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STRUCTURALLY UNSTABLE SOILS OF THE GANJA-GAZAKH FOOTHILLS AND ENGINEERING PROTECTION MEASURES AGAINST DEFORMATION PROCESSES OCCURRING IN THESE SOILS

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Abstract. In the article, the feature of settlement, subsidence, and other deformations in the ground foundations of various structures built in complex engineering-geological conditions with the joint participation of some soils with a stable structure (subsidence loess, silt, low-strength technogenic, soluble) are performed on the example of the Gazakh cement plant, the development of a conceptual solution for the methods of installation in foundations on such soils has been reflected. Analyzing the results of the complex engineer-geological research conducted at the research facility, priority was given to the principle of improving the working conditions of the soils that form the basis of the facilities in order to prevent the predicted large-scale deformation, the application of a foundation system consisting of piles with a large length, with the bottom placed in a layer of argillite-like very stiff clay, was accepted as the most correct technical solution, the dimensions of the pile are determined and installed in the study area and it was determined that the chosen solution was justified by monitoring the deformations.

Keywords: *Loess, loess-like soil, quicksand, subsidence, settlement, deformation, unstable structure, calculated value, pile.*

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1. Introduction

It can be said that the problems caused by soils with an unstable structure (subsidence loess and loess-like soils, quicksand, weak soils of organic origin, etc.) during engineering activities, it has been recorded in most parts of the world and despite the existence of numerous methods for solving those problems, it is likely that the need to conduct research in the direction of studying deformations and developing adequate measures against them will remain relevant for a long time.

Since the formation conditions of loesses belonging to soils with an unstable structure is different, the deformation processes occurring in them are also different and this diversity is formed under the influence of many factors this is also reflected in the results of studies conducted by some researchers (Balaev & Tsarev, 1964; Sergeeva *et al*., 1986; Olyansky, 2015; Trofimov, 2003).

The subsidence property of loess is mainly explained by the formation of special loess structures in these soils and other factors. In this regard, when designing industrial

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and civil construction facilities in the soil base of which there are subsiding loess soils, a thorough study of all characteristics of these soils, including their structures, should be accurately studied and, based on the results obtained, adequate protective measures should be developed to eliminate the negative effects that may arise from subsiding loess soils and its application before construction can ensure the reliability and longevity of the construction object to be built (Abelev, 1979; Kalachuk & Shin, 2016; Mokritskaya & Samoilich, 2017).

In the studies conducted by many researchers, the solution of issues such as the dependence of subsidence deformations in loess and loess-like soils on various factors, prediction of relative subsidence deformation, preparation of measures to prevent the subsidence process was reflected (Chernysh & Gubarev, 2018; Pantyushina, 2011; Frolov, 1988).

In the territory of the Azerbaijan Republic, loess and loess-like soils are widespread. Wetting these soils with water leads to the loss of stability of the soil base of various industrial and civil facilities and other construction objects built on them, the settlement of the foundations of the structures, and sometimes these soils slip out from under the foundation, and as a result, the complete or partial collapse of the construction objects. The mentioned negative situations also cause difficulties in the operation of the buildings built on those soils, as well as lead to additional repair and restoration works and detailed information about this has been given in studies conducted by some researchers (Alishzade, 1978; Aliyev, 2009; Heinrich & Schuster, 1996; Verdiev, 2019).

Loess and loesses-like soils with similar properties are widely distributed in the western region of the Azerbaijan Republic and are present as the soil base of infrastructure facilities of various purposes, including industrial facilities, hydrotechnical facilities, especially main canals, pumping stations, closed irrigation networks, and also civil construction facilities.

When these soils are wet, the subsidence deformations caused by the own weight of soils and the effect of additional loadings cause the operational conditions of construction objects to be difficult and in some cases lead to the termination of the operation of objects. In connection with the mentioned, the development of adequate engineering protection measures against the negative effects of subsidence deformations is of great scientific and practical importance, and the application of these measures will ensure reliable and safe operation of construction objects.

According to experts, 45% of the costs of industrial and civil construction projects built on loess and loess-like soils are spent on the development and implementation of a comprehensive plan of measures to prevent the deformation process caused by the subsidence of these soils (Pantyushina, 2011).

In connection with these factors, a comprehensive study of the subsidence properties of loess and loess-like soils and deformations arising from the joint participation of these soils with other structurally unstable soils in the geological environment, the determination of the negative impact of deformation processes on the bearing elements of construction objects, including hydro-technical structures and the development of appropriate engineering and protective measures to prevent these negative impacts are necessary.

2. Research object and research methodology

As a research object, subsidence loess and loess-like soils were selected which constitute the soil base of various facilities in the territory of Gazakh district, which is included in the Ganja-Gazakh foothill plain. Dangerous cases related to the subsidence deformation in the soil foundations of structures located in the research object occurred in the main water pipelines, pumping stations, in the 5-story residential building of the N military unit in the village of Aghkoynak, and at the Gazakh cement plant.

As a result of long-term subsidence deformations exceeding the permissible maximum indicators, some of those objects were demolished, and in other objects, repair and restoration works and foundation strengthening measures were carried out with the involvement of large amounts of funds.

The main purpose of the research is to study the features of deformation processes in the soil foundation of buildings and structures erected in areas with complex engineering and geological conditions, where structurally unstable soils of various lithological compositions are present, including subsidence loess and loess-like dust-clay soils, to determine the degree of influence of these processes on the integrity and safety of buildings and structures, provision of appropriate recommendations for the prevention of hazardous impacts on construction objects associated with deformation processes, providing recommendations to reduce the risk of safe operation of facilities, developing conceptual solutions for the construction of reliable foundations of projected and under construction facilities and strengthening the foundations of operated facilities in similar conditions.

In order to carry out the intended research works, generally accepted methods widely applied in engineering-geology research were used and relevant researches were carried out for settlement, subsidence and other deformations occurring in the geological environment, complicated by the presence of structurally unstable soils (loess subsidence soils, quicksand, weakly strong technogenic soils, soils containing soluble salts), which formed the engineering-geological conditions of the territory of the Gazakh cement plant and by the example of events, which are considered appropriate to prevent negative impacts from the above deformation processes.

In order to determine the normative and calculated values of the physical-mechanical properties of soils (moisture, density, porosity, plasticity indicators, granulometric composition, internal friction angle, specific adhesion force, deformation modulus, relative subsidence deformation, initial subsidence pressure, etc.), for mathematical-statistical analysis of laboratory test results, classification of soils, determination of soil conditions for subsidence and calculation of deformation processes of various nature, the following state standards were adopted as the basis:

1. AzDTN 2.15-2 Pile foundations. Design standards;
2. AzDTN 2.1-1 Loads and actions;
3. AzDTN 2.15-1 Soil foundations of buildings and structures.
4. GOST 20522-2012 Soils. Methods of statistical processing of test results;
5. GOST 25100-2011 Soils. Classification;
6. SNiP 2.02.01-83 Foundations of buildings and structures.
7. Guidelines for the design of foundations for buildings and structures (to SNiP 2.02.01-83);
8. SP 50-101-2004 Design and installation of bases and foundations of buildings and structures;

9. SNiP 2.02.03-85 (1995) Pile foundations;
10. MSP 5.01-101-2002 Design and installation of pile foundations;

The relevant mathematical-statistical analysis and calculations were carried out in accordance with the requirements of GOST 20522 and SNiP 2.02.01-83 standards to determine the calculated values for the first limit state of the physical-mechanical properties of the engineering-geological elements involved in the geological structure of the research area (reliability probability α=0.95 for load-carrying capacity) and the second limit state (reliability probability on deformation α=0.85).

The normative value X_n of all physical and mechanical properties of soils was taken equal to the arithmetic average $\overline{X}$ and calculated using the following formula:

$$X_n = \bar{X} = \frac{1}{n}\sum_{i=1}^{n} X_i,$$

here n – the number of characteristic definitions;

X_i – private values of the characteristics obtained from the results of individual i -th experiments;

In order to eliminate possible errors, a mathematical-statistical check of the experimental results was carried out and individual X_i quantities that did not meet the mentioned condition were excluded:

$$|X_n - X_i| > \nu \cdot S,$$

here X_n- the normative values of physical and mechanical properties of soils;

X_i – private values of the characteristics obtained from the results of individual i -th experiments;

ν – the statistical criterion accepted depending on the number of test results according to the table in the appendix of the relevant standard;

S – the mean square deviation of the characteristic, calculated by the formula:

$$S = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(X_n - X_i)^2}.$$

The calculation of the coefficient of variation of the characteristics, the accuracy index of its average value and the reliability coefficient for the soil is performed according to the formulas:

$$V = \frac{S}{X_n}\ ;$$

$$\rho_\alpha = \frac{t_\alpha \cdot V}{\sqrt{n}};$$

$$\gamma_{\mathrm{g}} = \frac{1}{1 \pm \rho_\alpha}.$$

here V – coefficient of variation of soil characteristics;

S – the mean square deviation of the characteristic;

X_n – the normative value of physical and mechanical properties of soils;

ρ_α – the indicator of accuracy (error) of the average value of the coefficient of variation;

t_α – the coefficient accepted according to the table in the applications of the corresponding standard, from the given one-sided confidence probability α and the number of degrees of freedom K = n - 1;

n – is the number of characteristic definitions;

γ_g – soil reliability factor;

The calculated value of the soil characteristics was calculated by the formula:

$$X = \frac{X_n}{\gamma_g}.$$

here X – calculated value of soil characteristics;

X_n – the normative value of physical and mechanical properties of soils;

γ_g – soil reliability coefficient;

Calculated values of the coefficient of internal friction and the specific cohesive force of soils were calculated using the following formulas in accordance with the relevant state standards:

$$tg\varphi_j = \frac{k\sum_{i=1}^{k}\tau_i \cdot \sigma_i - \sum_{i=1}^{k}\tau_i \cdot \sum_{i=1}^{k}\sigma_i}{k\sum_{i=1}^{k}(\sigma_i)^2 - \left(\sum_{i=1}^{k}\sigma_i\right)^2},$$

$$C_j = \frac{\sum_{i=1}^{k}\tau_i \cdot \sum_{i=1}^{k}(\sigma_i)^2 - \sum_{i=1}^{k}\tau_i \cdot \sigma_i \cdot \sum_{i=1}^{k}\sigma_i}{k\sum_{i=1}^{k}(\sigma_i)^2 - \left(\sum_{i=1}^{k}\sigma_i\right)^2}.$$

here $\mathrm{tg}\varphi_j$ – coefficient of internal friction of the j-th engineering-geological element (EGE);

k – the number of determinations τ at each point of the EGE;

τ_i – soil shear resistance at normal stress σi for the *i*-th sample in the corresponding EGE, MPa;

C_j – specific cohesive force of the j-th engineering-geological element (EGE), MPa.

To conduct operatively a mathematical and statistical analysis of laboratory results and simplify calculations, a special program was prepared in Excel using the indicated formulas and sequence, and this program was used in all calculations.

3. Research results, an analysis, and a discussion

Research was carried out in the construction area of the Gazakh Cement Plant.

Construction works on the site of the Gazakh cement plant were started in 2007. At the initial stage, the relief of the construction site with a slope of 1-3 degrees was leveled. For this, mainly clay soils were cut from the mountain slope in the upper part of the field and 0.5 m thick poured soil layers were gradually created in stages at the construction site, compacted with rollers weighing 15.0 tons and depending on the slope of the terrain, the terrain of the construction site was leveled by creating a layer of poured soil with a thickness of 0.0-7.0 m.

In order to clarify the engineering-geological conditions of the research area and to develop an engineering-geological and geotechnical model of the geological environment, complex field engineering-geological surveys were carried out in 2011-2013, and in the geotechnical laboratories of «Geoengineering» LLC and "AzHveM" EIB laboratory tests of 106 soil samples with undisturbed structure and determined the physical and mechanical properties of soils.

The geological structure of the object of study was studied on the basis of an analysis of the available fund and archival materials, as well as a result of analytical and statistical processing of the indicators of the relevant field and cameral studies.

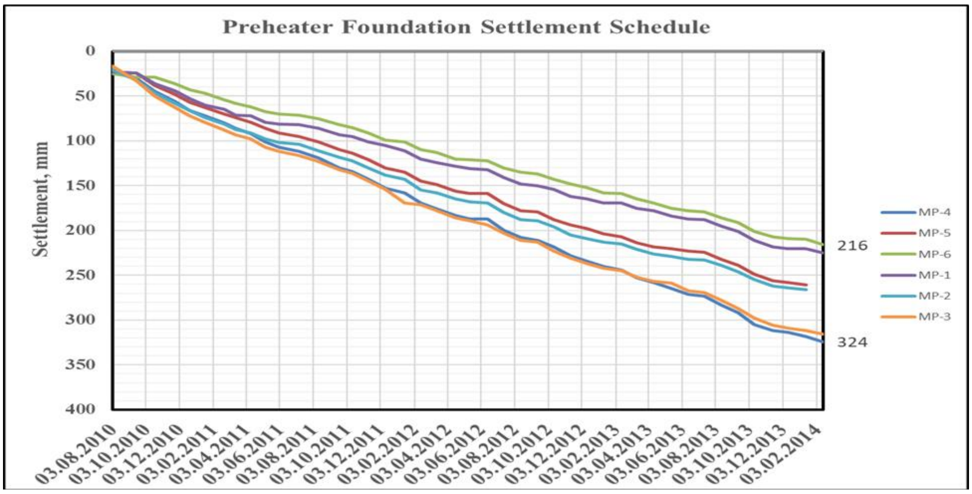

Figure 1. A deformation of the foundation of the Preheater built in the research area

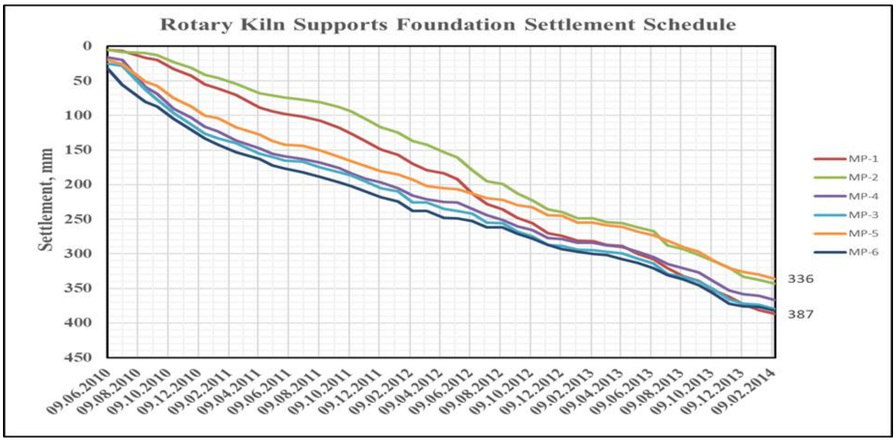

Figure 2. A deformation of the foundation of the Rotary Kiln Support built in the research area

Administratively, the research area is located approximately 2 km north-west of Dash-Salahli village, Gazakh district, Republic of Azerbaijan, it is bordered by the foothills of the Lesser Caucasus mountains on the west side. From the west, the site borders the foothills of the Lesser Caucasus, in the southwestern part of which the developed Dash-Salakhly limestone quarries are located. On the other sides, the construction site has no natural boundaries and is a single flat area (Figures 3 and 4).

The relief of the research area is flat, and the surface of the plain is slightly hilly, with small negative forms (suffusion funnels), slightly dissected by shallow gullies and ravines with gentle sides. In the lower and eastern parts of the site, there are small moist areas.

The research area is characterized as waterless, there are no natural surface watercourses in the area, but dry valleys are noted in the northern part of the site, which is filled with water flows during heavy rains. On slope of Mount Avey (889.7 m) adjacent to the west, at its foot, there are outlets of a group of springs.

In tectonic terms, an entire study area can be attributed to a large tectonic block, namely, to the Kura megasynclinorium. Geomorphologically, a study area covers a part of the Ganja-Gazakh plain (north-western part), directly adjacent to the foot of the slopes in the western part of the study area.

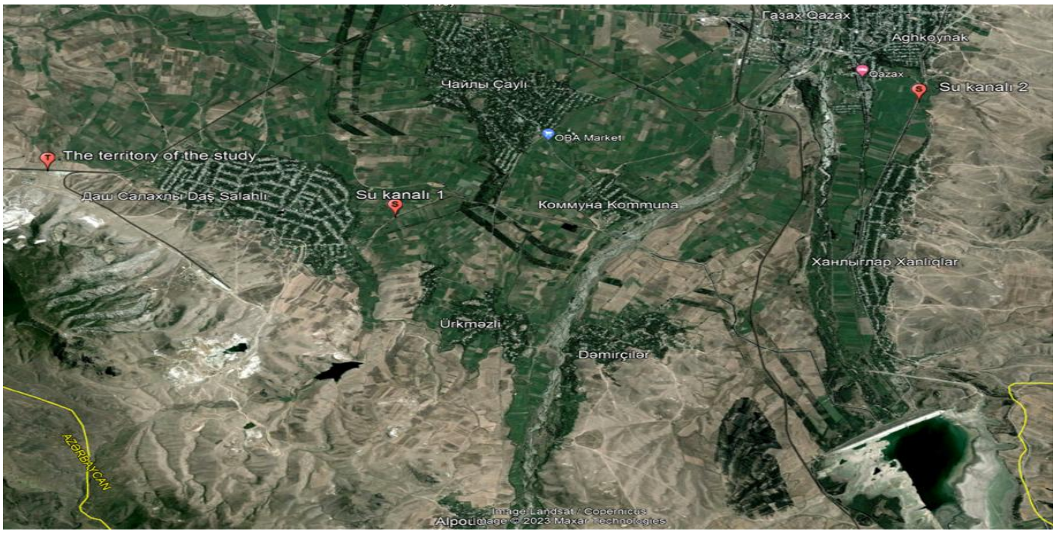

Figure 3. A satellite image of the research area

Figure 4. A satellite image of the study area and research site

The Ganja-Gazakh foothill plain has a very complex genetic form of relief. Cones of the tributaries of the rivers flowing from the Lesser Caucasus mountains played a key role in the formation of its geomorphological and orographic conditions. The sediments covering the surface of the plain almost in most areas consist of sediments of continental origin of these cones.

The research area, the absolute height of which ranges from 375.0-415.0 m, covers the southern side of the Kura depression with an inclination in the northern and north-eastern direction and also borders the slopes of the Lesser Caucasus Mountains on which the peaks of Avey (889.7 m) and Rizachal (605.8 m) are located. The geological structure of these mountains involves the strata of the Upper Cretaceous (K2), Maastricht (K2m), Campana (K2cp), and Upper Santon (K2st2).

In the geological structure of the study area involves deposits of the Quaternary system, brought from the slopes of the mountains as a result of erosion-denudation processes. As a result of accumulation of sedimentation products in the Kura basin, a thick layer of soft-friable materials of great thickness was formed.

Continental modern deposits of the quaternary system were found in the investigated geological structure in the depth interval of 0.0-81.6 m of the research area, they are composed of anthropogenic-technogenic (anQıv) and alluvial-deluvial-proluvial (adpQıv) deposits of genetic origin belonging to one geological formation.

According to the lithological composition, these deposits consist of the embankment, of a boulder-pebble-gravel layer of different granulometric composition (adQIV) with dusty-clay aggregates, of a layer of clay-buried soil (dpQIV), formed with the participation of plant remains and of weakly cohesive (clay and loams of different consistency), dusty-clay dispersed soils (dpQIV and adpQIV) characterized by variable facies.

The embankment soils of technogenic origin have a thickness of 0.0-6.7 m and are distributed throughout the study area.

In the engineering-geological section of the soil mass formed in the lower part of the technogenic soil layer of the geological environment, mainly dusty-clay soils (clays, quicksand, and sandy loam) of various genesis (Quaternary deluvial-proluvial and alluvial-deluvial-proluvial deposits) and coarse clastic soils (fragments of rocks, pebbles, gravel, gruss) and the lower part of these layers is limited by layers of argillite-like very stiff clay soil, formed in the depth interval of 58.1-81.6 m.

Deluvial-proluvial deposits of the quaternary period with clay, loam and sandy loams lithological composition have certain characteristics, including loess character and anisotropy.

The loess clays and loams involved in the lithological composition are not compacted naturally or poorly compacted, they are characterized by being macroporous and correspondingly high compressibility. During engineering-geological studies, it was determined that the dusty-clay soils of the study area have a high level of water absorption character.

In certain horizons of geological environment, sandy loams of plastic and solid consistency were formed. Plastic sandy loams were formed in depth intervals of 11.5-12.1 m and 49.0-54.0 m, and solid sandy loams - in the depth intervals of 12.4-16.9 m and 53.9-61.6 m.

Dispersed-cohesive (dusty-clay) and in some cases, weakly coherent coarse-grained soil layers with sandy filler were formed in the form of layers of different thicknesses in different horizons throughout the entire depth interval of the geological environment.

Layers of coarse-grained rocks that formed in the lower layers of the horizon (above the layer of argillite-like very stiff clays) are strongly cemented by clay particles and have the character of a conglomerate.

Argillite-like very stiff clays in the geological structure of the study area, were formed in the depth range of 58.1-81.6 m and have high density, strength and deformation.

Since the physical and mechanical properties of the buried layer involved in the geological structure of the studied soil massif are identical to the properties of hard clay and loam, this layer was not distinguished as a separate engineering-geological element.

The groundwater level in the study area is >28.0 m and was formed in Quaternary sediments. Underground water found in all areas of the study area have a hydraulic connection with each other and are characterized by aggressive properties in relation to concrete and reinforced concrete.

It should be especially noted that in the course of complex engineering and geological surveys carried out in 2011-2013, a decrease in the static level of groundwater in the study area by about 2.1-2.8 meters was recorded. It is assumed that the drop in water level is seasonal and presumably related to the feeding and discharge of gravitational waters.

At the same time, a change in the level of groundwater as a result of the transfer of carbonates, as well as easily soluble salts in the soil towards the drainage zone, led to the development of unfavorable physical and geological processes, including a deterioration in the physical and mechanical properties of the soil and, as a result, to the formation of additional deformation processes or settlement of the foundation after the dissolution of the above soluble substances.

In accordance with the requirements of regulatory documents, relevant standards and guidelines, in order to provide engineering and geological data, including the normative and design parameters of soils, work on the development of engineering and protective measures to prevent deformation processes and settlement of the foundations of actually constructed technological structures (Preheater, Rotary Kiln Support), a mathematical and statistical analysis of the results of laboratory studies was carried out and the main nomenclature indicators of the physical and mechanical properties of soils were determined (Table 1 and Table 2).

Table 1. Estimated values of the physical and mechanical properties of soils

Soil name	Soil density gr/sm³			Specific cohesion, kPa			Internal friction angle, degree			Modulus of deformation, MPa			
										In natural state		In a water-saturated state	
	X_n	α=0,85	α=0,95	X_n	α=0,85	α=0,95	X_n	α=0,85	α=0,95	X_n	α=0,85	X_n	α=0,85
	P	P_{II}	P_I	C	C_{II}	C_I	φ	$φ_{II}$	$φ_I$	E	E_{II}	Esat	$Esat_{II}$
Loam, very stiff, loess, subsidence, highly compressible	1,89	1,87	1,85	40	31	24	25°33′	23˚24'	21˚44'	17,1	13,8	9,5	10,1
Loam, stiff, loess, subsidence, highly compressible	1,99	1,97	1,96	84	50	42	28°08′	25˚34'	23˚33'	18,5	15,5	13,4	-
Loam, firm - stiff, highly compressible	1,98	1,97	1,96	36	36	24	29°15′	29˚15'	25˚06'	13,7	-	12,4	-
Loam, soft - firm, highly compressible	1,96	1,97	1,96	45	13	5	18°07′	18˚07'	15˚40'	12,8	-	11,5	-
Loam, very soft - firm, highly compressible	1,87	1,84	1,82	46	35	25	22°54′	20˚08'	17˚52'	6,1	-	5,6	4,02
Loam, very soft - soft	1,83	-	-	3	-	-	05°30′	-	-	3,3	-	-	-
Clay, very stiff	2,01	1,97	1,95	75	58	43	17°36′	15˚08'	13˚04'	28,8	-	18,2	-
Clay, stiff, subsidence, medium compressible	1,97	1,95	1,94	48	37	32	19°21′	18˚18'	17˚34'	20,4	16,3	16,6	13,5
Clay, firm - stiff, subsidence, highly compressible	1,96	1,95	1,94	18	16	15	21°18′	18˚18'	16˚26'	10,2	9,2	8,4	5,4
Sandy loam, very stiff	1,74	1,74	1,74	20	20	13	23°33′	23˚33'	20˚22'	12,3	-	11,3	-
Sandy loam, very soft - stiff	1,87	1,87	1,87	21	20	13	26°70′	23˚33'	20˚22'	7,4	-	-	-
Technogenic soil, embankment	1,88	-	-	50	-	-	25°52′	-	-	16,4	-	10,5	-
Pebbles (coarse, fine gravel) with clay	1,89	-	-	40	-	-	25°33′	-	-	17,1	-	9,5	-
Clay, very stiff, mudstone-like	2,00	2,00	1,99	141	126	116	20°51′	19˚12'	18˚11'	59,7	-	23,05	-

Table 2. The main nomenclature indicators of the physical and mechanical properties of soils

Soil name	Water content, %	Soil density gr/sm³		Void ratio	Degree of saturation	Liquidity index	Modulus of deformation, MPa		Pressuremeter deformation modulus, MPa		Internal friction angle, Degree	Specific cohesion, kPa	Relative subsidence	Initial subsidence moisture, %	Initial subsidence pressure, kgf/sm2
		Natural density	Dry soil density				In natural state	In a water-saturated state	In natural state	In a water-saturated state					
	W	ρ	ρ_d	e	Sr	I_L	E	Esat	Ep	Epsat	φ	C	ε_{sl}	W_{sl}	P_{sl}
Loam, very stiff, loess, subsidence, highly compressible	17,0	1,89	1,61	0,690	0,69	-0,32	17,1	9,5	18,7	10,13	25	40	0,012	37	4,9
Loam, stiff, loess, subsidence, highly compressible	23,0	1,99	1,63	0,676	0,91	0,15	18,5	13,4	20,4	11,6	28	84	0,017	25	2,1
Loam, firm - stiff, highly compressible	24,0	1,98	1,59	0,704	0,92	0,42	13,7	12,4	7,9		29	36			
Loam, soft - firm, highly compressible	24,0	1,96	1,58	0,721	0,91	0,61	12,8	11,5	17,4		18	45			
Loam, very soft - firm, highly compressible	32,0	1,87	1,42	0,920	0,95	0,91	6,1	5,61			23	46			
Loam, very soft - soft	44,2	1,83	1,27	1,13	1,06	1,84									
Clay, very stiff	18,0	2,01	1,70	0,612	0,81	-0,22	28,8	18,2	28,2	22,7	17,36	75			
Clay, stiff, subsidence, medium compressible	22,9	1,97	1,60	0,716	0,88	0,14	20,4	16,6	20,3	15,1	19,21	48	0,012		2,45
Clay, firm - stiff, subsidence, highly compressible	25,1	1,96	1,57	0,742	0,92	0,32	9,2	5,4	16,5		21,18	18	0,012		2,75
Sandy loam, very stiff	16,9	1,74	1,49	0,808	0,57	-0,58					26,7	21			
Sandy loam, very soft - stiff	26,4	1,87	1,48	0,836	0,85	0,17	7,4				27	21			
Technogenic soil, embankment	20,0	1,88	1,56	0,744	0,74	-0,05	16,4	10,7	15,6		25,52	50			
Pebbles (coarse, fine gravel) with clay	22,0	1,92	1,58	0,723	0,81	-0,04	17,1				25	40			
Clay, very stiff, mudstone-like	27,0	1,96	1,58	0,795	0,94	-0,14	59,7				21	141			

On the basis of the statistical, analytical and graphic interpretation of the geological data determined during the complex engineering-geological research conducted in the study area in 2011-2013, it was determined that the engineering-geological conditions of the part of the research area from the ground surface of the soil massif to the ceiling of the argillite-like very stiff clay layer involved in the geological structure, have category III (very complex) complexity and for the construction of industrial, civil construction, as well as infrastructure buildings and structures, does not have favorable engineering-geological conditions.

Dusty-clay soils involved in the engineering-geological structure of the geological environment and distributed at a depth from the earth's surface to the ceiling of the argillite-like very stiff clay layer have mainly unstable strength (subsidence, swelling, organomineral, organic, technogenic, etc.) and are characterized by heterogeneity and anisotropy.

Weak soils present in some horizons of the geological structure in the depth range of 7.5-61.6 m, especially quicksand and dusty-clay soils with a high fluidity index (soft-firm, very soft-firm), significantly complicate the geotechnical conditions of the territory and can lead to additional deformation processes.

Based on the data obtained from the DR measuring system, which makes it possible to determine the depths of propagation of deformation processes in the depth range of 0.0 > 50.0 m in the soil mass of the study area and to observe the dynamics of ongoing deformation processes at this depth, it can be stated that deformation processes cover the entire the thickness of the soil mass, from the surface of the relief to the roof of a layer of argillite-like very stiff clays (Figure 5).

The results of the interpretation of the indicators of the change in the magnitude of deformation with depth in time show that, with increasing depth, the deformation in time increased and these phenomena are observed mainly at depths of 30.5 m and >50.0 m of the geological environment.

Thus, highly compressible very soft-firm clays (E = 5.4 MPa) formed in different horizons of the geological environment of the study area (especially in the depth interval of 15.0-23.0 m, 27.0-33.0 m and 53.0 -57.0 m) are one of the main factors determining the deformation processes occurring in the soil massif.

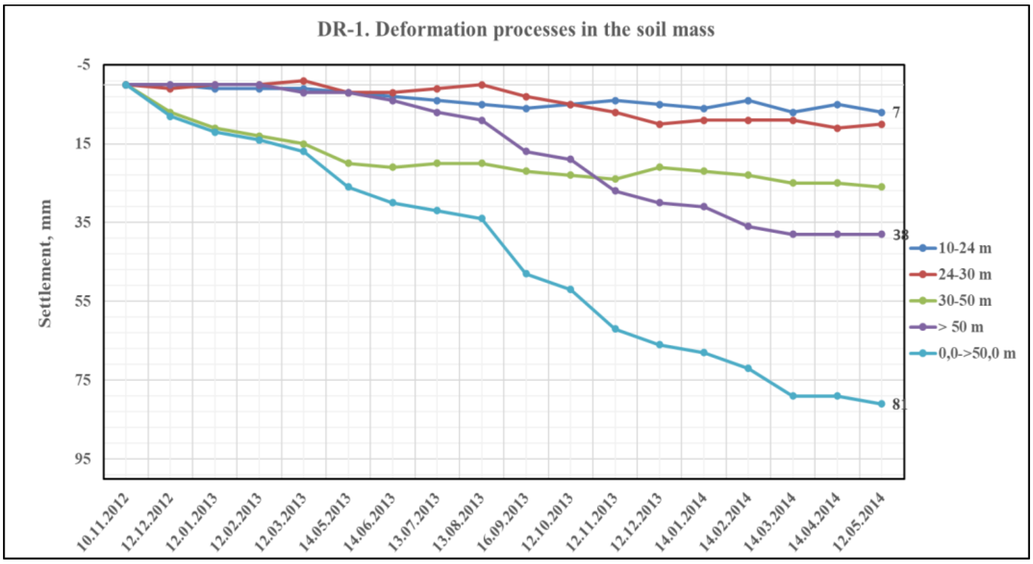

Figure 5. Variation of the deformations occurring in depth in the soil massif of the research area over time

Taking into account the peculiarities of the engineering and geological conditions of the study area, the design of buildings, the impact of technogenic soils, changes in the level of groundwater and an increase in soil moisture, as well as the actual values of settlement, the total settlement of the study area St was calculated using the following formula:

$$S_t = S_ç + S_{sl} + S_w + S_{cr} - S_1 - S_m,$$

here: S_t- the total settlement of the study area, cm;

$S_ç$- soil settlement, cm;

S_{sl}- soil subsidence deformation, cm;

S_w- additional soil settlement due to a decrease in the level of groundwater, cm;

S_{cr}- soil creep settlement, cm;

S_1- subsidence of the soil in its natural state due to the weight of the poured soil;

S_m- actual subsidence occurring in the soil massif during the construction, cm;

Settlements occurring in the soil foundation of the structures built in the study area were calculated using the following formula based on the layer summation method according to the half-space calculation scheme undergoing linear deformation:

$$S = \beta \cdot \sum_{i=1}^{n} \frac{\sigma_{zp,i} \cdot h_i}{E_i}$$

here β - dimensionless coefficient equal to 0.8;

$\sigma_{zp,i}$- the average value of the additional vertical normal density in the i-th layer of soil, equal to half the sum of the indicated stresses on the upper z_{i-1} and lower z_i boundaries of the layer along the vertical passing through the center of the foundation;

h_i və E_i- thickness and deformation modulus of i-th soil layer, respectively;

n - the number of layers into which the compressible thickness of the base is divided;

As a result of the calculation, the following foundation settlement values were determined: settlement in the ground base of the Preheater - S = 23,9 cm, ($S_ç$ + Ssl) = 23.9 + 21.9 = 45.8 cm, S_m=22.2 cm, S_1= 15.9 cm; settlement in the ground base of the Rotary Kiln Support - S = 23.7 cm, ($S_ç$+ S_{sl}) = 23.7 + 21.9 = 45.6 cm, S_m=29.1 cm, S_1= 19.6 cm; the settlement caused by the lowering of the water table - S_w =4.1 cm, soil creep settlement - S_{cr}=4.4 cm.

On May 14, 2013, the measured actual settlements of the foundations of the "Preheater" and the first support of the Rotary Kiln Supports were 175-265 mm, with an average value of 222 mm, the second and third Rotary Kiln Supports 262-313 mm, with an average value of 291 mm.

According to the generalized engineering and geological data, the thickness of loess and loess-like subsidence clay soils in the soil base of foundations in the study area is 26.2 meters, and the soil conditions for subsidence belong to type II. In accordance with this, it was necessary to carry out appropriate engineering protection measures against a subsidence, precipitation, and other deformations of the relevant structures, as well as the choice of an appropriate engineering solution.

There are many methods that can theoretically and practically be applied to settlement deformations in loess and loess-like soils, and these methods, taking into account specific geotechnical conditions, are implemented in the direction of increasing the physical and mechanical properties of the soil and improving the working conditions of the soil.

However, it was considered important to determine which of these methods is more applicable in real conditions, that is, it was necessary to choose the optimal scientific and technical solution in relation to subsidence deformations under the foundations of structures in the study area.

Interpretation of the results of all studies carried out in the area under study, including complex engineering and geological studies and geotechnical monitoring of deformation processes, experience in performing construction work in areas with difficult engineering and geological conditions, requirements of regulatory documents and recommendations for the application of engineering and protective measures in subsidence dusty clayey and loess soils and the generalization of literature data give grounds to state that in order to prevent or minimize deformation processes and settlement of foundations of buildings and structures erected in the research area, the most correct technical solution is the option of constructing foundations from deep pile systems with the condition of placing the base of the piles in the layer argillite-like very stiff clays.

Calculation of piles in the soil foundation of the structures. The application of large-sized pile-type foundations is determined by the following factors: the large value of the expected settlements and the large thickness of the compacted soil massif, the time factor, being very heavy of the technological equipment, strict demands for the permissible limit value of the technological settlements.

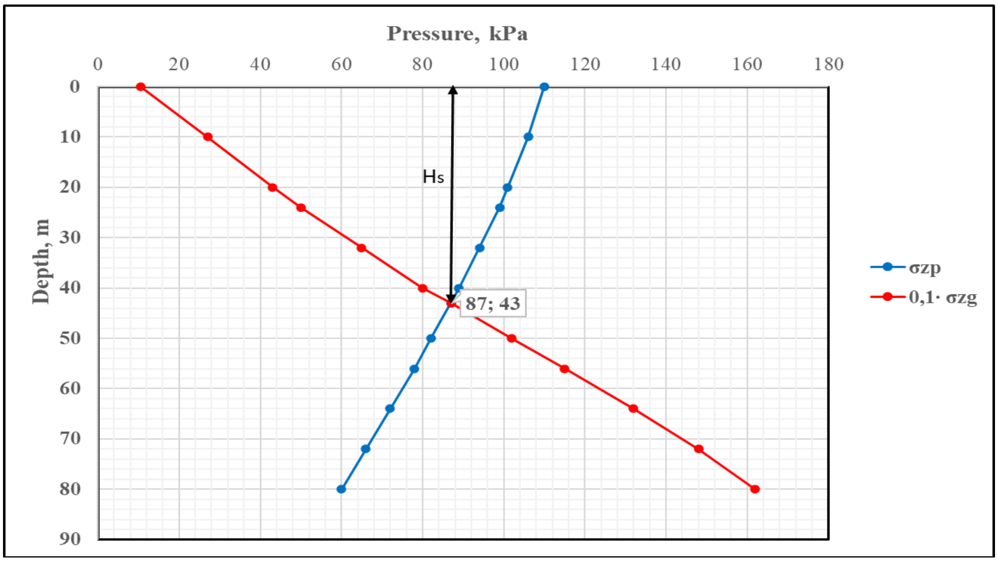

Figure 6. Determination of the depth of active compressible soil thickness ($\sigma_{zp}=0.1\cdot\sigma_{zq}$)

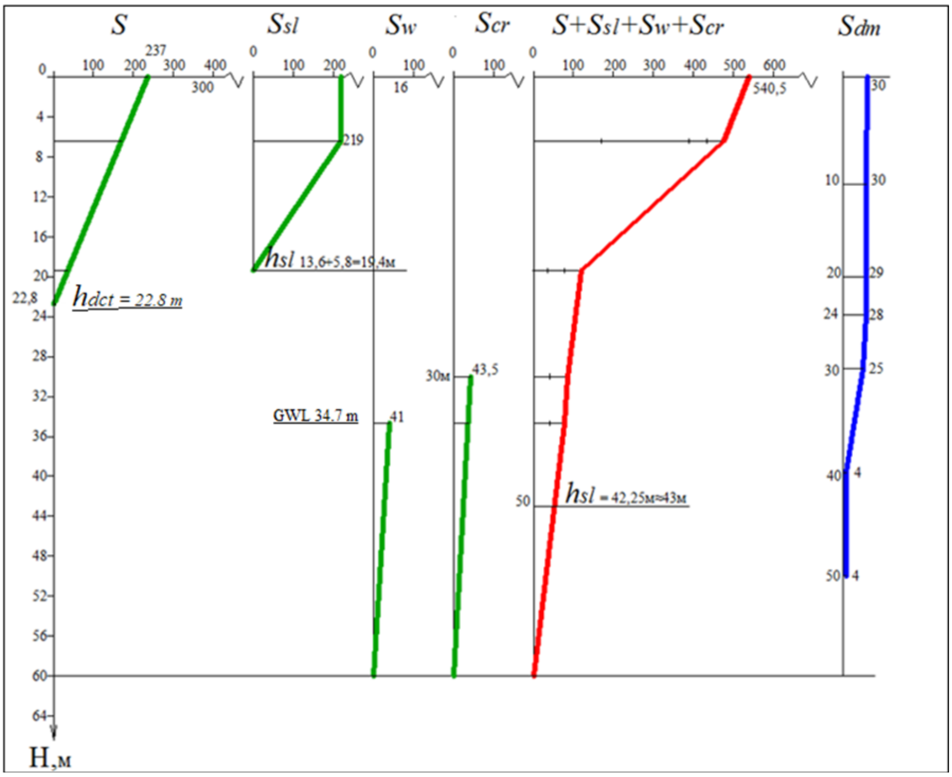

Figure 7. The scheme for determining the calculated depth hsl, to which the summation of lateral friction forces of the subsiding layers is performed: S – ground settlement; S_{sl} – ground subsidence; S_W – additional subsidence of the soil as a result of changes in the underground water level; S_{cr} – creep settlement of soils in the depth interval of 30-60 m; S_{dm} – the actual settlement of the depth benchmarks (marks);

Despite the fact that coarse-clastic soils with clay aggregates present in the engineering-geological structure of the geological environment are classified as non-compressive or weakly compressive soils, their low thickness did not allow them to be used as a reliable bearing layer.

According to the calculations performed, taking into account the accepted maximum permissible precipitation value (10 cm), the depth to which the resistance along the side surface of the piles is taken with a negative sign (negative friction) is 43 m.

It should be noted that in the relevant normative documents (AzDTN 2.15-2 Pile foundations. Design norms, SNiP 2.02.03-85 (1995) Pile foundations) provide for the calculation and design of piles with a length of 35.0-40.0 m.

When applying piles with a larger length, reference is made to the experience of calculating and designing piles in similar situations existing in world practice. In the calculation and design of piles applied in the research area, reference was made to similar conditions in the facilities built in the cities of St. Petersburg and Sochi of the Russian Federation.

On the basis of the performed calculations of settlement in the soil foundation of structures, a summary graph of the change in settlement and subsidence by depth was constructed, which shows the determination of the depth hsl = 43 m up to which the summation of the negative friction forces on the lateral pile surfaces (Figure 7).

To determine the negative friction forces along the lateral surface of the pile, the formula according to paragraph 9.10 of the AzDTN 2.15-2 Pile foundations, Design standards (or paragraph 8.11 of SNiP 2.02.03-85 Pile foundations) was used:

$$P_n = u \cdot \sum_{0}^{h_{sl}} \tau_i \cdot h_i.$$

here u – perimeter of the pile body part, m;

h_{sl} – design depth, m;

h_i – thickness, m, of the i-th layer of subsiding soil, settling during soaking and in contact with the side surface of the pile;

τ_i – design resistance, MPa (tf/m2), determined to a depth h = 6 m by the formula:

$$\tau_i = c_I + \zeta \cdot \sigma_{zg} \cdot tg\varphi_I$$

here τ_i– design resistance of the i-th layer of the soil, MPa;

ζ - lateral pressure coefficient, 0.7 is accepted;

φ_I and c_I – are the calculated values of the internal friction angle and specific adhesion force;

σ_{zg}- vertical stress from the own weight of water-saturated soil, MPa;

When taking into account the statistical analysis of the results of laboratory tests and the quantities determined as a result of calculations:

$$\tau_i = 3.4 + 0.7 \cdot 6.0 \cdot 1.98 \cdot 0.339 = 6.22\ \frac{tq}{m^2}.$$

When taking into account the calculated soil resistance τ_i and other quantities determined as a result of calculations:

$$P_n = 3.14 \cdot 1.5 \cdot (6.0 \cdot 0.5 \cdot 6.22 + 37.0 \cdot 6.22) = 1171.85 \text{ t.}$$

To determine the design load on the pile in accordance with the requirements of paragraph 9.9 of AzDTN 2.15-2 (SNiP 2.02.03-85 paragraph 8.10), the obtained value of the negative friction force should be multiplied by the coefficient of working conditions ɣc, equal in our case ɣc = 0.8. In this case:

$$P_n = 1171.85 \cdot 0.8 = 936.8 \text{ t.}$$

At the pile length section of 43-59.5 m, the design resistance on the side surface of the pile (*fi*), due to the lack of direct instructions in AzDTN 2.15-2, is determined for the maximum depth (35 m) given in Table 7.3 AzDTN 2.15-2, taking into account weighted average fluidity index in depth (43-59.5 m) and is taken equal to fi = 10 t/m^2.

To determine the design soil resistance in the depth range of 59.5-80.0 m to the side surface of the pile, the calculated resistance value was used, determined by the method of undrained cut of argillite-like very stiff clay in laboratory conditions (Su = 39.2 t/m2) and the calculated value of lateral friction on the pile surface at a depth of 59.5-80 m, used in world geotechnical practice (Kulhawy & Phoon, 1993):

$$\tau_i = fi = \alpha \cdot S_u, = 0{,}65 \cdot 39{,}2 = 25{,}5 \text{ tf/m}^2$$

here α is the adhesion factor, and α= 0.5 was taken for this case;

The resistance under the lower end of the pile R at a depth of 80 m was determined on the basis of the theory of undrained strength used in the world geotechnical practice (Rowe, 2012; Poulos, 1990):

$$f_b = R = N_c \cdot S_u + \sigma_{vb} = 9.0 \cdot 39.2 + 160.0 = 512.8\ tq/m^2.$$

here N_c- is the bearing capacity factor and for this case, $N_c = 9.0$ is accepted;

σ_{vb} – soil pressure from its own weight at the level of the pile heel, σ_{vb}= h · γ= 80,0 · 2,0=160,0 tf/m^2 (γ - soil density, t/m^3).

Thus, the bearing capacity of the pile below the depth hsl = 43.0 m, taking into account the known quantities (A=1.77 m^2, u= 4.71 m, γ_{cf} = 0.7, γ_c = 1.0, γ_{cR} =0.9) by taking account, it was calculated as follows:

$$F_d = \gamma_c \cdot \left(\gamma_{cR} \cdot R \cdot A + u \cdot \sum \gamma_c \cdot f_i \cdot h_i\right) = 1.0 \cdot [0.9 \cdot 512.8 \cdot 1.77 + 4.71 \cdot (0.7 \cdot 11.0 \cdot 16.5 + 0.7 \cdot 25.5 \cdot 20.5)] = 3139.0 \text{ tq}$$

The calculated load per pile is 952.0 taking into account the results of all calculations, the requirements of clause 9.9 of AzDTN 2.15-2 (8.10 of SNiP 2.02.03-85), and the weight of the pile (Ns=353.0 t):

$$N = \frac{F_d}{\gamma_k} - P_n - N_s = \frac{3139{,}0}{1.4} - 936.8 - 353.0 = 952.0 \text{ tf;}$$

here γ_k=*1.4* is the reliability coefficient and is taken from the relevant table;

P_n = 936.8 tq- is the negative friction force acting on the side surface of the pile above h_{sl} = 43.0 m;

Calculation of the settlement of a pile 80.0 m long and 1.5 m in diameter. Foundation 12.4 programme was used to calculate the pile settlement and was carried out according to the following calculation scheme (Figure 8).

Initial information:

The indicators of soil properties of the selected layers in accordance with the design scheme of pile settlement are given in Table 3.

Table 3. Indicators of soil properties of the selected layers

Layer numbers	Soil type	Liquidity index	Layer thickness, m	Modulus of deformation	Unit
Layer 1, 2, 3∗	Dusty clayey	$I_L = 0{,}7$	60	1200	tf/m^2
Layer 4	Clayey	$I_L \leq 0$	20	5970	tf/m^2

∗ - Due to the fact that silty-clayey soils, which make up a geological structure of the research area in the depth range of 0.0-60.0 m, consist of many alternating layers (very stiff clays, stiff clays, firm-stiff clays, firm clays, very stiff loams, stiff loam, very stiff sandy loam, very soft - stiff sandy loam), the calculated value was used in the calculations, determined as a result of generalizing the average values of the strength and deformation indicators of soils.

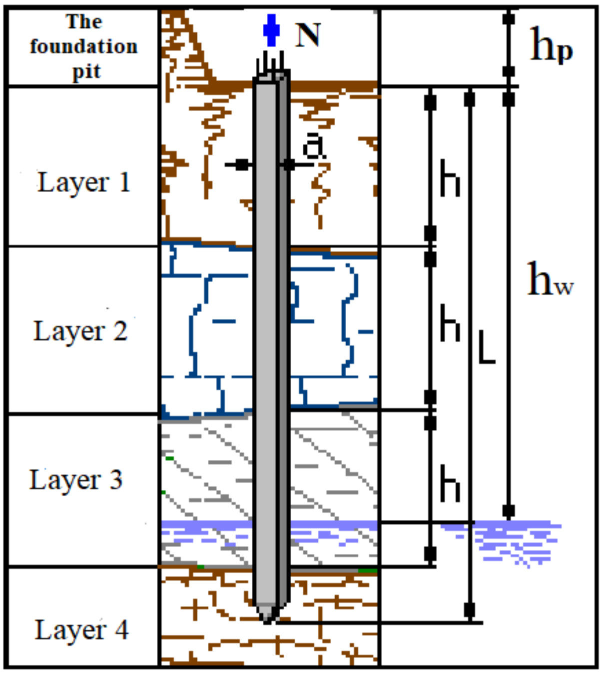

Figure 8. Calculation scheme of pile settlement

An initial data for calculation:
Type of pile bored;
Pile length 80.0 m;
Pile diameter 1.5 m;
Pit depth 0.0 m;

Loads on the pile;
Preheater N = 620 + 353 + 936 = 1909 tf;
Rotary Kiln Supports N= 250 + 353 + 936 = 1539 tf;

Bearing capacity of the pile;
Preheater Fd=3139 tf
Rotary Kiln Supports Fd=3139 tf

Estimated pile settlement: Design settlement of the pile:
Preheater S =35 mm;
Rotary Kiln Supports S =28 mm.

After strengthening the foundations of the Preheater and Rotary Kiln Supports structures in the second half of 2013 and the first months of 2014 with the use of foundations from deep pile systems, deformation processes, including foundation settlements, completely stopped.

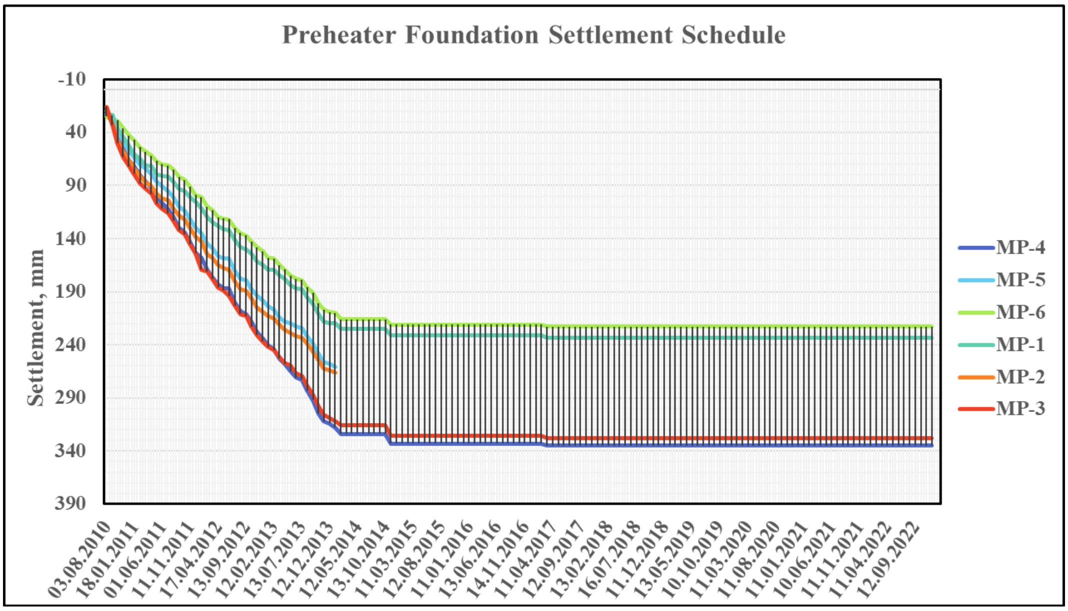

Figure 9. A deformation of the Preheater foundation after foundation reinforcement with deep piles 80 m long and 1.5 m in diameter in 2013-2014

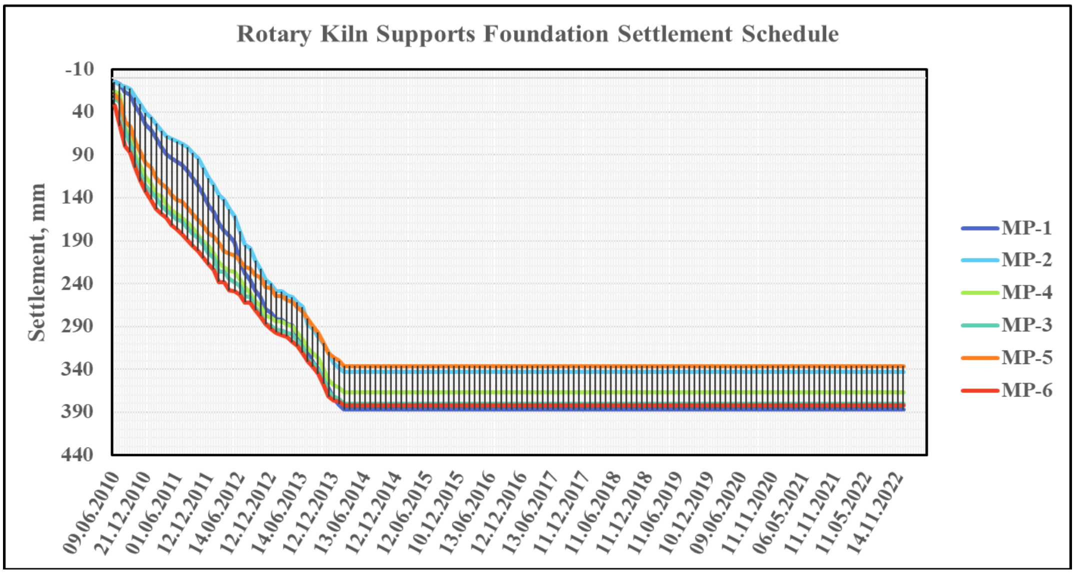

Figure 10. A deformation of the Rotary Kiln Supports foundation after foundation reinforcement with deep piles 80 m long and 1.5 m in diameter in 2013-2014

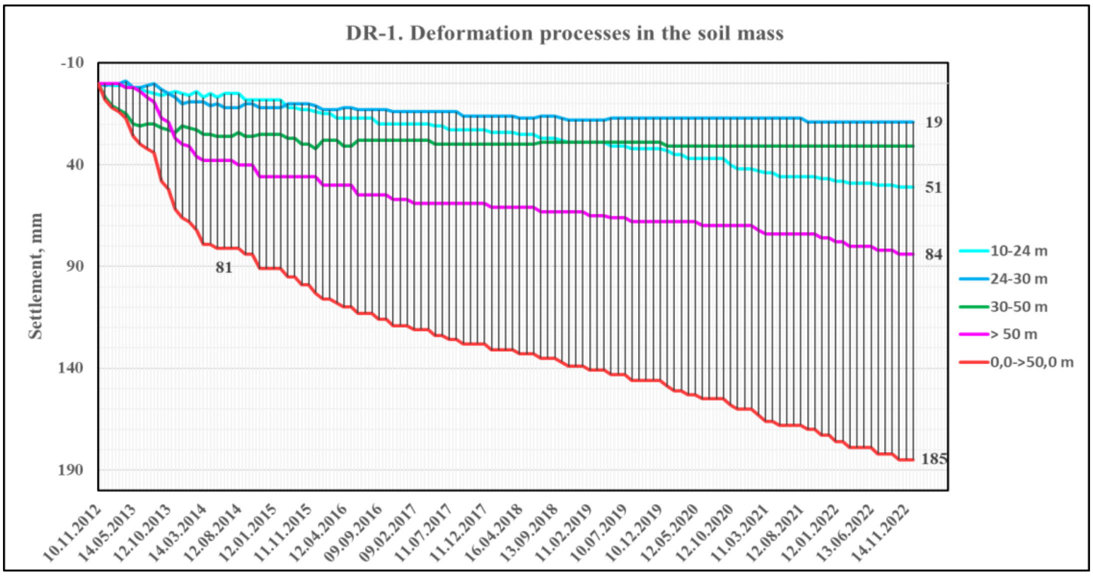

Figure 11. A continuation of deformations in depth in the soil mass forming the geological environment of the research area

The results of geotechnical monitoring show that in the study area, where work was carried out to strengthen the foundations, from February 2014 to the present, deformations in the soil foundation of the structures and settlements of the foundations have not occurred (Figure 9 and Figure 10).

However, in construction sites where appropriate measures are not taken to prevent deformation processes, in areas outside the boundaries of the research area, including in areas where "Depth Rappers" are installed, the process of deformation and settlement of the foundation continues, and the analysis of the results of the deformation processes suggests that these processes will continue for a long time (Figure 11).

4. Conclusions

- According to the mathematical-statistical and graphical analysis of the results of engineering surveys, the geological structure of the study area belongs to the III category of complexity and does not have favorable engineering-geological conditions for the construction of industrial buildings and structures up to the upper layer of argillite-like very stiff clay.
- Modern deposits of the Quaternary system of continental origin, studied in the depth interval of 0.0-81.6 m in the geological structure of the study area, are composed of technogenic (anQɪv) and alluvial-deluvial-proluvial (adpQɪv) deposits. According to the lithological composition, these deposits consist of the embankment, of the boulder-pebble-gravel layer of different granulometric composition (adQIV) with dusty-clay aggregates, of the layer of clay-buried soil (dpQIV), formed with the participation of plant remains, and of weakly cohesive (clay and loams of different consistency), dusty-clay dispersed soils (dpQIV and adpQIV) characterized by variable facies.
- The dusty-clay soils present in the geological environment up to the ceiling of the argillite-like very stiff clay layer are mainly characterized by subsiding loess and loess-like soils. Along with their inherent heterogeneity, anisotropy, and complex

engineering and geological properties, they are also characterized as specific soils (subsidence, swelling, organomineral, organic, technogenic, etc.) and are the main soils that determine deformation processes.

- Weak soils formed in the depth interval of 7.5-61.6 m in different horizons of the geological structure of the research area, especially quicksand, dusty-clay soils with a high liquid index (firm, soft-firm), significantly complicate the engineering and geological conditions of the area and lead to additional deformation processes.
- To prevent or minimize the formation of large-scale settlement and subsidence deformations in the soil base of structures built in the research area, the use of a foundation type consisting of piles of large length and diameter with the condition of placing the sole of the piles in a layer of argillite-like very stiff clay should be considered the most correct technical solution to this problem.
- After the work on strengthening the soil foundation using a system of foundations made of large-sized piles carried out in the second half of 2013 and the first months of 2014 in the foundations of structures built in the study area, deformation processes, including foundation settlements, stopped completely and in the specified area from February 2014 to the present time deformation processes and sediment the foundation did not happen.
- At the site where measures were not taken to prevent deformation processes and strengthen the foundations of structures and at the installation site of the "Deep rappers" (DR), as well as in areas that go beyond the boundaries of the study area, deformation processes and foundation settlements continue.
- In the course of geotechnical monitoring of deformation processes and interpretation of data obtained from the "deformation process measurement system", it was found that in many cases there is no linear relationship between deformation processes and static and dynamic load of structures, and the deformation process occurring in the soil mass depends mainly on the physical-geographical, physical-geological and geodynamic processes and in some cases from anthropogenic influences.
- Considering the results of this research work in the design and construction of industrial, civil, and infrastructure structures projects in areas with similar engineering and geological conditions, along with an increase in the reliability, strength, and durability of buildings and structures under construction, will save a significant amount of investment funds.

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