LARGE DIAMETER PILES UNDER AXIAL AND LATERAL LOADS

PIEUX DE GRAND DIAMETRE SOUMIS A DES FORCES AXIALES ET HORIZONTALES

СВАИ ВОЛЬШОГО ДИАМЕТРА ПРИ ВЕРТИКАЛЬНОЙ И ГОРИЗОНТАЛЬНОЙ НАГРУЗКАХ

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SYNOPSIS. The paper describes field tests on large diameter bored piles under axial and lateral loads. It is emphasized the necessity of taking into account the stresses which develop in the shaft, both for establishing the allowable load and for interpreting the test results. Because of difficulties related with the inspection of the quality of the concrete placed under water or under drilling mud, a limitation of the stresses in the shaft appear reasonable, which may lead to an incomplete use of the bearing capacity.

INTRODUCTION

The most signifieant aspect of the development in the field of foundation engineering in the last decades is probably represented by the proliferation and rapid extension of various procedures for constructing piles of large diameter and depths. A first and important consequence was the improvement of the performances of pile foundations, the bearing capacity of the large diameter piles reaching a different order of magnitude as compared with that of traditional piles i.e. hundreds or even thousands of tons. The ability of large diameter piles to resist important lateral loads has made possible the replacement of batter piles with vertical piles.

In Romania, large diameter piles drilled with the aid of bentonite mud have been constructed recently by means of a Romanian rotary-type drilling machine FA-12. In the same time, Benoto type bored piles have been extensively used. The following sections present the results of some field tests on large diameter bored piles and their interpretation.

BORED PILES UNDER AXIAL LOADS

For the foundation of a high chimney, piles bored under bentonite mud, of 38 m length and 1,27 m diameter have been used. Fig. 1 shows a geologic profile. From the ground level to a depth of 34,5o m the subsoil is made of recent deposited clayey and silty soils of low consistency, followed by a thick layer of gravel with sand into which piles have penetrated 3,5o m deep. Ground water was found at shallow depth below the ground level.

The point resistance of the piles has been computed according to the Russian Building Code (SNiP 1967). For the skin friction a value of 2,7 t/sqm has been employed, which was derived from pulling tests of precast reinforced concrete piles 25 m in length.

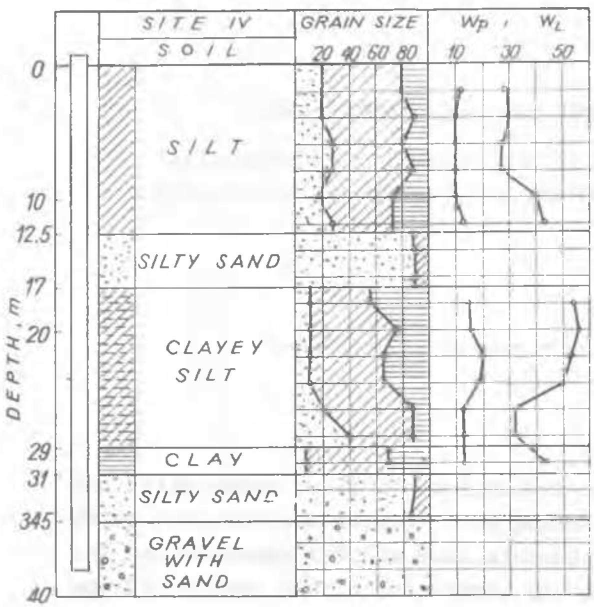


Fig. 1

The working axial load resulted of 510 tons from which, subtracting the own weight of the pile an allowable load of 440 tons was set up.

In tabel 1 load-settlement data for a bored pile on the site are given. Due to technical reasons the axial load on the pile could not exceed 600 tons. Under that load the measured settlement was of only 11 mm. Since a part of that deformation should be attributed to the compression of the shaft, it may be concluded that the actual bearing capacity of the pile is much larger than the computed one. Nevertheless, taking into account that piles are submitted also to bending and in order to avoid that the stress in the concrete exceeds 50 kg/sqcm, the axial load on the pile, including the own weight, was limited to 510 tons, for which the stress in the concrete is 40 kg/sqcm.

The results of this field test show, like in some other cases (Botea and Manoliu, 1971) that for the bored piles in noncohesive soils the bearing capacity computed with the

Table 1

P, tons	s, mm	P, tons	s, mm
50	0,19	350	2,10
100	0,14	400	2,81
150	0,68	450	4,73
200	0,97	500	7,08
250	1,28	550	8,80
300	1,70	600	11,17

SNiP formula is understimated. On the other hand, in the case of piles submitted to both axial and lateral loads, the limitation of stress in the concrete at 40-50 kg/sqcm which seems reasonable for cast in situ piles below the groundwater level, might limit also the bearing capacity of the pile.

BORED PILES UNDER LATERAL LOADS

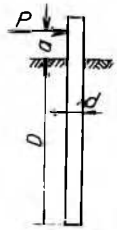
The study of the behavior of large diameter bored piles under lateral load is of major interest if one takes into account the fact that, due to the practical impossibility of constructing inclined shafts, the whole lateral load on the foundation has to be supported by the soil-pile interaction.

The field tests which will be described herein have been conducted on four different sites, named I..IV, located on the left bank of the Danube and characterized by recent alluvial deposits.

On the sites I, II and III piles have been constructed with a Benoto machine and on the site IV with a Romanian machine FA-12. In tabel 2 the geometrical characteristic of the piles are given.

In fig. 2, 3, 4, soil conditions for the sites I, II and, respectively III are given. Site IV is the same as the one described previously in connection to the axial loading test.

Table 2



Characteristics 1	I 2	II 3	III 4	IV 5
d,m	1,00	0,88	0,88	1,27
a,m	2,25	0,80	0,20	0,20
D,m	10,00	33,00	14,00	38,00
28 day concrete strength	B 200	B 300	B 200	B 300
Reinforcing steel area, sqcm	84	123	50,4	94
Steel type	PC 52	OB 38	PC 52	OB 38

Test results . The tests to be described have been routine tests, except the one performed on the site I where stresses in two reinforcing steel bars in the plane of the lateral load have been measured. For this purpose, electroacoustic transducers were used, which proved to be very reliable and stable in time (Constantinescu and Ţuţan, 1967).

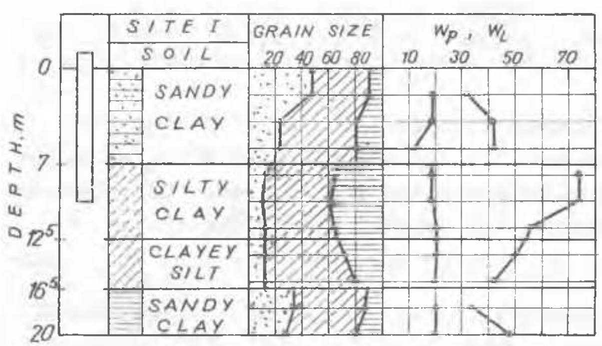


Fig. 2

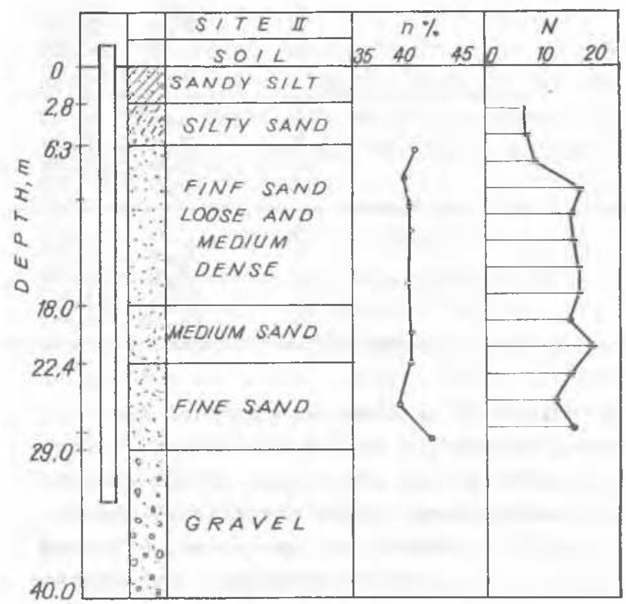


Fig. 3

In tabels 3 and 4 the measured deflections y_t and slopes s_t at the ground level are summarized.

DISCUSSION. A first observation pertains to the deflections measured at the piles on sites I and II which have been loaded two by two, simultaneously. The two piles on site I behaved differently from the first load, a fact which could be explained only by a local soil nonhomogenity, undetected by the borings or, partly, by the different age of the piles : 10 months for pile No. 1 and 4 months for pile No. 2. In contrast, piles on the site II exhibited a remarkable symmetrical behavior, proving that in the case of sand deposits the probability of local nonuniformity is lower.

For the interpretation of test results the Winkler model has been adopted for the soil, with a soil modulus increasing linearly to the depth :

$$k = m \cdot z \qquad (1)$$

where m is a factor of proportionality, in tons/cum and z is the depth, in meters.

Table 3

Site	Pile	P_t, tons	M_t, tm	Y_t, mm	$s_t\ 10^3$	Y_{tcalc}, mm	$s_{t\,calc}\cdot 10^{-3}$	$m\cdot 10^{-2}$ t/m³	$EI\cdot 10^{-4}$ tm²
1	2	3	4	5	6	7	8	9	10
I	1	2	4,5	0,34	0,145	0,34	0,186	91	11,5
		4	9,0	0,67	0,445	0,67	0,380	90	11,5
		6	13,5	1,88	0,690	1,88	0,772	28,5	11,5
		8	18,0	2,87	1,58	2,83	1,63	55	4,9
		10	22,5	4,23	2,51	4,22	2,25	40	4,9
		12	27,0	11,12	5,15	11,04	4,22	8,2	4,9
	2	2	4,5	0,64	0,24	0,64	0,27	28	11,5
		4	9,0	1,53	0,73	1,52	0,59	21	11,5
		6	13,5	4,67	2,10	4,63	1,62	5,5	11,5
		8	18,0	9,50	4,35	9,53	3,07	4,5	4,9
		10	22,5	14,89	6,14	14,69	4,59	3,8	4,9
		12	27,0	23,87	9,92	23,31	6,48	2,0	4,9
II	1	4	3,2	2,67	0,070	2,66	0,078	6,00	7,5
		8	6,4	7,29	0,175	7,36	0,164	3,35	7,5
		12	9,6	17,54	0,464	17,48	0,414	1,80	6,0
		16	12,8	32,49	0,780	32,55	0,760	1,26	4,5
	2	4	3,2	1,70	0,070	1,73	0,060	13,00	7,5
		8	6,4	5,99	0,185	5,97	0,167	5,00	7,5
		12	9,6	12,38	0,400	12,39	0,309	3,80	6,0
		16	12,8	21,41	0,570	21,46	0,568	2,65	4,5
		20	16,00	30,40	0,890	30,56	0,777	2,10	4,5

Relation (1) is commonly accepted for sandy soils. Its use is proposed also for the silty soils, of low and medium cohesion and consistency, which characterize sites I and IV. Computations have been performed by using diagrams of nondimensional coefficients (Reese and Matlock, 1956).

Measurements on instrumented pile on site

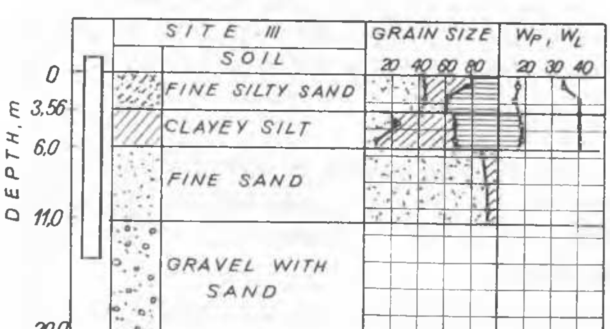


Fig. 4

I proved the significance of taking into account the variation of the flexural rigidity of the pile. Fig. 5 shows the relation between the applied moment and the stress in the reinforcing steel bars, at the trans-

Tab. 4

Site	P_t tons	M_t tm	y_t mm	$m\cdot 10^{-2}$ t/m³
	2	3	4	5
III	8	0,8	1,07	65
	12	1,2	1,65	62
	16	1,6	3,00	37,5
	20	2,0	4,64	26,0
	24	2,4	7,15	17,0
	28	2,8	9,85	13,0
	32	3,2	14,15	8,60
	36	3,6	28,7	3,85
	44	4,4	37,9	3,10
	48	4,8	49,1	2,80
IV	6	1,2	0,37	100
	10	2,0	0,90	55,5
	14	2,8	1,46	43,4
	18	3,6	2,29	40,6
	22	4,4	3,35	23,2
	26	5,2	4,86	16,0
	30	6,0	6,57	12,8
	34	6,8	9,18	8,8
	36	7,2	18,25	3,05
	37	7,4	50,65	0,32

duoers placed just above the ground level. The craoking of the concrete is olearly put into evidence by the moment when the stress in the tensioned bar increases more rapidly than the one in the compressed bar. In the test described, for loads exceeding 8 tons the concrete was cracked. Therefore, for the loading stages of 1o and 12 tons, computations have been made using a reduced value of the modulus of elasticity of the concrete as given by the transducer readings. The computations for the piles on site II have been made in a similar menner.

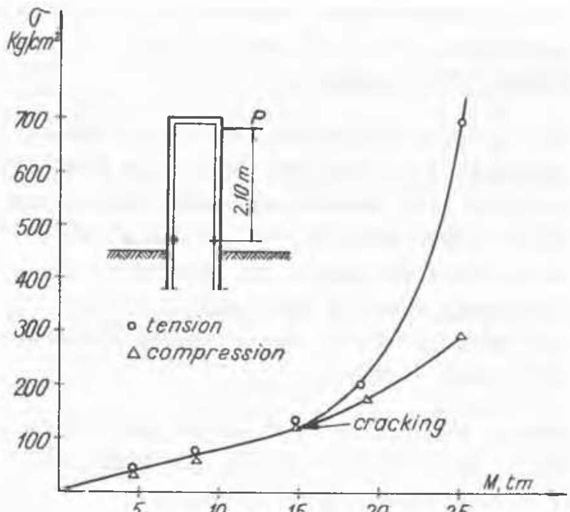


Fig. 5

Fig. 6 shows the diagram of bending moments obtained by means of transducer readings, in comparison with the computed diagram, for load P_t= 2 t, M_t= 4,5 tm, before the cracking of the concrete. The diagram in fig. 6 as well as the values of the deflections y_t and slopes s_t given in tabel 3 indicate a fairly good agreement between measurements and computations, thus confirming the validity of the assumption made with respect to the soil modulus variation. As a result, the same assumption was extended to the interpretation of the test results on sites III and IV where, lacking the measured slopes, no checking was possible.

As for the values of the coefficient m, it may be seen that m decreases as the pile deflection y_t increases, as a result of the

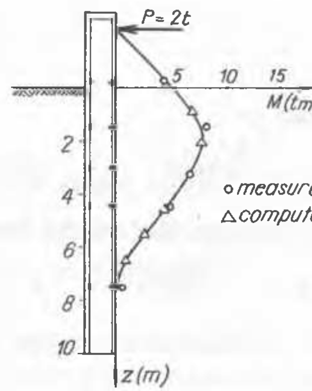


Fig. 6

nonlinear behavior of both soil and concrete. For the piles on site II, under loads of 12-16 tons which can be regarded as allowable, m has values ranging from 126 to 38o t/cum which agree well with those recommended for submerged loose and medium dense sand (Terzaghi, 1955).

The test described, especially those on the site I, show that in the case of long piles of large diameter, the pile behavior is largely controlled by the behavior of the concrete they are made of, under working load as well as under limit load. Usually the limit load is attained when the yielding of the pile section is reached. Under these circumstances, a correct estimation of the bending moments developping along the shaft and a proper selection of the reinforced concrete section are highly desirable.

The interpretation of the results of field tests on the four sites suggests for the preliminary design of a foundation on bored piles the possible use of a simple relation which does not require the measured values of y_t and s_t. The following relation is proposed for free-head piles :

$$M = P \cdot l \qquad (2)$$

where l ie the conventional depth of fixity, i.e. the length of a cantilever which, neglecting the soil pressure, would be submitted to a bending moment equal to the one developped in the pile. A relation similar to (2) is recommended for driven piles (SNiP 1967). The values proposed for l are given in tabel 5. Replacing in relation (2) M by the yield moment, the limit load P_{lim}

can be computed :

$$P_{lim} = \frac{M_{yield}}{l} \qquad (3)$$

The allowable load at the ground level :

$$P_{all} = \frac{P_{lim}}{2} \qquad (4)$$

In the tab. 6 a comparison between the values computed with the relation (3) and the experimental values is made. New field test data are needed to confirm the values given in tab. 5, which should be regarded as tentative ones.

Table 5

Type of soil	l
Loose sands, clay soils with $I_c < 0,5$	4 d
Medium dense sands, clayey soils with $0,5 < I_c < 0,75$	3 d
Dense sands, clayey soils with $I_c > 0,75$	2 d

Table 6

Site	M_{yield} tm	l	l m	P_{lim} tons	P_{all} tons	P_{lim} exp, tons
1	2	3	4	5	6	7
I	111	3d	3,o	37	18,5	-
II	11o	4d	3,52	32	16,o	-
III	72	2d	1,76	41	2o,5	44
IV	11o	3d	3,81	29	14,5	34

CONCLUSIONS. Field tests on large diameter bored piles have shown that under axial as well as under lateral loads, working loads on pilex can be restricted by the stresses which develop in the pile. The rational use of the bearing capacity of the pile requires to find out the reliable means to assure the quality of the concrete in the pile and to perform the in situ inspection of that quality.

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