
International Conference on Case Histories in Geotechnical Engineering (2004) - Fifth International Conference on Case Histories in Geotechnical Engineering

14 Apr 2004, 4:30 pm - 6:30 pm

Design of Axially Loaded Bored Single Piles in the Czech Republic

Jan Masopust
FG Consult, s.r.o. Praha, Czech Republic

Follow this and additional works at: <https://scholarsmine.mst.edu/icchge>



Part of the [Geotechnical Engineering Commons](#)

Recommended Citation

Masopust, Jan, "Design of Axially Loaded Bored Single Piles in the Czech Republic" (2004). *International Conference on Case Histories in Geotechnical Engineering*. 2.
<https://scholarsmine.mst.edu/icchge/5icchge/session01/2>



This work is licensed under a [Creative Commons Attribution-Noncommercial-No Derivative Works 4.0 License](#).

This Article - Conference proceedings is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.



DESIGN OF AXIALLY LOADED BORED SINGLE PILES IN THE CZECH REPUBLIC

Jan Masopust

FG Consult, s.r.o. Praha, Czech Republic
VUT Brno, Fakulta stavební, Ústav geotechniky

ABSTRACT

The paper deals with the design method of axially loaded bored single piles based on statistical analysis of more than 350 field static load tests (MLT) of bored piles made in Czech Republic and partially in Germany during the past 30 years. The limit state of serviceability is characterized by the construction of load-settlement curve. The first draft of this method was based on the theory of elasticity and on quasi-elastic relations between soil and pile and it was modified by means of back analyses of pile static load test results. Recently, the construction of the load-settlement curve is based on the non-linear pile settlement theory by means of pressiometric modulus of soil and rheological parameter of soil structure. Ten types of soils and weak rocks, typical for the geotechnical conditions not only in Czech Republic were chosen.

INTRODUCTION

First results of the statistical analysis of the set of 226 static load tests performed until then in Czech Republic were published 21 years ago. The method of construction of the limit load-settlement curve of a single bored pile was created /Masopust 1980, 1981, Bažant, Masopust 1981/ with the help of so far published works /Poulos, Davis, Mattes, 1977 – 1980/. From this time on this method became the most widespread method for computation of bearing capacity of bored piles based on the limit state of serviceability. It was proved simultaneously, that the classical calculations of limit bearing capacity are not reliable, as they do not take into account the real mechanism of mobilisation of bearing capacity of bored piles. This mechanism was verified by the observation at the sites especially during the performance of the static load test. The main principles are following:

- a) In the case of bored piles in soils and weak rocks (classes R5, R4 – see Table 1):
 - the pile deformation increases during the gradual loading and the skin friction is being mobilised up to its limit settlement 10 – 30 mm,
 - the tip resistance is being activated slowly with linear progress, while the limit base resistance corresponds with the settlement roughly equal to 10 % of tip diameter of the pile,
- b) The bearing capacity of piles in rocks (R3, R2, R1) is determined by the internal bearing capacity of the pile (permissible resistance of pile concrete, which is 5,0 to 6,0 MPa according to class of concrete),

- c) The method of installation of bored piles (boring, cleaning up of borehole, concreting) is of crucial importance to the bearing capacity of pile. It is possible to qualify some factors that influence the bearing capacity of pile (e.g. with the help of known technological rules – EN 1536), but it is impossible to compute them, as they are of statistical origin and relevant correlation are not known.

These facts caused the critical attitude of the author to the theoretical methods of calculation of bearing capacity of piles that do not include technological effects of pile installation. It is a case of various calculation methods of bearing capacity of bored piles according to the ultimate limit state. The both presented studies are of pragmatic nature and they are based on the real monitoring of data from the set of the field static load tests especially the tests arranged with load cells. Original set of 226 load tests was later extended by 90 tests performed in the last 20 years in the Czech Republic and by 40 tests performed in Germany /Dürrwang, 1977/. The set of static load tests is still opened. The author presumes that the results can become more exact depending on the increase of the number of tests.

LIMIT LOAD-SETTLEMENT CURVE OF SINGLE BORED PILE – THEORY OF ELASTICITY

Set of 226 static load tests includes bored piles of 0,6 to 1,5 m diameter, length 5,0 to 30,0 m. The piles were carried out by the method of intermittent unsupported or supported

excavation by either casing or stabilizing fluid. Part of tested piles (35 %) was fully instrumented, e.g. the distribution of normal stress in pile shaft was monitored. This monitoring together with the load-settlement curve of test pile describes fully the pile behavior /Feda, 1977, Masopust, 1994/. The example of this test is shown in Figure 1. For the purpose of this analysis the foundation soil was classified into 10 classes according to Table 1. In spite, that the geotechnical conditions in the Czech Republic are rather varied, it was found, that the classification given by Table 1 is sufficient for the setting up of the computing model.

The single bored pile is divided into the elements that are corresponding to the soil layers in the computing model (Fig. 2). The ultimate bearing capacity of the pile shaft Q_{yu} may be calculated by means of regression curves deduced from the statistical analysis of the field load tests:

$$Q_{su} = 0,7\pi \cdot \sum d_i h_i q_{si} \quad (1)$$

and

$$q_{si} = A - [B/(D_i/d_i)] \quad (2)$$

where A, B are the regression coefficients in Table 2, D, d, h according to Fig. 2.

Table 1. Classification and properties of foundation rocks and soils

Soil Type	Soil description
R 3	Moderately and slightly weathered rocks with $\sigma_C = 15 - 20$ MPa
R 4	Strongly weathered rocks and moderately weathered weak soils with $\sigma_C = 5 - 15$ MPa
R 5	Weathered and strongly weathered weak rocks with $\sigma_C = 1,5 - 5,0$ Mpa
C 10	Cohesive soils with $I_C \geq 1,0$
C 75	Cohesive soils with $I_C \cong 0,75$
C 50	Cohesive soils with $I_C \cong 0,5$
D 9	Cohesionless soils with $D_r = 0,8 - 0,9$
D 7	Cohesionless soils with $D_r = 0,7$
D 5	Cohesionless soils with $D_r = 0,5$
Y	Unsuitable soils

σ_C – axial compression strength, I_C - consistency index,
 D_r – relative density

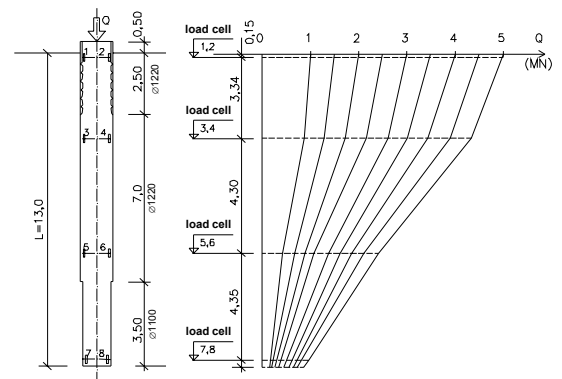
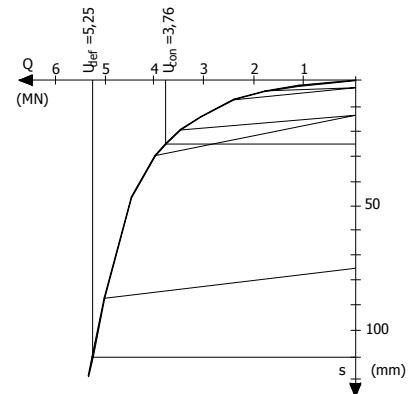


Fig. 1. Example of static load settlement curve

The tip resistance at the deformation corresponding to the full mobilization of skin friction equals:

$$q_t = E - [F/(L/d_0)] \quad (3)$$

where E, F are the regression coefficients in Table 2, L, d_0 according to Figure 2.

The mean skin friction along the pile shaft is:

$$q_s = (\sum q_{si} h_i d_i) / (\sum h_i d_i) \quad (4)$$

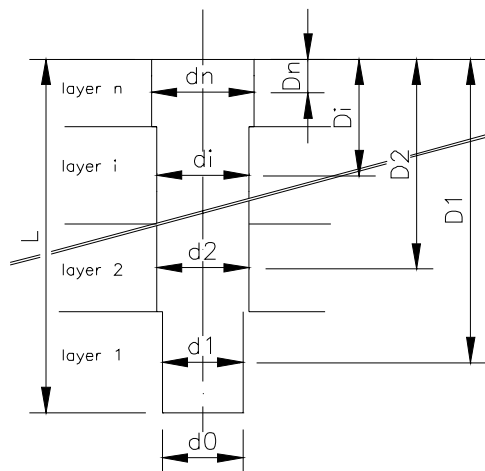


Fig. 2. Model of the pile in the layered soil

Table 2. Regression coefficients for various rocks and soils

Rock Soil	A /kPa/	B /kPa/	E /kPa/	F /kPa/
R3	246	226	2841	1299
R4	170	139	1616	1155
R5	132	95	958	704
C10	97	109	988	1084
C5	46	21	198	150
D9	154	116	1597	1399
D7	91	48	490	445
D5	62	16	268	175

The proportion of the total load carried by the pile tip:

$$\beta = q_0 / (q_0 + 4q_s L / d_0) \quad (5)$$

The pile load corresponding to the mobilization of skin friction:

$$Q_{yu} = Q_{su} / (1 - \beta) \quad (6)$$

and the pile settlement:

$$s_y = I [Q_{yu} / (dE_s)] \quad (7)$$

where I is the influence factor of pile settlement,

E_s is the value of the secant modulus of pile deformation according to Tables 3, 4 and 5 /Masopust, 1980/.

The influence factor:

$$I = I_1 R_k R_h \quad (8)$$

where I_1 , R_k , R_h are the coefficients according to Poulos /1972/.

Table 3. Secant deformation moduli E_s /MPa/ for piles in weak rocks

h /m/	Diameter d /m/								
	0,6			1,0			1,5		
	R3	R4	R5	R3	R4	R5	R3	R4	R5
1,5	50,3	28,2	20,0	72,3	35,0	24,7	85,5	33,5	22,3
3,0	64,5	43,1	30,8	106	57,3	41,0	138	58,8	41,2
5,0		58,2	41,3		75,3	54,8		87,9	63,7
10,0		87,5	61,6		115	83,2		133	97,0

Table 4. Secant deformation moduli E_s /MPa/ for piles in cohesionless soils

h /m/	Diameter d /m/								
	0,6			1,0			1,5		
	D_r								
	0,5	0,7	0,9	0,5	0,7	0,9	0,5	0,7	0,9
1,5	11,0	13,7	28,3	12,8	15,8	30,6	13,0	15,3	29,0
3,0	15,5	20,2	44,5	18,4	25,0	47,8	19,4	24,5	52,5
5,0	18,8	26,6	56,1	22,8	32,5	69,1	24,5	36,0	78,2
10,0	23,8	36,6	72,1	29,8	47,8	93,4	32,6	54,0	107

Table 5. Secant deformation moduli E_s /MPa/ for piles in cohesive soils

h /m/	Diameter d /m/					
	0,6		1,0		1,5	
	I_c					
	0,5	1,0	0,5	1,0	0,5	1,0
1,5	6,9	13,2	7,9	13,4	8,6	12,3
3,0	10,0	22,0	12,5	23,9	13,7	23,0
5,0	12,5	31,2	15,9	33,4	18,4	36,7
10,0	15,5	44,3	21,3	51,3	24,6	57,4

The first segment of the load-settlement curve (Fig. 3) is a parabola of the second order:

$$s = s_y(Q/Q_{yu})^2 \quad (9)$$

for the load interval $Q \in (0; Q_{yu})$, or for the settlement interval $s \in (0; s_y)$.

The second segment of the load-settlement curve is linear with the coordinates of the ultimate point ($s_{25} = 25 \text{ mm}$; Q_{pu}), where:

$$Q_{pu} = Q_{su} + Q_{bu} \quad (10)$$

and the pile tip-load is given by equation:

$$Q_{bu} = \beta Q_{yu} s_{25}/s_y \quad (11)$$

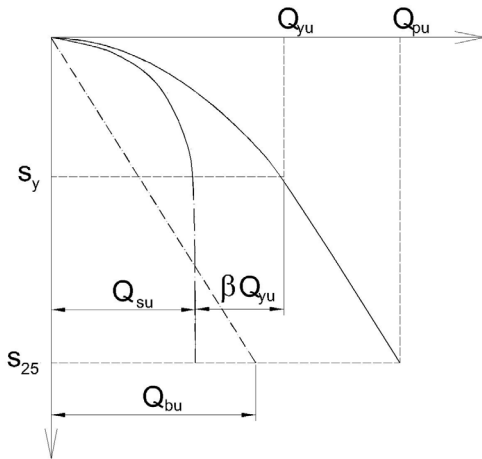


Fig.3. Ultimate load-settlement curve of bored pile

LIMIT LOAD-SETTLEMENT CURVE OF SINGLE BORED PILE – NON LINEAR THEORY

The relationship between load and deformation of bored pile especially for the settlement greater than 5 – 10 mm is distinctly non-linear. It was the reason for the acceptance of rheological model of foundation soil characterized by pressiometric modulus of soil deformation E_s and rheological coefficient of soil structure a according to Ménard /1965/. The rheological coefficient a depends on consistency index and consolidation degree for the cohesion soils; or on relative density for the cohesionless soils and finally on compression

strength and stress-strain relationship for the weak rocks. The typical values of a is given in Table 6 for the relevant classes of foundation soils. Mobilization of bored piles skin friction depending on deformation of pile shaft goes according to curves given by equation:

$$q_s = q_{s,lim} [1 - (1 - s/s_{s,lim})^{f(a)}] \quad (12)$$

where $q_{s,lim}$ is the limit skin friction of the respective layer of the foundation soil according to the equation (13), s is the settlement of pile head,

$s_{s,lim}$ is the limit pile head settlement according to equation (14),

$f(a)$ is the function value of the rheological coefficient a according to Table 6.

The limit skin friction is:

$$q_{s,lim} = 0,7 \cdot m_1 \cdot m_2 \cdot A \cdot \text{tgh}(D/(4 \cdot d)) \quad (13)$$

and limit settlement of pile in the respective soil layer is:

$$s_{s,lim} = A \cdot g(a, d) / (E_s \cdot d^{1/2}) \quad (14)$$

where A is the basic value of skin friction in the respective soil layer determined on the basis of statistical analysis of the field static load tests,

m_1 is the coefficient of the pile installation method (Table 7),

m_2 is the coefficient of the shaft insulation (Table 8) - if there is any,

D is the distance from the pile head to the middle of the respective soil layer

d is the pile diameter in the respective soil layer (Fig. 2),

$g(a, d)$ is the function value of the rheological coefficient a and pile diameter d according to Figure 4.

Table 6. Values of a , $f(a)$, A , X , Z

Soil classes	A	$f(a)$	A /kPa/	X /MPa/	Z /MPa/
R3	0,66 – 0,80	3,017 – 2,500	400	3,70	2,67
R4	0,66 – 0,75	3,017 – 2,670	280	2,60	1,89
R5	0,66	3,017	200	1,70	1,25
C10	0,50 – 0,66	4,500 – 3,017	150	1,35	0,98
C75	0,50 – 0,55	4,500 – 3,985	125	0,85	0,62
C50	0,50	4,500	90	0,36	0,26
D9	0,66 – 0,75	3,017 – 2,670	220	1,82	1,09
D7	0,66	3,017	140	1,15	0,69
D5	0,55 – 0,60	3,985 – 3,524	110	0,53	0,32

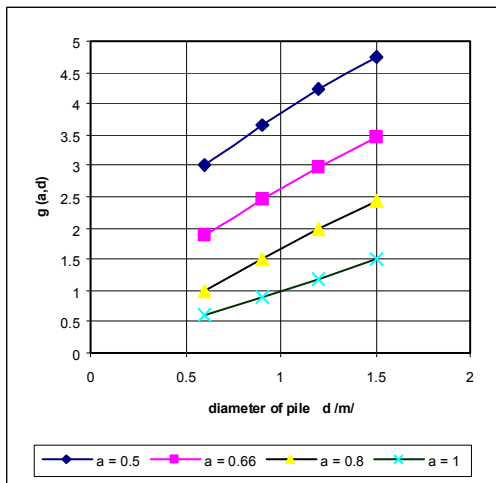


Fig. 4. Distribution of function $g(a,d)$

Table 7. Methods of pile installation

- Unsupported excavation with cleaning up, concreting in dry conditions
- Excavations supported by steel casings with cleaning up, concreting in dry conditions
- Excavations supported by steel casings with cleaning up, concreting in submerged conditions
- Excavation supported by steel casings without cleaning up, concreting in submerged conditions
- Excavations supported by fluids with cleaning up, concreting in submerged conditions
- Excavations supported by fluids without cleaning up, concreting in submerged conditions
- Boring with continuous flight augers (CFA)

Table 8. Methods of pile shaft insulation

- Piles without secondary shaft insulation
- Piles with secondary shaft insulation: PVC, PE-foil, thickness < 0,2 mm
- Piles with secondary shaft insulation: PVC, PE-foil, thickness 0,8 – 2,0 mm
- Piles with secondary shaft insulation: PVC, PE-foil, thickness > 2,0 mm
- Piles with secondary shaft insulation: wire netting and PVC/foil
- Piles with permanent casing of PVC, PE-tubes
- Piles with permanent casing of steel tubes

The tip resistance q_0 corresponding to the pile bottom settlement $s_0 = 10$ mm for the foundation soils R5, C10, C75, C50, D9, D7 and D5 is given:

$$q_0 = m_0 \cdot (X - Z/(L/d_0)) \quad (15)$$

The tip resistance for the weak rocks R4, R3 (according to Table1) is:

$$q_0 = m_0 \cdot (X - Z/(L/d_0)) \cdot t^{1/2} \quad (16)$$

on condition that minimum depth of pile bottom fixed in the rock is $t_{\min} \geq 0,5$ m,

where m_0 is the coefficient of pile installation method (Table 7),

X, Z are the basic values for the tip resistance in the respective soil layer, determined on the basis of statistical analysis of the field static load tests, according to Table 6,

L is total length of pile,

d_0 is pile diameter measured at the bottom.

Computing process

The pile is divided into the elements that are corresponding to the soil layers in the computing model (Fig. 2). The sequence of small gradually growing settlements v_i is given to the pile bottom and thus the settlement of the pile is modeled. The tip resistance $q_{0,1}$ and consequently the force $Q_{b,1}$ and the force ${}^1Q_{s,1}$ along the pile shaft is being mobilized during the first settlement of the pile bottom v_1 . The sum total of the forces ($Q_{b,1} + {}^1Q_{s,1}$) causes the elastic contraction of the first element w_1 (according to Hook's law). The element 2 of the pile moves by $(v_1 + w_1)$ and the skin force ${}^1Q_{s,2}$ is mobilized. The process is being repeated up to pile head and the settlement of the pile is computed $s_1 = v_1 + \sum w_i$ together with the corresponding pile force $Q_1 = Q_{b,1} + \sum {}^1Q_{s,i}$. The coordinates of the first point of the load settlement curve ($s_1, Q_{s,1}$) are obtained by this method. Next points of the load settlement curve are obtained accordingly for another value of v_i .

Examples of bored pile load - settlement curve

Figures 5 and 6 show the typical load settlement curves computed by the above-mentioned method for various soil, and technological conditions.

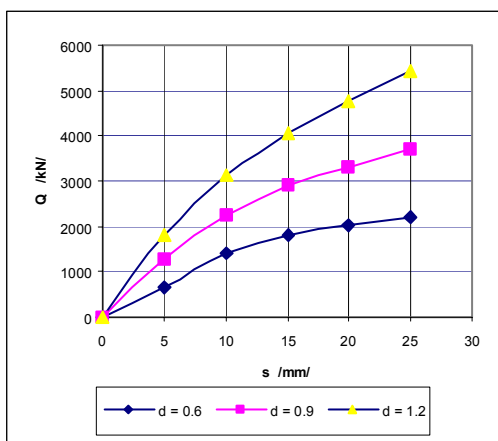


Fig. 5. Load-settlement curve of bored pile $L = 10$ m, $d = 0,6; 0,9$ and $1,2$ m with geotechnical profile: $0,0 - 2,0$ m – sandy loam $I_C = 0,5$ (C50); $2,0 - 7,0$ m sandy gravel $D_r = 0,7$; $7,0 - 15,0$ – firm clay $I_C = 1,0$

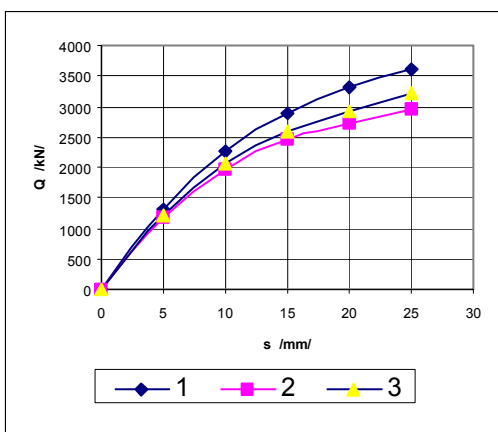


Fig. 6. Influence of pile installation method on bearing capacity of bored pile $L = 10$ m, $d = 0,9$ m, with geotechnical profile according to Figure 5. 1 – excavation supported by steel casing with cleaning up, concreting in dry conditions, 2 – boring with continuous flight auger, 3 – excavation supported by fluids with cleaning up, concreting in submerged condition

CONCLUSION

Design of cast-in-place bored piles based on field loading tests obtained in last 30 years in the Czech Republic and partially in Germany has been discussed. For the purposes of statistical analysis of field loading tests the foundation soil was divided into 10 classes. The non-linear relationship between load and settlement of bored piles was established

on the base of field test results. This relationship led to the acceptance of rheological model of foundation soil characterized by pressiometric modulus of deformation and by rheological coefficient of soil structure. The presented non-linear design method was developed for the construction of load-settlement curve of single bored pile depending on installation methods. The computer program PILOTA that was made for the design of bearing capacity of single piles based on limit state of serviceability enables to choose 7 types of pile installation (incl. CFA piles) and to choose 7 types of secondary pile shaft insulation in soils with corrosive water condition.

REFERENCES

- Bažant, Z., Masopust, J.: *Drilled pier design on load-settlement curve*. Proc. X-th ICSMFE, Vol.2, Stockholm 1981: p.615 – 618.
- Dürwang, R.: *Pfahltragfähigkeiten im Grenzbereich Lockerboden – Fels*. Trischler und Partner, Nr.2, September 1997.
- Feda, J.: *Interakce piloty a základové pudy*. Academia Praha, 1977, 156 p.
- Feda, J., Masopust, J.: *Design of axially loaded piles – Czech practice*. Proceedings of the ERTC3 Seminar, Brussels, April 1997, p.83 – 99.
- Masopust, J.: *Volba modulu deformace při výpočtu velkopřůměrových pilot*. Sborník konf. Výpočet pilotových základů, Karlovy Vary, 1980, p.33 – 41.
- Masopust, J.: *Návrhové parametry pro výpočet velkopřůměrových pilot*. Sborník konf. Zakladanie stavieb 81, Vysoké Tatry, 1981, p.165 – 169.
- Masopust, J.: *Vrtané piloty*. Čeněk a Ježek, Praha, 1994, 263 p.
- Masopust, J.: *Design of axially loaded bored single piles*. Proceedings of the 4th International Geotechnical Seminar, Ghent, June 2003, p.
- Ménard, L.: *Règles pour le calcul de la force portante et du tassement des fondations en fonction des résultats pressiometriques*. Proc. VI – ICSMFE, No. 2, Montréal, 1965, p.259 – 299.
- Poulos, H.G., Davis, E.H.: *Settlement behaviour of single axially loaded piles and piers*. Géotechnique, 18.3, p.351 – 371.
- Poulos, H.G., Mattes, N.S.: *The behaviour of axially loaded end-bearing piles*. Géotechnique, 19.2, p.285 – 300.
- Poulos, H.G.: *Load-settlement prediction for piles and piers*. JSMFD ASCE, Vol. 98, SM 9.