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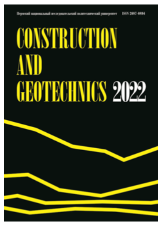

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CEMENTATION OF SOILS BY WRISTBAND-PIPE INJECTION TECHNOLOGY IN CONDITIONS OF THAWING PERPETUALLY FROZEN SOILS

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ABSTRACT

In the conditions of global warming of the climate in the territories of permafrost distribution it takes place the processes of increasing the temperature of perpetually frozen soils and increasing the depth of their thawing. These factors lead to a decrease in the structural safety of the buildings and constructions. Measures to return soils to the frozen state are not always feasible and effective. In such conditions the solution to the problem of decreasing the pile foundation bearing capacity can be the ground strengthening by the cementation using the wristband-pipe injection technology. In the article it is presented the experience of assessing the current technical condition and designing of bases and foundations strengthening of a three-storey residential building in Yamal-Nenets Autonomous District, village Muzhi. The object was given a limited-operational category of the technical condition due to the deficit of the pile foundation bearing capacity. To bring the constructions of the object to the working condition the project of pile foundation strengthening with the cementation by wristband-pipe injection technology was developed. While injecting the soil foundation with the hydraulic fracturing the additional (induced) tension state appears, and it changes the deformation and strength properties of soils, voids ratio, index of liquidity and modulus of deformation. Reinforced joints, soil compaction and stabilization appear due to the injection. While making the hydraulic test and fracturing in the soil body its stress and strain state is changing, foundation piles are stressed with the additional lateral pressure. Besides the soil is strengthening under the pile toe bulb. Analytical calculations are given for increasing the bearing capacity of a pile during cementation of soils due to the increase of horizontal stresses and the work of compacted foundation soil on the lateral surface and under the lower end of the pile.

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ЦЕМЕНТАЦИЯ ГРУНТОВ МАНЖЕТНОЙ ТЕХНОЛОГИЕЙ В УСЛОВИЯХ ОТТАИВАНИЯ МНОГОЛЕТНЕМЕРЗЛЫХ ГРУНТОВ

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АННОТАЦИЯ

В условиях глобального потепления климата на территориях распространения вечной мерзлоты происходят процессы повышения температуры многолетнемерзлых грунтов и увеличения глубины их оттаивания. Данные факторы приводят к снижению конструктивной безопасности зданий и сооружений. Мероприятия по возвращению грунтов в мерзлое состояние не всегда реализуемы и эффективны. Решением проблемы снижения несущей способности свайных фундаментов в подобных условиях может являться усиление грунтов цементацией по манжетной технологии. В статье приведен опыт оценки текущего технического состояния и проектирование усиления оснований и фундаментов трехэтажного жилого дома в Ямало-Ненецком автономном округе, с. Мужи. Вследствие дефицита несущей способности свайного фундамента объекту была присвоена ограниченно-работоспособная категория технического состояния. Для приведения конструкций объекта в работоспособное состояние разработан проект усиления путем цементации свайного основания по манжетной технологии. При инъецировании грунтового основания гидроразрывами появляется дополнительное (наведенное) напряженное состояние, которое меняет деформационные и прочностные свойства грунтов, изменяется коэффициент пористости, показатель текучести, модуль деформации. За счет инъекции появляются армированные связи, уплотнение и закрепление грунта. При опрессовке и создании гидроразрывов в массиве грунта изменяется его напряженно-деформированное состояние, сваи фундамента обжимаются с дополнительным боковым давлением. Кроме этого, упрочняется грунт под пятой сваи. Приведены аналитические расчеты увеличения несущей способности сваи при цементации грунтов за счет повышения горизонтальных напряжений и работы уплотненного грунта основания по боковой поверхности и под нижним концом сваи.

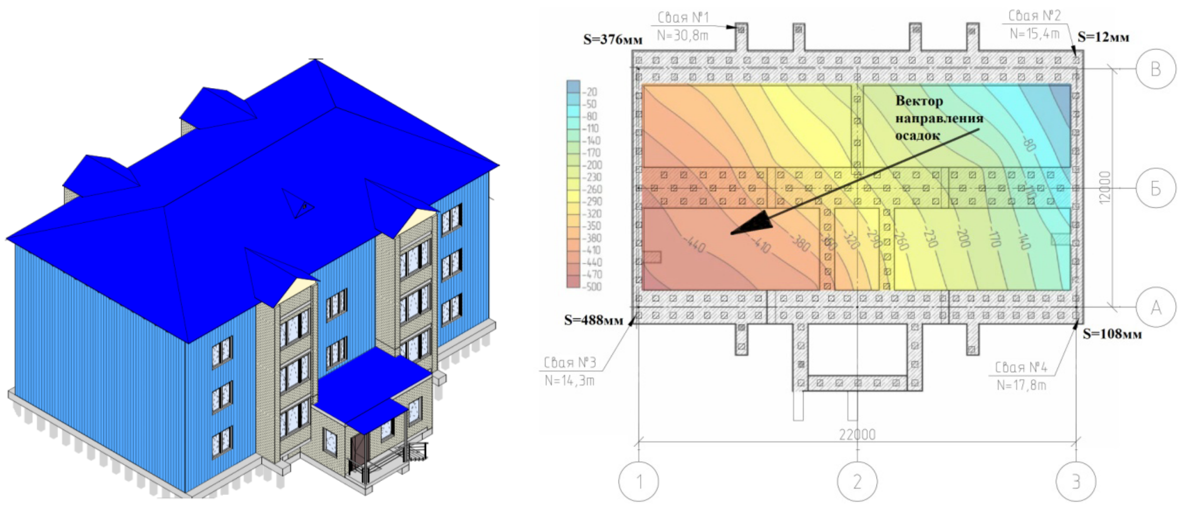
Introduction

The significant part of the territory of Russian Federation is located within perpetually frozen soils (PFS). At present, the processes of general climatic warming occurring in the Arctic zone of Russia have led to a significant increase in the temperature of perpetually frozen soils and an increase in the depth of their seasonal thawing (permafrost degradation) [1–3]. The results of long-term geocryological monitoring carried out by the foreign and Russian scientists have shown that these processes are global and in different ways refer to all the territories with PFS [4, 5].

The gradual thawing of the upper layers of PFS leads to a widespread decrease in the bearing capacity of frozen foundations in the built-up areas, and the area of freezing is actively reduced with increasing the depth of seasonal thawing [6, 7]. This process of PFS thawing leads to the differential settlement of buildings and constructions which were built according to the “first principle” of construction [8]. Measures of returning soil into the frozen condition often cannot be effective, moreover the freezing front can be formed in such a way that it will lead to the more differential settlement. And conversion of the foundation into the piled, strip foundation, isolated footing and etc. as well as installation of the additional piles may be difficult to realize in case of frequent and multi-row arrangement of existing piles, low sub-floor height and so on [9, 10].

One of the effective ways of strengthening pile foundations in conditions of PFS degradation is the reinforcement of foundations using pressure injection (PI according to wristband-pipe injection technology), based on the introduction of cement mortar into the soil mass by creating hydraulic fractures and partial impregnation of the soil [11, 12]. During this process, the characteristics of soils under the pile-toe bulb are becoming better and it is created additional pressure on the piles, formed due to the forced transverse deformation of soil. The development theory and practice of PI method is given in the works of V.A. Bogomolova, M.Y. Krickogo, V.V. Lushnikogo, B.N. Melnikogo, M.L. Nuzhdina, V.I. Osipova, Y.A. Pronozina, I.I. Saharova, M.A. Samohvalova, D.N. Davlatova etc. Formation of grid structure of soil compacted by hydraulic fracturing; creating of residual stress state; applicability in unfavourable and cramped conditions; workability; environmental friendliness at all stages [13, 14] are the advantages of PI method.

The cementation of pile foundation by wristband-pipe injection technology was used by the development team in the project of strengthening a three-floored building (YANAO, village Muzhi) (Fig. 1). Building dimensions are 22,0 × 12,0 m. The height of the building is 3,0 m. The structural scheme of the object is wall–based, with longitudinal and transverse load-bearing walls. There are tape pile foundations (piles 300 × 300 mm) with a high grillage and ventilated underground. Foundations are built according to the "first principle" of construction, i.e. saving the frozen state of soils.



a *b*

Fig. 1. Object diagram: general view of the building (*a*), schematic plan with the pile loads and the direction of building slope (*b*)

The analysis of engineering hydro meteorological conditions of the object has shown that the site being sought is located in a built-up area which is characterized by a well-developed network of engineering communications and a pronounced influence of anthropogenic activity. In the process of research the soils in the frozen state were not found, a part of the soils has been in the cooled condition, it means that there is a degradation of perpetually frozen soils of the foundation.

Summary engineering and geological section (Fig. 2): EGE 5-3 (N2-Q) – sandy loam is powdery, fluid, with a capacity from 1,00 to 7,10 m; EGE 3-4 (N2Q) – light sandy loam, highplastic, opened all around layer up to 12,50 m; EGE 4-3 (N2-Q) – loam is light, refractory, of power 0,90–3,60 m; EGE 1a (tQIV) – man-triggered soil, capacity from 1,40 to 1,90 m. The average value of the pile length from the sole of the grillage was ≈ 6,2 m, in the soil taking into consideration the height of the open crawl space was ≈ 5,0 m. The landing of the pile on the engineering-geological section is shown in Fig. 2.

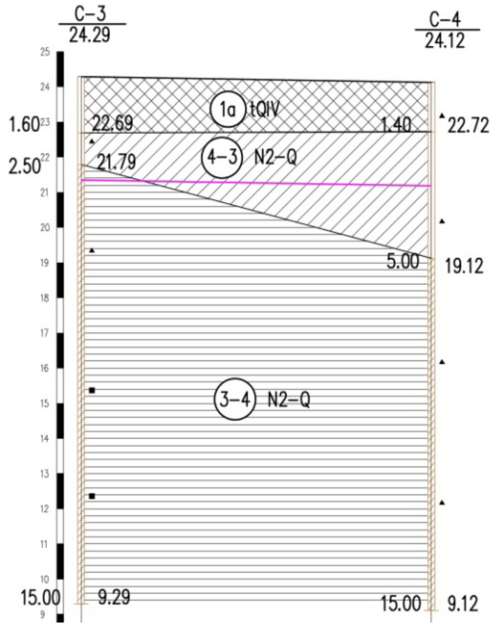


Fig. 2. Engineering-geological conditions of the object

The survey of the house residents has shown that the appearance of the defects in the form of cracks in the load-bearing and enclosing structures was recorded about two years ago before the examination after the replacement of the septic tank. During the examination of surrounding area in close proximity to the building (≈2,0 m) underground laying of the heat supply network and the presence of septic (10,5 m from building) were found. In conditions of general degradation of permafrost soils it was concluded that the septic and heat supply network have a direct thermal influence on the development of defects and existed deformations.

The instrument-aided structural survey revealed that the main defects of the grillages are represented by through and part-through cracks with an opening width of up to 3 mm. Places of regular flooding of the underground by sewage effluents due to violation of the tightness of the sewer system and its emergency technical condition have been established, which probably contributed to greater and transient warming and watering of the foundation soil and a decrease in its strength and deformation characteristics.

Numerous horizontal cracks with an opening width of up to 5 mm were found in the foundation piles. Such defects are caused by the creation of significant bending strain in the piles as a result of a differential settlement of foundations. It was found that the existing piles are mainly located in a layer of clay with a soft-plastic consistency. Based on the performed verification calculations of the bearing capacity of reinforced concrete piles with regard to the probable thawing of the soil base it was revealed that the estimated load on the existing w/w foundation piles exceeded their bearing capacity: the deficit in bearing capacity ranged from 8.3 to 29.5 t.

During the inspection of the walls it was found defects in the form of vertical and inclined cracks with an opening width of up to 5 mm. The most probable causes of these defects are the redistribution of efforts in the masonry caused by the development of uneven deformations of the foundations. According to the results of the survey, the foundations and walls of the facility were assigned a limited operational category of technical condition with the possibility of transition to an emergency state.

For a qualitative assessment of the deformation pattern, an engineering and geodesic survey was performed. As a result of the survey, it was found that the grillage of the building has a uniform overall slope (the difference in displacement is 488 mm), the maximum relative unevenness of vertical movements of the grillage foot is $\Delta S/L = 0.018$ which significantly exceeds the maximum permissible value $[\Delta S/L] = [0.0024]$. The maximum deviation of the building from the vertical was 200 mm. The revealed deformations are probably associated with a decrease in the bearing capacity of piles and an increase in the compressibility of the base as a result of the thawing of permafrost soils.

To assess the rate of deformation development, a phased geotechnical monitoring of the object was organized and carried out in 2 cycles: June – September 2022 (I monitoring cycle) and September – December 2022 (II monitoring cycle). The geotechnical monitoring program provided for geodetic monitoring of vertical displacements and precipitation, as well as technical monitoring of the dynamics of the opening of force cracks [15]. To implement the program, 13 geodesic marks along the perimeter of the building, as well as 8 gypsum lighthouses were installed in June 2022.

According to geodesic leveling made in September 2022 (1 cycle of monitoring) an average speed of settlement increment was 1,6 mm/month, maximum value –7…–11 mm. According to the second cycle of monitoring made in December 2022, an average speed of settlement increment was –2,8 mm/month, maximum value –13…–15 mm. Maximum settlement of the building during monitoring was –22 mm. Fig. 3 shows a graph of the movement of geodesic marks in time over two monitoring cycles. According to the results of the monitoring of the opening of force cracks the development of a crack by 1.1 mm was established. The revealed defects and their development indicate a continuous uneven precipitation of the foundations.

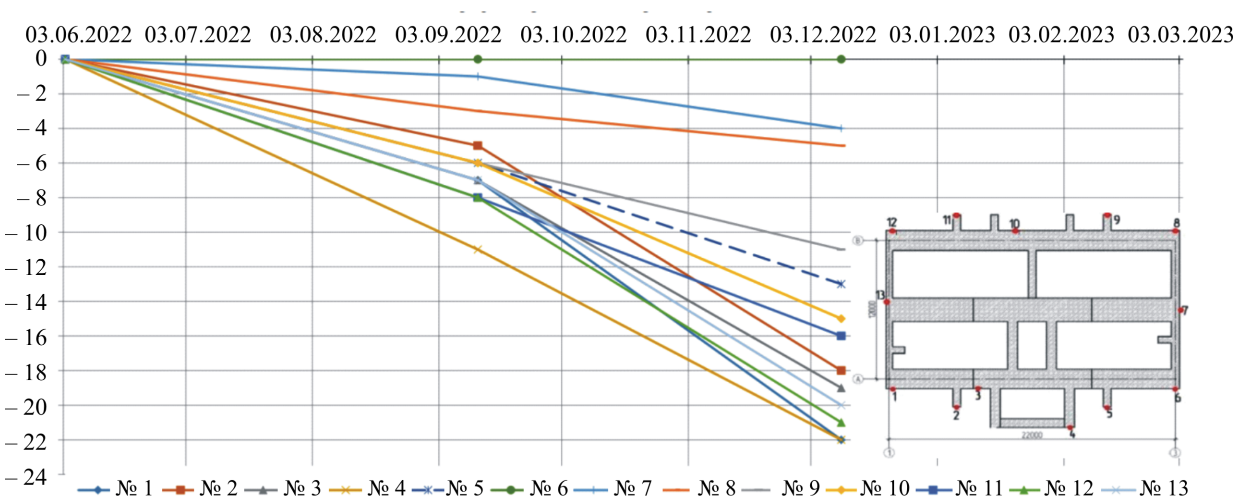


Fig. 3. Diagram of geodesic marks movement in time for 2 monitoring cycles

Based on a comprehensive study of the object it was found that the main factors influencing the degradation of PFS under the building were: general processes of global warming, influencing permafrost soils condition; underground laying of the heat supply network in the immediate vicinity of the building; regular leakage of sewage and, as a consequence, the ingress of sewage into the base; disturbed layout (deconverting) of the base under the building to discharge water to the surrounding terrain of the adjacent territory; sewing of the ventilated underground with a profiled sheet around the perimeter without holes for possible ventilation, which leads to a change in temperature and humidity the regime of the soil base; the thermal effect of the septic tank arranged in the immediate vicinity of the building.

Under the building there was a dominant temperature degradation of soils to a depth of more than 15 meters from the surface which was the main cause of significant uneven precipitation of pile foundations, and led to deformations, cracks and distortions of structural elements of the building.

In order to eliminate the progressive development of foundation sediments and their unevenness with the risk of a critical (emergency) situation it was developed a project to strengthen existing pile foundations from the positions of the device and operation of piles in the thawed foundation formed during the thawing of frozen soils by cementing the foundation, performed in order to increase the physical and mechanical properties of the soil foundation directly by side surface and heel of existing reinforced concrete piles [16, 17]. The schematic diagram of cementation is shown in the Fig. 4.

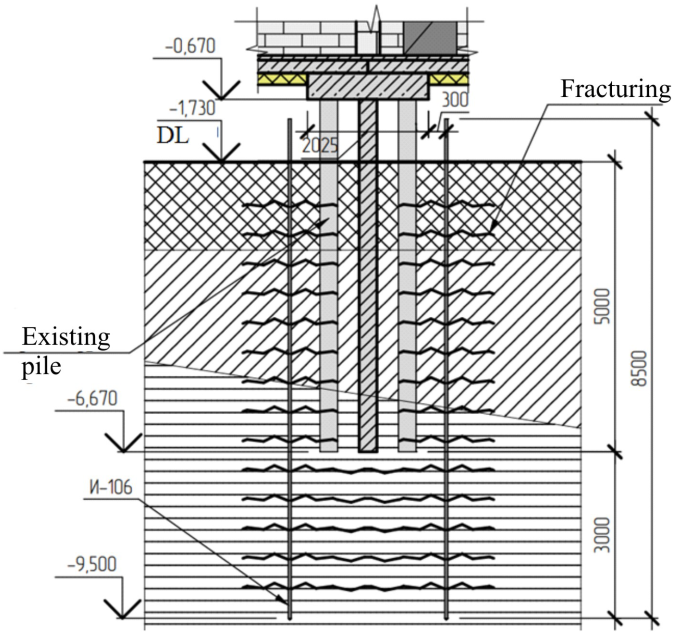


Fig. 4. Scheme of the pile foundation reinforcement by cementation according to wristband-pipe injection technology

In the process of injecting the soil base with hydraulic fractures an additional (induced) stress state appears, which changes the deformation and strength properties of soils [18, 19]. Thus, firstly: the porosity coefficient e and the yield index IL change, which in turn leads to the increase in the bearing capacity of piles. Secondly, due to injection, reinforced bonds appear, compaction and consolidation of the soil, and thirdly, the bearing capacity of piles increases due to the increase in lateral compression of the soil

To determine the change in the value of the porosity coefficient e and the yield index I_L, the calculation algorithm presented in [20, 21] was adopted. The pitch of the injectors according to the project is 0.75 m, V_d of the fixed soil array at a cementation depth of 8.5 m is 15.013 m^3, the total number of hydraulic fractures is 17. The calculation results with a change in the thickness of hydraulic fracturing t from 5 to 30 mm are presented in Table 1. The graph of the change in I_L from the difference (change) in porosity Δn is shown in Fig. 5. The change in natural humidity W from the thickness of hydraulic fracturing t is shown in Fig. 6. The dependence of the fluidity index I_L of the porosity coefficient is presented in Fig. 7.

According to the survey the initial values of the mechanical-and physical properties under the pile toe bulb are $e_{initial\ value} = 1.13$, liquidity index $I_L = 0.681$, $W = 0.347$, $W_L = 0.418$, $Wp = 0.195$.

When it is positioned the injectors with a step of 0.75 m and the hydraulic fracturing thickness of 5 mm, as a result of the injection of cement mortar due to the increase of soil density there is a decrease in the number of plasticity by 15 %, and the natural humidity by 7 %, i.e. the decrease in the natural humidity index depends on the volume of the injected solution into the soil. Consequently, a change in the porosity coefficient e and the yield index I_L during injection leads to the increase of piles bearing capacity.

Table 1

Calculation of changes in the values of the porosity coefficient e and the yield index I_L as a result of cementation

t	Cement mortar V_p, m^3	Δn	n_{final}	e_{final}	$W_к$	$I_{L\ final}$
0.005	0.1501	0.01	0.52	1.0833	0.323	0.57716
0.01	0.3002	0.02	0.51	1.0408	0.296	0.45602
0.015	0.4503	0.03	0.5	1	0.270	0.33972
0.02	0.6005	0.04	0.49	0.9607	0.245	0.22799
0.025	0.7506	0.05	0.48	0.9230	0.221	0.12055
0.03	0.9007	0.06	0.47	0.8867	0.198	0.01717

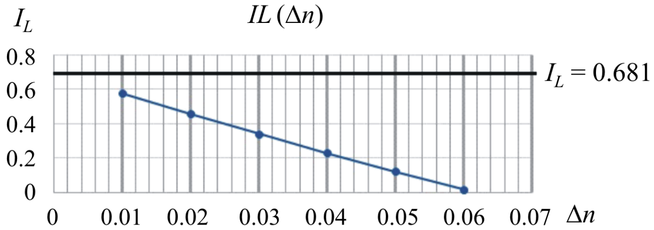


Fig. 5. Graph of the change in I_L from the difference (change) in porosity Δn

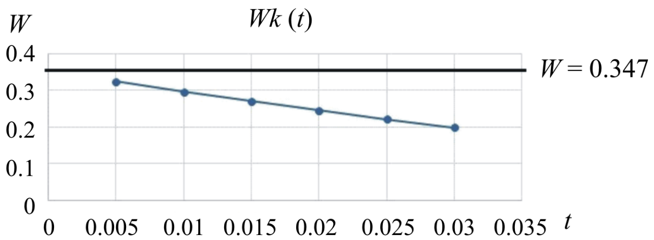


Fig. 6. The graph of W change from the width of hydraulic fracturing t

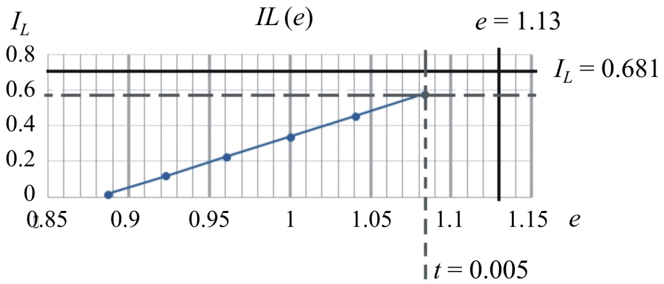


Fig. 7. The graph of I_L change from the void ratio e

Calculation algorithm was accepted to define changes in modulus of deformation after cementation of soil foundation [19, 22]. The calculation shall be made for the pile at the intersection of A/1 axes, where according to results of monitoring the greatest settlements were found.

Determination of the soil deformation modulus due to injection reinforcement:

$$E_a = \frac{f_a E_s E_d}{f_a k_a E_d + (1 - k_a) E_s}, \tag{1}$$

$$f_a = \exp\left[k_a (1 + e_d)\right], \tag{2}$$

where $k_a = \dfrac{V_s \cdot V_p}{V_d^2}$ – is the coefficient of soil reinforcement, $V_d = \pi r_{cp}^2 = 1,77$ м^3 – the volume of stabilized soil body (1 m.p.), r_{av} = 0,75 m – radius of sealing and fixing zone of the body, E_s = 8000 MPa – deformation modulus of cement mortar injected in to the soil, $V_p = V_d \cdot n$, V_s = 1,75 m^3/1m.p. – the volume of mortar for cementation.

Calculation of deformation modulus during soil compacting with the injection:

$$E_c = E_d \exp\left[e_d - e_c\right], \tag{3}$$

$$e_c = e_d - k_a (1 + e_d). \tag{4}$$

Deformation modulus after compacting and stabilization of the soil foundation:

$$E_{упл} = \frac{E_a + E_c}{2}. \tag{5}$$

The results of the calculation are shown in Table 2. The change of deformation modulus for the ground layers are shown in Fig. 8. Calculations show that after reinforcement and compacting of the soils with the injection, the middle-sized sand modulus of deformation has increased 1.62 times, low-plasticity loam soil – 2.6 times and high-plastic loam – 1.29 times. Thus, the effectiveness of the injection with the hydraulic fracturing is increasing with the growth of coefficient of soil porosity and is effective for weak soils with the soft fluid-plastic consistency.

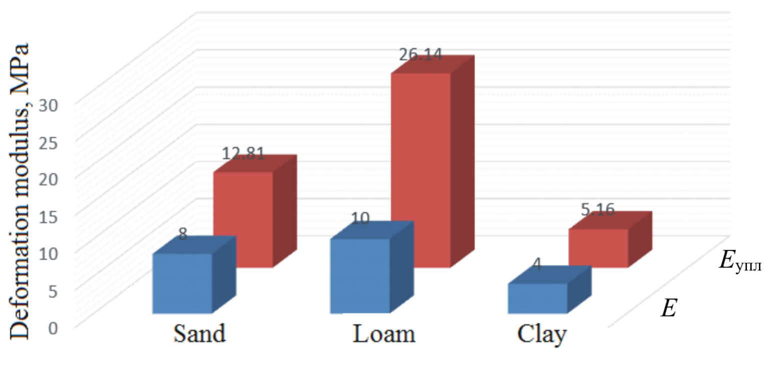


Fig. 8. Deformation modulus of the soil layers before and after the cementation, MPa

Table 2

Calculation of deformation modulus after compacting and stabilization of the foundation base by the injection

Layer number	Layer capacity, m	Type of soil	e	E_d, MPa	n, %	k_a	f_a	E_a, MPa	E_c, MPa	$E_{упл}$, MPa
1A	1.6	Sand	0.55	8	35.4	0.219	1.404	14.38	11.23	12.81
4-3	0.9	Loam	0.6	10	37.5	0.412	1.935	32.91	19.35	26.13
3-4	5.5	Clay	1.13	4	53	0.095	1.225	5.41	4.90	5.16

In the process of crimping and creating hydraulic fractures in the soil mass its VAT changes, the foundation piles are compressed with additional lateral pressure. In addition, the soil is strengthened under the fifth pile. In the design scheme (Fig. 11), the base along the length of the injector is divided into layers h = 500 mm high, equal to the distance between the injection holes of the injectors. As a result of hydraulic fracturing of the soil mass, a lens with a thickness of Δh = 5 mm is formed, by the amount of which the elementary layer is deformed.

The value of relative deformation:

$$\varepsilon = \Delta h / h = 0{,}01. \tag{6}$$

Vertical stresses:

$$\sigma_z = E_d \cdot \varepsilon. \tag{7}$$

The value of horizontal stresses:

$$\sigma_x = \sigma_z \cdot \xi, \tag{8}$$

where ξ – is the index of lateral pressure. The results of the calculations are shown in Table 3. The distribution of additional horizontal stresses relative to the lateral surface of the pile is presented in Fig. 9.

Table 3

Calculation of additional horizontal stresses relative to the side surface of the pile

Layer number	Layer capacity, m	Type of soil	e	φ, grad.	E, MPa	$E_{упл}$, MPa	σ_z, kPa	ξ	σ_x, kPa
1A	1.6	Sand	0.55	35	8	12.81	80	0.4	32
4-3	0.9	Loam	0.6	19	10	26.13	100	0.5	50
3-4	5.5	Clay	1.13	16	4	5.16	40	0.7	28

The increment of bearing capacity of the pile which occurs due to an increase in horizontal stresses and the work of the compacted soil of the base under the lower end of the pile and along the side surface:

$$F_d = \sum f_i \cdot h_i \cdot u + \sum \sigma_x^i \cdot \mathrm{tg}\varphi \cdot u \cdot h_i + \gamma_{cR} RA \tag{9}$$

Where the first term $\sum f_i \cdot h_i \cdot u$ is calculated resistance of the foundation soils along the side surface of the pile with improved indicators e = 1,0833 and I_L = 0,577 (see Table 1); second term $\sum \sigma_x^i \cdot \mathrm{tg}\varphi \cdot u \cdot h_i$ – is calculated soil resistance due to lateral pressure; the third term $\gamma_{cR} RA$ –

is calculated soil resistance under the fifth pile with improved indicators e = 1,0833 и I_L = 0,577; σ_x – is the side pressure voltage, kPa; h_i – is the height of the plot, м; u – is the perimeter of the pile section, m; R = 915 kPa (loam, $I_{L(final)}$ = 0,577); A = 0,3·0,3 = 0,09 m^2. Thus, F_d was 22,7 t.

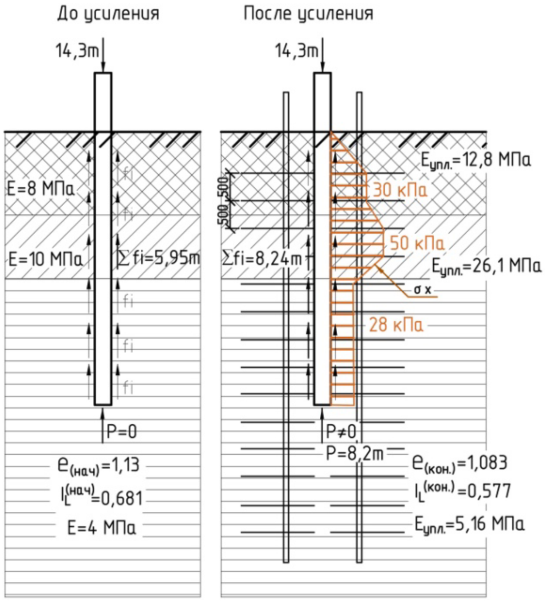


Fig. 9. Scheme of changing the work of the pile during the cementation of the soil base

According to the results of the verification calculations the actual bearing capacity of this pile at the time of the survey was 5.9 tons, i.e. the deficit of bearing capacity was 10.1 tons.

$$N = 14,3 > \frac{\gamma_0 \cdot F_d}{\gamma_n \cdot \gamma_k} = \frac{1,0 \cdot 5,9}{1,0 \cdot 1,4} = 4,2.$$

After cementation the reserve of pile-bearing load was 1,91t. (i.e., 13 %):

$$N = 14,3 < \frac{\gamma_0 \cdot F_d}{\gamma_n \cdot \gamma_k} = \frac{1,0 \cdot 22,7}{1,0 \cdot 1,4} = 16,21.$$

Thus, the bearing capacity of the pile during cementation increases due to increased horizontal stresses and the work of the flattened foundation soil on the side surface and under the lower end of the pile.

Финансирование. *Исследование не имело спонсорской поддержки.*
Конфликт интересов. *Авторы заявляют об отсутствии конфликта интересов.*
Вклад авторов. *Все авторы сделали равный вклад в подготовку публикации.*

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