BEARING CAPACITY AND METHOD OF PENETRATION OF PILES IN LOESS SOILS

LA FORCE PORTANTE ET LE METHODE DE L'ENFONCEMENT DES PIEUX DANS LE LOESS

НЕСУЩАЯ СПОСОБНОСТЬ И СПОСОБ УСТРОЙСТВА СВАЙ В ЛЕССОВИХ ГРУНТАХ

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SYNOPSIS. Large-scale field investigations of stress-deformation state of the pile-loess soil system have permitted to establish the essence of the pile behaviour under the vertical load in loess soils with different water content. On this basis the proposal calculation scheme for estimating the bearing capacity of the piles and calculation formula are given. The comparison of the calculated and experimental data yields satisfactory results. This part of the report is presented by A.A.Grigorian.
E.S.Ivanov describs the method of vibro-penetration of the big centrifugal concret piles into the loess·mass and presents the results of the settlements of the test piles under long-term wetting and overburden.

One of the main and difficult problems of soil mechanics is the establishment of the behaviour of depth footings in clayal soils with friction and cohision. In view of the absence of the rigorous theory the problem in question for piles in loess soils was solved on the basis of large-scale field investigations of the piles under static load. The present report is based on the results of static tests of 20 driven and 18 built-in-place piles, carried out with the wetting of the soil in their bases. The most complete was the set of tests of the piles of different constructions and sizes, performed on the site in Nikopol (the Ukraine).

Table I

The average values of the characteristics of the soils from the test site in Nikopol

Depth Ranges, m	Water Content %	γ_d g/cm^3	γ_s g/cm^3	Relative collapse under 2.5 kg/cm^2	Plasticity, %			Granulometric composition, %			Shear with water	
					W_L	W_p	I_p	>0.05 mm	0.05-0.005 mm	<0.005 mm	φ degrees	C kg/cm^2
0+8	6.5	1.36	2.68	0.07	26.0	18.0	8.0	21.0	66.2	12.8	20	0.05
8+23	5.0	1.46	2.67	0.02	23.0	17.0	6.0	32.2	58.1	9.7	18	0.06

In addition to the piles of usual construction, tests of the piles of special construction which permitted to estimate separately the ultimate value of the skin friction and that of the tip resistance and also the tensometric measurements were carried out. Two built-in-place piles had bulbs at the lower end. Two driven piles were provided with tenso-devices for measuring the values of the longitudinal forces and lateral pressures at different levels along the pile (A.A.Grigorian,1971).

Table II

The sizes and the ultimate loads of piles tested in Nikopol

Test	Kind of pile	Length, m	Cross-section, m	Degree of satur.	Ultimate load, t: skin fric.	lower end	whole
1	D.[x] special	6,0	0,3x0,3	0,33	20,0	10,0	33,0
2	D.special	6,0	0,3x0,3	0,80	10,0	6,0	15,0
3	D.tensometric	6,0	0,3x0,3	0,33	20,2	9,8	30,0
4	D.tensometric	6,0	0,3x0,3	0,80	10,5	3,5	14,0
5	D.tensometric	6,0	0,3x0,3	0,19	41,0	19,0	60,0
6	D.tensometric	6,0	0,3x0,3	0,80	13,0	4,5	17,5
7	B.[xx]	16,0	d=0,5	0,80	-	-	70,0
8	B.spesial	16,0	d=0,5	0,80	50,0	20,0	70,0
9	B.friction	16,0	d=0,5	0,80	55,0	-	-
10	B.with bulb	16,0	d=0,5 d_b=1,6	0,80	-	-	130,0
11	B.with bulb	22,0	d=0,5 d=1,6	0,80	-	-	155,0

D.[x]-driven; B[xx]- built-in-place.

The settlements of all the tested piles from the beginning of loading up to the disruption in loess soils of natural water content and fully saturated did not exceed 1 cm in most cases. The disruption was characterized by non-stabilized settlement under constant load. The load before the disruption was assumed to be ultimate load.

The principal calculation scheme for estimating the bearing capacity of the short driven pile was proposed earlier (A.A.Grigorian,1971). This article gives the calculation scheme in a more exact forme suitable for long (more than 10m) driven and built-in-place piles as well as for different states of water content of the soil.

The possibility of the collapse of the soil under overburden was excepted which makes the results more general.

The task of ultimate equilibrium of soil, capable of compaction at the base of the pile, was considered.

The failure of soil base of the pile occurs only after a negligible excess of the soil resistance forces at a certain ultimate surface. The shape and size of the ultimate surface are established on the basis of experimental investigations.

Within the part of the pile length l_1, this surface passed along the contact one between the pile and the soil, below this section the surface takes on the shape of a truncated cone ABEF with the bottom part in the shape of a lateral surface of a truncated cone BCDE and a sphere segment surface COD (Fig.1).

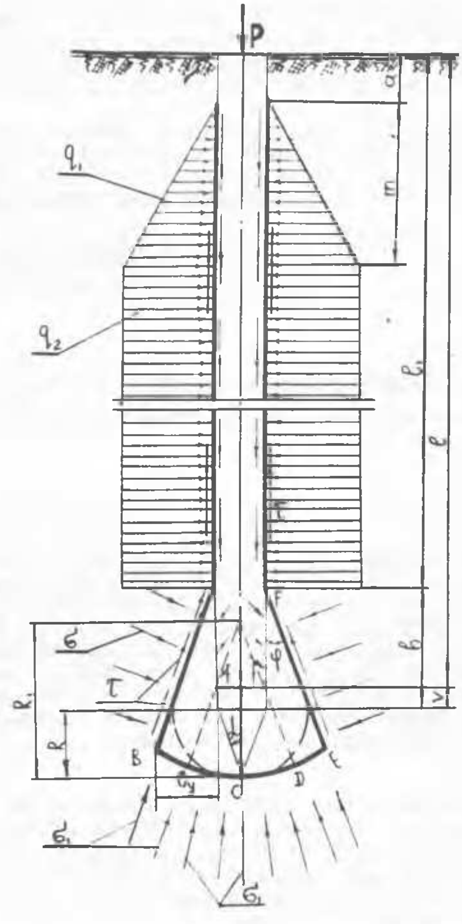

FIG.1. CALCULATION SCHEME OF THE BEARING CAPACITY OF A PILE

The lateral surface of the truncated cone ABEF is inclined to the vertical line at the angle of the internal friction of the soil φ . The lowest point O of the segment surface coincides with the acrid end of the compacted cone core. The compaction of the soil within the cone core under the driven piles occurs in the process of driving. For this reason the plastic deformations during the process of loading are practically negligible. The compaction of the core under the built-in-place piles occurs during the loading and in this case a certain settlement proportional to its diameter is observed.

Along the pile length at the ultimate state before the disruption two different sections should be distinguished: one within the length l_1- the section of sliding between the pile and the soil and the other, lower,the section of co-behaviour of the pile and the soil. At this lower section more significant forces (mostly normal)are transmitted to the soil than at the one above it,at the section of sliding of the pile along the soil.

The most part of soil within the volume ABCODEF is in a plastic state but the deformations with soil compaction take place only when the normal stresses along the ultimate surface BCODE reach the maximal value σ_1 . At the same time the shear of the soil occurs along the surface of the truncated cone ABEF. The deformations of compression develop at the angle of 90^o to the tangent at each point of the surface BCODE.Thus,after the ultimate equilibrium state is reached the soil deformations round the lower end of the pile take place downward along the pile and at a certain angle to it without upward deformations. This kind of failure process is typical of the pile base of the soil capable of compaction.
As a result of disruption some settlement of the pile will occur with the subsequent formation of another ultimate surface,similar to the previous one,but displaced downward.This process will continue repeatedly.
The proposed calculation scheme was experimentally confirmed. The deformations of the soil in the base of 6m long driven piles were investigated earlier (A.A.Grigorian,1969).The displacement of each indicator of deformation due to disruption and subsequent settlement of 25cm is given in Fig.2 which is a photo of the vertical plane in the soil mass exposed by cutting a hole.

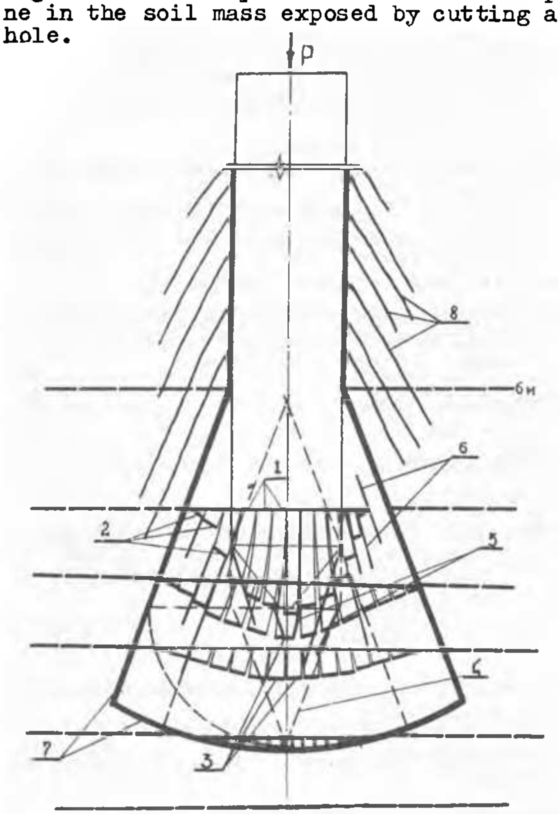

FIG.2 THE DEFORMATIONS,TRACES OF SHEAR PLANES AND THE ULTIMATE SURFACE AT THE LOWER AND OF THE PILE.

The picture of the displacement is obtained by connecting the ends of the wooden stakes (indicators in our case) in their initial 1 and final 2 states. The soil within the considered volume at the lower end of the pile begins to compact after the ultimate equilibrium state is reched. All the indicators of deformations,except the ones immediately adjoining the pile, at the depth above 6m did not change their positions and testified about the sliding of the pile to the soil at this section.The indicators at the axis of the pile 3 within the cone core were vertically displaced down.The traces of shear planes inside the core in its boundary 5 and also those outside the core 6 were observed.The ultimate surface of resistance is indicated by the heavy line 7. The system of parallel traces of shear planes 8 inclined at the angle to the vertical line are shown above the pile tip.They are also presented in the photo made from nature (Fig.3).

FIG.3. PHOTO OF THE TRACES OF SHEAR EXPOSED BY CUTTING A HOLE

The above traces were due to driving the pile and the lower ones due to the static loading. This is the evidence of the general nature of the soil resistance at the pile base under static and dynamic loading. As for the data of tensomeasurements they are presented by the curves of longitudinal forces at different levels along the tensometric pile (Fig.4).

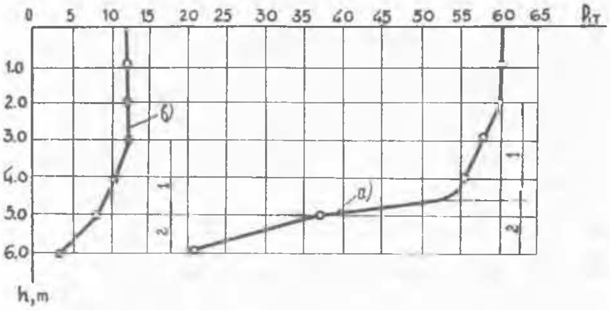

FIG.4. THE LONGITUDINAL FORCES P_i AS MEASURED BY MEANS OF THE DYNAMOMETRIC ELEMENTS OF THE TEST PILE

These forces are measured by means of dynamometric elements in two states of water content: a) natural and b) fully saturated. Each curve in Fig.4 permits to distinguish two different sections: 1 and 2 above mentioned, except the most upper vertical section where the resistance of soil is zero. Section 2 of the pile begins deeper than the five meter level in fully saturated soil but in the soil of natural water content the same section begins deeper than the four meter level. The significent forces (mostly normal) are transmitted from the pile to the soil at sections 2 especially in the soil with natural water content. The normal stresses were also fixed by means of pressure boxes placed at different levels along the pile (A.A. Grigorian,1971). The lateral pressure on the pile in fully saturated soil before loading increased linearly only within some part m (Fig.1). This pressure was equal to 0,34kg/cm^2 at the depth of 5,5m. The pressure boxes did not change their readings in relation to the state of rest in loading the pile in fully saturated soil. On the contrary the lateral pressure on the pile before loading and after the relaxation of stresses which are due to pile driving in the soil with natural water content equal to 0,2 along the whole length were zero. The presser boxes at the depth of 5,5m showed considerable increase of normal stresses in loading in the soil with natural water content. At the state of ultimate equilibrium this pressure was ~ 2kg/cm^2 while all the readings of the other boxes placed above remained zero. This measured value was close to the average value of the pressure obtained from the diference between the readings of dynomometric elements at the depth of 5 and 6m. The lower section of the pile in fully saturated soil begins at the level of the position of the lower pressure box which for this reason could not show the increase of pressure by loading.

The section of linear increasing of lateral pressure m at the state of rest on the basis of the results of testing the long built-in-place piles is approximately 10+15d,where d is the pile diameter.At deeper levels the lateral pressure takes the constant value q_2 corresponding to the value at the lower end of Section m.

From projecting all the forces at the ultimate surface on the vertical axis it follows:

$$P=P_1+P_2'+P_2''+P_2''' \qquad (1)$$

where P- ultimate load;
- P_1- resistance of soil at the section l_1 of sliding of the pile along the soil;
- P_2'- vertical components of stresses at the surface of the truncated cone ABEF;
- P_2'' -vertical components of stresses at the surface of the truncated cone BCDE;
- P_2''' -vertical components of stresses at the sphere segment surface COD.

$$P_1=U\{m[0,5\gamma\xi\, tg\varphi_c m+C_c]+q'(l_1-a-m)\}; \qquad (2)$$

where u-perimeter of the pile;
- a-section within which the resistance forces are not observed as overburden pressure is small; the value of a also depends on the degree of breach of the contact surface between the pile and the soil due to driving and for the short driven pile a=2+3m; for the built-in-place pile a=1m;
- γ - unit weight of soil,
- ξ - earth pressure coefficient equal to 0,4+0,6
- $\varphi_c C_c$ - angle of surface friction and adhesion between the pile and the soil respectively;
- m,d- the same as given above,
- q_2' - lateral soil resistance per unit of surface within the part of the length of the pile:l_1-a-m,

$$q_2'=\xi\gamma m\, tg\,\varphi_c+C_c, \qquad (3)$$

$$l_1=l+v-b,$$

where l is length of the pile, $v=\frac{d}{2}$;

$$b=\frac{d}{2}(Ctg\,\alpha-1)\,Ctg\,\varphi; \qquad (4)$$

where α - half of the angle of the core acrid which are determined by the fowllowing experimental formula, in degrees:

$$\alpha=\frac{\pi}{4}-\varphi(1+\frac{KC}{4}); \qquad (5)$$

where φ -angle of internal friction of soil in degrees,
C -cohesion of soil in t/m^2,
K—coefficient equal to 1, in m^2/t,
π-180°.

The limit of use of the formula (5):

$$\varphi(1+\frac{KC}{4})\leqslant 45°;$$

$$P_2'=Ql(\tau\, Cos\,\varphi-\sigma\, Sin\,\varphi); \qquad (6)$$

where $Q_1=\pi y\sqrt{1+Ctg^2\varphi}(y+d)$; (7)

$y= R_1Cos\varphi-(b-\frac{d}{2})\ Sin^2\varphi+b-\frac{d}{2}(1+Ctg\,\varphi)\ tg\varphi$; (8)

$R_1=\frac{d}{2}\ (Ctg\alpha+1+Ctg\varphi)$. (9)

$P''_2= Q_2\ \sigma_1\ Cos\ \varphi$, (10)

where $Q_2=\pi\sqrt{(y+\frac{d}{2}-R_1Sin\,\varphi)^2+[(b-\frac{d}{2})Sin^2\varphi]^2}\ \times$

$\times\ (y+\frac{d}{2}+R_1Sin\,\varphi)$. (11)

$P'''_2=\pi R_1^2,\ \sigma_1\ Sin^2\varphi$. (12)

The values of φ_c, C_c, φ and C are determined by the results of the consolidated direct shear of the soil at the laboratory. The shear stress τ in equation (6) is:

$\tau=\sigma\,tg\varphi+C$. (13)

The normal stress σ and the maximum main stress σ_1 in equations (6), (10), (12) and (13) are determined by the diagram of the shear of the soil where

$\sigma_3=\xi m\gamma$. (14)

NUMERICAL EXAMPLE

The ultimate load on a 16m long built-in-place pile with a diameter of 0,5m in the homogenous, highly collapsible soil in the saturated state has to be determined.

$a=1m$; $m=13d=6,5m$; $\gamma=1,6t/m^3$; $\varphi_c=\varphi=20^o$;

$C_c=C=0,5t/m^2$; $\xi=0,5$; $u=1,57m$.

$q_5^f = 0.5x1.6x6.5x0,364+0,5= 2,39t/m^2$ (Eq.3)

$\alpha=\frac{180}{4} - 20(1+\frac{1x0,5}{4}) = 22,5^o$ (Eq.5)

$b=0,25(2,414-1)2,747=0,95$ m (Eq.4)

$l_1=16+0,25-0,95= 15,3m$

$P_1=1,57\{6,5[0,5x1,6x0.5x0,364x6,5+0,5] + +2,39(15,3-1-6,5)\} =44T$ (Eq.2)

$R_1=0,25(2,414+1+2,747)= 1,54m$ (Eq.9)

$y=[1,54x0,94-(0,95-0,25)x0,108+0,95-0,25x x(1+2,747)]\ 0,364 = 0,50m$ (Eq.8).

$Q_1=3,14x0.50\sqrt{1+2,747^2}(0,50+0,50)=4,85m^2$ (Eq.7)

$Q_2=3,14\sqrt{(0,50+0,25-1,54x0,342)^2+[(0,95-0,25)x0,108]^2}(0,50+0,25+1,54x0,342)= =0,97m^2$ (Eq.11)

$\sigma_3=0,5x6,5x1,6= 5,2t/m^2$ (Eq 14)

$\sigma_1= 12,0t/m^2$; $\sigma=7,44t/m^2$;

$\tau = 7,44x0,364+0,5 =3,2t/m^2$ (Eq.13)

$P'_2= 4,85(3,2x0,94-7,44x0,342)=2,3t$ (Eq.6)

$P''_2= 0,97x12,0x0,94= 10,9t$ (Eq.10)

$P'''_2 =3,14x1,54^2x12,0,x0,342^2=10,4t$ (Eq 12)

$P=44+2,3+10,9+10,4=67,6t$ (Eq 1)

The ultimate load obtained in the static test N7 (Table II) is 70t. The deviation of the calculated value is -3.5%.

The usual methods of penetration of long driven piles into the soils with low water content (W=5%) by means of driving or vibro penetration proved to be ineffective. The investigation of making the pile foundations and their stability in collapsible soils in South Tadjikistan were carried out by E.S.Ivanov. The method of penetration of the centrifugal reinforced concrete piles 400mm in diameter to the depth of 24m was developed.

The installation for the penetration of piles consists of: 1. ВПП-I vibrator with low frequency, the exciting force of 18t the weight-carrying capacity of 6t, 2. the pump with the capacity of 30m^3/hour, pressure of 10-12 atm, 3, a source of power of 60 кvt, 4, a simple arrangement for supplying water and leading away the pulp (Fig.5).

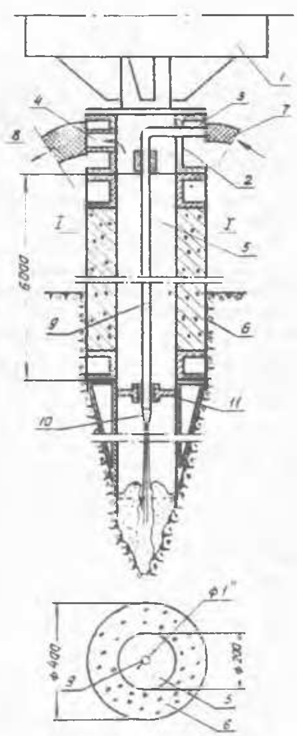

FIG.5. THE SCHEME OF PENETRATION OF THE FIRST LINK OF THE PILE
1-vibrator; 2-cap; 3 and 4-nipples for supplying water and leading away the pulp; 5- cavity of the pile, 6- the first link of the pile, 7 and 8- tubings; 9- wash-out tube; 10-cap, 11- nut, 12- cone tip with lower end opened.

The penetration of piles is done by separate 6m long links. The first link of the pile has a special metallic tag opened down and the central wash-out tube has a usual cap. The pump supplies water to the wash-out tube under the pressure of 10÷12 atm. and in 1-2 seconds the vibrator switchs on. The soil in the form of pulp is let out through the cavity of the pile and the tubing away from the construction site. Three workers perform all the technological process of penetration of the pile. The outlay of resource for the penetration of piles by this method are presented in Table III.

Table III

The outlay of resource on the pile penetration

The unit of calculation	the kind and outlay of resource: Amount of water, m^3	time of penetration, min	Power-consumption, K-h
1m of pile	0,5	1,25	2,5
1 link (6m of pile)	3,0	8,50	15,0

The piles were penetrated in loess soils at the depths of 6,12,18 and 24m below ground surface. The range of physico-mechanical characteristics of the soils down to the 25m depth are: the unit weight of dry soil 1,22-1,50g/cm^3, the porosity 54-41%, the plasticity index 7-15, the angle of internal friction 26-31°, the cohesion 0,10-0,17 kg/cm^2, unit collapse of soil mass under overburden pressure 8cm/m and under overburden plus 3kg/cm^2–16cm. The soil contains 12% sand fraction, 68% silt and 20% clay. In this soil condition the piles of different length were tested under static load and wetting. The critical loads changed their values as a result of wetting in the range from 50t to 25t for the 6m pile, from 80t to 49t for the 12m pile, from 100t to 60t for the 18m pile and from more than 100t to 80t for the 23m pile. The wetting of soil mass continued 4 months. The unit consumption of water per 1m^2 was approximatly 20m^3. The degree of saturation of the soil down to the depth 27m was 0,78. As a result of this wetting the soil settled together with the piles under overburden pressure. The settlement of the 6m long pile was 101cm, the 12m long-86cm, the 18m long-38cm and the 24m long-7cm. The settlement of the ground surface of the wetted mass was 130cm. The compaction zone occurs around the pile when the pile is penetrated. The pile base was exposed by cutting a hole. The density of the soil changed under long-term wetting in radial directions from the lateral pile surface (Fig.6). The investigations carried out on different soil mass levels showed, that the discompaction in the compaction zone and the compaction of soil in the remaining part due to overburden pressure occur simultaneously.

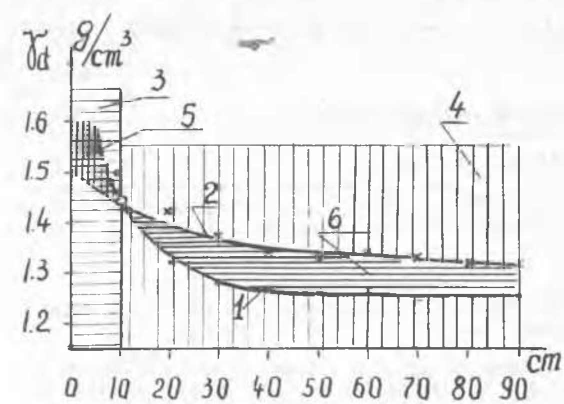

FIG.6. THE DEFORMATIONS OF SOIL AROUND THE PILE 5m BELOW GROUND SURFACE 1- the curve of soil density before wetting; 2-the same after wetting, 3-the zone of uncollapsible soils, 4- the zone of collapsible soils.

These phenomena connected with the negative skin friction in collapsible soils should be investigated in the future.

CONCLUSIONS:

1. The physical essence of the behaviour of the pils-soil system under vertical load has been established and has made possible to propose the calculation scheme and the method of calculating the bearing capacity of the piles in compacting soil with internal friction and cohesion.
2. The calculation of the ultimate load on the pile according to the proposed method may be done if the geometrical sizes of the pile and the value of the angle of internal friction, cohesion, unit weight of the soil and the earth pressure coefficient on rest at the uppear zone of soil massive are known. The comparison of the calculated and experimental data yields satistafctory results.
3. The method of the penetration of the tube form concrete piles into loess mass at the depth down to 24m is proposed.

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