Load Bearing Capacity and Deformation of Piled Foundations

Force portante et déformations des fondations sur pieux

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Summary

The authors give the results of theoretical and experimental investigations on the load bearing capacity of single piles in dense sand, carried out by V. G. Berezantzev. The equilibrium analysis of axial loading resulted in the development of a formula for the load bearing capacity of a pile footing.

Laboratory and field investigations were carried out by V. N. Golubkov on settlement of piled foundations. The settlement is proportional to the square root of the size of footing.

V. S. Khristoforov analysed the load bearing capacity and deformations of pile groups and pile trestles. The design of piled foundations with free length of piles should be based on the analysis of deformations of frames with rigidly anchored struts.

Sommaire

La première partie de l'exposé rend compte des recherches effectuées par V. G. Bérésantcev, sur la force portante des pieux isolés battus dans les sables compacts. La force portante de la pointe se détermine en appliquant la théorie de l'équilibre limite dans le cas d'une symétrie axiale.

Dans la deuxième partie sont exposés des résultats des expériences de laboratoire et sur chantiers sur le tassement des fondations sur pieux, effectuées par V. N. Goloubkov. Les tassements sont proportionnels à la racine carrée de la surface de transmission de la pression.

Dans la troisième partie sont exposées des recherches expérimentales sur le travail des groupes de pieux et des pieux à traction utilisés dans les constructions, effectuées par V. S. Khristoforov. Les calculs des fondations sur pieux avec hauteur libre sont à baser sur l'analyse des déformations des portiques à montants encastrés rigidement.

1. Bearing Capacity of Piles in Dense Sands

Modern methods of pile driving, more particularly those which make use of vibration, have brought about a considerable development of piled foundations. Investigations on the load bearing capacity and deformation of such structures are thus of great practical value.

The possibility of using long piles of large cross-sectional dimensions permits very heavy foundations to be constructed, such as those for bridge piers, using only a few piles sunk to dense sand or gravel. Experiments prove that the bearing capacity of sands, under the ends of such piles, considerably exceeds the values given by well-known methods of pile design. Better results can be obtained by using the following new method based on the limit equilibrium theory and on the experimental investigations described [1], [2].

If a deep foundation has a depth ratio greater than $3 \div 4$ $\left(\frac{D}{B} \geqslant 3 \div 4\right)$, sand failure may occur after considerable compaction, accompanied by the displacements of a small volume of soil. Hence, the load bearing capacity in this case is determined only by foundation settlement.

These special features of sand failure, however, are peculiar to such foundations, when, during the process of construction, no additional compaction of sand takes place within the limits of depth equal to or more than the width of foundation (foundations in trenches, sinking wells, caissons, etc.).

Substantially different conditions exist for the footing of a single pile sunk deeply by an ordinary or vibratory piledriver. During the sinking process, the pile displaces soil and forms around itself a relatively large compacted zone of soil, which changes failure conditions of the soil. The limit equilibrium under the end of the pile corresponds to the displacement of considerably developed sliding zones of compacted sand. These zones reach the horizontal plane below the foot of the pile. This phenomenon is illustrated in Fig. 1, showing sand deformation during model tests (the model was sunk to the depth equal to ten times its width). The model tests were carried out at the Leningrad Institute of Railway Engineers.

Hence, the load bearing capacity under the end of such single piles can be approximately determined using the scheme 2 (a) [1], given in Fig. 2. The surcharge of sliding zones on the level of the pile end is equal to the weight of cylindrical volume "bcda - $b_1c_1d_1a_1$" decreased by the value of internal friction T (Fig. 2) on the lateral surface of this volume. This internal friction appears during the displacement of the volume "bcda - $b_1c_1d_1a_1$" in the process of soi compaction under the pile end.

The value of internal friction on the depth Z can be approximately calculated as a product of tang φ_D (φ_D - angle of internal friction) and e_z - the lateral pressure on the surface "bc, b_1c_1" with radius $l_0 = l + R$.

By means of the analysis of lateral pressure on the cylindrical surfaces in the axial symmetrical problem of limit equilibrium theory [2], e_z is found by the following formula :

$$e_z = \frac{\tan\left(\frac{\pi}{4} - \frac{\varphi_D}{2}\right)}{\lambda - 1} \left\{ 1 - \left[\frac{1}{1 + \frac{Z}{l_0} \tan\left(\frac{\pi}{4} - \frac{\varphi_D}{2}\right)} \right]^{\lambda - 1} \right\} \gamma_D l_0 \quad (1)$$

φ_D and γ_D correspond to the soil of overburden,

$$\lambda = 2 \tan \varphi_D \tan \left(\frac{\pi}{4} + \frac{\varphi_D}{2} \right)$$

φ and γ denote the same characteristics for the soil of footing.

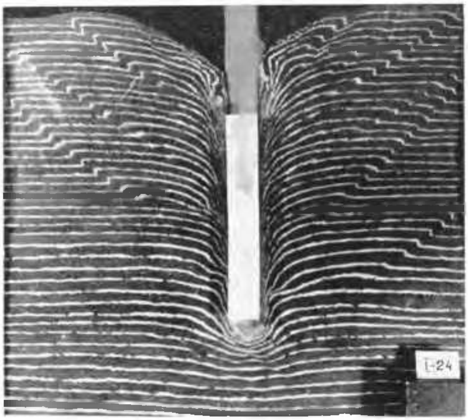

Fig. 1 Sand deformations after deep sinking of a pile model. Déformation du sable pendant l'enfoncement du modèle réduit du pieu en profondeur.

Corresponding to the form of sliding surfaces [2] "l_0" is calculated from expression :

$$l_0 = R \left[1 + \frac{\sqrt{2} \cdot e^{\left(\frac{\pi}{2} - \frac{\varphi}{2}\right) \tan \frac{\varphi}{2}}}{\sin \frac{\pi}{4} - \frac{\varphi}{2}} \right] \tag{2}$$

Using the expression (1), the following formula gives the average value of the surcharge :

$$q_T = \alpha_T \gamma_D D \tag{3}$$

Coefficient "α_T" is a function of ratio $\frac{D}{B}$ and of angle φ_D (see table 1).

Table 1

$\frac{D}{B}$ \ φ_D	26°	30°	34°	37°	40°
5	0·75	0·77	0·81	0·83	0·85
10	0·62	0·67	0·73	0·76	0·79
15	0·55	0·61	0·68	0·73	0·77
20	0·49	0·57	0·65	0·71	0·75
25	0·44	0·53	0·63	0·70	0·74

The axial symmetrical limit analysis [1] gave the formula for average value of ultimate bearing capacity :

$$q_f = A_k \gamma B + B_k q_T$$

or, corresponding to the (3) :

$$q_f = A_k \gamma B + B_k \alpha_T \gamma_D D \quad .... \quad (4)$$

where A_k and B_k depending upon φ, are given by the curves on Fig. 2.

The result of bearing capacity calculations for dense sands under the single piles, after (4), show that theoretical data are in reasonable agreement with the experiments. These values are higher than those obtained by existing methods.

For example, for the pile $B = 63$ cm, driven to the fine sand ($\varphi = 34°$, $\gamma = 1$ t/m^3) at 26 m ($\varphi_D = 26°$, $\gamma_D = 1$ t/m^3) formula (4) gives $q_f = 586$ t/m^2, by the tests $q_f = 600$ t/m^2, by technical conditions $q_f = 340$ t/m^2.

Formula (4) may be used only for the computation of bearing capacity of soil under the end of the pile; the lateral friction, if necessary, can be determined by conventional methods.

The load bearing capacity of foundations with a large number of piles is determined by settlement. The results of investigations on soil deformations and foundation settlements are discussed below.

2. Settlement of pile foundation and deformations of subsoil

Features typical for the development of soil deformations and settlements of piled foundations were investigated in field and model pile tests. The results show that the settlement "w" of piles and piled foundations is a compound function

$$w = f(P, D, \sqrt{A}, E, H_a)$$

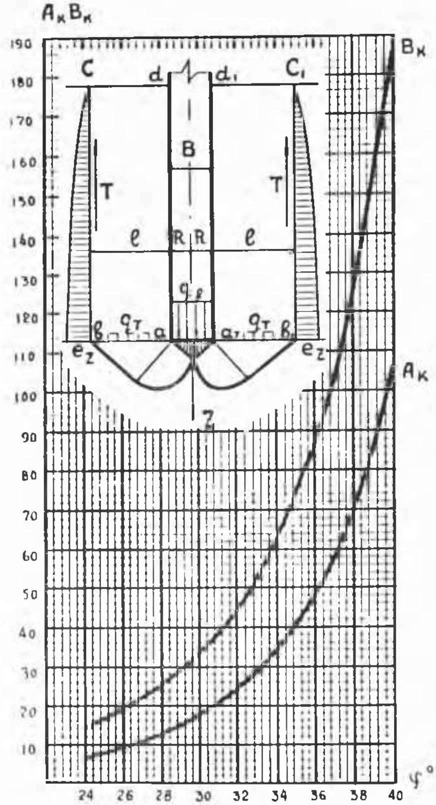

Fig. 2 Sand failure and curves of coefficients in formula (4). Schéma de l'état-limite du sol sous le pieu et graphique donnant les coefficients dans la formule (4).

where P is the load on a pile, D the depth of pile driving and A is the area transmitting the load from the piled foundation to the soil, E is the modulus of deformation of soil, H_a is the depth of the deformation zone.

The depth of the deformation zone H_a was determined in field conditions for a particular case at the footing of a group of 16 piles, 5·6 m long, in fine silty sand. The soil deformation was observed on nine marks, each mark being driven in a bore-hole to a predetermined depth. After the marks had been driven into the soil, the borehole was filled with sand and the casing pipe was withdrawn. The deformation of the marks was recorded by string deflection devices to an accuracy of 0·1 mm.

This test proved that the depth of the deformation zone depends on the value of the load; when changing the load on a pile from 14 tons to 22·5 tons or under a unit pressure of 1·4 - 2·2 kg/cm^2 on the soil in the plane of the pile ends, the depth of the deformation zone H_a changes from $0{\cdot}4\sqrt{A}$ to $0{\cdot}7\sqrt{A}$.

Tests with model piles at the depth ratio of 20 (without a compaction zone around the piles) were carried out in fine dense sands. Small-shot was used as a depth indicator of deformation. Test results showed :

1. The volume of the deformation zone depends on the value of unit load which a pile and a group of piles transmit to the soil, and on the size of the bulb of pressure.
2. With equal settlement, the value of unit pressure on the soil is more for a single pile, but the depth of soil deformation is greater for a group of piles.

Series of field tests with piles driven to a depth of 3·6 m 5·6 m and 7·5 m also confirmed that the load transmitting area influenced the value of general deformation. The investigations of single piles and piled foundations were carried out under the same soil condition, namely, in a silty fine sand layer of 10·7 m, depth. In table 2 are shown data of the mechanical properties of the soil.

Table 2

Indicators of mechanical properties of soils	*Dimension*	*Value*
Water content	per cent	23-28
Bulk density	t/m^3	2·02
Density of soil particles	t/m^3	2·65
Porosity	per cent	from 45 to 34
Modulus of deformation of soil at the top of layer	kg/cm^2	75-100
Modulus of deformation of soil at the foot of layer	kg/cm^2	350
Angle of internal friction	degrees	31°
Content of sand grains with size from 0·25 mm to 0·05 mm	per cent	92
Grains less than 0·005 mm	per cent	4

Friction in the soil along the sides of the pile forms a compacted volume of soil around the pile during its settlement [3], [4]. This volume, taking part of the load, transmits it to the soil in the plane of the pile ends; the rest of the load is transmitted by the pile ends to the ground. The size of the bearing volume of soil is determined by the angle "α" between the external faces of the bearing volume of soil and the lateral surface of the pile. Angle "α" changes from 0° to 7° depending on types and density of soil.

The relationship between settlement, load transmitting area and number of piles is shown by Figs. 3 and 4.

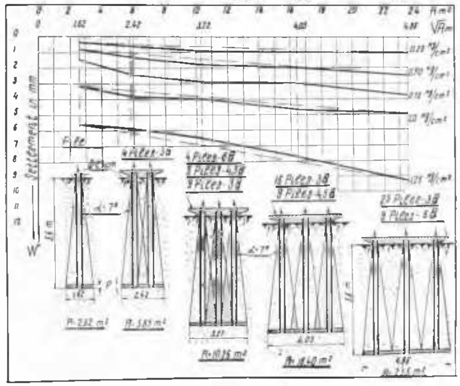

Fig. 3 Results of field tests of piles and piled foundations with piles driven to a depth of 5·6 metres.

Résultats des essais sur chantier des pieux et des fondations sur pieux avec une profondeur d'enfoncement de 5,6 m.

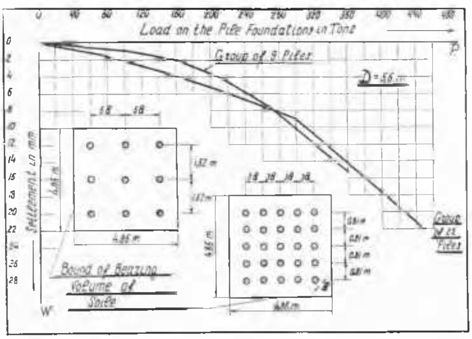

Fig. 4 The relationship between loading and settlement of piles.

Graphique du tassement d'une fondation sur pieux en fonction d'un nombre de pieux variable, mais avec une surface constante de transmission de la charge sur la fondation.

Maximum resistance of piles is provided by the contact of their bearing volumes. This is the limit position for joint déformation of soil and piles. The settlement of piled foundations is proportional to the square root of the load-transmitting area. Under conditions of equal loading, minimum settlements were observed with single piles and the maximum settlements with piled foundations (Fig.3). Piled foundations with an equal load transmitting area at the level of the pile ends, but with a different number of piles have practically equal settlements under equal loads (Fig. 4).

The results of field tests of piled foundations with the maximum size of load transmitting area 4·86 × 4·86 m show that the bearing capacity of piled foundations depends mainly upon settlement. Conventional methods of settlement computation may be used.

3. Work of piles and pileworks

Experimental investigations with large models of piles were carried out in the Laboratory of Soil Mechanics of the All-Union Research Institute named after Vedeneev. The aim of these tests was to assemble the data relating to stresses in piles when they are driven in groups, pile trestles and both vertical and trestle piles in rigid pileworks.

General characteristics of the test structures are shown in Fig. 5.

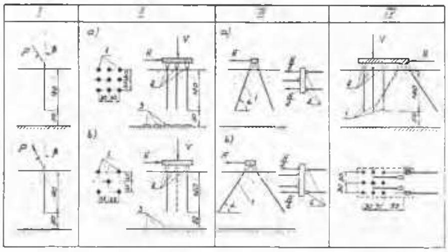

Fig. 5 Arrangement of test installation :
I. single piles ; II. pile groups : (a) of nine piles, (b) of five piles ; III. pile trestles : (a) of three piles, (b) of four piles ; IV. pile assembly ; 1. piles, 2. measuring heads, 3. hydraulic capsules.
Schéma des installations d'essai :
I. pieux isolés ; II. groupe de pieux : (a) de neuf pieux ; (b) de cinq pieux ; III. pieux à traction : (a) trois pieux ; (b) quatre pieux ; IV. Grillage en pieux ; 1. les pieux ; 2. têtes mesuratrices ; 3. capsules hydrauliques.

Steel tubes, 9 cm in diameter, furnished with caps and sunk in soil to 1·2 m were used as model piles. The ultimate load on the test pile reached from 1 to 1·5 tons. The maximum vertical component of the load on the foundation composed of 13 piles (see Fig. 5, IV) reached 12·6 tons and the horizontal component was 3·8 tons. Each test structure was provided with special equipment for measuring the stresses in the heads of the piles. The general arrangement of a measuring head is shown in Fig. 6. The measuring head was designed so that any load exerted on a pile created longitudinal forces in the pile head drafts which were measured by means of resistance strain gauges.

Measuring of stresses in soil at the bottom of the tank was made by means of string and pick-up hydraulic capsules.

The tests were carried out in sands.

The main results of these investigations are as follows :

The bearing capacity of piles subjected only to axial force is less than that of piles subjected to the combined action of axial force and bending. For instance, it was found that the resistance of single piles to drawing could be increased more than 1·5 times, if the drawing force formed the angle β ($\tan\beta = 1:3$) with the axis of the pile.

Under the initial loading, the settlement of a pile group proves to be greater than that of single piles. However, the ultimate load on a pile group under which its settlements greatly increase is much greater than the sum of the ultimate bearing capacities of single piles of which each pile group is composed. If reloaded, the relationship between settlement and load for the pile group does not differ appreciably from that of single piles. After reloading, the value of the ultimate bearing capacity of a pile group increases slightly.

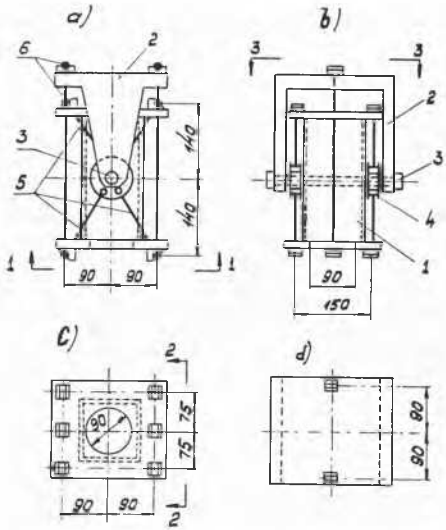

Fig. 6 General scheme of measuring head :
(a) front, (b) view 2-2, (c) view 1-1, (d) 3-3. 1. housing, 2. bearing plate with cheeks, 3. central bolt, 4. washers, 5. drafts, 6. cylindrical hinges.
Schéma principal de la tête mesuratrice :
(a) élévation, (b) vue 2-2, (c) vue 1-1, (d) vue 3-3. 1. corps, 2. dalle d'appui avec des joues, 3. boulon central, 4. rondelles, 5. tirants, 6. articulations cylindriques.

The character of work of pile groups differs from that of single piles. Under the initial loading, soil compaction occurs around the pile group. This compaction greatly influences the behaviour of piles under repeated loading; settlements greatly decrease. While loads are kept within permissible limits, the relationship between settlement and load is practically linear.

Piles in pile trestles are subjected both to axial forces as well as bending stresses.

The load bearing capacity of pile trestles increases with the increase of rigidity of pile connections and it is always greater than the design values allowing for a hinged system. With rigid pile connections the ultimate bearing capacity of a pile trestle may be twice as much as the design values. The load bearing capacity of such piles proves to be considerably higher than that which is determined by standard pile tests. It is generally observed in piles under tension.

The angle of pile trestle inclination should not be limited to the value $\tan\alpha_k = 3:1$ in every case. With the decrease of tangent of this angle up to the value of $\tan\alpha_k = 1·5:1$ the bearing capacity of the pile trestle due to horizontal load more than doubles and the construction becomes less liable to deformation.

The simplified computation of a pile assembly with free length of piles according to the scheme of a frame with a continuous collar, and elastically-yielding hinge supported struts sometimes results in reducing the bearing capacity to half its real value.

Experimental data show that satisfactory results can be obtained with the help of a scheme of frame with rigidly fastened struts. The forces so obtained were in reasonable agreement with experiments if a correct selection of fastennings for piles in soil is made.

The tests showed that the design of the piling should be carried out in accordance with the second ultimate state (calculation of deformations) and with the third ultimate state (fissure formation).

The distribution of stresses between piles obtained by tests is in agreement with calculations based on elasticity.

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