computer program established by Prof. Tadatsugu Tanaka. It uses the mechanical properties of soil ground for the numerical calculation of bearing capacity and settlement. For analyzing bearing capacity working as friction, CFA and Casing piles were modeled and compared with results of static load test.

Fig. 8. SLT and designed bearing capacity.

Taking advantage of the axi-symmetric nature of the problem, only half domain of the model ground and pile was analysed. The soil ground and pile were discretized into four noded quadrilateral elements. Number of nodal points are 675, number of finite elements are 606, number of materials are 4 (1 is sand with gravel, 2 is hard clay, 3 is clay, 4 is bored pile).

During CFA pile installation the question of over-expenditure of concrete was appeared. The actual volume of borehole was about 1.3-1.4 times more than theoretical volume of borehole. After the determination of preliminary average radius \( r + \sqrt{r} \), increasing diameter of CFA piles and remodeled numerical mesh FEM analysis was repeated. It results in increasing bearing capacity of CFA piles “load-settlement” results of field static load test and stress and strain of soil around of single CFA pile through FEM computer program. The results of “load-settlement” from FEM are illustrated in Fig. 9.

Fig. 9. Results of FEM analysis.

5 COMPARISON OF DDS AND CASING BORING (TRADITIONAL) PILES

Comparison of DDS and Casing boring piles bearing capacities by SLT are presented in Fig. 10. There is big difference between DDS and Casing boring piles bearing capacities.

Significant differences between bearing capacities of DDS and casing boring piles indicate the incomplete usage of DDS technology resources.

Fig. 10. Comparison of DDS and Casing boring piles bearing capacities by SLT.
Classically there are two stages of pile under vertical loads: during the first stage, the ultimate state of stress-strain condition of soil is developed, and during the second stage, the slippage of pile through the soil takes a place. DDS pile works identically, but if the DDS pile is surrounded by soil subjected to compaction, the bearing capacity is increased. In this case the coefficient of soil work condition will be different. In the case of traditional bored pile no compaction is occurred, therefore coefficient of shaft work of pile equal to 0.7, and in the case of DDS piles surrounded by soil subjected to compaction this coefficient will be increased. Moreover in case of DDS pile surrounded by soil subjected to compaction only the sides undergoes compaction, under the pile no compaction. From results of static load tests of DDS and case piles, the inverse problem was performed to the definition of coefficients. Obtained coefficients of DDS piles shaft work for different EGE (engineering geological elements) are presented in Table 2.

After reprocessing of obtained data by statistic analysis, several results of elastic modulus were rejected. It is necessary to take into account elastic modulus, angle of internal friction and cohesion increase due to compaction during the design DDS pile.

| Depth, m | EGE2 | EGE3 |
|---------|------|------|
| 5       | 1.23 | 1.31 |
| 6       | 1.18 |      |
| 7       | 1.15 | 1.21 |
| 5       | 1.45 | 1.23 |
| 6       | 1.37 | 1.37 |
| 7       | 1.32 | 1.23 |
| 5       | 1.67 | 1.23 |
| 6       | 1.56 |      |
| 7       |      | 1.21 |
| 5       | 1.33 |      |
| 6       | 1.26 |      |
| 7       | 1.21 |      |

6 FEM ANALYSIS OF DDS PILES

The main goal of this research is to determine the influence of soil compaction on the bearing capacity and settlement of pile. FEM analysis was performed to analyse the stress-strain development of traditional and DDS pile in different geological elements of Astana. The FEM model included two soil clusters. In case of traditional pile simulation the two clusters have identical soil characteristics, in case of DDS pile simulation of upper cluster was changed. The change took into consideration the compaction of soil surrounding the DDS pile. Only surrounding soil condition, but not under the pile, was changed; since there is nothing occurs under the pile, namely no compaction.

First of all, it becomes important to predict bearing capacity of traditional and DDS pile. Next the obtained bearing capacities is compared with the calculated ones by Kazakhstan Standard. By substituting this relationship in Kazakhstan Standards equation of bearing capacity, the coefficient of surrounding soil work of DDS pile can be obtained by Eq. 1 and Eq. 2.

The results of FEM analysis for different geological elements and different DDS pile diameters are presented in Fig. 11.

$$k = \frac{F_{D,PL,X}^{DDS}}{F_{D,PL,X}^{TR}}$$  \hspace{1cm} (1)

$$\gamma_{D,PL,X}^{DDS} = \frac{kF_{D,PL,X}^{TR} - \gamma_{D,PL,X}^{TR}RA}{u\gamma_e \sum f_i h_i}$$  \hspace{1cm} (2)

where: $F_{D,PL,X}^{DDS}$ – predictable by Plaxis bearing capacity of DDS pile; $F_{D,PL,X}^{TR}$ – predictable by Plaxis bearing capacity of traditional pile.

In case of DDS pile surrounded by soil subjected to compaction, and no compaction under the pile, the obtained coefficients of DDS piles shaft work for different EGE (engineering geological elements) equal to: EGE2 – 1.38, EGE3 – 1.26, EGE4 – 1.2.
7 COMPARISON OF SLT RESULTS FROM DIFFERENT TYPES OF PILES

SLT of different types of pile was performed with a view to compare bearing capacity of traditional (namely, boring casing pile and driving pile).

Unfortunately, most of the tested piles did not achieve ultimate settlements prescribed by Kazakhstan Standard – 24 mm, and so, for bearing capacity comparison, it was chosen to use 3mm settlement criteria, since all the piles achieved this settlement. All the piles were designed to the criteria of 2200 kN bearing capacity. Designed parameters of piles (length and cross section) by Kazakhstan Standards are presented in Table 3. Results of comparison are presenting in Fig. 12.

All of these coefficients show incapacity of accurate design of modern pile technology by out-of date Standards, otherwise this coefficients tending to 1. The results of SLT showed entirely expected regularity. CFA piles showed highest bearing capacity as long as during CFA pile installation it was expended much more concrete than during Casing pile installation.

Differences between Driving and Casing pile are too small and can be neglected. The reason for these differences might be the empirical coefficients required by Standards.

Table 3. Designed pile characteristic.

| Type of pile | Required quantity, e.a. | Length of pile (m) | Diameter or cross section (m) |
|--------------|------------------------|-------------------|-----------------------------|
| CFA          | 1                      | 10                | 0.5                         |
| DDS          | 1                      | 10                | 0.5                         |
| Casing       | 1                      | 10                | 0.5                         |
| Driving      | 2                      | 12                | 0.3 x 0.3                   |

8 CONCLUSION

The installation process of CFA piles is less time-consuming compared to traditional rotary bored piles. Stability of borehole is ensured by the soil in continuous auger and therefore no other stabilization elements are needed. Suitable geological conditions for CFA piles are fine-grained soils of stiff consistency, weathered limestone and sandstone, residual fine-grained soils and medium dense to dense well-grained sands. Numerical analysis has been performed in form of a parametric study. Very good results were achieved using the numerical modelling in comparison with results of static load tests. These investigations are important for understanding the behaviour of boring piles on problematical soil ground.

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