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ANALYSIS OF PILE'S BEHAVIOUR USING ULTIMATE STRESS ZONES

LE CALCUL DES PIEUX COMTE TENU DES ZONES DE'LEQUILIBRE LIMITE

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Synopsis. Natural non-linear relations between horizontal displacements and settlements of piles and pile foundations can be succesfully discribed in taking into account the compression of the soil in the ultimated stress zones. The size of these zones increases due to load value. While using the fundamental laws of the Theory of the Ultimated Stress Conditions, one can take into account the influence of the depth of foundations on the level of deformations of the base. In this case there is no reason to change deformation characteristics with depth. The main principles of bearing capacity and deformations of pile foundations evaluation with consideration of ultimate stress zones development and internal and external bulging are presented herein. The method of calculations of short rigid piles on lateral loads and vertically loaded pile foundations are presented in this report. Experimental data proves reliability of these methods.

A lot of publications on the analysis of the laterally loaded piles was made during last several years. The main theoretical studies are based on the Theory of Elasticy or on the Beding value Theory. From our point of view one of the most complex studies on this score was conducted by Levacheva & Fedorovsky [1] in which the solution based on the assumption that beding value is changing with depth as a fraction-linear function was received.

Experiments both on scaled models and full-size instrumented piles were conducted. The results of investigations shows that the intensity of soil resistance increases with depth as linear function. The equations of the theory of Ultimated Stressed Conditions can easy discribe direct behaviour of laterally loaded piles.

This method needs less engineering assumptions than calculations, based on the other theories. The experiments show that for the laterally loaded piles there is no share failure at the soil surface. It means that local soil bulging and compression of the soil around the ultimated stress zones is the main reason for deformations. Such expiremental data can be used for solving the problem of deformations of pile foundations using the method of changing equilibrium states. The usage of this method can be shown on solving the problem for short rigid laterally loaded piles. In this case we can image the pile as short vertical beam the length of which l, width b & N-lateral strength in upper part of pile. The gorizontal displacements are Δ_1 and Δ_2 for upper and lower part of pile respectively. Under the load the pile rotates around the zero point situated on b from its bottom. As the result we can obtain two linear surfaces of resistance (fig. 1). The first surface can be caracterized with height h_1 and the second with characteristics h_2 and b. The value of soil resistance must be taken due to maximum value of pressure. The intensity of soil rsistance will lineary increase with depth, due to experimental data. The main idea of our method is to use Ultimated Stress Conditions equations not for calculations of the stability of piles in soil, but for the definition of horizontal displacements of laterally loaded piles. To solve this problem we received 7 equations for uncohesive soils. These equations can easily be transmitted for cohesive soils, but it is not the aim of our article

The total soil resistance of the upper part if the soil weight lineary increase with depth :

$$P_1 = \int_0^{h_1} b\gamma z\zeta^2 \, dz = 0.5 b\gamma h_1^2 \zeta^2 \qquad (1)$$

where b-pile width;
z-vertical coordinate;
h_1 - maximum height of the cone of resistance in the upper part of the pile;
γ - unit weight of soil;

The value of soil resistance in lower part of pile, can be determined if soil weight in all points of cone of resistance could be considered as maximum:

$$P_2 = b\int_0^{h_2} \gamma l \xi^2 dz = b\gamma l h_2 \xi^2 \qquad (2)$$

where h -maximum height of the surface of resistance in lower part of pile;

l-the length of the pile;

From the equations of eqilibrium on axis X one can receive:

$$N - P_1 + P_2 = 0 \qquad (3)$$

where N-the value of lateral strength;

The equation of moments will be:

$$N(l-\lambda/2) - P_1(l - 2h_1/3 - \lambda/2) = 0 \qquad (4)$$

From geometry one can receive:

$$\Delta_1/(l-\lambda) = \Delta_2/\lambda$$

$$\xi = tg(45+\varphi/2) \qquad (5)$$

φ -angle of internal friction;

The equations of equilibrium for the volume of pressing soil in the cone of resistance in the upper part and the volume of moving soil will be:

$$b\Delta_1(l-\lambda)/2 = \int_0^{\xi h_1} \frac{\beta}{E} \frac{\int_0^{h_1 - x/\xi} b\gamma z \xi^2 dz}{b(h_1 - x/\xi)} b(h_1 - x/\xi)dx \qquad (6)$$

β-coefficient which is influenced by Puasson Ratio;

E-modulus of deformation ;

x-lateral coordinate;

On analogy for the lower part of pile:

$$b\Delta_2 \lambda/2 = \int_0^{\xi h_2} \beta/E\ \gamma l\ \xi^2 b\ (h_2 - x/\xi)dx \qquad (7)$$

We can find the solution of the problem after solving the system of these 7 equations. Really for Δ_1 and Δ_2 we can receive:

$$\Delta_1 = \frac{2\beta}{3b(l-\lambda)E} P_1^{1.5} [2/(b\gamma)]^{1/2} \qquad (8)$$

$$\Delta_2 = \frac{\beta}{b^2 \gamma \lambda \xi l E}(P_1 - N)^2 \qquad (9)$$

and may be determined from equations:

$$\frac{\lambda}{l} = \frac{6(P-1)^2 - 2(P-1)P^{3/4}(6U)^{1/2}}{6(p-1)^2 - 4UP^{3/2}} \qquad (10)$$

$$\frac{16P^3}{9U^2} = (P-1)^2[1-(P-1)/U^2]^2 \qquad (11)$$

where:

$$P = \frac{P_1}{N} \quad \text{and} \quad U = \xi l(2b\gamma/N)^{1/2} \qquad (12)$$

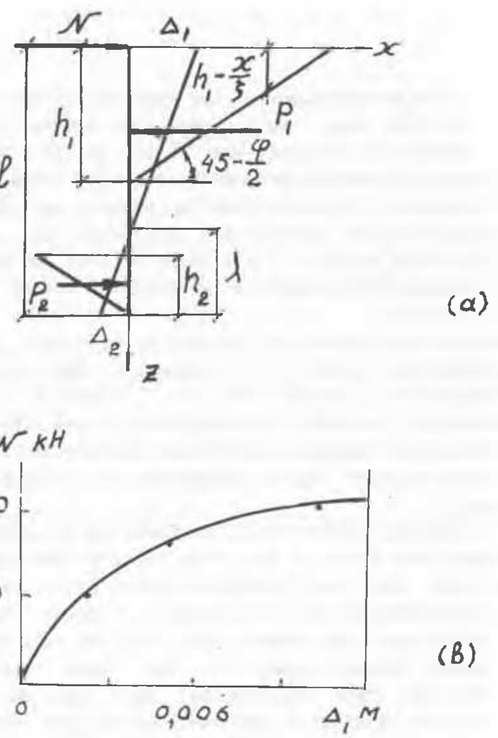

Figure 1. Soil-pile system & rezults of calculations

On fig. 1b we plotted the results of calculations from this method for pile with length 3m b=0.3m, angle of internal friction - 30°. The value of lateral strength changed from 0 to 20 Kn. One can see that we have obtained non-linear curve. We have recived new method with which we can recive non-linear correlation between lateral strength and displacements.

The results of tests of piles foundations shows that the value of bearing capacity may exceed the sum of the test values of bearing capacities of piles in the foundation[2] in one case and be less in the other.

We used new non-linear solution for bearing capacity of the earth-sheltered foundations[3] to generalize theoretically these tests . This solution is given in dimentionless formula:

$$\frac{\sigma_K}{\tau b} = \left[1 + \frac{F}{\tau b} K_\gamma\right]\left[\frac{h}{b} K_h + K_\varphi\right] + \frac{c}{\tau b} K_c \qquad (13)$$

where the bearing capacity of the ground under the pile base;

b-width of the base of the foundation;

h-the depth of the foundation base;

K_γ, K_h, K_φ K_c-bearing capacity factors;

$F=2(f_s+f)$, where f_s - skin resistance of the foundation;

f-skin resistance of the uplifting soil mass.

For clays the value of interaction between the ground under the foundation and the ground above the foundation base is $H=h/b<=5$. If $h/b>5$ the resistance of the ground does not depend on the depth. When $H>5$ $\sigma_K/\tau b$ is equal for $H=5$. The value of F is supposed as the average value within the range of 5b above the pile base. It makes it possible to take into consideration the heterogeneity of the ground and the shear failure under the conditions of internal and external soil bulging.

The results of the comparison of values of σ_K calculated according to the formula 13 for the ground under the base of the driven pile with with actual of σ_f are given in the Table 1. The deviation of α values from the average value of α =1.70 does not exceed 15%.

Table 1

N	Author	soil	h m	b m	φ	c Mpa	τ T/m³	f_s Mpa	σ_f/σ_K=α
1	Bartolomey	tight plastic loam	12	0.3	19	0.02	1.97	0.38	1.63
2.	Bartolomey	semi-solid loam	6	0.3	17	0.02	1.9	0.4	1.68
3.	Bartolomey	soft plastic loam	12	0.3	20	0.024	1.98	0.28	1.94
			5	0.3	19	0.019	1.96	0.22	1.68
4.	Condrashov	tight plastic loam	4.2	0.3	19	0.42	2.18	0.49	1.51

For the calculations we suppose f= 0.5 f . For above mantioned ground conditions results of calculations are satisfactory. In general the values of σ must not exceed the values , that we obtain under the conditions of the soil bulging, taking into consideration the compressibility of the ground in the elastic and plastic core.

The bearing capacity of the pile foundation can be calculated as a massive which's horizontal dimensions can be limited by the internal surfaces of the the pile and vertical as the driven length by the formula(14):

$$F_K = 2dh\{[(n-1)K+1]+[(n-1)K+1]\}f_K + d^2[(n-1)K+1][(m-1)K+1]\sigma_c \qquad (14)$$

where d-side length of the crosssection of pile

n-number of piles in rows;

m-total number of piles in the row;

K-number of d between axes of the piles

f -frictional forces at the lateral surfaces of the pile foundation;

σ -is defined by formula (13) according to the width of the base of the massive foundation.

$\sigma_c = \alpha\sigma_K$ α=1.5

We determine:

$$A = \frac{[(n-1)K+1]+[(m-1)K+1]}{2m} \frac{f_K}{f_o} \qquad (15)$$

$$B = \frac{[(n-1)K+1]+[(m-1)K+1]}{m} \frac{\sigma_c}{\sigma_o} \qquad (16)$$

where σ_o-the bearing capacity of the ground under the base of the single pile.

Formulas (15),(16) describe the ratio of the bearing capacity of the ground at the lateral surface and under the base of the single pile. Formula (15) was used to determine the value of f_K/f_o acording to testing of pile foundations. As the basis it was used Bartolomey's tests, where values of f_K and f_o were detrmined as the results of the simple measurments. We used the results of 3 tests of singlerow pile foundations (K=3) and 3 tests of doublerow pile foundations to obtain the value of f_K/f_o. Piles in singlerow foundations were driven to h=12m. In double row pile foundations piles were driven to h=5 and h=12m. Bearing stratums were loam semi-solid clay, tight-plastic clay, soft-plastic clay . For singlerow pile foundations the following values were obtained:

f_K/f_o = 0.57; 0.55; 0.52

for doublerow pile foundations:

f_K/f_o = 0.59; 0.8; 0.75

The analysis of the alteration of the values of σ_K depending on the values of b for piles, driven to the tight-plastic loam, has been done under the following conditions: φ =20, c=0.02 Mpa, τ=18 Kn/m³, F=14Mpa, h=20 and h=6m The distance between the axis of n=1, 2, 3, 4, 5, 6, 10, 20. The values of σ are given in the Table 2

Table 2

Number of rows	h=20m b(m)	H	σ_K MPA	h=6m H	σ_K MPA
1	0.3	66.7	24.6	20	24.6
2	1.2	16.7	23.3	5	23.3
3	2.1	9.5	25.7	2.8	19.6
4	3.0	6.7	28.9	2.0	20.5
5	3.9	5.1	32.3	1.5	-
6	4.8	4.2	32.1	1.25	-
10	9.3	2.2	27.4	0.64	-
20	17.4	1.15	27.1	0.34	21.9

According to the data of the table 2 for the analysis of the alteration of the bearing capacity of the pile foundations in comparison with the bearing capacity of the single pile was considered to be constant . This conclusion follows also from [4]. After simple transformation we can obtain:

$$\mu = \frac{F_K}{F_C} = \frac{A\eta + B}{1+\eta} \qquad (17)$$

where F_C -the bearing capacity of the single pile ,

η -the ratio of skin resistance of the single pile to the bearing capacity of the soil under the pile's base.

The values A and B depending on the values of "n" if m=10 are given in the table 3.

Table 3

n	A	B
1	0.72	2.8
2	0.4	5.6
3	0.29	6.5
4	0.2	7.0
5	0.18	7.5
10	0.15	8.1

The values of μ for different values of η and n were calculated according to formula (17) and table 3. The results are given in Table 4.

Table 4

n	η=1	η=5	η=10
1	1.73	1.02	0.86
2	3.0	1.27	0.87
3	3.45	1.32	0.85
10	4.1	1.48	0.87

As it is shown in the table 4 the average value of the bearing capacity of the pile for the pile foundation can be more or less then the bearing capacity of the single pile. The tests[2] has been done for values η=1÷2. The value of η >5 can be expected for piles with high length, driven into clayey ground. Master's tests was conducted in such conditions. We compare our and his results in table 5.

Table 5

N piles	η	μ	μ_c	μ/μ_c
4	15	0.7	0.81	0.86
8	15	0.63	0.72	0.88
9	23	0.59	0.68	0.88
16	23	0.4	0.62	0.64

Table data proves that theoretical study can give satisfactory value of bearing capacity of pile foundation.

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