Study of the bored pile side resistance in rock

Estudio de la resistencia lateral del pilote perforado en roca

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ABSTRACT. Bored piles embedded into rock or cutting through them are often used in foundations of high-rise buildings. As practice shows, the bored piles resistance is mainly realized by the side surface. Pile side surface resistance values depend on rock type, rock strength, and strain level during testing. Most bored pile resistance techniques proposed in literature take into account unconfined compressive rock strength and do not consider the pile strain under load and regional features of rock soils. The article contains the analysis of results over 200 large-scale pile-load tests in various rock soils on construction sites in Russia, the USA, Canada, Hong Kong, Turkey, and other regions. Test loads were as high as 33.3 MN. A strain index classification of piles as a function of differential pile displacement was proposed. The ranges of mobilized side resistance of piles for different strain index were established. The dependence of side resistance of bored piles as a function of unconfined compressive rock strength and strain value was obtained.

KEYWORDS: bored pile, load transfer, side resistance, statistical analysis

1 INTRODUCTION

The bearing capacity of bored piles takes into account their side surface and base resistances. As pile testing practice shows, the bored piles resistance is mainly realized by the side surface. The base resistance is realized after significant pile displacement followed by side surface failure. Horvath et al., 1979, 1983 showed that mobilization of the side surface resistance of large diameter piles could be achieved when the displacements equal 5-6 mm. Ng et al., 2001 showed that at $S/D \leq 0.4$ % an elastic socket (pile) behavior at working load is observed. Furthermore, load-displacement relationship becomes nonlinear and pile side resistance is fully mobilized at $S/D \approx 1\%$. However, the pile base resistance starts to mobilize at $S/D \approx 5\%$ (O'Neill et al., 1996). Numerous pile test results show that such displacement values are almost never implemented in rocks. In this regard, the bearing capacity of piles in rock masses is obtained from their side surface resistance.

The article contains the analysis of results over 200 large-scale pile-load tests in various rock soils on construction sites in Russia, the USA, Canada, Hong Kong, Turkey, and other regions. Test loads were as high as 33.3 MN. A strain index classification of piles as a function of differential pile displacement was proposed. The ranges of mobilized side resistance of piles for different strain indices were established. The dependence of side resistance of bored piles as a function of rock uniaxial compressive strength and strain value was obtained.

2 DETERMINATION OF THE SIDE SURFACE RESISTANCE OF BORED PILES

The side surface resistance of bored piles in rock f_s can be found according to the following simple relationship:

$$\frac{f_s}{p_a} = \alpha \left(\frac{R_c}{p_a}\right)^m \quad (1)$$

where R_c is unconfined compressive strength, p_a = 0.1 MPa is atmosphere pressure, α and m are empirical coefficients.

This empirical relationship was proposed by Rosenberg and Journeaux, 1976 on the base of large-scale pile tests. Horvath, 1978 analyzed databases of pile and anchor test results mainly performed in sedimentary rocks. Later Horvath et al., 1983 proposed a relationship between side resistance f_s and the strength of weaker material (pile concrete or rock) based on experimental data analysis. Rowe and Armitage, 1984 analyzed more than 70 results of bored pile testing with diameters ranging from 0.064 m to 1.300 m. On the base of dataset (Rowe and Armitage, 1984), Carter and Kulhawy, 1988 obtained empirical coefficients α and m as a lower bound with significance level 0.9. Reese and O'Neill, 1999 analyzed described dependencies above and suggested their correlation. Zhang and Einstein, 1998 also reviewed the available databases and the correlations between f_s and R_c obtained by other authors and proposed their coefficients. Zhang and Einstein, 1999 complemented their study, in consequence of which the correlation approached coefficients (Carter and Kulhawy, 1988). Ng et al., 2001 reviewed the available databases as well as conducted some tests on bored piles in Hong Kong, which allowed them to propose their own relationship. Kulhawy et al., 2005 performed another evaluation of the above dependencies and concluded that most authors had m=0.5 and based on their additional research they estimated the coefficient α=1.0. Table 1 summarizes dependencies presented by the authors mentioned above.

Table 1. Empirical coefficients α and m for equation (1)

Authors	α	m
Rosenberg и Journeaux, 1976	1.19	0.52
Horvath, Kenney, Kozicki, 1983	0.63-0.95	0.5
Rowe и Armitage, 1984	1.42	0.5

Carter и Kulhawy, 1988	0.63	0.5
Reese и O'Neill, 1999	0.65	0.5
Zhang и Einstein, 1998	1.26	0,5
Zhang и Einstein, 1999	0,63	0.5
Ng, Yau, Li, 2001	0.6	0.5
Kulhawy, Prakoso, Akbas, 2005	1	0.5

As analysis shows, most of the existing methods allow estimating side surface resistance of piles depend on unconfined compressive rock strength and do not consider the pile strain. Practice shows that pile tests in rocks are performed at rather low loads which does not allow to estimate their real (maximum) bearing capacity. In this regard, most pile solutions have significant design margins and open up the opportunities for project optimization.

3 GENERAL DESCRIPTION OF INVESTIGATED ROCKS AND PILES

Experimental data included the results of a series of pile tests on compression and pull-out loads collected at different construction sites in Russia, Turkey, the USA and other countries. Table 2 shows the rock and pile characteristics of different construction sites. The following rocks were carried out in this study: shale, limestone, sandstone, gabbro, granite, porphyrite, etc. The unconfined compressive strength was 0.23-55.0 MPa. The tested bored piles were 0.43-1.83 m of diameter and 0.5-22.2 m of embedded length. Test loads were as high as 33.3 MN. The maximum displacement of piles S was 0.12-130.0 mm which allowed to measure mobilized side surface resistance f_{sm} = 0.01-5.40 MPa.

4 METHODS AND RESULTS

The correlation and regression statistical data analysis technique was employed using MS Excel and IBM SPSS Statistics. The following bearing capacity parameters were analyzed:

- mobilized side surface resistance f_{sm};
- normalized mobilized side surface resistance f_{sm}/p_a;
- ratio of mobilized side surface resistance to unconfined compressive strength f_{sm}/R_c;
- ratio of pile displacement to pile embedment in rock (relative pile strain) S/L_r;
- the ratio of pile displacement to pile diameter S/D.

Experimental studies were performed in two phases.

At the first stage, the experimental data were incorporated into the total sample. The Pearson's correlation coefficient ρ, significance level (using P-value) and sample correlation ratio η

Table 2. Summary of pile test results in rocky soils summarized by the authors

Site location	Rock type	Rock	N	d, m	L, m	R_c, MPa	Q, MN	S, mm	f_{sm}, MPa
Russia, Ekaterinburg	Magmatic	Gabbro	9	0.8	0.50-4.50	6.3-27.8	2.7-4.4	8.04-54.56	0.24-2.90
Russia, Ekaterinburg		Granite	33	0.62-1.0	0.51-5.19	2.2-37.1	4.1-4.4	0.12-21.69	0.06-3.61
Russia, Ekaterinburg		Porphyrite	12	0.43-0.8	0.50-2.06	2.00-25.20	0.4-4.4	4.71-51.81	0.06-2.91
Russia, Ekaterinburg	Metamorphic	Shale	31	0.62-0.8	1.4-22.2	2.5-28.4	0.6-7.4	0.84-111.16	0.03-1.58
Russia, Moscow	Sedimentary	Limestone	36	0.9-1.5	0.6-14.8	3,0-55,0	12.0-33.3	1.33-130.0	0.10-5.40
Russia, Kemerovo		Sandstone	4	0.75	6.0-7.6	28.5	7.1-7.3	5.2-11.85	0.40-0.50
Turkey (Akguner and Kirkit, 2012)	Sedimentary, metamorphic	Argillite, marl, sandstone, shale, phyllite	7	0.8-0.9	1.5-16.0	0.8-2.2	5.5-12.0	3.05-42.1	0.09-1.18
USA (Castelli and Fan, 2002)	Sedimentary	Limestone, marl	5	0.92-1.83	4.75-6.58	0.7-7.5	-	5.8-24.0	0.38-1.24
USA (Brown D.A., Turner J.P., Castelli R.G., 2010)	Sedimentary	Siltstone, argillite, sandstone	25	0.51-1.83	0.6-11.0	0.15-54.9	-	0.51-53.09	0.1-2.6
Great Britain, USA, Australia, Canada (M.W. O'Neill et al, 1996)	Magmatic, sedimentary, metamorphic	Siltstone, basalt, chalk, marl, shale, argillite, sandstone	63	0.46-1.60	0.9-18.0	0.23-14.74	-	0.76-10.16*	0.01-1.51

Note: n – number of tests; d – diameter; L – rock embedment; R_c – unconfined compressive rock strength; Q – maximum pile test load; S_{max} – maximum displacement of the tested pile; f_s – measured mobilized side surface pile resistance; *– precipitation is given for 50% of the maximum load

(Gmurman V., 2004) were calculated via statistical analysis.

The Pearson's correlation coefficient ρ is widely used in statistical analysis. It evaluates the correlation relationship among the parameters and lies in the ranges from -1 to +1. The closer its value is to +1 (or −1), the stronger is the degree of linear relationship between parameters. If the ρ value is close to zero, it indicates a weak linear strength of relationship.

The correlation parameter η is the ratio of a between-group dispersion to the total dispersion. It estimates the strength of the non-linear correlation relation between the parameters and ranges from zero to one. If η is close to zero, the strength of relationship is weak or does not exist; if it is close to one, the relationship is strong. The correlation ratio and the Pearson's correlation coefficient satisfy the condition η≥ρ.

The correlation analysis revealed the most significant factors and nature of the relationship (linear or non-linear). Relationship in correlation interaction was analyzed at the significance level α=0.05. This corresponds to GOST 20522 requirements for calculating soil safety factor.

The following factors that can affect the bearing capacity of bored piles in rocks were analyzed:

- unconfined compressive rock strength R_c;
- normalized unconfined compressive rock strength R_c/p_a;
- relative depth of pile embedment in rock L_r/D;
- pile displacement in rock S;
- ratio of pile displacement to pile embedment in rock S/L_r;
- ratio of pile displacement to pile diameter S/D.

The main results of correlation analysis for the total data sample are shown in Table 3. It can be noted that the relationships between the analyzed parameters are significant and highly non-linear.

Table 3. Correlation parameters for the total sample

Bearing capacity parameter		Factor					
		R_c	R_c/p_a	L_r/D	S	S/D	S/L_r
f_{sm}	ρ	0.720*	0.720*	-0.462*	0.110	0.075	0.386*
	η	0.937	0.937	0.974	0.938	0.950	0.999
f_{sm}/p_a	ρ	0.720*	0.720*	-0.462*	0.110	0.075	0.386*
	η	0.937	0.937	0.974	0.938	0.950	0.999
f_{sm}/R_c	ρ	-0.315*	-0.315*	-0.118	-0.086	-0.095	0.070
	η	0.981	0.981	0.790	0.901	0.935	0.997
S/L_r	ρ	0.191*	0.191*	-0.328*	0.668*	0.616*	–
	η	0.754	0.754	0.841	0.967	0.969	–
S/D	ρ	0.028	0.028	-0.043	0.897*	–	–
	η	0.624	0.624	0.941	0.999	–	–

* – correlation is significant at the significance level α=0.05

Sample size N=228

The analysis revealed that unconfined compressive rock strength R_c and R_c/p_a (Fig. 1) as well as the length of embedment in rocky soils L_r/D greatly influence mobilized pile side surface resistance f_{sm} and f_{sm}/p_a. Non-linear behavior prevails here, and η exceeds ρ by 1.3-2.1 times.

The relationships between mobilized side surface resistance and pile displacements S and S/D are weak and not statistically insignificant. At the same time, the value of S/L_r affects the mobilized resistance f_{sm} and f_{sm}/p_a, and is also inversely proportional to its embedment in the rock L_r/D.

In general, the following conclusion can be drawn based on the performed analysis of the total sample:

- mobilized side surface resistance of piles depends on soil strength, the length of pile embedment in rock and the value of S/L_r ratio;
- pile displacement S and displacement-to-diameter ratio S/D insignificantly influence the mobilized pile resistance.

In the second stage, the influence of rock strength and pile displacement on mobilized side surface resistance of piles were studied in detail.

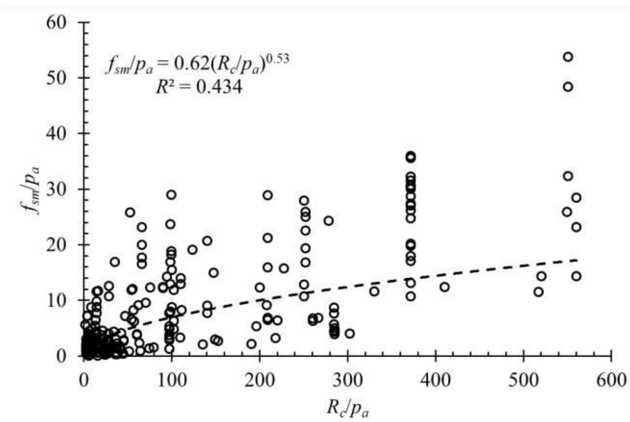

Figure 1. Relationship between normalized mobilized side surface resistance f_{sm}/p_a and normalized unconfined compressive strength R_c/p_a

Randolph, 1994 indicated that full side surface mobilization occurs at S/D =1 %. Taking into account the specified ratio, a tested pile classification on the base of displacement index (DI) was proposed by Ng C. et al., 2001. This is an approximate measure of the pile "local" displacement and degree of mobilization of side resistance for a given maximum side resistance value. Index A is assigned to pile tests at $S/D \geq 1$ % and observed f_{sm}/S ratio <30 kPa/mm. These values of f_{sm} which are close to the fully mobilized value of side resistance (Ng C. et al.,2001). Index B is assigned to the results at 0.4 % ≤ S/D < 1 % and with f_{sm}/S ratio < 200 kPa/mm. Index C is assigned to the remaining results. The disadvantage of this approach is that there are insignificant bored pile displacements in rock. Most of the strains occur due to the compression of pile material (especially in the case of relatively long piles) (Gotman A. and Gavrikov M., 2021). In addition, DI does not take into account the pile material strain. It is also not always acceptable to use f_{sm}/S for categorization as the strain rate when observed is often higher.

The correlation analysis showed that S/D ratio does not assess an significant influence on f_{sm}. In addition, the pile displacement and the measured f_{sm} resistance significantly depend on the length of pile embedment in the rock mass. According to the authors, the relative pile strain S/L_r is more suitable for categorizing the strain level. It is applied in classical definition of strain of a pile since the width of the interaction zone of a single pile with the soil is comparable to the length of its embedment (Randolph, 1994, Fleming K. et al., 2009). In this regard, the following pile strain classification (strain index SI) is proposed: I – $S/Lr \geq 1\%$; II – $0.5\ \% \leq S/Lr < 1\ \%$; III – remaining results. Such classification will enable one to assess pile bearing capacity more accurately and exclude data with insignificant strains.

Figure 2 shows the results of the joint effect of R_c/p_a and SI on f_{sm}/p_a values. The higher is S/L_r, the higher is the resistance f_{sm}/p_a; lower values of f_{sm}/p_a generally correspond to minimum values of strains. It indicates that the relationships based on these data have excessive safety reserves.

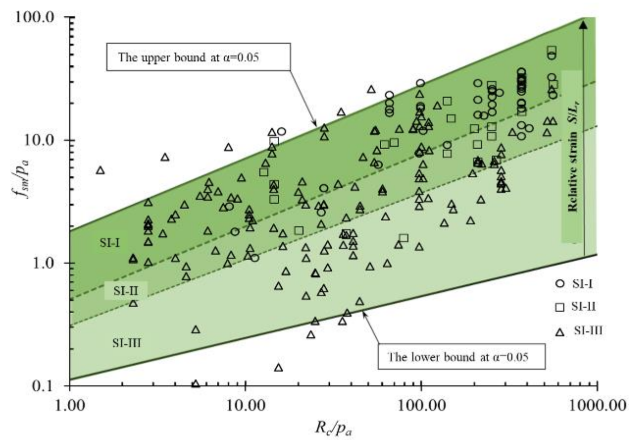

Fig. 2. Influence of normalized unconfined compressive strength R_c/p_a on normalized mobilized side surface resistance f_{sm}/p_a as a function of strain index

Figure 2 provides quantitative and qualitative assessment of the effect of R_c/p_a on the f_{sm}/p_a, depending on deformation level. The equation obtained may be useful for practical applications:

$$\frac{f_{sm}}{p_a} = \alpha\left(\frac{R_c}{p_a}\right)^m \quad (2)$$

where α and m are empirical coefficients. The values of α and m depend on strain index, their values are respectively 0.77 and 0.6 for SI-I (R^2=0.68); 0.75 and 0.54 for SI-II (R^2=0.51) and 0.93 and 0.34 for SI-III (R^2=0.22).

The obtained empirical coefficients do not fundamentally differ from those obtained by other authors (Table 1). However, the empirical coefficients and strain level increase simultaneously. Considering the processed data of large number of pile load tests performed in rocks, the following semi-empirical dependence was proposed:

$$\frac{f_{sm}}{p_a} = a\left(\frac{S}{L_r}\right)^b\left(\frac{R_c}{p_a}\right)^c \quad (3)$$

Empirical coefficients *a, b* and *c* depend on rock genesis and are given in Table 4.

Equation (3) provides side surface resistance of a pile depending on the expected pile strain level that can be calculated using established solutions (Fedorovsky V., 1974; Randolph M. and Wroth C., 1978). However, pile load testing is still necessary even with the above approach. Moreover, the dependence enables to reduce bearing capacity excessive reserves of piles that were used up in empirical dependencies obtained from underloaded pile testing results.

Table 4. Empirical regression coefficients for equation (3) depending on rock type

Empirical coefficients	Total sample	MG	MT	SD
a	2.81	3.89	5.88	3.4
b	0.223	0.225	0.201	0.185
c	0.526	0.474	-0.723	0.468
R^2	0.665	0.750	0.514	0.56
N	228	62	71	95

Rock type: MG – magmatic; MT – metamorphic; SD – sedimentary

5 DISCUSSION

Bearing capacity of bored piles as tested is often determined by mobilized side surface resistance value. The latter depends on the maximum load applied during testing and pile strain caused by loading.

The proposed strain index allows to analyze mobilized resistance and to estimate the pile's failure load level. Such index can also be used to evaluate f_{sm} of long and large-diameter piles in dispersed soils when the test load causes minor displacements.

It follows that bored pile bearing capacity in rock is often underestimated. This results in significant safety reserves when designing pile foundations and, as a consequence, to higher construction costs for the zero cycle works. In addition, the value of f_{sm} in rocks depends on pile embedment length. This is due to the fact that side stress distribution along the pile in rock does not exceed the specified limits of stress transfer depth. Once the limits are exceeded, side surface of piles practically ceases to transfer the load.

In order to exclude excessive stockpiles, it is advisable to determine the maximum value of f_{sm}. For this purpose, 2-3 m long pile fragments can be produced at construction sites. The pile fragments will be subsequently brought to failure. For example, such an approach was employed during the construction of the Ice Arena in Yekaterinburg (Sharafutdinov R. et al., 2022), which allowed reducing the number of piles by 2 times.

The proposed equation (3) can also be used to estimate the maximum value of f_{sm}. Considering pile displacement calculations via known methods, more accurate values of resistances can be iteratively determined. The dependence allows to get a better understanding of the load transfer through the borehole pile shaft on the basis of pile displacement and the embedment depth in rock.

6 CONCLUSIONS

This paper presents the analysis of results over 200 large-scale pile-load tests in various rock soils on construction sites in Russia, the USA, Canada, Hong Kong, Turkey, and other regions.

Based on correlation analysis it is established that mobilized side surface resistance of piles depends on soil strength, pile embedment length and differential pile displacement value S/L_r. At the same time, absolute displacement values S and displacement-to-diameter ratio S/D have little influence on the value of mobilized pile resistance.

A classification of bored piles by strain index depending on differential pile displacement is proposed: SI-I - $S/L_r \geq 1\%$; SI-II - $0.5\ \% \leq S/L_r < 1\ \%$; SI-III - other results. Such classification enables to assess accurately the bearing capacity of piles in rocks and exclude underestimated values of f_{sm} with insignificant strains. The ranges of mobilized side surface resistance of piles for various strain indices are established.

The combined dependence to estimate mobilized side surface resistance considering rock strength and pile strain is proposed. The dependence allows to get a better understanding of load transfer through borehole lateral walls in consideration of pile displacement and the embedment depth in rock.

7 ACKNOWLEDGEMENTS

The authors acknowledge the staff of the Laboratory No. 35 of Gersevanov Research Institute of Bases and Underground Structures (NIIOSP) for aid and the materials provided.

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