Field measurements investigating the stress strain behavior of driven pile foundations on hard soil

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ABSTRACT: Analyzed field tests of Perm hard argillite clays. Calculated bearing capacity of driven piles based on this type of soil differs from results of field measurements as CPT, static loading test of full-scale and reference piles. Measured pile resistances are compared with theoretical calculations utilizing five analytical approaches: K. Terzaghi`s method, B.J. Hansen`s method, A.S. Vesic`s method N. Janbu`s method and Berezantzev`s method.

KEYWORDS: static loading test, hard soil, settlement, bearing capacity, driven pile.

1 INTRODUCTION

In the last 10 years heigh of the buildings in Perm grew constantly, as well as loads transmitted to foundations are increased. In this case alluvial deposits can not bear the loadings, so Lower Permian deposits such as sandstone and argililte are interact with high-loaded deep foundations.

This type of soils classified as soft rocks or hard soils, and there is to less in-situ information about strengh and deformation properties of this hard soils. On the one hand according to recent studies (Ponomaryov et al. 2013, Sytchkina et al. 2011, 2013), becomes clear, that new methodologies of laboratory test and in-situ tests for this type of soils are needed. On the other hand – pile tip bearing capacity calculation methods existing in Russian building standards also needed to be adjusted.

1.1 *Initial information*

Experimental construction site located in one of the district of Perm. The surface of the site is covered in man-made ground partly with a soil-vegetative layer. Strength and deformation characteristics presented at the Table 1, geological profile, consist of four boreholes, is presented in Figure 1.

Table 1. Soil characteristics.

Soil layer	Soil description	γ, (kN/m^3)	E_{oed}, (MPa)	c, (kPa)	φ, (°)
1	Loam hard to plastic	18.82	11.8	31.0	21
2	Sandy loam hard to plastic	17.15	30.0	0.0	32
3	Sand fine	17.52	28.0	0.0	32
4	Hard clay	18.15	10.0	28.0	18
5	Sandstone	19.11	12.8	11.0	33
6	Argillite clays	19.31	11.6	30.0	26

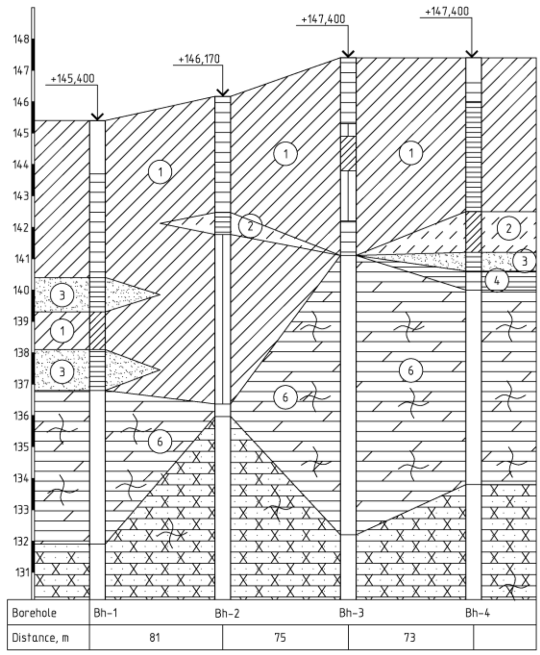

Figure 1. Geological profile of experimental construction site.

Designed foundations are pile foundations with monolithic grillage. Piles are driven, reinforced concrete of solid square section 0.3x0.3 m. The length of piles is 10 m. The design load per pile accepted in the project is 826 kN.

In this case pile foundations interact with upper layer of argillite clays and sandstones. This is the upper heavy weathered and weak layer between Lower Permian and alluvial deposits.

2 ANALYTICAL APPROACH

Analytical approach for bearing capacity calculating presented in Russian building standards SP 24.13330.2011 “Pile foundations’ using formula:

$$F_d = \gamma_c \left(\gamma_{cR} RA + u \sum \gamma_{cf} f_i h_i\right) \quad (1)$$

Where F_d is a bearing capacity of a pile; $\gamma_c, \gamma_{cR}, \gamma_{cf}$ - are coefficients; A - is a square of pile tip, m^2; u - is a pile section perimeter, m; $f_i h_i$ - pile skin friction, kN/m; *R* is a pile tip ultimate resistance. *R* value tabulated in standard and depends on cohesion or non-cohesion soil type, liquidity index I_L and depth of pile tip bearing soil layer tabulated in standard. But, there are no values for weak soil types such as heavy weathered argillite clays and sandstones.

Using different analytical approaches of K. Terzaghi (Terzaghi, 1943), B.J. Hansen`s method (Hansen, 1970), A.S. Vesic`s (Vesic, 1977), method N. Janbu`s (Janbu, 1976) method and V.G. Berezantzev`s (Berezantzev, 1970) method for pile tip bearing capacity q_{ult} to estimate the value of *R* for heavy weathered argillite clays and sandstones.

Inspite of differences between approaches all of them use original equation proposed by Terzaghi:

$$q_u = cN_c s_c + qN_q + 0{,}5\gamma BN_\gamma s_\gamma \quad (2)$$

Where N_c, N_q, N_γ - are bearing capacity factors, c - cohesion, q - is effective pressure at the pile tip, s_c, s_γ - are shape factors.

The Hansen`s general bearing capacity equation contains shape, depth and other factors, that`s made wide application of this equation:

$$q_u = cN_c s_c d_c + qN_q s_q d_q + 0{,}5\gamma BN_\gamma s_\gamma d_\gamma \quad (3)$$

The Vesic likens the problem of failure at the pile tip to that of the expansion of cylindrical cavity immersed in an elastoplasic medium. So that even the compressibility of the medium is taken in account by reduced rigidity index I_{rr}. Initial data for I_{rr} are taken from previous studies (Sytchkina et al. 2013).

For Janbu`s method $\psi = 90°$ angle is used.

Berezantzev`s method using equation:

$$q_u = A_k \gamma_0 d/2 + B_k q_n + C_k c \quad (4)$$

Where A_k, B_k, C_k - are coefficients, those depend on internal friction angle φ.

The results of the pile tip bearing capacity calculations were carried out for four boreholes and are presented in Table 2 (also see Figure 1). E. Sytchkina pointed to the similarity of heavy weathered argillite clays and hard consistency clays, so the *R* value for clays with a I_L=0.1 from SP 24.13330.2011 also indicated in the table.

Table 2. Pile tip bearig capacity calculated by different analytical approaches.

Approach	q_u (kPa)			
	Bh-1	Bh-2	Bh-3	Bh-4
Terzaghi	3842	6757	3460	2400
Hansen	3928	7926	3527	3338
Vesic	5068	7905	4878	2405
Janbu	3301	6313	2962	2797
Berezantzev	3683	8190	3323	2271
SP24.13330.2011	7033	7700	7300	6200

As the table shows, the methods give close values, thus it is necessary to make some comments on each method.

Terzaghi`s and Berezantzev`s methods quite simple and their use allows for the least initial data to estimate pile resistance at the tip of the pile.

Hansen's method is most convenient to use because it gives the result in less than the required input data. Besides the considerable quantity of different factors allows using one equation in case of different types of pile foundations.

To use the Vesic`s and Janbu`s methods specific data is required, that cannot be obtained by standard engineering-geological surveys (I_{rr} reduced stiffness index and the angle ψ).

However, it is recommended to use the Vesic`s method in the case of pronounced anisotropy of soils.

3 IN-SITU TESTS

On the experimental construction site in addition of boreholes, cone penetration test (CPT), static loading test of full-scale piles and static loading test of reference piles were performed. Boreholes, points of CPT and static loading tests ranged from 10-15 m apart.

3.1 *Cone penetration test*

On the site were carried out CPT by equipment named C-832 with a mechanical penetration system of the cone. Used cone type II according to GOST 19912-2001 classification, i.e. the probe with the cone, and a friction clutch.

Probe indentation was carried out with maximum force indentation 30 kN, and measurement of the resistance of the soil under the tip (cone) of the probe and the skin friction resistance of the probe. Cone penetration tests carried out in 33 points, penetration depth was 8,4-14,0 m.

3.2 *Static loading pile tests*

Static loading full-scale pile tests and static loading test of reference piles were carried out according to GOST 5686-94. Test points of full-scale and reference piles ranged from 15-20 m apart.

3.2.1 *Static loading reference pile test*

On the site were carried out 4 static loading reference pile tests. The soil under the pile tip – argillite clays and sandstone (see Figure 1).

Reference pile is a steel pipe with a diameter of 114 mm, and pile tip cone with an apex angle of 60 °. Reference pile huddled in a predrilled hole lead free drop hammer weight of 4 kN. Harvesting will be stopped at the number of strokes over the last 50 to 0.1 m dive.

A hydraulic thruster with the load-carrying capacity of 1000 kN was used as a loading device. Each load step was registered by a manometer.

Pile loading was done evenly by a pumping unit with load steps equal 20 kN. The rate of settlement 0.1 mm per hour was accepted as the criterion of conditional stabilization. The final criteria of the static loading reference pile test included condition: the general pile settlement could not be less than 0.02 m.

Thereafter, to assess the limit of resistance of the soil under the tip of the pile carried separately indentation cone of the reference pile, and then the indentation of reference pile shaft.

3.2.2 *Static loading full-scale pile test*

On the site were carried out 4 static loading full-scale pile tests. The soil under the pile tip – argillite clays and sandstone (see Figure 1).

Full-scale piles were driven, reinforced concrete of solid square section 0.3x0.3 m. The length of full-scale piles was 8-10 m.

A hydraulic thruster with the load-carrying capacity of 2000 kN was used as a loading device at this time. Each load step was registered by a manometer.

Pile loading was done evenly by a pumping unit with load steps equal 100 kN.

The rate of settlement 0.1 mm per hour was accepted as the criterion of conditional stabilization. The final criteria of the static loading reference pile test included condition: the general load could not be less than design values 1100 kN and 1200 kN correspondingly.

4 IN-SITU TESTS RESULTS

According to test results were calculated values of ultimate pile tip resistance in accordance with the requirements of SP 24.13330.2011.

According to the results of CPT the ultimate pile tip resistance R_{CPT} was calculated using average value of measured tip resistances q_s of the cone according to the formula:

$$R_{CPT} = \beta_1 q_s \qquad (5)$$

Where β_1 - is a correlation coefficient between q_s and R_{CPT}.

According to the results of static loading reference pile tests the ultimate pile tip resistance R_{RP} was calculated using tip resistance measured value R_{sp} of the reference pile according to the formula:

$$R_{RP} = \gamma_{cR} R_{sp} \qquad (6)$$

Where γ_{cR} is a correlation coefficient between R_{sp} and R_{RP}.

According to the results of static loading full-scale pile tests the ultimate pile tip resistance R_{FP} was back-calculated using the equation (1), obtained bearing capacity of the full-scale pile F_d using formula:

$$R = (F_d / \gamma_c - u \sum \gamma_{cf} f_i h_i) / \gamma_{cR} A \qquad (7)$$

Analysis of the calculation results is shown in Table 3 (also see Figure 1), as a comparison, the table shows the pile tip bearing capacity calculation according to Hansen`s method.

Table 3. Pile tip bearing capacity calculation results based on in-situ tests.

In-situ tests	R_i (kPa)			
	Bh-1	Bh-2	Bh-3	Bh-4
Full-scale pile	10170	10770	10170	10940
Reference pile	5680	5680	5680	5680
CPT	4200	4496	3145	4304
Hansen	3928	7926	3527	3338

As the table shows the ultimate pile tip resistance calculated from full-scale pile tests significantly higher than those obtained by reference pile tests and CPT.

In front of this ultimate pile tip resistance calculated from reference pile tests and CPT give good agreement with the analytical methods, such as Hansen`s method.

Comparing the results of static loading reference pile test and CPT it becomes clear that β_1 correlation coefficient value between q_s and R_{CPT} may be increased. However this correction requires more in-situ tests data.

5 CONCLUSION

The paper reviewed results of ultimate pile tip resistance calculations by different analytical approaches and in-situ tests data. Analysis of these results follows conclusions:

1. For an initial assessment of the ultimate pile tip resistance can be used method of B.J. Hansen.
2. If there are anisotropy of soils Vesic method, taking into account the horizontal deformation, can be used.
3. Analytical approaches gives the closest R values to the calculations results based on CPT data.
4. Comparison between results of static loading reference pile test and CPT shows that correlation coefficient β_1 value may be adjusted.
5. Back-calculating of the ultimate pile tip resistance using bearing capacity formula from SP 24.13330.2011 shows significantly higher results than calculating results those obtained by reference pile tests and CPT. Therefore this approach shouldn't be used even as initial assessment.

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