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DETERMINATION OF THE BEARING CAPACITY OF PILE FOUNDATIONS BASED ON CPT TEST RESULTS

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Abstract: The method implementing CPT results in order to determine pile's bearing capacity is presented. In this method, the shaft and base bearing capacities contribute to the determination of whole load–settlement curve making use of respectively chosen load–transfer function. The method has been developed on the basis of filed measurement of piles being applied to loading tests as well as CPT testing in the area of piling. Totally, tests for 94 Vibro and large bored piles installed under differentiated soil conditions were analyzed. The test results have been interpretated statistically by estimating the method parameters and by verification of various statistical hypotheses.

1. INTRODUCTION

Reliable assessment of the work of engineering constructions transferring the loads into the subsoil is still a serious challenge from both theoretical and engineering points of view. A deformable construction interacts with flexible subsoil generating the respective cross-section forces (bending moments, shear and normal forces) that are strictly related to mechanical properties of the soil. During the foundation process, particularly for deep foundations, some of geotechnical parameters change. This mostly concerns pile foundations, where in a direct vicinity of piles, geotechnical parameters may improve or get worse, depending of the technology applied. The geotechnical reconnaissance is usually carried out prior to foundation works, thus becomes additional problem in engineering calculations. Recently, it is assumed that the best results of soil properties may be achieved based on in situ tests. Such a situation can also be observed in Poland where in situ investigations of the subsoil are widely conducted. The best argument for the last statement is the present conference. Coherent summary of such investigations can be found, e.g., in the paper by TSCHUSCHKE [11].

The most common in situ investigations are the following: dynamic penetration tests, static cone penetration tests (CPT), standard penetration tests (SPT), dilatometer tests (DMT), pressuremeter tests (PMT), vane shear tests (VST), and geophysical tests. Despite some disadvantages, recently in Poland the design of pile foundations is mostly based on CPT tests.

In the analysis presented in the paper, two opposite technologies are considered, i.e., full displacement piles of Vibro-Fundex and Vibrex type versus large diameter bored piles with soil extraction. In both cases concerned, static penetration tests were carried out prior to pile installation.

In the paper, the method used for the assessment of bearing capacity of piles on the basis of the results of static penetration tests (CPT) is presented. It allows the determination of shaft and base bearing capacities of the piles as well as it enables the construction of a complete load–settlement curve using the respective load–transfer functions.

The method proposed has been developed based on in situ investigations consisting of load tests on piles supplemented by CPT tests made at the place of pile installation. In the analysis, totally 94 results of tests on Vibro-Fundex, Vibrex and large diameter bored piles installed under various soil conditions were applied.

The test results used for verification of the assumptions were subjected to statistical inference [10], which covered two basic procedures, i.e., estimation of the parameters and verification of statistical hypotheses.

The estimation of the parameters, i.e., an assessment of expected value, variance and standard deviation, was made using the method of confidence intervals at the defined probability (confidence level).

The results obtained by correlation method were next verified using the procedure of the verification of statistical hypotheses. It consisted in the verification of the assumptions accepted for both the parameters investigated (parametrical hypotheses) and shapes of distributions (non-parametrical hypotheses). The hypotheses were verified in terms of significance and consistency tests for the assumed probability (significance level).

An assessment of the parameters of linear and non-linear regression functions was carried out by regression analysis. In order to determine the influence of significant factors on the parameters of load–transfer functions, the multiregression analysis was made (constructing the pairwise correlation matrix of coefficients and partial correlation matrices).

2. PILE BEARING CAPACITY

A direct determination of the bearing capacity of piles is based on the cone resistance of the penetrometer (q_c) from CPT test. Bearing capacity of the pile can be determined according to equation (1), whereas unit ultimate resistance under the base and along the shaft of the pile – from equations (2) and (3), respectively [4]:

$$R_u = R_{bu} + R_{su} = A_b \cdot q_{bu} + \sum A_{si} \cdot q_{sui} \quad \text{[MN]}, \tag{1}$$

$$q_{bu} = \psi_1 \cdot \overline{q}_c \quad \text{[MPa]}, \tag{2}$$

$$q_{sui} = \frac{\overline{q}_{csi}}{\psi_{2i}} \quad [\text{MPa}], \tag{3}$$

where:

R_u – ultimate load of the pile head [MN], corresponding to the virtual settlement of pile base,

R_{bu} – ultimate soil resistance under the pile base [MN],

R_{su} – ultimate soil resistance along the pile shaft [MN],

q_{bu} – unit, ultimate soil resistance under the pile base [MPa],

$\overline{q}_c$ – average unit cone resistance of the penetrometer at the pile base [MPa],

q_{sui} – unit, ultimate soil resistance along the pile shaft within the i-th calculation layer [MPa],

$\overline{q}_{csi}$ – average, unit cone resistance of the penetrometer within the i-th calculation layer [MPa],

A_b – surface of the pile base [m^2],

A_s – surface of the pile shaft [m^2],

ψ_1 – bearing capacity factor of the base,

ψ_{2i} – bearing capacity factor for the i-th calculation layer.

In order to divide a subsoil into the calculation layers, the Harder–Bloh procedure can be used [7]. Filtration of the direct CPT readings is made by stepwise statistical analysis which enables us to choose uniform soil layers from the CPT test results.

2.1. UNIT ULTIMATE SOIL RESISTANCE (q_{bu}) UNDER THE PILE BASE

2.1.1. AVERAGED, UNIT CONE RESISTANCE OF THE PENETROMETER

Average, unit cone resistance ($\overline{q}_c$) of the penetrometer within the zone near the pile base is determined according to the following formula:

$$\overline{q}_c = \frac{1}{l_1 + l_2} \int_{h-l_1}^{h+l_2} q_c(h)dh \quad [\text{MPa}]. \tag{4}$$

In the method proposed, the ranges of zones l_1 and l_2 are estabilished on the basis of the schemes dependent on the arrangement of the soil layers near the pile base. Totally, three basic schemes have been distinguished [5]:

1. Scheme I: $l_1 = 4D_b$, $l_2 = 1D_b$ (where D_b is the diameter of the pile base). Scheme I is divided into

Ia – uniform soil.

Ib – non-uniform soil (the base is sunk in the soil of higher cone resistances and above there are weaker soils).

Ic – special case of the scheme I: (the base is sunk in the soil of higher cone resistances and above there are very weak soils such as mad and peat; in such a case, the range of the zone l_1 does not contain very weak soil).

2. Scheme II: $l_1 = 2D_b$, $l_2 = 4D_b$; non-uniform soil (the base is sunk in the soil of lower cone resistances, whereas above there is the soil of higher parameters).

3. Scheme III: $l_1 = 4D_b$, $l_2 = 4D_b$; non-uniform soil (the base is sunk in the soil of higher cone resistances, whereas above and below there are soil layers of lower parameters).

2.1.2. BEARING CAPACITY FACTOR OF PILE BASE

The factor of the bearing capacity of the pile base ψ_1 is determined in terms of $\psi_1(\overline{q}_c)$ function which has been assumed based on the selection of the respective regression function model. Linear and non-linear regression functions are considered. The latter may be directly transformed into the linear form or transformed in terms of finding the logarithm. Finally, the power function was assumed which possessed the highest correlation factor, the highest determination level and the lowest loss function (assumed as a sum of square residuum deviations).

2.1.2.1.Vibro-Fundex and Vibrex piles

The value of base bearing capacity factor ψ_1 is determined according to equations (5) and (6), figure 1 (P_A = 1.0 MPa):

$$\psi_1 = 1 \quad \text{for} \quad \frac{\overline{q}_c}{P_A} \le 4, \tag{5}$$

$$\psi_1 = 1.901 \cdot \left(\frac{\overline{q}_c}{P_A}\right)^{-0.455} \quad \text{for} \quad 4 < \frac{\overline{q}_c}{P_A} \le 40. \tag{6}$$

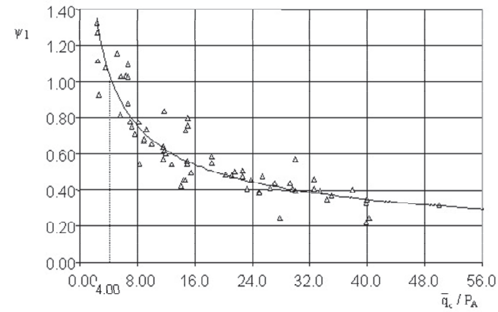

Fig. 1. Relation between ψ_1 and $\frac{\overline{q}_c}{P_A}$ for Vibro piles

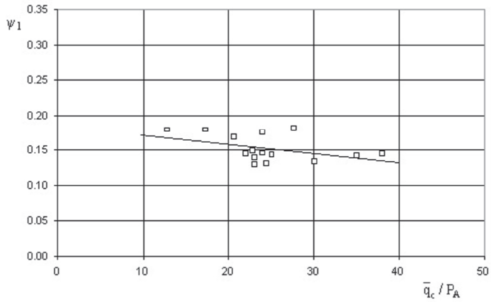

Fig. 2. Relation between ψ_1 and $\frac{\overline{q}_c}{P_A}$ for large diameter bored piles

In the case of Vibrex piles, the factor ψ_1 should be additionally increased by 10%, which results from somewhat different technology of pile installation. This was confirmed by verification analysis making use of consistency coefficients determining the ratio of calculated to measured magnitudes.

2.1.2.2. Large diameter bored piles

The value of base bearing capacity factor ψ_1 is determined according to the following formula (figure 2, $P_A = 1.0$ MPa):

$$\psi_1 = -0.001\frac{\overline{q}_c}{P_A} + 0.177 \quad \text{for} \quad 10 \le \frac{\overline{q}_c}{P_A} \le 40\,. \tag{7}$$

2.2. UNIT ULTIMATE SOIL RESISTANCE (q_{su}) ALONG THE PILE SHAFT

The values of q_{sui} established according to the method proposed should be taken at the depth of 5.0 m and greater (measured from the soil surface). At the depths smaller than 5.0 m, the values q_{sui} should be determined using interpolation procedure between zero and the values obtained from equation (3). If very weak soil occurs directly below the soil surface, a substituted interpolation level can be determined according to [8].

2.2.1. AVERAGED UNIT CONE RESISTANCE OF THE PENETROMETER

Averaged unit cone resistance $\overline{q}_{csi}$ of the penetrometer is assumed for the i-th calculation layer as:

$$\overline{q}_{csi} = \frac{1}{\Delta h}\int_{h_{i-1}}^{h_i} q_c(h)\cdot dh \quad [\text{MPa}]. \tag{8}$$

2.2.2. BEARING CAPACITY FACTOR OF THE PILE SHAFT

Bearing capacity factor ψ_2 of the pile shaft has been taken in the form of linear load–transfer functions [ψ_2 ($\overline{q}_{cs}$)], depending on the type of the pile and the soil.

In order to carry out the statistical analysis of the significance of both regression coefficient and a free term, the linear regression function parameters were evaluated. The evaluation was carried out using critical significance level assumed by t-Student's distribution.

2.2.2.1. Vibro-Fundex and Vibrex piles

Bearing capacity factor ψ_2 of the pile shaft values is determined from equations (9)–(12), depending on the soil type (figure 3):

- for clays, sandy clays as well as clayey sands (C, CS, SC) within the range of $0.5 \le \frac{\overline{q}_{cs}}{P_A} \le 12.0$:

$$\psi_2 = 17.96 \cdot \frac{\overline{q}_{cs}}{P_A} + 9.38\,; \tag{9}$$

- for silty sands (S-M) within the range of $4.0 \le \frac{\overline{q}_{cs}}{P_A} \le 40.0$:

$$\psi_2 = 10.30 \cdot \frac{\overline{q}_{cs}}{P_A} + 82.57\,; \tag{10}$$

- for fine sands within the range of $4.0 \le \frac{\overline{q}_{cs}}{P_A} \le 40.0$:

$$\psi_2 = 7.0 \cdot \frac{\overline{q}_{cs}}{P_A} + 86.34\,; \tag{11}$$

- for coarse and medium sands within the range of $4.0 \le \frac{\overline{q}_{cs}}{P_A} \le 40.0$:

$$\psi_2 = 5.91 \cdot \frac{\overline{q}_{cs}}{P_A} + 58.63\,. \tag{12}$$

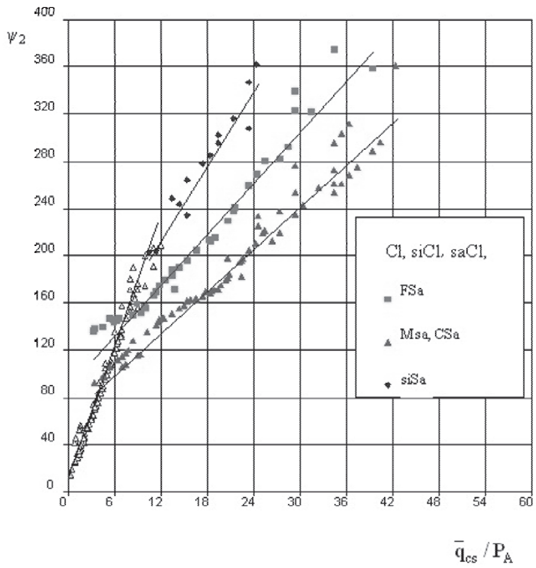

Fig. 3. ψ_2 versus $\frac{\overline{q}_{cs}}{P_A}$ for Vibro piles

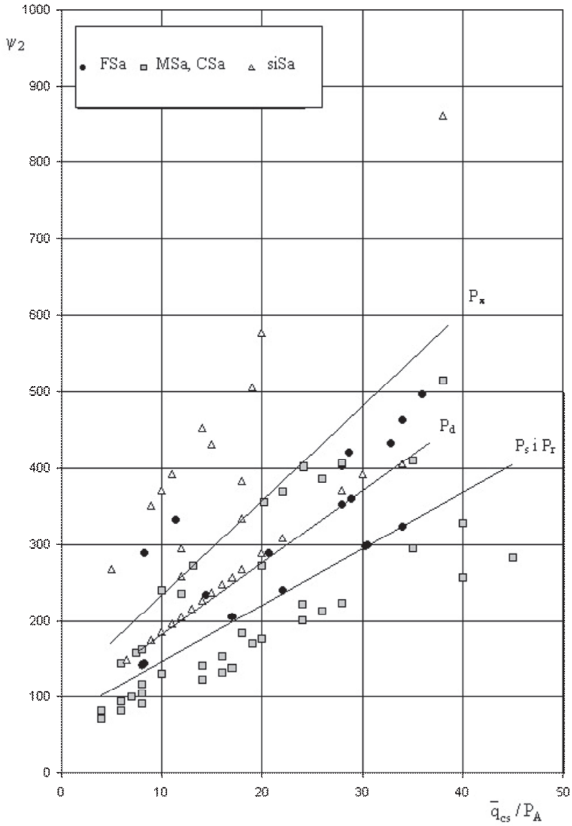

Fig. 4. ψ_2 versus $\dfrac{\bar{q}_{cs}}{P_A}$ for large diameter bored piles

2.2.2.2. Large diameter bored piles

The value of bearing capacity factor ψ_2 of the pile shaft is calculated from equations (13)–(16), depending on the soil type. The application range of the equations varies: $4.0 \le \dfrac{\bar{q}_{cs}}{P_A} \le 40.0$, (figure 4):

- silty sand (S–M):

$$\psi_2 = 12.40 \frac{\bar{q}_{cs}}{P_A} + 109.77\,; \tag{13}$$

- fine sand (S):

$$\psi_2 = 9.41 \frac{\bar{q}_{cs}}{P_A} + 88.15\,; \tag{14}$$

- coarse and medium sand for the diameter $D \geq 1.5$ m:

$$\psi_2 = 10.32 \frac{\bar{q}_{cs}}{P_A} + 125.57\,; \tag{15}$$

- coarse and medium sand for the diameter $D < 1.5$ m:

$$\psi_2 = 6.34 \frac{\bar{q}_{cs}}{P_A} + 68.90\,. \tag{16}$$

3. DETERMINATION OF THE LOAD–SETTLEMENT CURVE

In the method proposed for a determination of load–settlement curve of the pile, depending on the load level, load–transfer functions are used [2]. The functions which can be applied to elastic pile installed in arbitrarily layered subsoil are curvilinear functions describing the dependence of pile shaft resistance on the displacement of any pile point (the curves *t–z*) and the dependence of the resistance of the pile base on its displacement (the curve *q–z*). Complete load–settlement curve may be constructed using both functions *t–z* and *q–z* together with pile internal deformability.

For the shaft of the pile the power function was assumed:

$$t = q_{su} \cdot \left(\frac{z}{z_v} \right)^{\alpha} \quad \text{for} \quad z \leq z_v, \tag{17}$$

where:

t – the shaft resistance,

z – the shaft displacement,

z_v – the pile displacement at which mobilisation of maximum friction resistance along the shaft takes place (defined as a percentage of the pile's diameter along the shaft).

The coefficient α which is an exponent of the power function for the pile shaft was preliminary defined as the function of the soil and pile type. After selecting the pile groups for which the shaft was embedded in soil characterized by similar conditions, mean values of the coefficient α were assumed. In order to check whether the differences between mean values for various pile groups are due to different soils and pile types or rather due to by random factors, the verification procedure of the parametrical hypotheses was carried out. For that purpose an evaluation of the significance for mean values by *t*-Student's test and the significance of the variance by Fisher–Snedecor's test were carried out.

In order to increase the accuracy of the method applied (by a decrease of the scatter around the mean value), the influence of significant factors on the value α was established using multi-regression. In the above procedure, a proper selection of explanatory variables, which should be statistically significantly related to explained variable (the coefficient α in our case) and simultaneously should not be statistically interrelated to each other, was of a great importance. To do that the correlation matrices of coefficients for pairs of variables were constructed eliminating those explanatory variables, whose correlation coefficients were insignificant for relations with explained variable (the coefficient α) and simultaneously significant for relationships between explanatory variables (in the case of searching for multi-regression function).

Since the correlation coefficients which are the measure of covariance are not always a measure of a factual relationship between variables, additionally partial correlation matrices have been constructed. The latter allowed us to find hidden relationships between the variables analysed and to show some apparent relationships (partial correlations determine correlations between the pairs of variables when the other variables are controlled).

The parameters being easily accessible during the design stage are assumed to be the factors which may exert an essential influence on the value α. For individual groups of piles (separated as a function of pile and soil type along the shaft) geometrical parameters of piles and their bearing capacities together with combinations of both were being considered.

For the base of pile the following power function is assumed:

$$q = q_{bu} \cdot \left(\frac{z}{z_f} \right)^{\beta} \quad \text{for} \quad z \le z_f, \tag{18}$$

where:

q – the base resistance,

z – the base displacement,

z_f – the base displacement causing maximum mobilisation of soil resistance under the base (defined as a percentage of pile base diameter).

The coefficient β being the exponent of the power function q–z has been determined analogically as the coefficient α (considering the respective factors that significantly influence its value in multi-regression process).

3.1. VIBRO-FUNDEX AND VIBREX PILES

The displacement z_v of pile was assumed as corresponding to 3% of pile diameter, $z_v = 0.03D$.

For Vibro-Fundex piles, whose shaft is installed in cohesive soils, the coefficient α can be defined in the following way:

$$\alpha = 0.654 - 0.809 \cdot \frac{R_{su}}{R_u} \quad \text{for} \quad \frac{R_{su}}{R_u} < 0.5, \tag{19}$$

$$\alpha = 0.250 \quad \text{for} \quad \frac{R_{su}}{R_u} \geq 0.5\,. \tag{20}$$

For Vibro-Fundex piles, whose shaft is installed under varying soil conditions (in cohesive and non-cohesive soils), the coefficient α is equal to 0.243.

For Vibro-Fundex and Vibrex piles, whose shaft is installed in non-cohesive soils, $\alpha = 0.190$.

The displacement z_f of the pile base was assumed as corresponding to 10% of base diameter, $z_f = 0.10D_b$.

For Vibro-Fundex piles, whose base is installed in cohesive soils, the coefficient β can be expressed by:

$$\beta = 0.671 - 0.104 \cdot \frac{R_u}{R_B} \quad \text{for} \quad \frac{R_u}{R_B} < 5.0, \tag{21}$$

$$\beta = 0.150 \quad \text{for} \quad \frac{R_u}{R_B} \geq 5.0, \tag{22}$$

where R_B is expressed in MN (R_B = 1.0 MN).

For Vibro-Fundex and Vibrex piles, whose bases are installed in non-cohesive soils, the coefficient β can be defined as follows:

$$\beta = 0.370 - 0.257 \cdot \frac{R_{bu}}{R_u}\,. \tag{23}$$

3.2. LARGE DIAMETER BORED PILES

The displacement z_v of pile was assumed to be equal to 0.015 m.

The coefficient α can be determined from the following formula (H_c = 1.0 m):

$$\alpha = 0.604 - 0.005\frac{Dh}{H_c^2}\,. \tag{24}$$

The displacement z_f of the pile base was assumed as corresponding to 10% of base diameter: $z_f = 0.1D_b$ for $D_b \leq 1.0$ m and $z_f = 0.10$ m for $D_b > 1.0$ m.

The coefficient β is defined according to the following formula:

$$\beta = 0.388 + 0.016\frac{R_u}{R_B}, \tag{25}$$

where R_u is given in MN (R_B = 1.0 MN).

The values higher than $\beta_{\max} = 0.700$ should not be considered.

4. PRACTICAL APPLICATION OF THE METHOD FOR DETERMINATION OF LOAD–SETTLEMENT CURVE

The method has been exemplified by three piles:

• Vibro-Fundex pile, 22.0 m long, 0.508 m in diameter along the shaft and 0.65 m in diameter at the base (figure 5).

• Vibrex pile, 18.0 m long, 0.457 m in diameter along the shaft and 0.65 m in diameter at the base (figure 6).

• Large diameter bored pile, 17.0 m long, 1.5 m in diameter along the shaft (figure 7).

In figures 5–7, predicted and factual load–settlement curves together with bearing capacity distributions are shown.

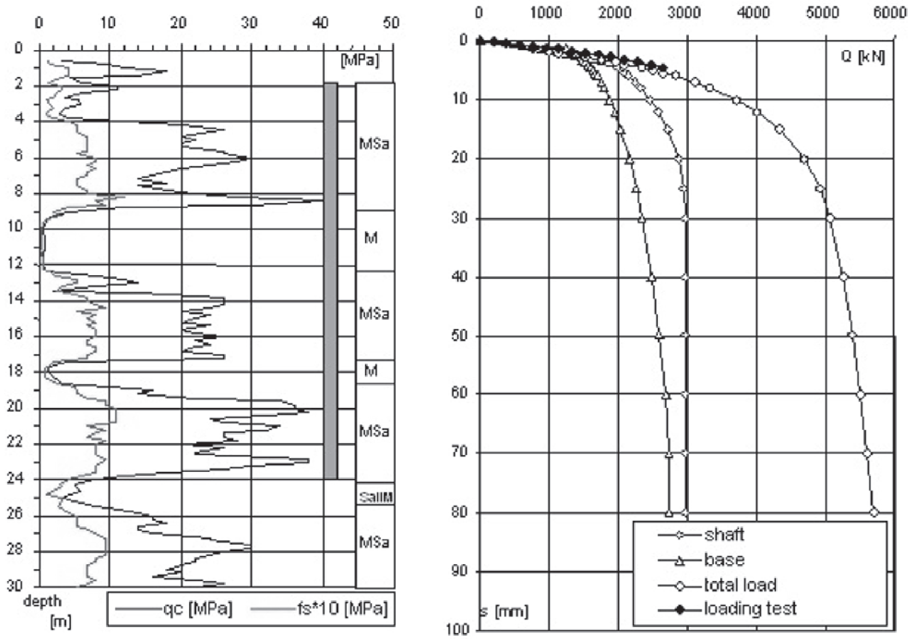

Fig. 5. Load–settlement curve representing Vibro-Fundex pile,
$h = 22.0$ m, $D = 0.508$ m, $D_b = 0.650$ m

5. SUMMARY

The method presented in this paper allows the determination of load–settlement curve of the pile until the pile achieves its ultimate bearing capacity and potential deviations from its expected value occurs. Additionally, a complete relation of load–

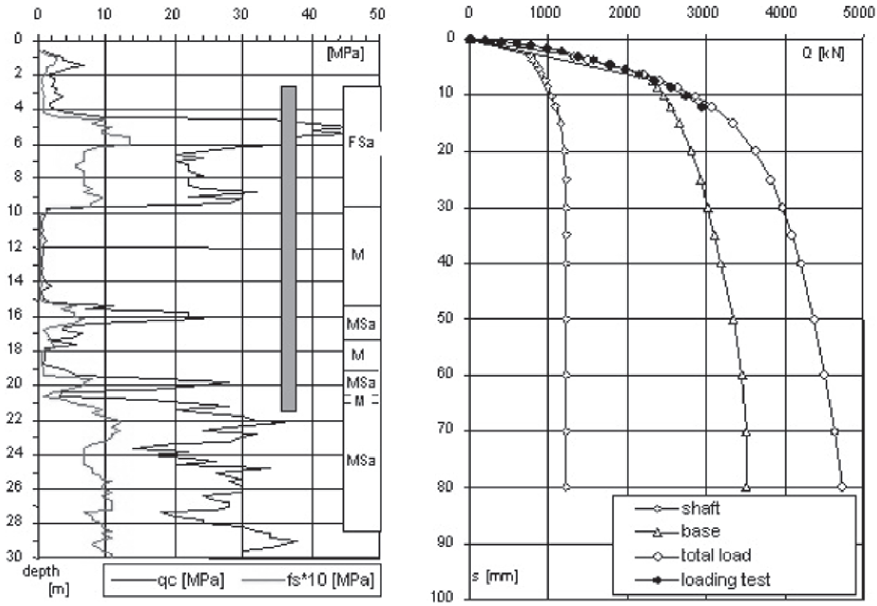

Fig. 6. Load–settlement curve representing Vibrex pile,
h = 18.3 m, D = 0.457 m, D_b = 0.650 m

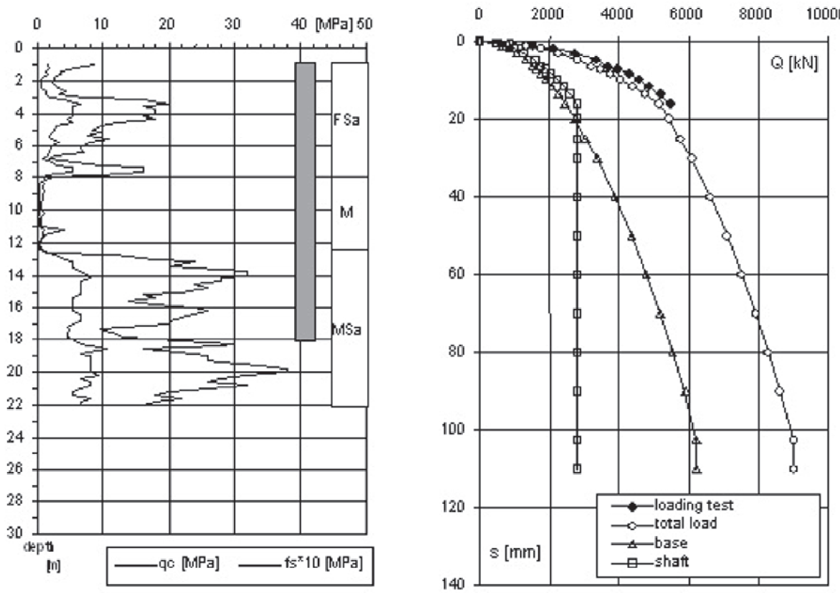

Fig. 7. Load–settlement curve representing large diameter bored pile,
h = 17.0 m, D = 1.5 m

settlement for the pile at the design stage enables a realistic assessment of the ultimate serviceability state together with the margin of safety assumed by a designer.

The bearing capacity of pile can be determined directly from CPT test results. In the respective calculations, the cone resistance q_c is used which has to be averaged near the pile base and along its length for selected soil layers.

In order to determine a complete load–settlement curve, load–transfer functions are used which may be chosen in terms of the following parameters:

- unit, ultimate resistances along the shaft and under the pile base,
- pile displacements at which mobilisation of maximum friction resistances along the shaft and under the base of the pile occurs (determined by the corresponding percentages of pile diameters along the shaft and at the base),
- exponents of power of load–transfer functions α and β.

In the calculations verifying the correctness of the method, the conformity coefficients for ultimate loads and loads for selected displacements were close to unity. Coefficients of variation for ultimate load are equal to 0.1 for Vibro piles and 0.137 for large diameter bored piles. For selected settlements of trial piles, these coefficients did not exceed 0.20 which is accurate enough for engineering calculations.

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