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The Analysis of Pile's Bearing Capability for a Pile Field of Short Grid Pitch in Hydrocompactive Soil of Type II: Technique and Simulation Results

A.I. Rusakov

The Solver for the Equation of HS State. In the developed technique the state of HS is represented by vectors $\vec{\tau}$, $\vec{\sigma}_z$, $\vec{s}_{sl}$, which are obtained by discretization of corresponding functions $\tau(z)$, $\sigma_z(z)$, $s_{sl}(z)$. The discretization points z_i for quantities τ, σ_z are determined by separating the HS into sub-layers; the discretization points z_i' for quantity s_{sl} are determined by using experimental table of relative subsidence $\varepsilon_{sl}(z_i', p_j)$. To come to discrete form of the HS equation, replace the integrals (7) and (14) in the Part 1 of this paper [1] by finite sums; the equation (4) itself turns into a set of algebraic equations with respect to unknowns $\tau(z_i)$. At that, the pass from the points z_i' to points z_i, required for substitution of the quantity (9) into formula (11) of Part 1, is made by linear interpolation.

The state of HS is obtained in two stages. At 1st stage we take the long enough preliminary step of the PF L_{p0} and solve the equation (4) from Part 1 [1] by iteration method. Starting solution for initiating the iteration process is the state of watered HS with no PF. For 2nd stage we have to specify a small increment, or variation of the PF step ΔL_p, and organize the stepwise solution process based on diminishing the step from starting value L_{p0} to wanted design value L_{p1}. After each diminishing the step of PF, the equation (4) from Part 1 is solved by minimization of quadratic residual:

$$\delta^2(L_c, \vec{\tau}) = \left\| \tau(z) - \tau_0(z, \quad \hat{\sigma}_z \tau(z'), \quad \hat{s}_{sl} \hat{\sigma}_z \tau(z')) \right\|^2 . \qquad (1)$$

Here $\|\cdot\|$ is Euclidean norm of vector; the difference of functions is taken as the difference of vectors obtained by discretization of these functions. Minimization is realized by the gradient method. The initial value of argument is the vector $\vec{\tau}$ obtained with solving the equation of state at the preceding computation step, when the PF step was greater then the current by the variation value.

The peculiarity of the state equations after discretization is their possible insolubility for small step L_p = 3D÷4D (D is the pile diameter). In practice it reveals itself like impossibility to zero the residual (1) by taking an appropriate vector $\vec{\tau}$. Given increase of number of discretization points, when the thickness of sublayers reduces and the number of lines in relative subsidence table rises, the minimal residual of the state equation decreases. For large enough number of discretization points due to the PF step L_{p1} = 3D÷4D it is usually possible to secure the value of solution relative error about $\delta(L_{p1},\vec{\tau})/\|\vec{\tau}\|$ ~ 0,02.

The necessity of residual's consecutive minimization with the drift of parameter L_p is defined by impossibility of minimization of the goal function (1) from any initial point $\vec{\tau}_{init}$ because of the complicated form of this function. The initial point must be located in the neighborhood of the minimum point of the objective function — to gain the latter, the consecutive minimization of the array of functions replacing the objective one is applied [2].

The FEM Simulation of the Pile Basement on the Hydrocompactive Soil. The finite element model (FE-model) enables to determine the parameter s_1 of the state equation of the HS and next to verify the results of computing the lateral friction forces on a pile. Below, we solve the first of these problems by means of "LIRA 9.x" bundled software.

The model is built on the soil parallelepiped comprising a pile, limited by vertical planes of symmetry of the infinite PF. The planes are situated in the middle between pile rows and taken as the closest to selected pile. The boundary conditions upon the nodes of these planes make the displacements along the normal lines to these planes impossible as well as rotational displacements of the normal lines.

The pile is simulated by the “spar” FEs. The trunk of a pile is the chain of linear-elastic spars (segments), the nodes of which are located in the middle between horizontal edges of volumetric soil FEs. From each intermediate node of the trunk the physically non-linear spar FE initiates downward and connects this node with an adjacent soil’s node. This FE — the physically non-linear brace — simulates the lateral resistance of soil upon the trunk portion in the segment bounds. Connection of pile segments with soil by using the braces enables to account sliding of soil along the pile trunk in simulation procedure — the important factor in analysis of side resistance [3]. Only the lower node of trunk is a soil node.

The brace implements almost rigid connection of trunk and soil until the internal longitudinal force becomes equal to limit force of lateral resistance over the trunk segment. The tension-compression deformation of brace, with magnitude about mutual displacements of trunk and sliding along it soil, occurs when the internal longitudinal force goes over the limit force of side resistance by a negligible margin. Thus, the brace functioning is defined by the diagram of elastic-plastic material close to the diagram of perfectly plastic material.

The watered hydrocompactive soil has been simulated with elastic volumetric FEs having Young’s modulus E_{sl} established at given depth from experimental relation of relative subsidence versus pressure $\varepsilon_{sl} = \varepsilon_{sl}(z, p)$ as follows. Take the unwatered soil as linear-elastic medium of deformation modulus due to natural dampness E_{nd}. On watering, besides elastic component of total deformation, there arises the component of relative subsidence, and for vertical compression strain we have:

$$\varepsilon_\Sigma = \psi_{nd}\sigma_z + \varepsilon_{sl}(z, \sigma_z); \quad \psi_{nd} \equiv \frac{1}{E_{nd}} \frac{1-\nu \ -2\nu^2}{1-\nu}. \tag{2}$$

Here ψ_{nd} is the compliance characteristic of soil of natural dampness during tighten compression. By considering the watered soil as elastic with elasticity modulus E_{sl}, introducing the compliance characteristic for it ψ_{sl}, we obtain:

$$\varepsilon_\Sigma = \psi_{sl}\sigma_z; \quad \psi_{sl} \equiv \frac{1}{E_{sl}} \frac{1-\nu-2\nu^2}{1-\nu}. \tag{3}$$

The equalities (2) and (3) produce equation for unknown E_{sl}:

$$\psi_{nd}\sigma_z + \varepsilon_{sl}(z, \sigma_z) = \psi_{sl}(E_{sl})\sigma_z,$$

the solution of which has the form:

$$\frac{1}{E_{sl}} = \frac{\varepsilon_{sl}(z,\sigma_z)}{\sigma_z}\left(\frac{1-\nu\ \ -2\nu^2}{1-\nu}\right)^{-1} + \frac{1}{E_{nd}}. \tag{4}$$

The approximate value of vertical pressure σ_z at the given depth we evaluate by SP 22.13330.2011 “SNIP 2.02.01-83 The Bases of Buildings and Constructions”, i. 5.6.40.

Note. The elasticity modulus E_{sl} can be made more precise. To take into account the difference between soil stiffness during initial loading and unloading, we can employ the expression (6) and the comment to hypothesis 3 of the paper [1] for founding the next substitution in formula (2): $\psi_{nd}\sigma_z \;\to\; \psi_{nd}\sigma_{z1} + 0.2\psi_{nd}(\sigma_z - \sigma_{z1})$. Thus formula (4) has been conversed to the form:

$$\frac{1}{E_{sl}} = \frac{\varepsilon_{sl}(z,\sigma_z)}{0.8\sigma_{z1}+0.2\sigma_z}\left(\frac{1-\nu\ \ -2\nu^2}{1-\nu}\right)^{-1} + \frac{1}{E_{nd}}. \tag{5}$$

After estimation of parameter s_1 by using formula (4) and solving the HS state equation, one can substitute the obtained pressure $\sigma_z(z)$ into (5) and get the corrected s_1. But this correction doesn’t disturb the estimate s_1 prominently.

The developed model of the pile basement has two local loadings, producing the wanted SSS in stepwise scheme with small enough substep of loading:

1. Loading soil and pile by their own weight.

2. Loading the upper node of pile by given displacement — the settlement s_u.

The process of real soil deformation supposes the next staging of bringing the SSS on:

1. Loading the unwatered soil and the pile trunk by gravity force (and also by loads compressing both the trunk of simulated pile and the soil in the same way, see below).
2. The design settlement of pile.
3. Watering: the deformation characteristics of hydrocompactive layer are changed in stepwise manner to final values according with experimental table of relative subsidence.

In simulation, the stage 3 appears being replaced to the first place, and adequacy of model is based on hypothesis about independence of ultimate state from the sequence order of loadings and material properties variation. This hypothesis can be taken in given case, because the FE-model under consideration is nonlinear elastic. — In nonlinear elastic systems the internal forces are conservative, and, therefore, the stress state is uniquely defined by displacements in a system with no regard of their development's trajectory [4, i.4.4].

The trait of suggested model is an absence of deformations before loads, whereas *there are* initial deformations in a real pile basement: the piles, sunk in unwatered soil before foundation erection, are practically not deformed, while the soil is compressed under own weight. With no special precautions, the simulated deformation of soil caused by gravity would be distorted by stretching forces from a pile.

To remove fictitious tension of soil bulk, we have to introduce additional loads in the gravity loading to get the pile compressed in conformance with compression of unwatered soil thickness. To do this, specify additional weight of pile and apply the upward load to lower pile's node, equal to additional weight, to get coincidence of pile and soil deformation. The specific gravity of pile γ_p is obtained from the condition:

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$$\varepsilon_z = \frac{1}{E_{nd}} \frac{1-\nu \quad -2\nu^2}{1-\nu} \gamma_{nd} z = \frac{\gamma_p z}{E_p},$$

whence it holds:

$$\gamma_p = \frac{E_p}{E_{nd}} \frac{1-\nu \quad -2\nu^2}{1-\nu} \gamma_{nd}.$$

Here E_p is the elasticity modulus of pile's material; the subscript "*nd*" points to a soil of natural dampness; the specific gravity of soil is taken uniform along vertical line. Together with additional weight, the load from weight of pile's material must be applied to the pile.

The vertical section of deformed soil bulk resulted in simulation is shown in Fig. 1. In the plot one can see the physically nonlinear braces, connecting the pile trunk FEs with the soil FEs. We lay next input data in this analysis: the height of collapsible layers is 15 m; the pile length 20 m; the pile grid pitch 2 m; the height of liner-deformed layer under conventional foundation 5 m; the pile settlement 12 cm. The dimension of the soil FE is taken as 0.25×0.25×0.5 m. The maximum subsidence of soil is obtained as 14 cm. In calculation series with input data similar to presented ones we have established the settlement of soil in the foot of conventional foundation between piles in the range s_1 = 0.4s_u÷0.6s_u. For preliminary analysis, the value of parameter in the HS state equation as s_1 = 0.5s_u is tolerable to take.

The Example of Calculation of the Soil SSS and Safe Design Load on a Pile by Solving the Equation of HS State. As the example of analysis we have taken the construction site in Rostov-on-Don, where under the humus layer there are layers EGE-1—EGE-3 — the loess-like loams, and under them the layer of Scythian clay EGE-4. The layers EGE-1, EGE-2 are collapsible, the hydrocompaction subsidence caused by own weight on watering is 41 cm. The lower boundary of collapsible strata is at the depth about 20—25 m. The next basement design has been considered: the field of drilled piers is on the square grid of pitch 2 m; concrete piles of length 28 m, the trunk diameter 0.63 m, the widening of pile's lower end of diameter 1.3 m.

The results of safe design load evaluation in accordance with SP

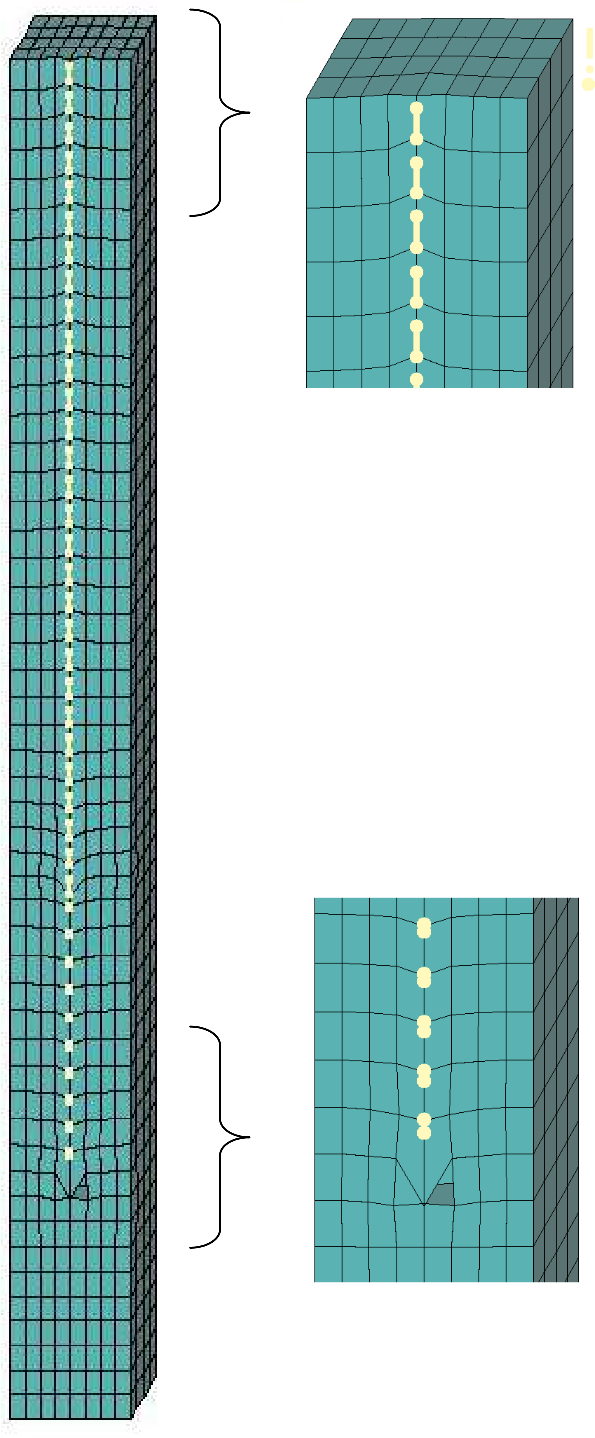

Fig. 1. The sectional view of deformed soil bulk (the section plane includes the pile centerline)

24.13330.2011 “SNIP 2.02.03-85 Pile Foundations” are as follows:

$$N = \frac{F_d}{1.4} - 0.8P_n \ = 2118 \text{ kPa} - 1380 \text{ kPa} = 738 \text{ kPa}.$$

The side component of total bearing capability F_d appeared to be as

$$F_{d\ lat} = u\sum 0.7\, f_i h_i \ = 560 \text{ kPa}; \tag{6}$$

the component of bearing capability F_d due to resistance of supporting layer appeared to be as

$$F_{d\ drag} = RA = 2406 \text{ kPa}.$$

The diagrams of subsidence, collapse strain, and vertical pressure for case under consideration are shown in Fig. 2, *a*. The subsidence value is 41 cm.

The results of evaluation of safe design load on a pile for the same well by the method of solving the HS state equation are as follows:

$$N = \frac{F_d}{1.4} - P_n \ = 1846 \text{ kPa} - 854 \text{ kPa} = 992 \text{ kPa}.$$

Here $P_n = u\sum \tau_i h_i$ is the force of negative friction, the sum is taken over all sublayers such that $\tau_i > 0$, τ_i is calculated by the formula (11) from paper [1]. The components of bearing capability F_d appeared to be as follow:

$$F_{d\ lat} = u \sum_{z_i > H_{sl}} 0.7\, f_i h_i + u\sum_{\tau_i < 0} 0.7 |\tau_i| h_i \ = 363 + 246 = 609 \text{ kPa}; \tag{7}$$

$$F_{d\ drag} \ = 1977 \text{ kPa}.$$

The first sum in formula (7) is the component of lateral resistance force at the bearing thickness. It is evaluated in the similar way like component (6), but summation is to be made over all layers below the HS (z_i is the level of sublayer’s midst), and the congruent pit’s level mark must be used instead of the actual one for HS lower bound (when $z = H_{sl}$). The second sum is the component of lateral resistance force at the HS. For its evaluation the distributed loads τ_i are employed, which have been obtained from the equation of HS state. The component of friction along lateral surface $F_{d\,lat}$ has been varied a little.

The frontal resistance component $F_{d\,drag}$ was evaluated by using the Table 7.8 of SP 24.13330.2011 and congruent mark of a pit for $z = H_{sl}$. Its value has decreased as a result of decreasing the vertical pressure of soil between piles at the level of lower end of piles. In whole, the safe design load on a pile has increased owing to the negative friction force reducing.

For comparison, in Fig. 2, *b*, there are the diagrams of the same characteristics as in Fig. 2, *a*, but taking into account the HS's interaction with the PF. The inflection point *A* of vertical pressure diagram corresponds to the sign change of friction force over the pile's lateral surface. The subsidence in given case is of 16 cm.

The calculations reveal that pile portion, where friction force τ is no greater then limit value τ_{max}, has short enough extension about $0.1l$, i.e., the error of tangent force estimation by function (10) of paper [1] is negligent on calculation of required forces N and P_n.

Resume. In evaluation of the safe design load on a pile, the interaction of the HS with a pile field of short grid pitch is taken into account on basis of setting up and solving the equation of HS state (equilibrium). The height distribution of vertical pressure in watered hydrocompactive thickness in the case of PF of short grid pitch essentially differs from the pressure distribution in the case of single pile. This results in the decrease of subsidence and the increase of safe bearing load on a pile.

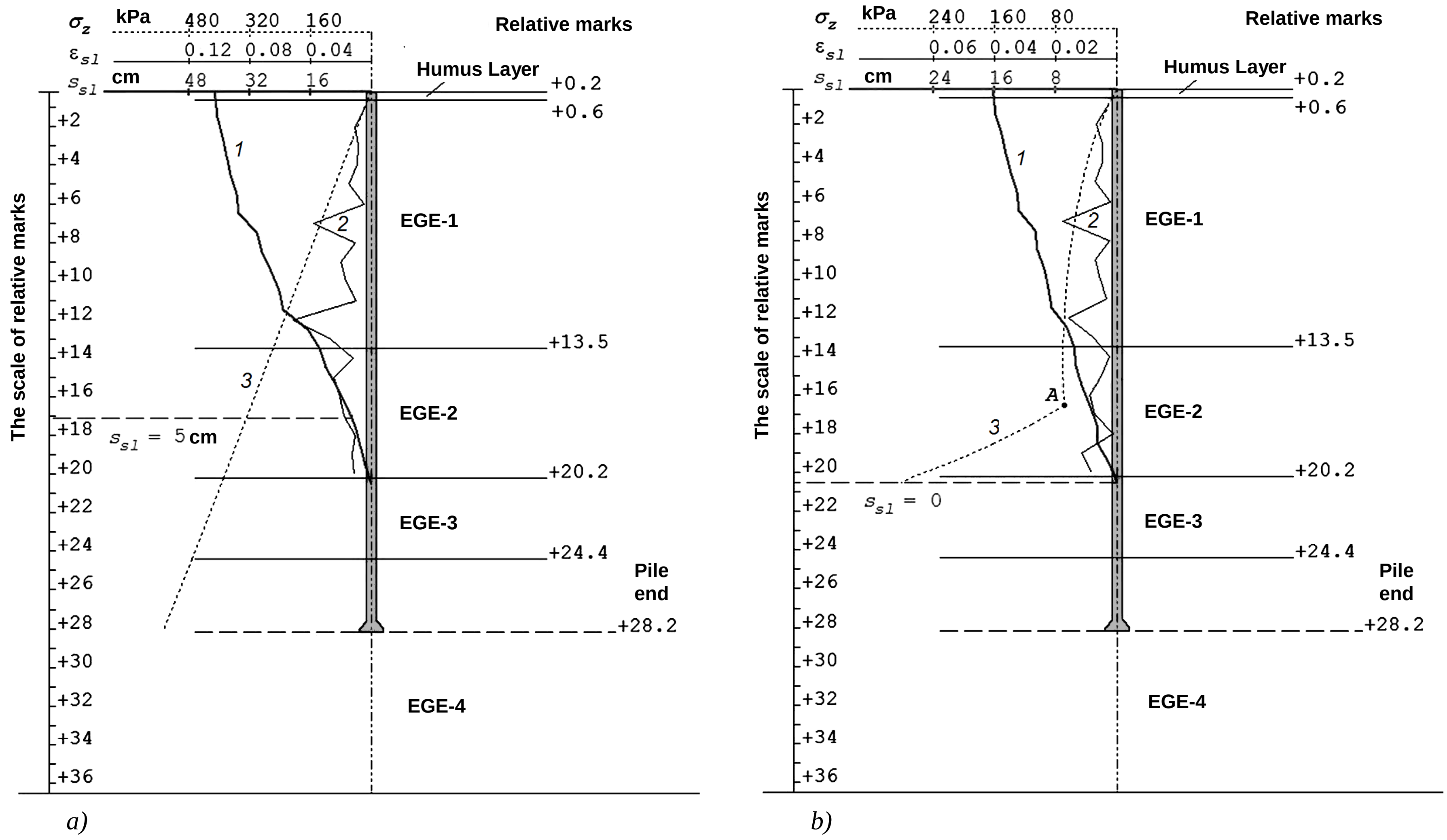

Fig. 2. Diagrams in the HS: 1 — subsidence s_{sl}; 2 — collapse strain ε_{sl}; 3 — vertical pressure σ_z

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