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RESULTS OF THE CALCULATED PREDICTION FOR INTERACTION OF DRILLING-INJECTION PILES, HAVING CONTROLLED BROADENING, WITH DUST-CLAY GROUND BASIS

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ABSTRACT

A new method for constructing a drilling-injection pile according to a cuff technology with injection of a cement slurry in the formation mode of "fracturing" and a device for controlled broadening at the end is described. This method is aimed at high-quality and efficient compaction of the soil foundation under the ribbon foundation of the reconstructed building. The results of field studies of the interaction of the piles under consideration with the base are analyzed. The radius of the compacted zone of the near-land massif is established. Geometric parameters of broadening, pile trunks and hydraulic fractures are determined. An algorithm for calculating the main parameters of the proposed piles has been developed to predict their interaction with a dusty clay soil base under static loading conditions. A comparison is made with classical analytical solutions in the field of soil mechanics, the theory of elasticity and plasticity.

Key words: Reconstruction, Drilling-Injection Piles (Micro-Piles), broadening At the End of the Pile, Providing Control, Dusty-Clay Soils, and Stress-Strain State.

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1. INTRODUCTION

There is a large number of buildings and structures (including cultural heritage sites) that need to be reconstructed, restored and modernized in accordance with modern requirements associated with the development of their underground space in order to accommodate social and engineering infrastructure around the world [2 , 4, 5, 12, 14]. All of them, as a rule, are located within dense urban development in complex engineering-geological and hydro-geological conditions [6, 8, 12, 14]. In this regard, it is urgent to develop an effective and reliable method of reinforcing ribbon foundations with compaction of the soil base [1, 4, 5, 9, 10, 14].

According to the results of the analysis, the leading positions are taken by the reinforcement methods associated with the formation in the soil mass of reinforcing elements of various shapes - piles with broadening at the end, drilling injections (injection) piles with increasing diameters of their trunks using the cuff technology of cement slurry injection in the "fracturing" 1, 3-5, 9]. At the same time, all these methods to one or another degree unite the main disadvantage, which consists in the unpredictable and uncontrolled spread of the solution in a silty-clay soil massif and, as a consequence, in the probability of occurrence of an uneven base in terms of compressibility and other adverse consequences [2, 6 , 8, 12]. In this regard, in order to improve the processability and efficiency of reinforcing the foundation foundations, a combined method is proposed, consisting in the construction of drill-hole piles for the cuff technology with the injection of cement mortar in the "fracturing" regime and creation of controlled broadening at the lower ends of the piles [7] (Fig.1).

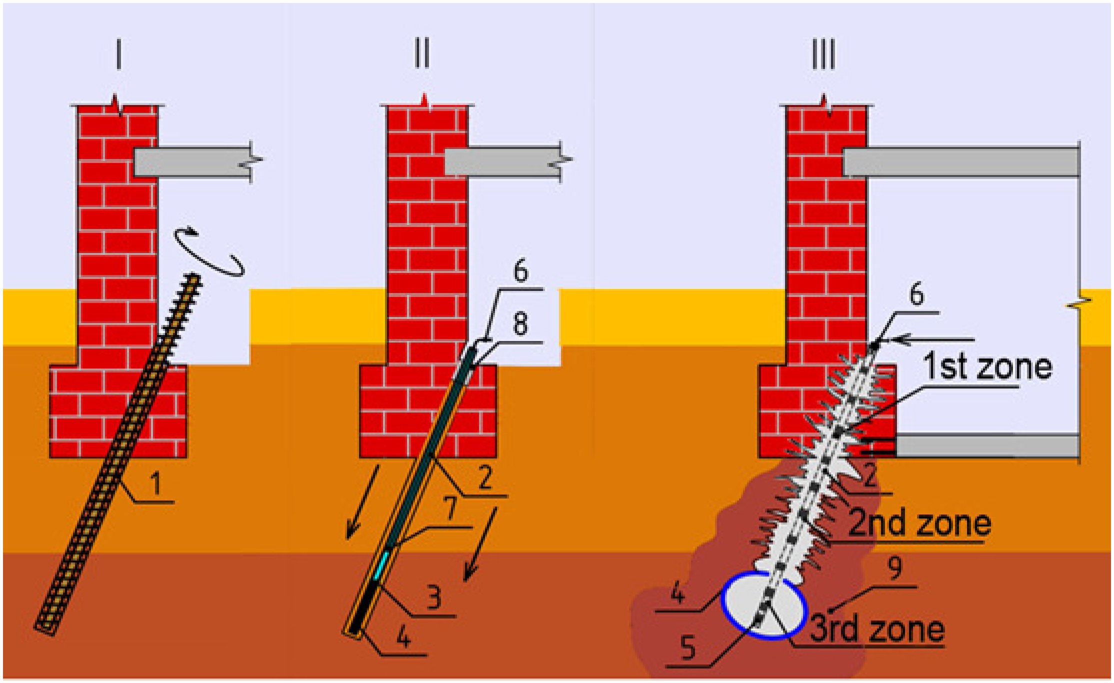

Figure1 Scheme of a drilling-injection pile device with controlled broadening at the lower end: 1 - well; 2 - pipe-injector; 3 - collar; 4 - membrane-glass; 5 - rubber cuffs; 6 - hose; 7 - packer; 8 - concrete plug; 9 - compacted zone of soil massif

For the pile device, a pipe-injector with three zones of perforations is used. The first is located at the end of the membrane-glass (elastic rubber shell, which covers the lower end of the injector), the second - in the zone of strengthening the soil surrounding the trunk of the pile, the third - in the reinforcement zone of the foundation material. In the process of injection into the first zone, the membrane-glass is stretched and forms a broadening in the soil massif, the dimensions of which can be controlled by the flow and pressure of the cement slurry (according to the gauge of the solution of the mortar pump). Then the packer moves to the second and third zones of the holes to form the pile stem, injecting the solution in the "fracturing" regime and "healing" the cracks in the structures of the reinforced foundation [7].

2. RESEARCH METHODS

Field studies were carried out at two sites in Tyumen (the physical and mechanical characteristics of the base grounds are given in Table 1). At the first site, piles with a length of 6 m were installed with injection of the solution into a membrane-glass to form a controlled broadening in the solution volume of 10-20 liters, on the second - piles 2 m long in a solution volume of 30-40 liters.

Table 1 Physical and mechanical characteristics of the soil base of the field for field research

characteristic*	**Area № 1**		**Area № 2**	
	sandy loam plastic	**loam soft-plastic**	**clay semi-solid**	**loam soft-plastic**
z, m	2.0-5.0	5.0-9.0	0.5-1.4	1.4-6.6
γ_{gr}, kN /m^3	19.6-19.8	18.4-18.8	17.6-18.2	18.4-19.6
γ_d, kN /m^3	17.8-18.2	14.4-15.2	14.7-15.4	13.9-15.9
W, %	22.0-24.0	28.0-30.0	18.0-20.0	26.0-32.0
e, unit fraction	0.6-0.7	0.8-0.9	0.76-0.84	0.7-0.8
I_P, %	5.0	16.0	18.0	9.0
I_L, unit fraction	0.71	0.69	0.15	0.62
φ, gr	19.0-20.0	15.0-16.0	14.7-21.5	14.7-19.2
c, kPa	7.0-8.0	11.0-12.0	24.0-28.0	19.0-23.0
E_{comp}, MPa	3.2-3.3	1.4-1.5	3.3-3.4	2.2-2.3
E, MPa	11.2-11.6	4.2-4.5	18.5-19.0	7.7-8.0

* z - vertical depth from the surface; γ_{gr} - specific gravity of soil; γ_d - specific gravity of dry soil; W - moisture; e - coefficient of porosity; I_P - plasticity number; I_L - yield index; φ - angle of internal friction; c - specific grip; E_{comp} - compression strain modulus; E – modulus of deformation, corrected with the help of the coefficients adopted in Table 5.1 SP22.13330.2011.

To determine the geometric parameters of the proposed piles and to change the physico-mechanical characteristics of the soil massif, field-level excavation was carried out after field surveys and piles were made (Fig. 2). It was found that when an injection of 10-20 liters of solution is injected into the membrane-beaker at the lower end of the pile, an ellipsoidal broadening with a diameter of 250-350 mm elongated horizontally is formed with a ratio of the horizontal dimension to the vertical dimension of about 1.3-1.4. When injecting 30-40 liters of solution, a ellipsoidal broadening, 360-390 mm in diameter, elongated along the vertical, with a horizontal to vertical ratio of about 0.7-0.8 is formed. The discrepancy in the values within the specified ranges is explained by the different lengths of the piles produced (6 and 2 m, respectively).

When the barrel of the drilling-injection pile is formed along the cuff technology with injection of the solution in the "hydraulic fracturing" regime due to the constantly acting pressure on the walls of the well with an average value of 0.2 MPa, the diameter of the trunk increases by an average of 2 times. During the first injection of the solution, hydraulic fractures up to 40 mm thick extend a considerable distance from the pile up to 3 m. In the second case, hydraulic fractures up to 130 mm thick are localized in the near-suture zone of the soil with a radius of up to 0.5 m. Sakharov discovered two types of hydraulic fracturing textures [1, 2, 10]:

- A - in the form of cracks of greater opening with the formation of continuous lenses 10-130 mm thick;
- B - in the form of numerous thin cracks up to 5 mm thick, poorly filled with cement mortar.

Figure 2 Geometric features of the proposed piles: a, b - controlled broadenings of 10-20 liters; c, d - controlled broadening of 30-40 liters; e - increase in diameter of the pile stem; f - formation of hydraulic fractures; g, h - the nature of the spread of hydraulic fractures during the first and repeated injection of the solution, respectively; I,j- the types of fracture textures A and B, respectively; k - horizontal hydraulic fractures

3. RESULTS AND DISCUSSION

3.1. Calculation of the main parameters of the proposed piles

The radial displacement of the membrane-cup wall u1 during injection is determined, depending on the required depth of the widening in the soil mass (pile length), pressure and volume of the injected solution, as well as the existing stress-strain state (SSS) of the foundation under the foundation, on the basis of analytical solutions of O. Hoffman, Z.G. Ter-Martirosyan and others. [13, 14] (Figure 3):

$$V = \frac{4}{3}\pi(r_b + u_1)^2 \; ; \qquad (1)$$

$$u_1 = \left(\frac{3V}{4\pi}\right)^{\frac{1}{2}} - r_b \; , \qquad (2)$$

where r_b - initial radius of the drilled well (depends on the screw diameter), m; r_b=0,04 m; V- volume of cement mortar injected into the membrane-glass, m^3.

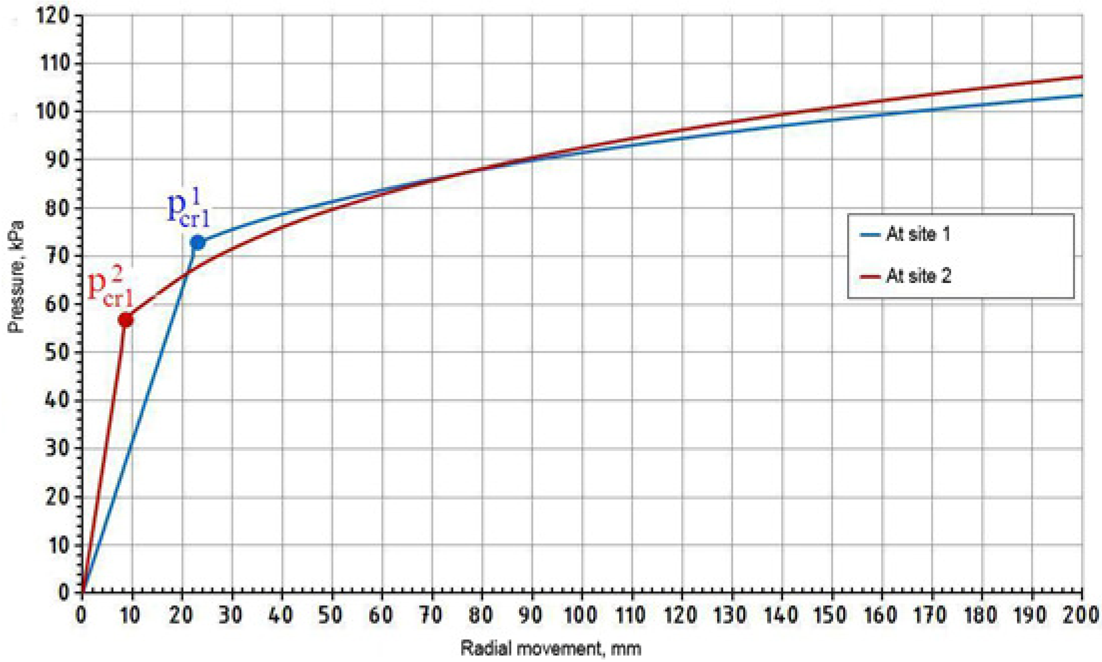

Figure 3 The dependence of the radial displacement of the wall of the membrane-cup u1 on the acting pressure of the injected solution (p^1_{cr1} , p^2_{cr1} , is the critical pressure exerted by the membrane on the soil massif during the injection, in which the elastic deformations of the soil become plastic, for sites 1 and 2, respectively) on the elastic expansion stage at $p_{\text{in1}} \leq p_{\text{cr1}}$:

$$u_1 = (p_{\text{in1}} - \sigma_{01}) \cdot \left(A_{21} + A_{22} \cdot \left(\frac{1+w}{2} \right) \right) \cdot \frac{1}{E}; \tag{3}$$

on the elastoplastic expansion stage at $p_{\text{in1}} > p_{\text{cr1}}$:

$$u_1 = \frac{1}{E} \cdot A\, p_{\text{cr1}}\, k_{r1}^{\beta} + \frac{1}{E} \cdot M \cdot \frac{2\,(1-k_{r1}^{1-\alpha+\beta})}{1+\beta-\alpha} - \frac{1}{E} \cdot N \cdot \frac{1-k_{r1}^{1+\beta}}{(1+\beta)}, \tag{4}$$

where p_{in1} - the pressure of the injected solution on the walls of the membrane-glass, kPa; p_{cr1} - the critical pressure exerted by the membrane-glass on the surrounding soil during the injection, at which the elastic deformations of the soil become plastic, kPa; A_{21} , A_{22} - dimensionless coefficients; E - modulus of deformation of the soil massif in the zone of formation of broadening, kPa; σ_{01} - lateral pressure from the ground weight of the soil in the zone of formation of the broadening, kPa.

At the initial stage, the expansion of the spherical cavity is hindered by the horizontal lateral reactive pressure from its own ground weight:

$$\sigma_{01} = \frac{(2\sigma_h + \sigma_z)}{3} = \frac{2\left(\gamma_{\text{gr}} z \cdot \frac{v}{1-v} \right) + \gamma_{\text{gr}} z}{3}, \tag{5}$$

where σ_h, σ_z - respectively, the horizontal and vertical stress components σ_{01}; γ_{gr} - specific gravity of soil , kN/m^2; z - vertical depth from the surface, m; v – coefficient of transverse strain.

The coefficients α, β, A, M, N are determined by the formulas:

$$\alpha = \frac{12\sin\varphi}{3 + 3\cos\varphi}, \qquad (6)$$

where φ - angle of internal friction;

$$\beta = \frac{2(1-\sin\varphi)}{1+\sin\varphi}; \qquad (7)$$

$$A = A_{21} + A_{22}\left(\frac{1+w}{2}\right); \qquad (8)$$

$$M = (\beta_1 + \beta_2(1-\alpha)) \cdot (p_{cr} + s), \qquad (9)$$

where

$$\beta_1 = A_{11} + \beta A_{21}, \qquad (10)$$

$$\beta_2 = A_{12} + \beta A_{22}; \qquad (11)$$

$$N = (\beta_1 + \beta_2) \cdot s, \qquad (12)$$

where s - auxiliary coefficient:

$$s = \frac{\beta}{\alpha - 1}, \qquad (13)$$

when solving the problem of expanding a spherical cavity $s = 1$.

The critical pressure p_{cr1}, exerted by the membrane on the soil mass, is determined during the injection of the solution, at which the elastic deformations within the condensed zone become plastic:

$$p_{cr1} = \frac{4(\sigma_{01} \cdot \sin\varphi + c \cdot \cos\varphi)}{1 - w - \sin\varphi(3+w)}, \qquad (14)$$

where w - basic coefficient, calculated through additional coefficients A_{11}, A_{12}, A_{21} and A_{22}:

$$w = \frac{A_{12} - 2A_{21} - 2A_{22}}{2A_{22}} - \frac{\sqrt{4A_{21}^2 - 4A_{21}A_{12} + A_{12}^2 + 8A_{22}A_{11} + A_{22}^2 + 2A_{22}A_{12} + 20A_{22}A_{21}}}{2A_{22}}; \qquad (15)$$

$$A_{11} = 1; \qquad (16)$$

$$A_{12} = -2\lambda v; \qquad (17)$$

$$A_{21} = -v; \qquad (18)$$

$$A_{22} = \lambda(1-v), \qquad (19)$$

where λ - diversity factor (ratio of the strain modulus at compression E to the tensile strain modulus E_t), which allows you to take into account the elastoplastic properties of the dusty clay soil and varies from 1 to 5 depending on the type and humidity of the soil.

Then the radius of the packed zone R_{com1} is determined depending on the value of u_1:

$$R_{\text{com1}} = (r_b + u_1)\cdot k_{r1}, \tag{20}$$

where k_{r1} - coefficient reflecting the increase in the radius of the compacted zone around the controlled broadening during injection:

$$k_{r1} = \left(\frac{p_{in1} + s}{p_{\text{cr1}} + s}\right)^{\frac{1}{\alpha}}. \tag{21}$$

The value of the residual voltage $\sigma_{\text{r}}^{\text{res}}$ is determined depending on the values of u_1 and R_{com1}:

$$\sigma_{\text{r}}^{\text{res}} = \sigma_{\text{r}} - \sigma_{\text{r}}^{*}, \tag{22}$$

where

$$\sigma_{\text{r}} = -\frac{p_{\text{in}}(r_{\text{b}} + u_1)^n}{R_{\text{com1}}^n - (r_{\text{b}} + u_1)^n}\cdot\left(\frac{R_{\text{com1}}^n}{r^n} + 1\right), \tag{23}$$

$$\sigma_{\text{r}}^{*} = -\frac{p_{\text{in}}(r_{\text{b}} + u_1)^2}{R_{\text{com1}}^2 - (r_{\text{b}} + u_1)^2}\cdot\left(\frac{R_{\text{com1}}^2}{r^2} + 1\right), \tag{24}$$

where r is the distance from the wall of the membrane-glass of the controlled broadening within the compacted zone of radius R_{com1}; n is a coefficient that varies in the range 0-2 depending on the type of soil and its water saturation and is determined on the basis of indications of mesodosis during the formation of the broadening (in our case, n = 1.55) on the basis of the power law of deformation under shear:

$$\tau = \tau_0 \gamma^{\frac{n}{2}}, \tag{25}$$

where τ_0 - initial shear stress value; τ - current shear stress in the process of loading the soil massif; γ – shear deformation of soil massif.

3.2. Design parameters for the formation of the pile shaft

The radial movement of the well wall u_2 is determined by the pressure of the injected solution p_{in2} (kPa) at the stage of the pile stem formation, depending on the length of the pile and the value of pin2, when the condition $p_{\text{req}} > p_{\text{in2}} > p_{\text{cr2}}$ (where p_{req} is the pressure on the well wall at which Hydraulic fractures are formed, kPa, p_{cr2} is the critical pressure corresponding to the beginning of the formation of plastic deformations, kPa) on the basis of analytical solutions. Ter-Martirosyan, V.G. Fedorovsky, A.I. Polishchuk [6, 14] and others:

on the elastic expansion stage at $p_{in2} \le p_{cr2}$:

$$u_2 = (p_{in2} - \sigma_{02})\cdot(A_{21}^{*} + A_{22}^{*}(1 + w^{*}))\cdot\frac{1}{E}, \tag{26}$$

where σ_{02} - lateral reactive pressure of the soil during the formation of the pile shaft, kPa;

on the elastoplastic expansion stage at $p_{req} > p_{in2} > p_{cr2}$:

$$u_2 = \frac{1}{E} \cdot A^* p_{\mathrm{cr}2} k_{\mathrm{r}2}^{\beta^*} + \frac{1}{E} \cdot M^* \cdot \frac{1 - k_{\mathrm{r}2}^{1-\alpha^*+\beta^*}}{1+\beta^* - \alpha^*} - \frac{1}{E} \cdot N^* \cdot \frac{1 - k_{\mathrm{r}2}^{1+\beta^*}}{1+\beta^*}, \quad (27)$$

At the initial stage, the expansion of the cylindrical cavity is hindered by the horizontal lateral reactive pressure from its own weight of the ground:

$$\sigma_{02} = \gamma_{\mathrm{gr}} z \cdot \frac{\nu}{1-\nu}. \quad (28)$$

The coefficients α^*, β^*, A^*, M^*, N^* are determined by the formulas:

$$\alpha^* = \frac{2\sin\varphi}{1+\sin\varphi}; \quad (29)$$

$$\beta^* = \frac{1-\sin\varphi}{1+\sin\varphi}; \quad (30)$$

$$A^* = A_{21}^* + (1+w^*) \cdot A_{22}^*; \quad (31)$$

$$M^* = (\beta_1^* + \beta_2^*(1-\alpha^*)) \cdot (p_{\mathrm{cr}2} + s^*), \quad (32)$$

where

$$\beta_1^* = A_{11}^* + \beta^* A_{21}^*, \quad (33)$$

$$\beta_2^* = A_{12}^* + \beta^* A_{22}^*; \quad (34)$$

$$N^* = (\beta_1^* + \beta_2^*) \cdot s^*, \quad (35)$$

where s^* - auxiliary coefficient, which when solving the problem of expansion of a cylindrical cavity is determined by the formula:

$$s^* = \frac{c \cdot \cos\varphi}{\sin\varphi}. \quad (36)$$

The critical pressure $p_{\mathrm{cr}2}$ is determined corresponding to the onset of the formation of plastic deformations on the surface of the borehole wall under the pressure of the injected solution:

$$p_{\mathrm{cr}2} = \frac{-2(\sigma_0 \cdot \sin\varphi + c \cdot \cos\varphi)}{((w^*+2) \cdot \sin\varphi + w^*)}; \quad (37)$$

where w^* - basic coefficient, calculated through additional coefficients A^*_{11}, A^*_{12}, A^*_{21}, A^*_{22}:

$$w^* = \frac{-A_{21}^* + A_{12}^* - 2A_{22}^* - \sqrt{A_{21}^{*2} - 2A_{21}^* A_{12}^* + A_{12}^{*2} + 4A_{22}^* A_{11}^*}}{2A_{22}^*}; \quad (38)$$

$$A_{11}^* = 1 - \nu^2; \quad (39)$$

$$A_{12}^* = -\lambda\nu(1+\nu); \quad (40)$$

$$A_{21}^* = -\nu(1+\nu); \quad (41)$$

$$A_{22}^{*} = \lambda(1-\nu^{2}) \,. \tag{42}$$

Then, the radius of the compacted zone of the near-land massif R_{com2} is determined by increasing the diameter of the pile stem under the pressure of the injected solution:

$$R_{\mathrm{com2}} = (r_{\mathrm{b}} + u_{2})k_{\mathrm{r2}} \,, \tag{43}$$

where k_{r2} - coefficient reflecting the increase in the radius of the compacted zone of the near-land massif under the influence of the pressure of the injected solution on the walls of the well:

$$k_{\mathrm{r2}} = \left(\frac{p_{\mathrm{in2}} + s^{*}}{p_{\mathrm{cr2}} + s^{*}} \right)^{\frac{1}{\alpha^{*}}} . \tag{44}$$

3.3. Design parameters for the formation of hydraulic fractures

The pressure on the borehole wall is p_{req} (kPa), at which hydro-fractures are formed depending on the depth of the injection horizon z, the specific gravity of the soil γ_{gr}, its strength characteristics (specific adhesion c and the internal friction angle φ), strength values of the concrete under compression (σ_c) and tension (σ_t), which must be taken into account when re-injecting the solution in the "fracturing" regime [1, 3]:

$$p_{\mathrm{req}} = \gamma_{\mathrm{gr}} \cdot z \cdot (1 + \sqrt{3} \cdot \mathrm{tg}\varphi) + \frac{2}{\sqrt{3}} \cdot c \,. \tag{45}$$

Changes in mechanical characteristics are estimated mainly from changes in the modulus of deformation and the porosity coefficient by the method of A.L. Lanis, according to which the average value of the changed soil characteristics is determined by the volume of the injected solution per 1 m of the pile length [3]:

$$E_{com2} = \frac{E_c + E_a}{2} \,; \tag{46}$$

$$E_{\mathrm{a}} = \frac{f_{\mathrm{a}} E_{\mathrm{s}} E_{\mathrm{d}}}{f_{\mathrm{a}} k_{\mathrm{a}} E_{\mathrm{d}} + (1 - k_{\mathrm{a}}) E_{\mathrm{s}}} \,; \tag{47}$$

$$f_{\mathrm{a}} = \exp[k_{\mathrm{a}}(1 + e_{\mathrm{d}})] \,; \tag{48}$$

$$k_{\mathrm{a}} = \frac{V_{\mathrm{s}} V_{\mathrm{p}}}{V_{\mathrm{d}}^{2}} \,; \tag{49}$$

$$E_{\mathrm{c}} = E_{\mathrm{d}} \cdot \exp[e_{\mathrm{d}} - e_{\mathrm{c}}] \,; \tag{50}$$

$$e_{\mathrm{c}} = e_{\mathrm{d}} - k_{\mathrm{a}}(1 + e_{\mathrm{d}}) \,; \tag{51}$$

$$e_{\mathrm{a}} = \frac{(1 - k_{\mathrm{a}}) \cdot e_{\mathrm{c}}}{1 + k_{\mathrm{a}} e_{\mathrm{c}}} \,; \tag{52}$$

$$e_{com2} = \frac{e_c + e_a}{2} \,, \tag{53}$$

where E_{com2} - modulus of deformation of soil, reinforced and compacted by hydraulic fractures; E_c - modulus of deformation of a compacted soil massif between fractures, MPa; E_a - modulus of deformation of the ground mass reinforced with hydraulic fractures as a whole, MPa; E_s - the deformation module of the solution injected into the soil massif after its solidification (it is assumed equal to 8,000 MPa); E_d - the initial modulus of deformation; f_a – coefficient of exponential dependence of soil pore filling with solution; k_a - coefficient reflecting the proportion of solidified solution in frac fractures (minus pore filling) in the allocated volume of the soil massif; V_p - pore volume in the investigated volume of the soil massif,m^3; V_d - the investigated volume of the soil massif, m^3; e_c - coefficient of porosity of the compacted soil massif between fractures; e_a - coefficient of porosity of the ground mass reinforced by hydraulic fractures as a whole; e_d - initial coefficient of soil porosity;e_{com2} – coefficient of soil porosity, reinforced and compacted by hydraulic fractures.

Specific cohesion of soil in the compaction zone c_{com2} is proposed to be determined according to a SP 50-101-2004 with substitution of the value of $e_{com2.}$

3.4. Determination of the regularity of sediment development under static loading conditions

The modulus of deformation of compacted soil under the end of the proposed pile is determined by the formula:

$$E_{\mathrm{com1}} = \frac{(1+\nu)\cdot\left(1-\dfrac{\nu}{1-\nu}\right)}{\left(\dfrac{e_{\mathrm{com1}}\cdot(1+e_0)}{p_{\mathrm{in1}}-\sigma_{01}}\right)}\cdot\frac{\sigma_{\mathrm{r}}^{\mathrm{res}}\cdot\mathrm{tg}\varphi}{\left(\sigma_{\mathrm{r}}^{\mathrm{res}}\cdot\mathrm{tg}\varphi-\sigma_{01}\cdot\dfrac{\left(1-\dfrac{\nu}{1-\nu}\right)}{\sqrt{3}}\right)}, \qquad (55)$$

where e_0 **-** initial coefficient of soil porosity.

On the basis of the elastoplastic model, S.P. Tymoshenko and analytical solutions Z.G. Ter-Martirosyan [11], taking into account the changes in the mechanical characteristics of the compacted zone of the near-land massif along the lateral surface (formula (46)) and under the end of the pile (formula (55)), as well as the preservation of residual stresses $\sigma_{\mathrm{r}}^{\mathrm{res}}$ (formula (22)) to the load pressure), which does not exceed $\sigma_{\mathrm{r}}^{\mathrm{res}}$, the draft of piles S under consideration from the existing static load is proposed to be determined by the formula:

$$S = \frac{N_{\mathrm{st}}}{\pi(r_b+u_1)\cdot G_{com1}+2\pi l\, G_{com2}}\cdot\ln\left[\frac{\tau^*\left(\dfrac{R_{com1}}{r_b+u_1}+\dfrac{R_{com2}}{r_b+u_2}\right)-\tau_u}{\tau^*-\tau_u}\right], \qquad (56)$$

where G_{com1}, G_{com2} - the soil shear modulus in the compacted zone under the end and along the lateral surface of the pile, respectively; l - length of pile; τ_u - tangential stress in the soil massif from the actual vertical static load N_{st}:

$$\tau_{\mathrm{u}} = \frac{N_{\mathrm{st}}}{\pi(r_{\mathrm{b}}+u_1)^2+2\pi l(r_{\mathrm{b}}+u_2)}, \qquad (57)$$

τ^* - tangential stress limit value:

$$\tau^* = \gamma_{\mathrm{cr}}\left((\sigma_{\mathrm{r}}^{\mathrm{com1}} + \sigma_{\mathrm{r}}^{\mathrm{res}}) \cdot \left(\frac{1+\sin\varphi}{1-\sin\varphi} \right) + 2c_{\mathrm{com1}} \cdot \left(\frac{\cos\varphi}{1-\sin\varphi} \right) \right) + \gamma_{cf}\,(\sigma_{\mathrm{r}}^{\mathrm{com2}} \cdot \mathrm{tg}\,\varphi + c_{\mathrm{com2}}), \tag{58}$$

where γ_{cr}, γ_{cf} - coefficients of ground conditions under the end and along the lateral surface of the pile, respectively (taken in accordance with clause 7.6 of SP 24.13330.2011); c_{com1}, c_{com2} - specific cohesion of soil in the compacted zone under the end and along the lateral surface of the pile, respectively; $\sigma_{\mathrm{r}}^{\mathrm{com1}}$ - pressure of radial compression of the soil mass at the end of the pile:

$$\sigma_{\mathrm{r}}^{\mathrm{com1}} = p_{\mathrm{cr1}} + \sigma_{01}; \tag{59}$$

$\sigma_{\mathrm{r}}^{\mathrm{com2}}$ - pressure of radial compression of a soil massif along the lateral surface:

$$\sigma_{\mathrm{r}}^{\mathrm{com2}} = p_{\mathrm{cr2}} + \sigma_{02}. \tag{60}$$

On this basis, the dependence of the sediment on the load was plotted and compared (Fig. 4).

Thus, the calculations performed according to formula (56) without taking into account the influence of residual stresses and changes in the mechanical characteristics of the near-soil massif within the compacted zone and the technique of SP 24.13330.2011 "Pile foundations" allow determining the draft of the proposed piles under conditions of static loading with a significant margin up to 45%). For a more reliable determination of the draft (with a deviation of up to 25%), using formula (56), when calculating the design resistance, it is necessary to take into account residual stresses according to formulas (22-24) and changes in mechanical characteristics when forming the compacted zone of the soil mass by formulas (46, 56).

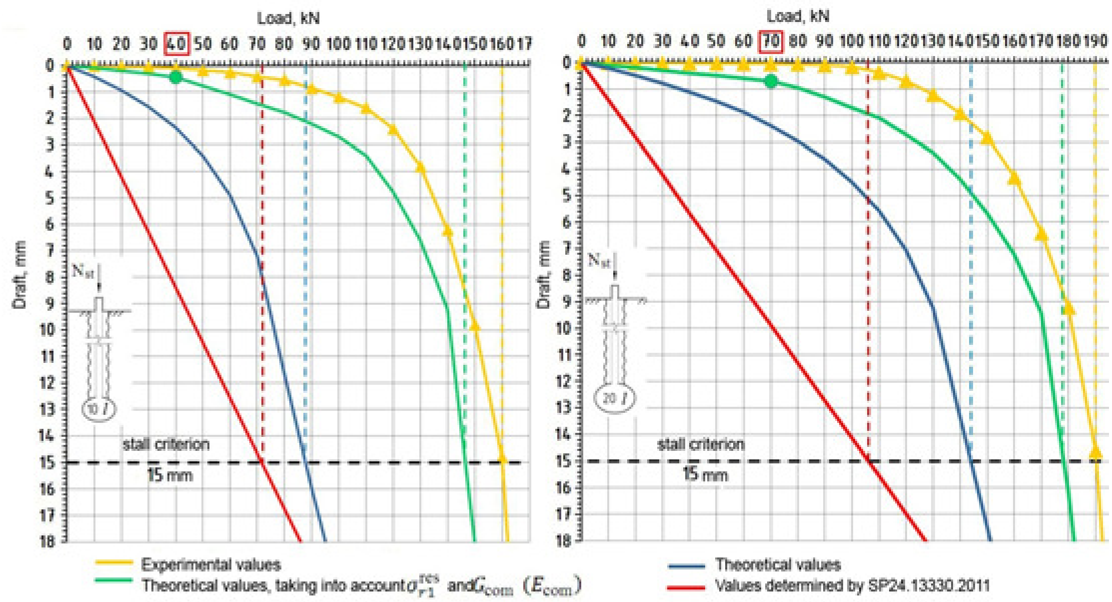

Figure 4 Graphs of dependence of rainfall on load for drilling pile piles with controlled broadening and repeated injections of solution in the regime of "hydraulic fractures": a, b - at site 1 with the volume of injected solution of 10 and 20 liters, respectively; c, d - on the site 2 with the volume of the injected solution of 30 and 40 l, respectively. The letters: $\sigma_{\mathrm{r}}^{\mathrm{res}}$ - residual voltage; E_{com}, G_{com} is, respectively, the deformation modulus and shear modulus of the near-land massif within the compacted zone; N_{st} - vertical static load

4. CONCLUSION

1. A new effective method for reinforcing ribbon foundations of buildings and structures on dusty-clay grounds has been developed, which allows to combine the device of drilling-injection piles with controlled broadening at the lower ends, the use of the cuff technology for injecting cement mortar in the regime of "hydraulic fracturing" (including repeated) along any horizon of the soil massif and restoration of the foundation construction into one technological operation.

2. It has been established that when forming a controlled broadening at the end of a drilling-injection pile with a volume of 10-40 liters, its minimum diameter is 250-390 mm. At the same time, within the average distance (radius) from the injector pipe of 0.15-0.60 m, a compacted zone of the ground mass is formed with significantly changed physicomechanical characteristics (the density increases by an average of 25%, the humidity decreases by 37%, the strain modulus increases by 64%), which in the future must be taken into account when calculating the basic parameters of controlled broadening.

3. It has been established that during the formation of the borehole of drilling-injection pile, it is necessary to apply only re-injection of the solution in the "hydraulic fracturing" regime, in which an increase in the number of hydraulic fractures up to 130 mm thick and their localization in the near-soil subsoil within 0.5 m from the injector tube, as a result of which the bearing capacity of the pile increases by an average of 20%. It was also revealed that due to the constantly acting pressure on the walls of the well, an average of 0.2 MPa, an increase in the diameter of the pile stem is on average 2 times, which together with the formation of hydraulic fractures leads to the formation, within the average distance (radius) injector 0.2-0.4 m of the compacted zone with changed physicomechanical characteristics (density increases by an average of 17%, humidity decreases by 28%, the strain modulus increases by 35%), which in the future should be taken into account in the design calculation dividing the parameters of the pile stem.

4. Based on the results of field studies, a calculation algorithm has been developed that allows:

- determine the radius of the compacted ground zone around the controlled broadening and pile stem with an accuracy of 7%;
- determine the radial movements of the walls of the membrane-glass and the well with an accuracy of 10 and 14%, respectively;
- predict the dependence of the sedimentation of the proposed piles in the dusty clay soils under conditions of static load loading with an accuracy of up to 25%.

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