Bearing Capacity of Hollow Piles Driven by Vibration

Force Portante de Pieux Creux Enfoncés par Vibration

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SYNOPSIS The factors influencing the deformation and bearing capacity of hollow piles with earth core, with big diameter, driven by a vibrating rammer are analysed. Test results from the driving and statical test loading of 20 m long piles with an external diameter of 135 cm are examined. The evaluation of the test results confirms that during the initial stages of loading the force is transferred as skin friction with insignificant and rapidly damped deformations. With the increase of the force, the ground under the pile bottom is loaded as well. A complex interaction occurs between the stressed zone under the bottom, the walls and the earth core within the pile. The core exerts a considerable influence on the deformations in case of piles with big diameters. It is found that its contribution depends on the strength and deformation properties of the soil, the possibility of consolidation, wall friction, etc. The core contribution to the total bearing capacity of the pile is determined. The bearing capacity of the pile is compared with the expected one calculated after the vibration dynamic formulae.

INTRODUCTION

Hollow piles of big diameter are successfully used for the foundation of heavily loaded harbour structures and bridges. When the works have to be carried out in the water vibrating rammers are used to achieve rapid driving and avoid heavy carrying structures for the pile drivers.

The theory of pile driving by vibration has been developed and applied in the foundation of a number of projects. A wide variety of vibration pile drivers of various weight, vibration frequency, power, etc., are available. The bottom end is most frequently open to facilitate driving. Thus a soil core is formed during the driving, which participates in varying degree in the total deformation and bearing capacity of the pile depending on the stress and strain properties of the soil and on the diameter of the hollow.

This foundation method is applied in the execution of structures along the Black Sea coast. It was considered in the initial design due to overrating the work of the core, that the soil in it, compacted during vibration driving would contribute essentially to the bearing capacity of the piles. The operational loading of one pile was expected to be more than 4 MN.

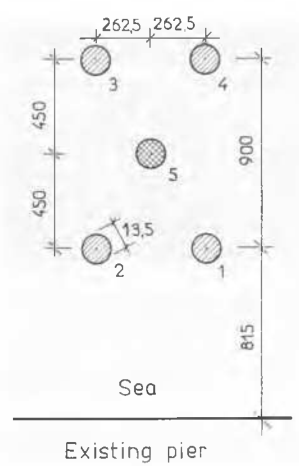

Fig.1 Layout of test pilots

DYNAMIC LOADING

For the purpose of the investigation test piles were driven after the pattern indicated on Fig.1. They are 20 m long (consisting of 4 sections, each 5 m long), with 135 cm external diameter, 111 cm internal diameter and 12 cm wall thickness. They are made of concrete class '400' (strength 4 MPa) and are prestressed with 24 steel bars 18 mm dia.

The strength of the steel is 600 MPa, the yield point - 400 MPa and Young's modulus 200 GPa. The geological profile is shown on Fig.2.

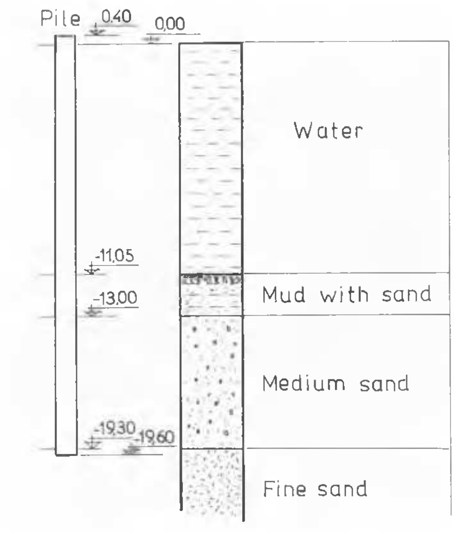

Fig. 2 Geological profile along the pile axis

The pile sections were fit together on the coast. Then the whole pile was raised at the place of driving by a floating crane. It sunk 0.80 m due to its own weight, which was 232 kN. Then the vibrating rammer weighing 80 kN was mounted. This additional loading caused a further settlement of 0.10 m. Ten minutes after the vibrating rammer was put in operation at a voltage of V = 420 V and a current intensity of I = 100 A, the pile sunk another 3 m. After that began the underwashing until the bottom sunk to level - 17.50 m. After the underwashing ceased the pile sunk 0.10 m at a current intensity of 110 A during 15 min. The driving to the bottom design level of - 19.60 m was conducted at a current intensity of 180 A.

The bearing capacity of piles driven by vibration can be established after SNiP II-B.5-67 (1968) after the formula

$$P = \lambda \left(\frac{25.5N}{A_o n} + Q \right), \qquad (1)$$

where P - bearing capacity of pile in kN

N - power used by vibration rammer to drive the pile in kW

A_o - vibration amplitude of pile in cm

n - rotation frequency of vibrator eccentric in Hz

Q - total weight of pile and vibration rammer in kN

$\lambda = 5$ - coefficient considering the influence of vibration driving on the soil properties.

For the characteristics of the vibration driver, at current intensity I = 110 A and voltage V = 420 V,

$$N = \eta \sqrt{3} \frac{I\ v}{1000} \cos \phi - 0.25 N' =$$
$$= 0.85 \sqrt{3} \frac{110 \times 420}{1000} \times 0.70 - 0.25 \times$$
$$\times 100 = 22.6 \text{ kW}$$

where $\eta = 0.85$ - efficiency of vibration rammer

$N' = 100$ kW - rated power of vibration rammer

$\cos \phi = 0.70$

$A_o = 0.60$ cm at bottom level - 17.50 m

$n = 6.8$ Hz

$Q = 232 + 80 = 312$ kN - weight of pile and vibration rammer

After formula (1) P = 2260 kN.

At the end of driving:

N = 53 kW, and the bearing capacity, obtained after the same formula, at $A_o = 0.75$ cm, is P = 2880 kN.

BEARING CAPACITY AFTER EMPIRICAL FORMULAE

The bearing capacity of the piles can be evaluated also by using the geological data. For this purpose several methods have been developed which give results differing considerably. The Soviet norms include the following formula:

$$P = (R_p F_D + u \sum m_f R_f l_i) \qquad (2)$$

For this case:

$F_D = 0.463$ m^2 - cross sectional area of pipe (ring)

$$R_p = \beta \left(\gamma d A_k^o + \alpha \gamma_1 h B_k^o \right) \qquad (3)$$

where $\gamma = \gamma_1 = \gamma' = 10$ kN/m^3 - unit weight of submerged soil

D = 1.35 m - external diameter of pile

h = 8.55 m - depth of driving, measured from the ground level.

The following coefficients are read from SNiP II-B.5-67 (1968) for h/D= 6.33 and angle of shearing resistance ϕ = 30°:

$$A_k^o = 17.3;\ B_k^o = 32.8;\ \alpha = 0.75;\ \beta = 0.227$$

After replacing in formula (3), the ultimate point resistance of the pile is:

R_p = 0.227 (10x1.35x17.3+0.74x10x8.55x32.8)= = 524 kPa

u = 4.24 m - pile perimeter

m_f = 1 - coefficient depending on the soil and the method of pile driving

R_{f1} = 54 kPa - average rated value of skin friction (after statistical data) for medium sand and at depth from -12.00 to -19.30 m (l_1= 7.30 m).

R_{f2} = 43 kPa - average rated value for fine sand and at depth -19.30 to -19.60 m (l_2 = 0.30 m).

Since underwashing is conducted during driving, the value of the skin friction is multiplied by 0.9.

For these data formula (2) gives P_{lim}=1800 kN

The value of P_{lim} calculated after other methods reaches 6500 kN.

TEST STATICAL LOADING

Because of the considerable scattering of the results and in order to evaluate the reliability of the methods for calculating the bearing capacity, a statical test loading of Pile No.5 was carried out, as shown on Fig. 1. The remaining piles were used as anchoring piles. The distances between them were bridged by steel section bundles. Expecting a greater bearing capacity, the loading was carried out by four 3 MN jacks fed by pumps from the coast. The deformations were also observed from the coast by precise levelling instruments. The loading was applied in stages, whereby after each stage the conditioned damping of the deformation was awaited, that is until the increase of the settlement became less than 0.1 mm during the last hour. Fig.3 shows the general view of the loading system.

Fig.4 indicates the relation between loading and settlement, and Fig.5 - the increase of deformations in time for some stages of loading.

Fig.6 gives the rate of the variation of deformations for some stages.

The pile was loaded until complete exhaustion of its bearing capacity P_{lim} = 2050 kN. At this load the prevailing deformations are residual S_r = 70 mm. The time-settlement curves S(t) for different loads P show a rather slow damping of the deformations, especially at the last load of 2050 kN. The speed diagrams (Fig.6) give a non-monotonous broken line with gradually decreasing amplitudes. It is evident that the compression of the core plays a certain role for the settlement.

Fig. 3 General view of the statical test loading system

CONTRIBUTION OF THE CORE

The contribution of the core can be determined using Fig.7.

The equilibrium condition for the element of height dy at a distance y from the bottom end of the pile reads:

$$F\ (\sigma_y + d\sigma_y) + F\gamma dy = F\sigma_y - \sigma_x f\ u\ d\ y, \quad (4)$$

where $F = \frac{\pi d^2}{4}$ - internal cross sectional area

γ - unit weight of soil

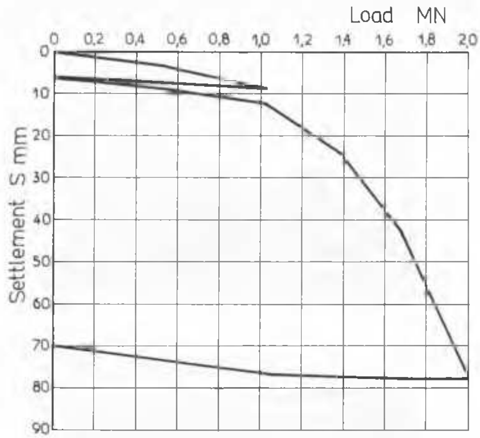

Fig. 4 Relation between loading and settlement of Pile No.5, obtained at the statical test loading

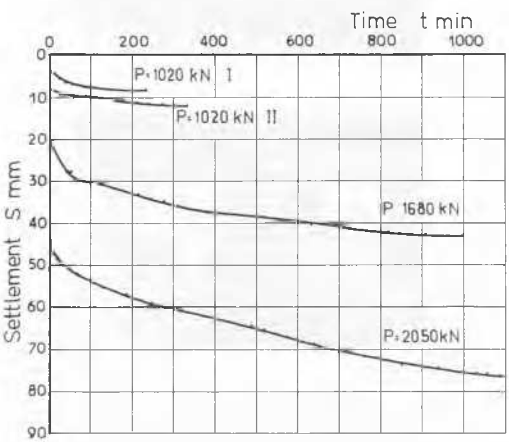

Fig. 5 Increase of deformations in time for different loading

σ_y - average vertical stress at distance y from the bottom end

σ_x - horizontal stress on pile wall

f - coefficient of friction between core soil and pile wall

d - internal pile diameter.

The stress σ_x is often defined as σ_y multiplied by the coefficient of earth pressure K. This is rigorously valid only if σ_y and σ_x are the principal normal stresses. In fact since there is also friction along the internal wall of the pipe, this plane is not the principal one. The stress σ_x is obtained more accurately by using Fig.8. Without violating the general validity of the solution the angle $\varnothing$ is accepted to be the limit angle.

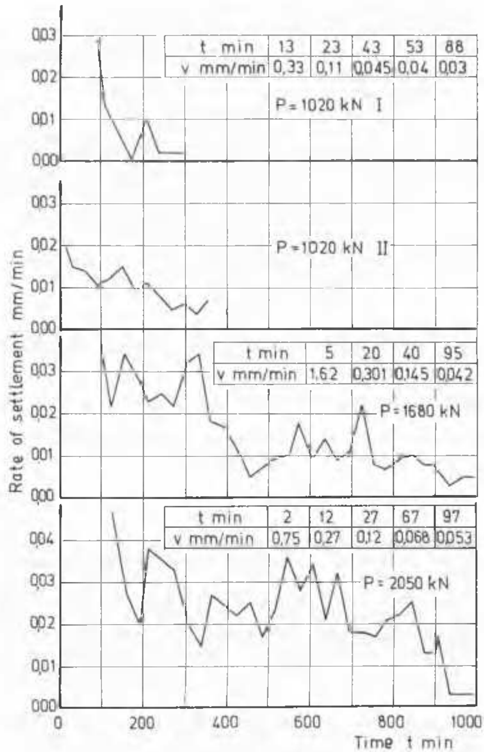

Fig. 6 Rates of variation of deformations in time for different loading

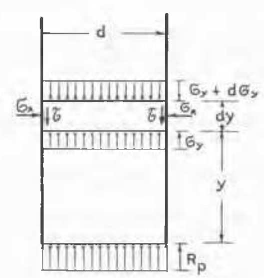

Fig. 7 Scheme for evaluating the statical function of the core

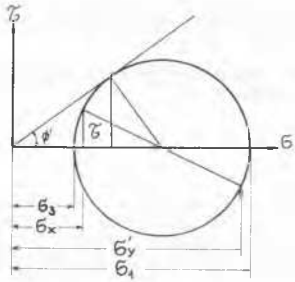

Fig. 8 Mohr's circle for evaluation the relations of the stresses in the core

The geometric relations give:

$$\left(\frac{\sigma_y' - \sigma_x}{2}\right)^2 + \tau^2 = \left(\frac{\sigma_y' + \sigma_x}{2}\right)^2 \sin^2\phi, \quad (5)$$

where σ_y' - vertical stress at the wall of the pile

σ_x - could be also designated by σ_x'

$\tau = f\sigma_x$ - specific friction between the soil of the core and the wall of the pile.

The coefficient $m_1 = \frac{\sigma_x}{\sigma_y'}$ is introduced.

After transformation of (5):

$$m_1 = \frac{(1+\sin^2\phi) - 2\cos\phi\sqrt{tg^2\phi - f^2}}{\cos^2\phi + 4f^2} \quad (6)$$

The solution requires that σ_x be defined with relation to the average vertical stress σ_y. For the purpose the greatest vertical stress in the core axis is used, which is the principal normal stress σ_1. It is accepted by approximation that the decrease of this stress towards the wall of the pile up to the value σ_y' is linear.

After applying the axial symmetry

$$\sigma_y = \sigma_y' + \tfrac{1}{3}(\sigma_1 - \sigma_y') \quad (7)$$

From Mohr's relation it is well known that

$$\sigma_y' + \sigma_x = \sigma_1 + \sigma_3; \quad (8)$$

$$\sigma_3 = \sigma_1 tg^2(45 - \tfrac{\phi}{2}) = \sigma_1 m_o. \quad (9)$$

From equations (7), (8) and (9)

$$\sigma_x = m\sigma_y, \text{ where } m = \frac{3m_1(1+m_o)}{3 + 2m_o + m_1} \quad (10)$$

The quantities participating in the expression for m are constant, therefore m is also constant. This simplifies the integration of the differential equation (4). By applying the limiting condition that the stress at the bottom end of the pile is R_p, i.e. for y = o, $\sigma_y = R_p$, the integral of the equation is

$$Y = \frac{d}{4mf}\ln\frac{\gamma d + 4mfR_p}{\gamma d + 4mf\sigma_y}. \quad (11)$$

where d is the internal diameter of the pile.

The height where the stresses caused by R_p are damped is determined by assuming $\sigma_y = 0$ in (11)

$$H = \frac{d}{4mf}\ln\left(1 + \frac{4mfR_p}{\gamma d}\right). \quad (12)$$

Equation (11) can be solved also with relation to σ_y. Then the expression for the variation of σ_y as a function of y is obtained in explicit form

$$\sigma_y = \left(\frac{\gamma d}{4mf} + R_p\right)\exp\left(-\frac{4mf}{d}y\right) - \frac{\gamma d}{4mf}. \quad (13)$$

The settlement of the core is

$$S_1 = \int_0^H \frac{\sigma_y\,dy}{E_o} \quad (14)$$

Assuming that the modulus of deformation E_o = const. and replacing in formula (14) σ_y from formula (13) and after integration

$$\Delta H = \left(\frac{\gamma d^2}{16E_o m^2 f^2} + \frac{R_p d}{4E_o mf}\right) \times$$
$$\times\left[1 - \exp\left(-\frac{4mfH}{d}\right)\right] - \frac{\gamma dH}{4E_o mf}. \quad (15)$$

Making use of formulae (10) and (13) the resulting force of friction T between the pile and the soil of the core can be calculated as follows:

$$T = \int_0^H \pi d\tau\,dy = \int_0^H \pi d f m\sigma_y\,dy \quad (16)$$

After replacing σ_y from (13) into (16) and solving the integral

$$T = \left(\frac{\pi\gamma d^3}{16mf} + \frac{\pi d^2 R_p}{4}\right) \times$$
$$\times\left[1 - \exp\left(-\frac{4mfH}{d}\right)\right] - \frac{\pi\gamma d^2 H}{4}. \quad (17)$$

It follows from the condition for statical equilibrium of the core that

$$R_p \le \frac{T}{F} \quad (18)$$

In case $R_p > \frac{T}{F}$ the height H will be increased or the core will slide within the pile.

In case the height of the core is smaller than the value of H determined after formula (12), then the actual height of the core is put in the expression (17) for T in place of H. The value obtained for T is the maximum force which can be transferred from the core to the pile.

Besides, due to deformation of the core, the pile settles also because of the deformation of the stressed zone underneath. This deformation is obtained after Schleicher

$$S_r = \frac{wR_p \sqrt{F_D}\,(1-V^2)}{E_o} \qquad (19)$$

where $F_D = \frac{\pi D^2}{4}$.

NUMERICAL SOLUTION

Sand characteristics:

Angle of shearing resistance $\phi = 30^o$
Angle of wall friction $\delta = 20^o$
Modulus of deformation E_o = 35000 kPa
Internal diameter of pile d = 1.11 m
External diameter of pile D = 1.35 m
Limit force, determined by test loading P_{lim} = 2050 kN
Skin friction at 4 mm limit sliding deformation read from Fig.4 P_f = 600 kN
Point resistance P_P = 1450 kN

After formulae (6), (9) and (10)

$m_1 = 0.37 \quad m_o = 0.333 \quad m = 0.366$

$$R_p = \frac{P_p}{F_D} = \frac{1450}{1.43} \simeq 1000 \text{ kPa}$$

After formulae (12) and (15)

$H = 8.10$ m $\quad S_1 = 54$ mm

Fig. 9 indicates the variation of the vertical stresses σ_y along the height of the core.

After formula (19)

$S_2 = 24$ mm

Total settlement

$S = 78$ mm,

which coincides with that obtained at the loading test. This settlement was considerable above that admissible for the structure, which imposed an elongation of the pile.

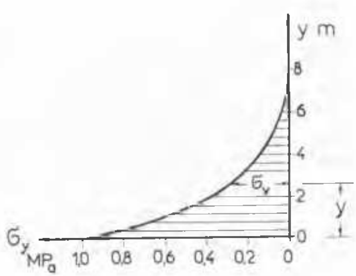

Fig. 9 Variation of average vertical stresses along the height of the core

The resulting force of friction calculated after formula (17) is T = 890 kN, approximately equal to $R_p F_d$ = 960 kN.

The ultimate bearing capacity, determined after Berezantsev (1970) is

$$R_p^{lim} = 1430 \text{ kPa} > R_p = 1000 \text{ kPa}.$$

CONCLUSIONS

1. The bearing capacity of hollow piles driven by vibration calculated after 'dynamic' formula (1) and after empirical formula (2) based on statistical data, corresponds satisfactorily with the results of the loading test, without considering the settlement of the pile. Formulae derived on the basis of a non-deformable soil core give a bearing capacity many times higher.

2. After overcoming the skin friction, during which the pile is subject to relatively small vertical displacements, the stresses are transferred to the bottom and a great part of them are taken over by the soil core. Depending on the shearing resistance and the deformation properties of the soil and on the internal diameter of the pile, the deformation of the core can be considerably greater than the settlement of the soil underneath.

3. The settlement of the core is relatively slow and its duration depends on the consolidation characteristics of the soils.

4. The main criterion for the admissible loading of hollow friction piles of big diameter is the deformation of the core. This deformation together with the settlement of the soil under the pile should not exceed the admissible deformations of the structures.

5. The formulae (12), (15) and (17) derived here allow the evaluation of the active height and settlement of the soil core, as well as the resulting force of friction between core and pile.

REFERENCES

Berezantsev, V.G. (1970) "Calculation of the Earth Foundation of Structures". Stroiizdat, Leningrad, pp. 186-187 (in Russian).

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