The Work of Pile Foundations and the Surrounding Soil

Interaction des Fondations sur Pieux et du Sol

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SYNOPSIS The stressed-and-deformed state,which is established both theoretically and on the basis of field experiments,of the soil near piles and pile foundations is considered. The study has shown that both the pile lateral surface and grillage base soil pressure is largely dispersed in the solid of the soil,and exerts practically no effect on the strained state of the soil below the pile point and also on the pile foundation settlement. This makes it possible to refine the calculation scheme which is used in forecasting the pile foundation settlement. Formulae for calculation of floating and end-bearing piles settlement with due regard to their interaction in the pile group are also given. The formulae are obtained by processing the results of pile work mathematical modelling using the elasticity theory technique. The work of filling piles in expulsive soil has many salient features. Experiments with large models have revealed time-dependent regularities in the expulsive forces change pattern. The analysis enabled us to work out technique for calculation of cylindrical and pedestal piles lifting.

Determination of pile foundation foot calculated area is necessary for calculating settlement of floating pile foundation. In this case the area of conventional foundation foot is generally determined,as well as the intensity of additional pressure which deforms soil below the pile foundation. Due to lateral surface friction the floating pile tranfers a part of the pressure on to the soil layers being cut through. This pressure is dispersed in the solid of the soil,therefore at some distance from the pile the tensions reduce to the intensity at which the soil cannot compress and experiences elastic deformations only. They are often an order below the compressive strains.

When projecting it may be assumed for long piles that specific friction in the lower part of the pile (f) is distributed evenly. In this case,according to Pati (1963),the vertical additional tensions at the pile point (Fig.1) in the horizontal plane are:

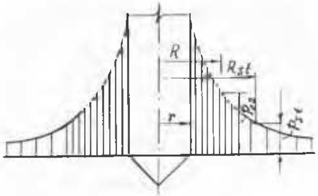

Fig.1. Epure $P_{z,s}$ in the plane of pile point

$$P_{z,s}=\alpha f \qquad (I)$$

where α-R/r ratio function factor. $P_{z,s}$ stress sheet extends to infinity (Fig.1).

According to Tsytovich (1973),however, the soils can compress only at pressure being higher than their structural strength P_{st}, i.e. when $P_{z,s} \geqslant P_{st}$.The parts of the pure where this inequality is not held are not to be taken into account, since the soil at such pressures undergoes small elastic deformations which do not significantly affect the value of predicted settlement. Epure $P_{z,s}$ can be approximated according to Dalmatov & Lapshin (Dalmatov et al.,1975) by the following expression:

$$\bar{\bar{P}}_{z,s}=1.141(R/r)^{-1.237}f \qquad (2)$$

From this by integration we find the force compressing the soil at pile base from efforts transferred by its lateral surface.

$$P_s=50.46r^2f(0.310+r/R_{st})(r/R_{st})^{0.763}-1.310 \qquad (3)$$

Pile settlement developing due to base soil compression,being higher than shift settlement values (Dalmatov et al.,1975), pile's friction against the soil is:

$$\underline{\underline{P}}_{s\ max} = u\sum f_i l_i \qquad (4)$$

where u is pile perimeter, f_i is specific friction of pile against the soil of i-th layer; l_i is thickness of i-th layer. The dispersed part of the effort transferred by the pile lateral surface

$$\underline{\underline{P}}d=\underline{\underline{P}}s\ max - \underline{\underline{P}}s \qquad (5)$$

We find the value of R_{st} (Fig.1) proceeding from the approximation (2):

$$R_{st}=r(1.148\ f/P_{st})^{0.808} \qquad (6)$$

Along the foot of conventional foundation we may assume even distribution of pressure $\bar{\bar{P}}$ adm,which causes base soil compression:

$$P_{ad,m}=(P - P_d) / \pi R_{st}^2 \qquad (7)$$

Here P is load on the pile.

In foundations consisting of group or rows of piles the load on the surrounding soil is transferred via pile point, lateral surface and grillage foot. As this takes place friction significantly reduces against lateral surfaces inside group of piles as compared with that of a single pile (Doroshkevitch 1973). Therefore, dispersion of the pressure, which is transferred by pile lateral surfaces, can be considered in corner piles only within $0.5P_d$ while that of piles of border rosw (not corner ones) - $0.25P_d$.

Then the load, producing compression of base soil at 3-4d distance between piles (where d is pile cross section), is transferred via the foot of centrally loaded corner pile foundation:

$$N_e=\sum P_e - (0.25n_{ed}+2) P_d \quad N_p \qquad (8)$$

where n_{ed} is quantity of piles located along the perimeter of group of piles not taking into account corner piles; $\sum P_e$ is the sum of loads transferred along lateral surfaces of all piles in the group with due regard to reduce of frictional forces operating in the group (Doroshkevitch, 1973); N_p is weight (mass) of all piles in the group.

It may be assumed that this load is evenly distributed within the area of conventional foundation (Fig.2,a): $F_{ec}=b_e \cdot l_e$ where $b_e=a_e(n_b-1)+eR_{st}$; $l_e=a_e(n_e-1)+2R_{st}$

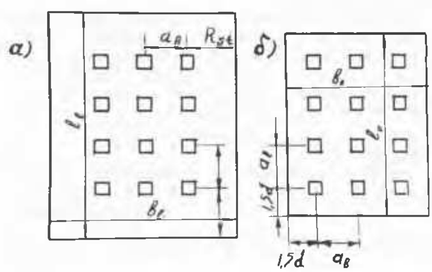

Fig.2. Layouts of compression distribution in the plane of pile point; a-plan, b-section

It may be assumed that the pressure, which is transferred by the piles points on to the base is distributed in the compressed layer below piles points (Fig.2,b) within the area: $F_{oc}= b_o l_o$ where $b_o=a_b(n_b-1)+3d$

$$l_o=a_e(n_e-1)+3d$$

In this case mean pressure is:

$$P_o =(\sum P_o)/F_{oc} \qquad (9)$$

Based on $R_{st} \leq 1.5$ d, the area of convention foundation foot may be taken equal to $F_{o,c}$. Then the intensity of pressure upon its foot deriving from all forces causing soils compression below piles is:

$$\bar{\bar{P}} = (Nl + \sum P_o)/F_{oc} \qquad (10)$$

However if $R_{st} > 1.5d$, it is expedient to determine pile foundation settlement at the expense of soil deformation below piles points depending on two areas of load: F_{oc} and F_{ec}, which are loaded in accordance with $\bar{\bar{P}}_o$ and $\bar{\bar{P}}_e$ loads. This being so,

$$\bar{\bar{P}}_e=N_e/F_{eo} \qquad (11)$$

Knowing intensity of $\bar{\bar{P}}_o$ and $\bar{\bar{P}}_e$ pressures areas and penetration depth, calculation of settlement can be made with a method known.

Calculation of piles settlement considering their interaction in the group, suggested by Fedorovsky (1975), is based on presenting soil bedding as homogenous or heterogeneous elastic semi-space.

Formulae are given below for calculation of cylindrical pile settlement on the assumption that the surrounding soil is homogenous in radial direction (this is performed, i.e. for bore-filling piles and tubular piles, being driven with soil extraction out of the hollow, as well as for driven piles in some types of soils). Formulae for calculation of single pile settlement are aplicable for load not exceeding proportionality limit, while for evaluation of pile interaction in the group - up to losses of supporting power.

Assume that the pile (length l, diameter d, body compression stiffness EF) cuts through a soil layer of thickness l with shear modulus G_I and Poisson's ratio V_I and rests upon homogenous semi-space with elastic characteristics G_2 and V_2. It is obvious that by the use of depth averaging the majority of cases of pile application can be reduced to this scheme. Settlement of pile crown loaded by axial force $\bar{\bar{P}}$ (in linear phase) is:

$$\bar{\bar{W}} = \beta \bar{\bar{P}}/G_I \, l \qquad (12)$$

where

$$\beta=\beta_*[\lambda(\varkappa_1)+(1-\beta_*)/\alpha_*] \qquad (13)$$

values, which appear in (13) are physically authentic. α_* is proportionality factor for settlement of absolutely rigid pile ($EF = \infty$) in homogenous base

where $\alpha_* =0.171 \ln (K_v \, l/d)$ (14)

$$\bar{\bar{K}}_v = 2.816 - 3.783 V + 2.175V^2 \qquad (15)$$

($\bar{K}_v$ for (14) is calculated at $V=V_I$);

β_* is an analogous factor for absolutely rigid pile in two-layer base

$$\beta_*=0.171 \ln (\bar{\bar{K}}_v G_I \, l/G_2 d) \qquad (16)$$

where $\bar{\bar{K}}_v$ is determined from formula (15)

$\varkappa_I$ being the relative rigidity of pile

$$\varkappa_I =EF/G_I l^2 \qquad (17)$$

The estimation of pile body compressibility, as it has been possible to demonstrate by similarity theory technique, is performed with the aid of single function of $\varkappa_1 - \lambda(\varkappa_1)$ parameter (Fig.3).

Comparison between the formula (12) and the results of computation is given in the report

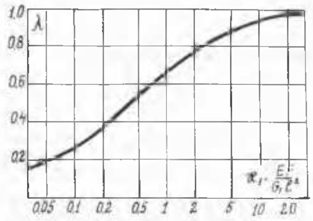

Fig.3. Diagram of $\lambda = f(\varkappa)$ values (Fedorovsky,1975), it shows a good fit with calculations of Mattes and Poulos (1968).

Calculations of piles interaction in the group have shown that tangential stresses τ_{rz} around the pile reduce inversely with distance to the pile axis, while soil settlement $\bar{W}$ according to logarithmic low. In this case the settlement in the upper layers the soil at $Z \angle 0.25\ l$ increase more slowly than the load ($d^2\ \bar{W}/dp^2 < 0$), while at $Z > 0.75\ l$ more rapidly ($d^2 W/d\ p > 0$) since owing to pile's slip across the soil (the zone of slip usually extends from above downwars) ever increasing part of load P is transferred to the soil in the lower part of the pile and via its heel. As for average settlement along the pile length, it increases directly with P load. It is convenient to accept this value as a measure of piles interaction in the group. It may be assumed for calculation that additional settlement of the pile, separated by ρ from the loaded pile, equals to

$$\bar{\bar{W}}\ ad = \delta\ P/G_I l \qquad (18)$$

where

$$\delta \begin{cases} = 0.17I \ln (K_v\ G_I l/\ 2G_2 \rho) & \text{at } \bar{K}_v\ G_I\ l/2\ G_2 \rho > I \\ 0 & \text{at } K_v G_I\ l\ /2\ G_2 \rho \leq I \end{cases}$$

To study distribution of soil tensions and deformations experiments near pile foundations were carried out in field conditions. Soil tensions in inter-pile space and active zone have been measured, as well as soil laminar deformations at variable depths as the foundation is loaded and its settlement develop. For measuring tensions and deformations load cells and depth marks were inserted into the solid of the soil before driving in piles.

The experiments were carried out with single piles, one- and two-row pile foundations, groups of piles at separation between piles equalling to 3 and 6 diameters. The piles were 25x25cm and 30x30cm in section and 5-12m long. Some results of this study is given in Bartolomey's work (1975).

Observations have shown that the soil under grillage was engaged in work at 1-4mm settlement of foundation. At the degrees of loads causing foundation settlement over 10mm the tension in inter-pile space does not increase and the piles and the soil in between shifts as solid mass. Tensions deriving from friction forces applied along the lateral surface are dispered in the surrounding soil. The vertical tension above the plane of pile point constitutes only 10-20% of inter-pile tension below its point-at transferring via pile lateral surface as much as 50% of the load. Tensions under pile foundations and laminar deformations of soil compression increase, as load rises, approximately proportionally.

As an example, in Fig.4 there are given

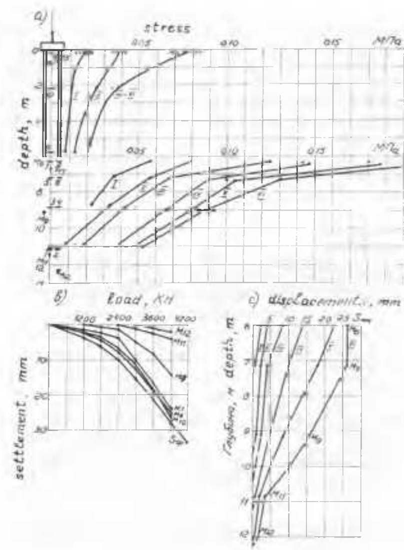

Fig.4. Diagram of change in vertical tensions (a), settlement of depth marks ( ) and shifts of depth marks (b): 1-15 - load cells; M_{1-12}-depth marks; I-IV - epures of tensions (a) and marks ettlement (b) at loads of 1.200, 1.800, 2.400 3.000, 3.600, 4.200 KN, applied upon foundation.

changes in tensions and vertical shifts in inter-pile space, as well as in the active zone of two-row pile foundation consisting of 12 piles of 30x30cm section, 6m long; which have been driven with spacing of 3 diameters into tight-plastic loams, It is obvious from the diagrams that, as load rises, there increase tension, settlement of depth marks and laminar deformations of soil in the active zone in the range of 6-7.2m and then in the range of greater depth. At load closely approximating the limiting one soil deformation developped to the depth of 12m.

A salient feature of the pile work in expulsive soil is that while wetting of the latter there emerge tangential and normal forces of expulsion along the pile's lateral surface and under its butt. Under these forces pile lifting takes place, the value of which depends on pile length, thickness of layer of the expulsive soil, external load. Time- dependent regularities in expulsive forces change pattern while wetting of soil were studied at experimental piles in field conditions (Sorochan,1974). In time those forces grow up to maximum value,after that they reduce to the value which remains constant in spite of continious soil lifting. The change pattern of tangential expulsive forces which work at the lateral surface is similar to change in summary forces. In distinction to the above the normal expulsive forces working at the butt increase as the soil swells, and their stabilization sets in after lifting of soil layers ceases. Maximal value of expulsive forces conforms to mobilization of all internal forces of soil resistance, while minimal- to work of friction forces.

Change in tangential expulsive forces in time is dependent on soil shift at swelling (Fig.5). It is established that as

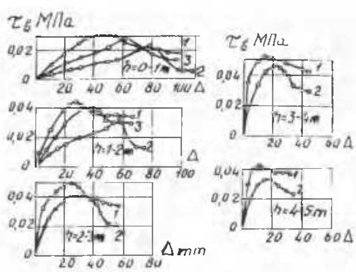

Fig.5. Diagrams of tangential expulsive tensions (τ_6). at soil shift Δ at depths h in clays:
I - Aralian; 2- Sarmatian; 3-Khvalynian

depth increases the maximal value of tangensional tensions emerges at lesser shifts of soil layers.

The value of lifting of pile at soil swelling can be determined according to the formula

$$\Delta_c = \Omega \Delta_n - W P / u \qquad (19)$$

where Ω -coefficient, dependent on pile length and depth-dependent change pattern of lifting of soil layers;

Δ_n -lifting of free surface of soil at swelling;

W -depth-related coefficient depending on the mode of tangential expulsive tensions.

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*Individual contribution to the report is as follows: Dalmatov and Doroshkevitch (Part I); Fedorovsky (Part 2); Bartolomey (Part 3); Sorochan (Part 4).