Determination of the Bearing Capacity of Piles Using the Cone Penetration Test

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Abstract. The article is devoted to the problem of discrepancy between the data of bearing capacity of piles, determined by the data of cone penetration tests (CPT) and static tests. An example of this discrepancy is shown for one of the construction sites of the Kazan city. The reason of discrepancy for the investigated area is revealed and the method of solving the problem by creating a new correlation table for alluvial Sands, which are often the basis of piles in the city of Kazan, is proposed.

1. Introduction

Modern construction involves the use of complex structural solutions of buildings and structures. If the geological conditions are not quite favorable, the most reliable option is to use a pile Foundation.

Since the cost of pile foundations is usually much higher than the cost of shallow foundations, it is important to determine the accuracy of the bearing capacity of the piles in specific geological conditions.

Among the field methods of soil testing in natural conditions, CPT occupies one of the leaders, as it allows exploring the soil massif to a greater depth at a relatively small material cost compared to other field methods. For example, full-scale static load tests [1].

Limit of application of CPT for the calculation of the bearing capacity of piles is due to a significant discrepancy between the calculation data and the real strength values. This is because when processing the data of CPT uses a table (SP 24.13330) constructed on the basis of correlations for the soils of the entire territory of Russia.

In practice, such a mismatch leads to improper selection of piles and, as a result, to large material costs. On the territory of Kazan city, there are hundreds of cases when the design organization recommended the use of piles of a certain length, which then on the construction site could not be driven to the required depth because of their underestimated bearing capacity.

The main purpose of this study is to find the cause of the discrepancy between the bearing capacity of the piles, determined by CPT data and the real strength of the pile Foundation on the example of one of the construction sites in the city of Kazan. We also propose a way to solve this problem by creating regional correlation tables.

2. Review

The method of calculating the bearing capacity of the piles in the domestic practice is given in the current SP 24.13330.2011 "Pile foundations" [2] and is to determine the private value of the limit
resistance of the driving pile \( F_u \) cross-sectional area \( A \) (bearing area of the pile is taken over cross-sectional gross or the largest diameter) and the perimeter \( U \) (outer perimeter of the cross-section of the pile) at the sensing point as the sum of the resistance on the frontal surface \( R_s \) and the resistance to friction on the side surface of the pile \( f_i \) (unit shaft resistance):

\[
F_u = R_s A + f_i U
\]  

(1)

where \( h \) is the depth of pile immersion from the ground surface, m.

The relevance of the method of CPT in Russia is confirmed by a large number of studies in this area [3-13].

Talking about foreign experience, in this case, the calculation of the bearing capacity of piles according to CPT data is performed by two approaches – direct and indirect [7]. In the indirect approach, first, the strength characteristics of the soil are determined from the sensing data, and then they are used to calculate the bearing capacity of the pile. The direct approach involves the calculation of the resistance of the soil under the tip of the probe \( q_b \) and ground resistance aside surface of the friction clutch of the probe \( f_i \) using the parameters of CPT.

The most common methods of calculation include:

1. methods in which only \( q_{eq} \) values are used to calculate \( q_b \) and \( f_i \):
   - method Bustamante and Gianessi (LCPC, Laboratoire Central des Chaussees Pontset method) [14]. Resistance under the lower end of the pile \( q_b \) (the unit toe resistance) is defined as the product of the coefficient of bearing capacity, depending on the type of soil and method of the device of the pile, \( k_b \) (coefficient as governed by the magnitude of the cone resistance, type of soil, and type of pile) to the average value of drag, \( q_{eq} \) (average cone resistance):

\[
q_b = k_b \times q_{eq}
\]  

(2)

The lateral resistance \( f_i \) is defined as the quotient of the average resistivity of the soil by the friction coupling of the probe (unit sleeve friction resistance), \( q_{eq} \) by a coefficient depending on the type of soil, the type of pile and the method of pile arrangement \( k_s \) coefficient as governed by magnitude of the cone resistance, type of soil, and type of pile:

\[
f_i = q_{eq} / k_s
\]  

(3)

   - Aoki and De Alencar method [15]. Determination of resistance \( q_b \) is the division of the average value of the resistivity of the soil around the tip of the pile \( q_{ca} \) (the unit toe resistance) by a factor that takes into account the type of pile \( F_b \) (coefficient of type pile):

\[
q_b = q_{ca} / F_b \leq 15 \text{ MPa}
\]  

(4)

Determination of the resistance on the lateral surface of the pile unit shaft resistance \( f_i \) is to divide the works of the average values of the resistivity of the soil around pile toe \( q_{ca} \) the unit toe resistance and the coefficient that takes into account the kind of soil as coefficient of type of soil on the coefficient that takes into account the type of piles \( F_c \) coefficient of type of pile:

\[
f_i = q_c \times \alpha_s / F_z \leq 120 \text{ kPa}
\]  

(5)

2. methods in which \( q_c \) and \( f_s \) are used for calculation:
   - Schmertmann and Nottingham method [16, 17]. Determination of the resistance of the soil under the lower end piles, \( r_t \) (unit toe resistance) is the product of the coefficient of compaction of the soil, (dimensionless coefficient; a function of pile type, ranging from 0.8% through 1.8%) and the average resistivity of the soil in the influence zone, \( q_{ca} \) (average cone resistance is determined in an influence zone extending from 6b through 8b above the pile toe (b is the pile diameter) and 0.7b through 4b below):

\[
r_t = C \times q_{ca}
\]  

(6)

Determination of the resistance of the soil on the lateral surface of the pile, \( r_s \) (unit shaft resistance) is the product of the dimensionless transition coefficient, \( K \) (dimensionless coefficient) on the resistivity of the soil along the friction sleeve of the probe, \( f_s \) (sleeve friction):

\[
r_s = K \times f_s
\]  

(7)

   - method Tumay and Fakhro [18]. Cone resistance is determined by the method of Schmertmann and Nottingham [equation 6].
Determination of resistance on the side surface of the pile \( f_i \) (the unit shaft resistance) is the product of a dimensionless coefficient \( m \) (dimensionless coefficient) on the average value of the specific resistance of the soil on the friction coupling of the probe (sleeve friction) (for type II probe) \( f_{sa} \):

\[
f_i = m \times f_{sa}
\]  

(8)

3. a method in which the data obtained by the piezocone testing (the piezocone) are used for calculations:

- **Eslami and Fellenius method** [19]. Determination of resistance under the pile toe, \( r_t \) (unit toe resistance) is the product of a correlation coefficient equal to 1 with the diameter or side of the pile less than 0.4 m, \( C_t \) (the correlation coefficient of 1 provided on the side of the pile is less than 0.4 m) on the average value of the "effective" resistance of the soil on the cone in the zone of influence, \( q_{et} \) (geometric average of the cone point resistance adjusted to effective stress):

\[
r_t = C_t \times q_{et}
\]

(9)

Determination of resistance on the side surface of the pile, \( r_s \) (unit shaft resistance) is to determine the product of the coefficient depending on the type of soil, \( C_s \) (shaft correlation coefficient, which is a function of soil type determined from the soil classification) and the average value of the effective drag \( q_{es} \) (geometric average of the cone point resistance adjusted to effective stress):

\[
r_s = C_s \times q_{es}
\]

(10)

4. a method in which the soil resistance under the tip of the qc probe and the parameters of the natural stress state of the soil massif (the total vertical stress from the soil's own weight \( \sigma_{vo} \), the effective vertical stress from the soil's own weight \( \sigma'_{vo} \)) are used for the calculation:

- **Almeida method** [20]. Determination of the resistance \( q_b \) is to determine the division of the difference between the \( q_c \) and the total vertical stress \( \sigma_{vo} \) on the coefficient depending on the type of pile and its material \( k_2 \):

\[
q_b = \frac{q_c - \sigma_{vo}}{k_2}
\]

(11)

Determination of the resistance on the lateral surface of the pile \( f_i \) lies in the definition of dividing the difference between the frontal resistance \( q_c \) and the vertical full voltage \( \sigma_{vo} \) reduction factor applied to piles \( k_1 \):

\[
 f_i = \frac{(q_c - \sigma_{vo})}{k_1}
\]

(12)

3. **Geological engineering aspects of the construction site**

Exploring site is located on the right Bank of the Kazanka River in Kazan city [figure 1]. In 2012, the Palace of water sports was built there. It is a complicated structure with dimensions of 187.5x74 m and a height of 25 m. Reinforced concrete piles with a cross section of 0.3x0.3 m length of 12 m was taken as foundations.

Geological structure of the site is characterized by Quaternary alluvial-deluvial deposits based on Neogene deposits [figure 2].

Geomorphologically, the research site is located within the I terrace of the Kazanka river.

The surface of building site is relatively flat, with a general slope of 2-3 degree in the South-West direction, characterized by absolute altitude from 56.9 m in the Northern part to 53.9 m in the South-West.

Based on CPT data, the length of the piles was taken to be 12 m, however during the construction piles managed to score at a depth from 6 to 9 m.

4. **Study procedure**

To obtain real values of the bearing capacity of the piles at the site, control tests of the piles by static load were carried out. The test points are shown in figure 1. The results of field trials are presented in the table 1 and 2.
Figure 1. Scheme of building elements and field work points.

Figure 2. Geological engineering section on the line I-I (The location of the section line is shown in Figure 1). 1- earth; 2a- very stiff clay; 3a-very stiff loam; 6a- fine sand; 7a- medium sand; N2a- neogene very stiff clay.

Table 1. Values of physical properties of the soils.

| Engineering geological element | Soil type                  | Density, g/sm³ | Void ratio, arb. units | Water content, arb. units | Liquidity index |
|--------------------------------|----------------------------|----------------|------------------------|---------------------------|-----------------|
| 2v                             | firm clay                  | 1.87           | 0.93                   | 0.32                      | 0.6             |
| 3b                             | firm-stiff loam            | 1.97           | 0.71                   | 0.24                      | 0.4             |
| 3v                             | soft-firm loam             | 1.9            | 0.86                   | 0.3                       | 0.6             |
| 3g                             | very soft-firm loam        | 1.9            | 0.87                   | 0.31                      | 0.9             |
| 6a                             | fine saturated sand        | 2.06           | 0.56                   | 0.21                      |                 |
| 7a                             | medium saturated sand      | 2.07           | 0.56                   | 0.2                       |                 |
| № pile | section (cm) | length (m) | ultimate resistance of pile of CPT $F_d$, kN | ultimate resistance of pile of static load test $F_d$, kN | divergence | soil under the lower end of the pile |
|--------|--------------|------------|---------------------------------------------|--------------------------------------------------|------------|-------------------------------------|
| 16     | 30x30        | 7.7        | 500                                         | 1200                                             | 2.2        | saturated sand                      |
| 21     | 30x30        | 8.68       | 1077                                        | 1330                                             | 1.4        |                                     |
| 513    | 30x30        | 8.58       | 800                                         | 1500                                             | 2.3        |                                     |
| 769    | 35x35        | 8.6        | 1200                                        | 1600                                             | 1.6        |                                     |
| 2062   | 35x35        | 8.71       | 932                                         | 1100                                             | 1.25       |                                     |
| 4739   | 35x35        | 7.3        | 1040                                        | 1480                                             | 1.7        |                                     |

5. Conclusion

By authors it was analyzed the data of CPT of alluvial Quaternary Sands at the construction site of the Palace of water sports in Kazan city. The study showed that the average value of the limit resistance of the pile according to the results of CPT was 924.8 kN, and the average value of the limit resistance of the pile according tests by static load amounted to 1368.3 kN. That is, the discrepancy is on average 1.7 times.

The similar discrepancy was revealed for other ground areas of the Kazan city and the Republic of Tatarstan. Now the data of several thousand CPT points were analysed.

The analysis of obtained results shows that the main reason for the discrepancy is using of the recommended SP 24.13330 coefficients of transition from the probe resistance under the probe tip to the limit resistance of the soil under the lower end of the pile $\beta$. For example, according to SP 24.13330, $\beta$ values for piles of the investigated site are in the range of 0.26-0.40. At the same time, to obtain real values of pile resistance, the specified range should be 0.43-0.60.

Table 3 is showing the values of coefficients $\beta$ for driving piles according to SP 24.13330 and proposed by the authors for alluvial Quaternary Sands.

| Average value of soil resistance $q_s$, kPa | Coefficient values $\beta$ | According to SP 24.13330 | Proposed by the authors |
|--------------------------------------------|-----------------------------|---------------------------|-------------------------|
| <1000                                      | 0.90                        | 0.90                      |
| 2500                                       | 0.80                        | 0.80                      |
| 5000                                       | 0.65                        | 0.75                      |
| 7500                                       | 0.55                        | 0.70                      |
| 10000                                      | 0.45                        | 0.65                      |
| 15000                                      | 0.35                        | 0.55                      |
| 20000                                      | 0.30                        | 0.45                      |
| >30000                                     | 0.20                        | 0.35                      |

The most promising way to solve this discrepancy is the creation of private regional tables according to CPT for different types of soils, and in the future – territorial building standards for
Kazan and the Republic of Tatarstan, which will improve the accuracy of engineering-geological surveys and increase the economic profitability of construction.

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